21

Spillways, Energy Dissipators, and Spillway Gates

21.1. Introduction

A spillway is a structure constructed at a dam site, for *effectively disposing of the surplus water from upstream to downstream.* Just after the reservoir gets filled up, up to the normal pool level, water starts flowing over the top of the spillway crest (which is generally kept at normal pool level). Depending upon the inflow rate, water will start rising above the normal pool level, and at the same time, it will be let off over the spillway. The water can rise over the spillway crest, upto the maximum reservoir level, which can be estimated from the inflow flood hydrograph and the spillway characteristics, by the process of flood routing, explained earlier. Therefore, it is only the spillway, which will dispose of the surplus water and will not let the water rise above the maximum reservoir level. Had there been no such structure, over which the water would have overflown, the water level must have exceeded maximum reservoir level, and ultimately would have crossed the freeboard and thus overtopped the dam, causing the failure of the dam. *Hence, a spillway is essentially a safety valve for a dam.* It must be properly designed and must have adequate capacity to dispose of the entire surplus water at the time of the arrival of the worst design flood.

Many dams have failed (especially the earthen dams) because of the improperly designed or inadequate spillways.

21.2. Location of a Spillway

A spillway can be located either within the body of the dam, or at one end of it or entirely away from it, independently in a saddle. If a deep narrow gorge with steep banks, separated from a flank by a hillock with its level above the top of the dam (such as shown in Fig. 21.1), is available, the spillway can be best built independently of the dam.

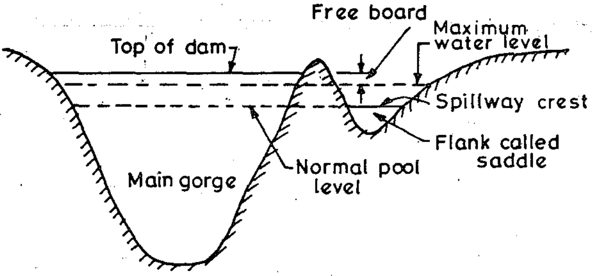


Fig. 21.1

Under such circumstances, a concrete or an earthen dam can be constructed across the main valley and a spillway can be constructed independently into the saddle. Sometimes, a concrete or a masonry dam along with its spillway can be constructed in the main valley, while the flank or flanks are closed by earthen dikes or embankments. The top level of such an embankment is kept at maximum reservoir level. The materials and designs of these embankments are such that they fail as soon as water overtops them. Hence, if by chance, either due to excessive flood above the design flood or due to failure of gates of main spillway, etc., the water rises above the maximum reservoir level, it shall overtop such embankment, which at once fails ; providing sufficient outlet for the disposal of excessive water. This type of a secondary safety arrangement is generally provided on large dams especially on earth and rockfill dams, and is *known as Subsidiary Spillway* or *Emergency Spillway* or *Breaching Section.*

The main spillway is constructed to dispose of the designed flood above the normal pool level and upto the maximum reservoir level. It is situated either within the dam, or at one end of it, or independently in a saddle away from the main dam. A separate independent spillway is generally preferred for earthen dams, although due to non-availability of sites, a concrete spillway is sometimes constructed within or at one of the ends of an earth dam. If the main spillway is situated in a flank, a secondary emergency spillway may be situated in another flank as shown in Fig. 21.2.

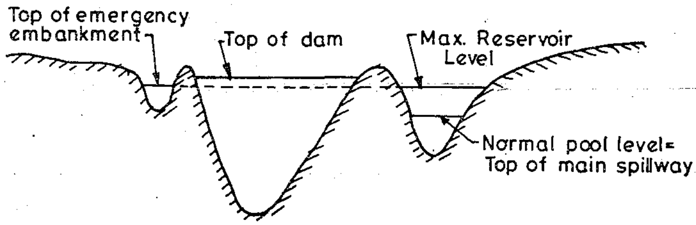


Fig. 21.2

21.3. Design Considerations for the Main Spillway

The main spillway, often called the spillway, is properly designed so as to dispose of the excess water without causing any damage to the dam, or to any of its appurtenant structures. The spillway structure should be structurally and hydraulically adequate and must not give way under worst and variable loading conditions.

The required discharging capacity of the spillway should be as closely estimated as possible. The underestimation will lead to overtopping of the main dam and its consequent damages ; while the over estimation will lead to unnecessarily costly constructions which shall never he utilised during the life of the dam, and hence, will remain a waste investment. However, on large dams, a conservative view is always preferred because the failure of a single dam due to inadequate capacity may result in the loss of numerous human lives to which no cost allocation can be made. Moreover, an emergency spillway or a breaching section is generally provided, the failure of which under necessary circumstances, may though cause serious erosion on the downstream, but shall protect the main dam from failure.

The water passing over the spillway and falling on the downstream side must not be allowed to erode the downstream soil, and hence, arrangements must be made for effectively dissipating the energy of the falling water.

21.4. Controlled and Uncontrolled Spillways

The flow of water over a spillway may be controlled by installing gates over the spillway crest. In such a case, the spillway is known as a controlled spillway. The out flow can be controlled in such spillways and hence, preferred in modern days. However, some spillways are left just by constructing their crest at normal pool level. As water will flow over such a spillway, depending upon the reservoir level and the corresponding head over the spillway, such uncontrolled spillways are guided only by the available water head, and hence, are called uncontrolled spillways. The comparative advantages and disadvantages of controlled and uncontrolled reservoirs have been discussed in chapter 18.

VARIOUS TYPES OF SPILLWAYS

Depending upon the type of the structure constructed for disposing of the surplus water, the spillways can be of the following major types :

(1) Straight Drop Spillway.

(2) Overflow Spillway generally called *Ogee Spillway.*

(3) *Chute Spillway* often called *Trough Spillway* or Open channel Spillway.

(4) Side Channel Spillway.

(5) Shaft Spillway.

(6) Syphon Spillway.

The various types of spillways enumerated above are described below along with the design details of 'Ogee Spillway' and 'Chute Spillway'.

21.5. Straight Drop Spillway or Overfall Spillway

This is the simplest type of spillway and may be constructed on small bunds or on thin arch dams, etc. It is a low weir and simple vertical fall type structure, as shown in Fig. 21.3. The downstream face of the structure may be kept vertical or slightly inclined. The crest is sometimes extended in the form of an overhanging lip, which keeps small discharges away from the face of the overfall section. The water falls freely from the crest under the action of gravity. 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. The design of such a spillway is done as that of a weir which was explained in the chapter on

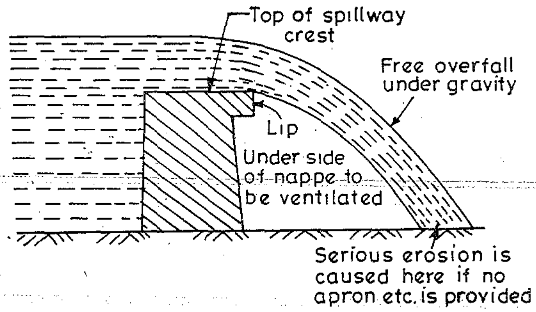


Fig. 21.3. (*a*) Straight drop spillway without d/s protection.

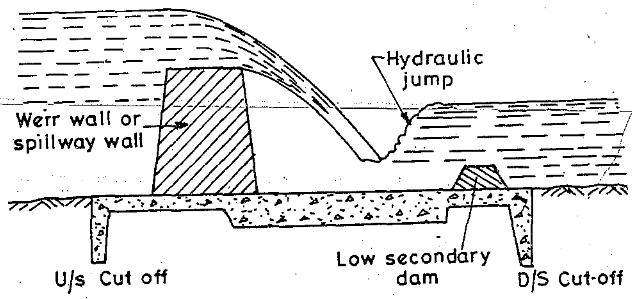


Fig. 21.3. (*b*) Straight drop spillway with d/s protection works.

weirs. Sometimes, a secondary dam of low height is constructed on the downstream side to create an artificial pool of water so as to dissipate the energy of the falling water.

21.6. Ogee Spillway or Overflow Spillway

Ogee spillway is an improvement upon the 'free overfall spillway, and is widely used with concrete, masonry, arch and buttress dams. Such a spillway can be easily used on valleys where the width of the river is sufficient to provide the required crest length

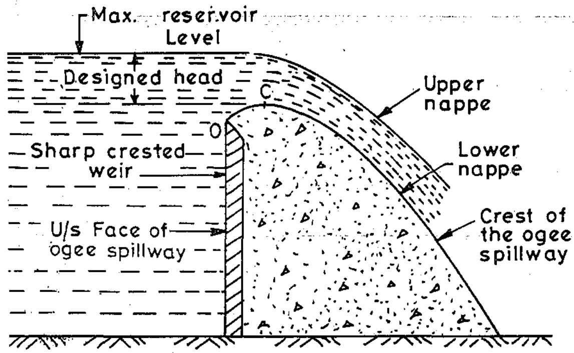


Fig. 21.4. (*a*) Section of an ogee spillway with vertical u/s face.

and the river bed below can be protected from scour at moderate costs. *The profile of this spillway is made in accordance with the shape of the lower nappe of a free falling jet, over a duely ventilated sharp crested weir,* as shown in Fig. 21.4 (*a*).

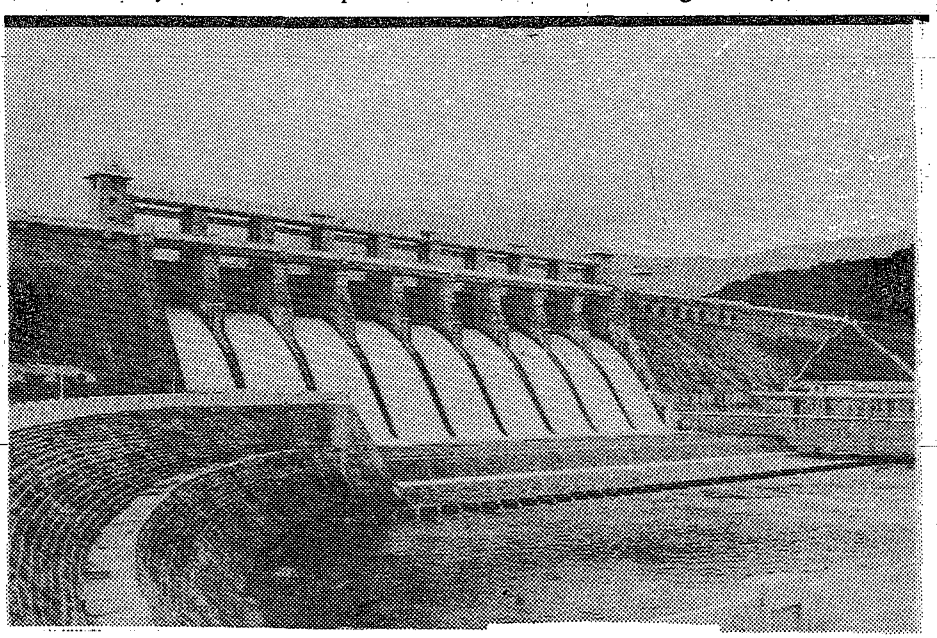


Fig. 21.4. (*b*) Photoview of an Ogee Spillway of Amaravathi dam (earthern cum masonry dam) located on river Amaravathi (a tributary of river Cauvery) in Coimbatore District in Tamil Nadu state.

The shape of the lower nappe of freely falling jet over a sharp crested weir can be determined by the principle of projectile. It generally rises slightly (to point *C*) as it originates from the crest (*O*) of a sharp crested weir and then falls to make a parabolic form. Now, if the space between the sharp crested weir and the lower nappe is filled with concrete or masonry, the weir so formed will have a profile similar to an 'ogee' (*S*-shaped curve in section), and hence called an 'ogee weir' or an 'ogee spillway'. This lower nappe, will then become the crest of the spillway. Since the lower nappe of the free falling jet will be different for different heads over the crest of the sharp crested weir, *the profile of the ogee weir is generally confined to the lower nappe that would be obtained for maximum head over the spillway (i.e. upto the maximum reservoir level).*

In a free overfall spillway, the water jet falls clearly away from the face of the spillway, and the gap between the jet and the face is kept ventilated. While in an ogee spillway, the falling water glides over the curved profile of the spillway, and there is no space between water and crest of the spillway, under normal design conditions.

Normally, the upstream face of the spillway is kept vertical and the crest shape confirms to the lower nappe of a vertical sharp crested weir under maximum head. But if the upstream face of the spillway is kept sloping, the crest shape should also confirm to the lower nappe that would be obtained for an inclined sharp crested weir (Fig. 21.5).

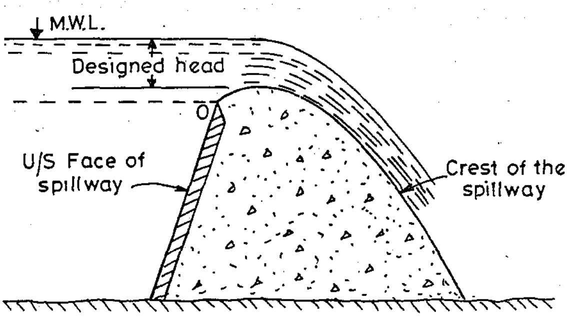


Fig. 21.5. Ogee spillway with inclined u/s face.

21.6.1. Cavitation. The crest of the ogee spillway can be made to confirm only to one particular nappe that would be obtained at one particular head. This head is called the *designed head* and represented by, say, H_d. But in practice, the actual head of water on the spillway crest, called the *operating head,* may be less or more than the designed head. *If this operating head on the spillway is more than the designed head, the lower nappe of the falling jet may leave the ogee profile, thereby generating negative pressure at the point of separation.* The generation of vacuum or negative pressure (*i.e.* pressure below the atmospheric pressure) may lead to formation of bubbles or cavities in the water. These cavities or bubbles filled with air, vapour and other gases are formed in a liquid, whenever the absolute pressure (*i.e.* atmospheric pressure—vacuum pressure) of the liquid is close to its vapour pressure, so as to commence evaporation. Such a condition may arise when the head of water is more than the designed head and the consequent high velocity jet causes reduced pressures or negative pressures in the lower region of the water jet.

Such cavities, on moving downstream, may enter a region where the absolute pressure is much higher (*i.e.* more vacuum). This causes the vapour in the cavity to condense and return to liquid with a resulting implosion or collapse of the cavity. When the cavity collapses, extremely high pressures are generated. The continuous bombardment of these implosions will thus take place near the surface of the spillway, causing fatigue failure of its material. The small particles of concrete or masonry are thus broken away, causing formation of pits on its surface and giving the surface a spongy appearance. This damaging action of cavitation is called 'pitting'. The cavitation plus the vibrations from the alternate making and breaking of contact between the water and face of the spillway, may thus result in serious structural damages to the spillway crest. *Hence, it can be concluded that if the head of water over the spillway is more than the designed head, cavitation may occur. On the other hand, if the head of water over the spillway is less than the designed head, the falling jet would adhere to the crest of the ogee spillway, creating positive hydrostatic pressures and thereby reducing the discharge coefficient of the weir.*

21.6.2. Designing the Crest of the Ogee Spillway. The ogee spillways were being designed in the earlier periods, in accordance with the theoretical profile obtained for the lower nappe of a free falling jet. The profile was known as Bazin's profile. Theoretically, the adoption of such a profile, should cause no negative pressures on the crest under designed head. But in practice, there exists a lot of friction due to roughness on the surface of the spillway. Hence, negative pressure on such a profile seems inevitable. The presence of negative pressure causes the danger of cavitation and sometimes fluctuations and pulsations of the nappe. Hence, while adopting a profile for the spillway crest, the avoidance of negative pressures must be an objective along with consideration of other factors such as practicability, hydraulic efficiency, stability and economy. Depending upon research work based on these objectives, various modified profiles have been proposed these days.

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 d/s profile can be represented by the equation

$$x^n = K \cdot H_d^{\,n-1} \cdot y \qquad \text{...(21.1)}$$

where (x, y) are the co-ordinates of the points on the crest profile with the origin at the highest point C of the crest, called the apex.

H_d is the *design head including the velocity head.*

K and n are constants depending upon the slope of the upstream face. The values of K and n are tabulated in Table 21.1.

Table 21.1

Slope of the u/s face of the spillway	K	n
Vertical	2.0	1.85
$1:3\ (1H:3V)$	1.936	1.836
$1:1\frac{1}{2}\ (1H:1\frac{1}{2}V)$	1.939	1.810

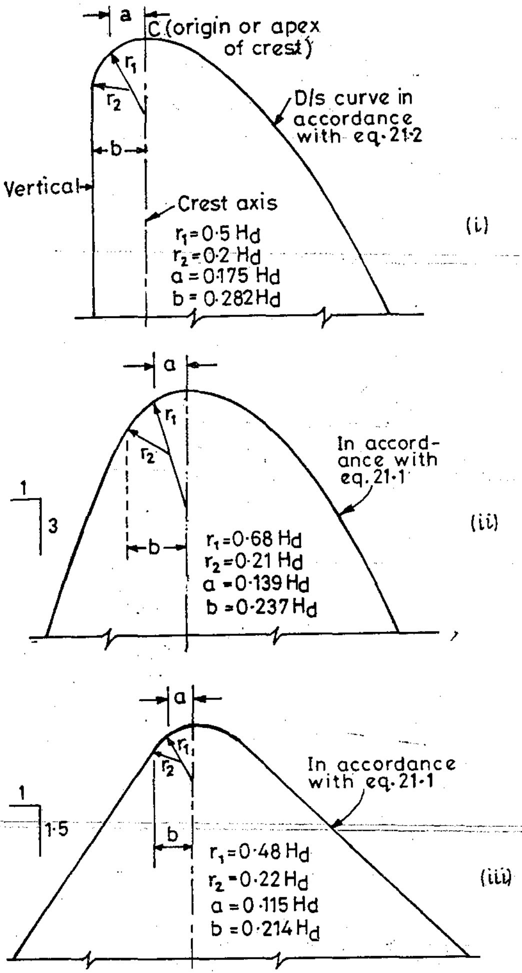


Fig. 21.6. WES profiles for Ogee Spillways for different u/s slopes.

Thus, for a spillway having a vertical u/s face, the d/s crest is given by the equation

$$x^{1.85} = 2 \cdot H_d^{0.85} \cdot y \qquad ...(21.2)$$

Different upstream curves were given by WES for different slopes, as shown in Fig. 21.6.

According to the latest studies of U.S. Army Corps, the u/s curve of the ogee spillway having a vertical u/s face, should have the following equation :

$$y = \frac{0.724\,(x + 0.27\,H_d)^{1.85}}{H_d^{0.85}} + 0.126\,H_d - 0.4315\,H_d^{0.375} \times (x + 0.27\,H_d)^{0.625} \qquad ...(21.3)$$

The u/s profile extends up to $x = -0.27\,H_d$

Co-ordinates for the upper nappe for various WES shapes of ogee spillway are also available and can be utilised in the design of training walls and spillway bridges, etc.

The profile for an ogee spillway, having a vertical upstream face, can be determined on the basis of its WES profile, or Table 21.2 may be used for making the ogee profile.

Table 21.2. Table for marking Ogee profile

$\frac{x'}{H_d}$	$\frac{y}{H_d}$	
	Lower Nappe	*Upper Nappe*
0	– 0.125	0.831
0.10	– 0.033	0.807
0.25	– 0.000	0.763
0.50	– 0.034	0.668
0.75	– 0.129	0.539
1.0	– 0.283	0.373
1.5	– 0.738	– 0.088
2.0	– 1.393	– 0.743
3.0	– 3.303	– 2.653
4.0	– 6.013	– 5.363
5.0	– 9.523	– 8.873

(x', y) are the co-ordinates of a point on the profile as shown in Fig. 21.7.

The co-ordinates of the lower nappe determine the crest profile, while the plotting of the upper nappe is useful in determining the clearance for the spillway deck bridge and the top levels of training walls on the sides of the spillway.

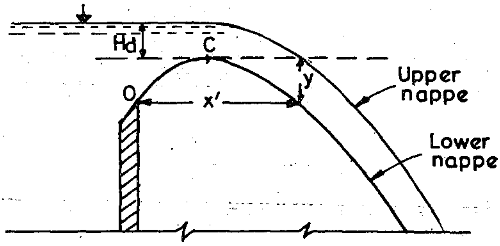


Fig. 21.7

After having plotted most of the profile of the ogee spillway, a smooth gradual reverse curvature is provided at the bottom of the downstream face, as shown in Fig. 21.8. The reverse curve turns the flow into the apron of a stilling basin or into the spillway discharge channel. A radius of about one-fourth of the spillway height is satisfactory for this reverse bottom curve.

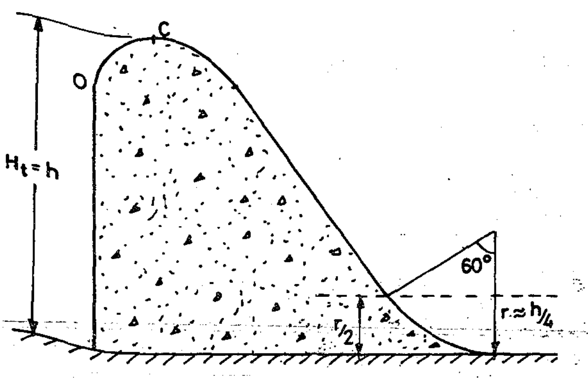


Fig. 21.8

21.6.3. Practical Profile Consistent with the Profile of Gravity Dam. When the profile for the crest of the ogee spillway is plotted over the triangular profile of the gravity dam (non-overflow section), it is found that it goes beyond the downstream face of the dam, thus requiring thickening of the section for the spillway (Fig. 21.9). This extra concrete required, can be saved by shifting the curve of the nappe in a backward direction until this curve becomes tangential to the downstream face of the dam (Fig. 21.10). Hence, a saving can thus be affected, by providing a projecting corbel on the upstream face of the spillway section (Fig. 21.10). The construction of the spillway is thus carried out as if it was a non-overflow dam. Only the slight modifications are made after reaching the required height (up to O) at

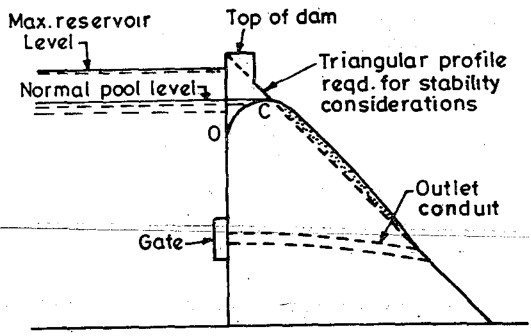


Fig. 21.9

which corbel is provided, and a smooth required curve *OCA* is given, as shown in Fig. 21.10.

If the spillway is provided with gates on the upstream face to control the flow in outlet conduits (Fig. 21.9), then the corbel will interfere with gate operation and hence the above concrete saving cannot be affected.

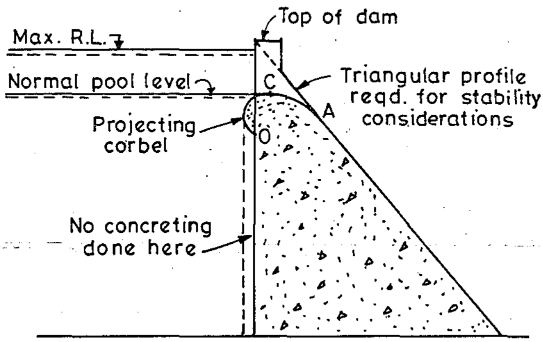


Fig. 21.10. Provision of Corbel.

The structural design and stability requirements of ogee spillway are exactly the same as that of a gravity dam. The forces acting on a gravity dam also act on the ogee spillway and remain predominant. The pressure exerted on the crest of the spillway by the flowing water and the drag force caused by the fluid-friction are usually negligible as compared to other forces which are acting on a gravity dam and also act on an ogee spillway. Hence, the design and construction of the ogee spillway is consistent with that of the gravity dam. Due to this reason, the spillway is sometimes called overflow portion of the dam, and the real dam section is termed as non-overflow section of the dam.

However, the change in momentum of the flow in the vicinity of the reverse curve may create a dynamic force on the spillway, in addition to the forces acting on the gravity dam. This force must be considered in designing bottom of the d/s face of spillway or the bucket type energy dissipator. This is dealt separately while discussing the design of energy dissipators for the spillways.

21.6.4. Discharge Formula for the Ogee Spillway. The discharge passing over the ogee spillway is given by the equation :

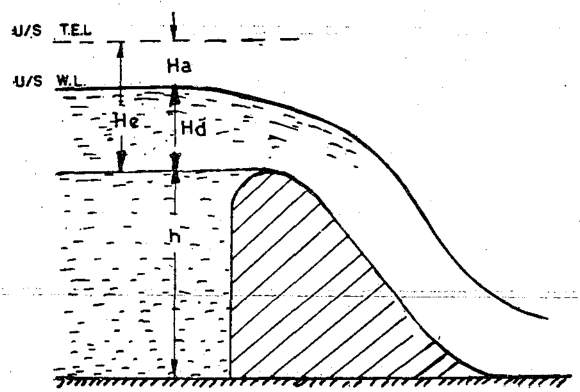


Fig. 21.11

$$Q = C \cdot L_e H_e^{3/2} \qquad \text{...(21.4)}$$

where Q = Discharge

L_e = Effective length of the spillway crest

C = Coefficient of discharge which depends upon various factors such as relative depth of approach, [*i.e.* d/H_d ratio (Fig. 21.11), relation of actual crest shape to the ideal nappe shape, slope of upstream face, downstream apron interference, and submergence, etc.

H_e = Total head over the crest including the velocity head.

If the discharge Q is used as the design discharge in Equation (21.4), then the term H_e will be the corresponding design head (H_d) plus the velocity head (H_a). In such a case, $H_e = H_d + H_a$. For high ogee spillways, the velocity head is very small, and $H_e \approx H_d$.

21.6.5. Variation of Coefficient of Discharge with Various Factors. This is discussed below in details :

(*i*) *Depth of approach.* The coefficient of discharge firstly depends upon the depth of approach. In other words, it depends upon the height of the ogee weir (h) to the design head over the weir (H_d). If the height of the weir is more than 1.33 times the design head, the velocity of approach has been found to have a negligible effect upon discharge, and as such H_d becomes equal to H_e or $\frac{H_e}{H_d} = 1.0$. In such a case, the coefficient of discharge, say $C = C_d$, has been found to *be 2.2 in M.K.S. or S.I. Units* and 4.03 in *F.P.S. Units.*

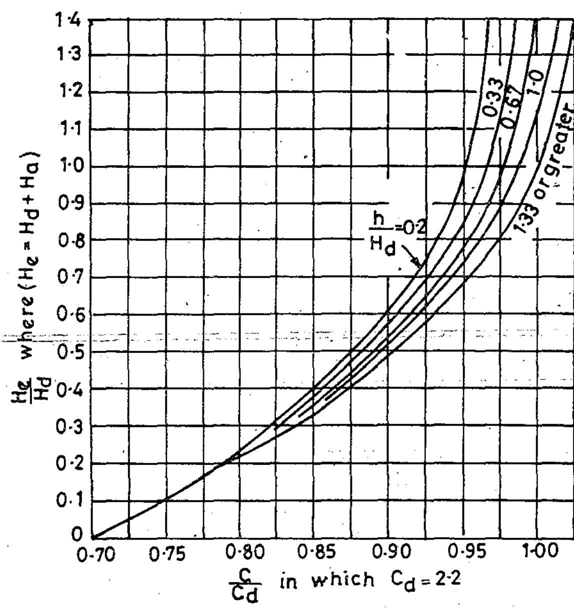


Fig. 21.12

However, in low spillways, with $\frac{h}{H_d} < 1.33$; the approach velocity is having an appreciable effect. The curves given in Fig. 21.12 can be used in such cases, to evaluate the coefficient of discharge C, using $C_d = 2.2$.

(*ii*) *U/s Slope.* The coefficient of discharge is also affected by the slope of the u/s face of the ogee weir. The values of C_d and C found up to now were for a vertical upstream face. If the u/s face is sloping, a correction factor by which the above values of C should be multiplied can be obtained from the curves given in Fig. 21.13.

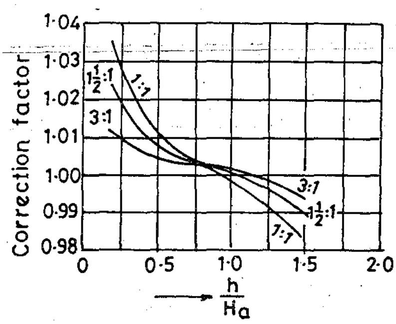


Fig. 21.13

(*iii*) *D/s apron interference and submergence effects.* The third important factor affecting the discharge coefficient of an ogee weir is the effect of downstream apron interference and downstream submergence.

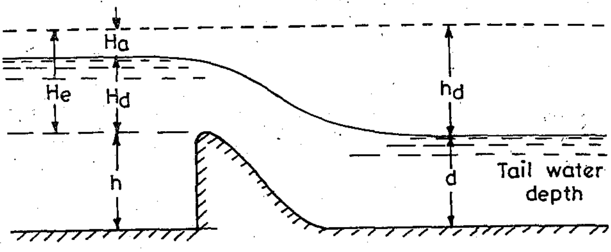


Fig. 21.14. Maximum tail water depth for a non-submerged weir.

When the tail water 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 a **submerged weir.**

Where the hydraulic jump occurs, the coefficient of discharge may decrease due to back pressure effect of the downstream apron and is independent of the submergence effect. When the value of $\frac{h_d + d}{H_e}$ (Fig. 21.15) exceeds 1.7, the downstream apron is found to have negligible effect on the coefficient of discharge. But there may be a decrease in the coefficient due to tail water submergence. The correction factor, by which the value of C should be multiplied in order to get the modified or correct value of coefficient of discharge, can be obtained from the curve of Fig. 21.15.

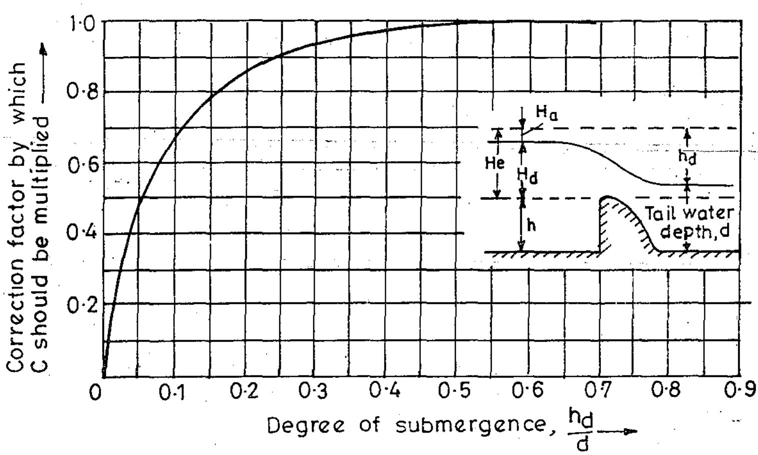


Fig. 21.15. Effect of submergence on C.

21.6.6. Effects of Actual Prevailing Head on the Discharge Capacity of a Spillway. It was pointed out earlier that when once a spillway has been designed and constructed for a design head (H_d), and for a corresponding coefficient of discharge (say C), it will not always find the same head over its crest in its actual operations. The actual operating head (H) i/c velocity head, may be less or more than the designed head. Since the design is done for maximum head, the possibility of a head more than the designed head is very meagre. When the actual operating head passing over the spillway is less than the designed head, the prevailing coefficient of discharge (C_d) tends to reduce, and is given by the equation

$$C_d = C \cdot \left(\frac{H}{H_e}\right)^{0.12} \qquad ...(21.5)$$

where H_e is the designed head including velocity head.

Since an overflow spillway is sufficient in height (*i.e* $h > 1.33\, H_d$) ; the coefficient of discharge C at designed head can be taken as 2.2. The prevailing coefficient of discharge at 50% head (*i/c* velocity head) will then be

$$C_d = 2.2 \times \left(\frac{0.5\, H_e}{H_e}\right)^{0.12} = 2.2 \times 0.92 = 2.02.$$

Similarly, for still lower heads, the coefficient of discharge goes on reducing and tends to become constant at about 1.7. (Because at very low heads, the velocity head becomes the governing factor, which tries to make H a constant).

The effective length (L_e) *of ogee spillway.*

The effective length of the spillway crest (L_e) to be used in equation (21.4), is given by the equation

$$L_e = L - 2\left[K_p \cdot N + K_a\right] H_e \qquad ...(21.6)$$

where L = the net clear length of the spillway crest

K_p = Pier constraction coefficient

K_a = Abutment contraction coefficient

N = Number of piers

H_e = Total design head on the crest including velocity head.

The values of K_p and K_a depend mainly upon the shape of the piers and that of the abutments. The greater is the divergence from streamlined flow, the greater is the constraction coefficient and lesser is the effective length of the crest. A 90° cut water nose pier is most efficient and has quite a low value of K_p, and is generally preferred. Values of K_p and K_a are given in Tables 21.3 and 21.4. Various shapes of piers are shown in Fig. 21.16.

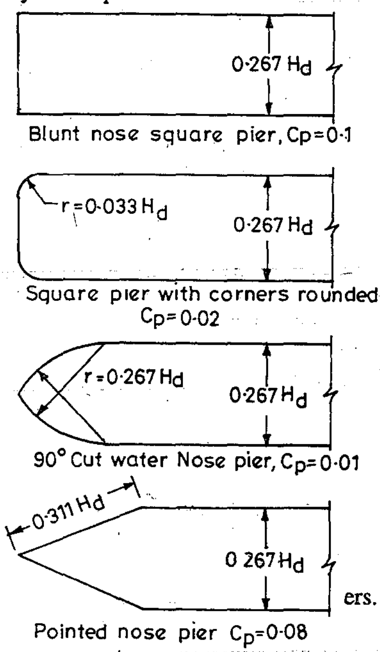


Fig. 21.16. Various shapes of piers.

Table 21.3

S. No.	*Pier Shape*	*Contraction coefficient* K_p
1.	Square nosed piers without any rounding	0.1
2.	Square nosed piers with corners rounded on radius equal to 0.1 of pier thickness	0.02
3.	Rounded nose piers and 90° cut water nosed piers	0.01
4.	Pointed nose piers	0.0

Table 21.4

S. No.	*Shape of abutment*	*Contraction coefficient* K_a
1.	Square abutment with head wall at 90° to the direction of flow	0.2
2.	Rounded abutment with head wall at 90° to the direction of flow	0.1

21.6.7. Aeration Arrangements in Gated Ogee Spillways. The entire design of an overflow spillway has been done with the assumption that upper and lower nappe are subjected to full atmospheric pressure. But in practice, due to insufficient aeration, development of negative pressures takes place beneath the nappe due to removal of air by the falling jet. The development of negative pressures causes the danger of cavitation and induces fluctuation and pulsation effects on the jet, which may be very objectionable if the spillway is to be used as a discharge measuring device.

To control the development of negative pressures, aeration pipes 25 mm dia at say 3 m centre to centre may be provided along spillway face below the gate lip. These pipes can be connected to a bigger sized header. Formulas have been developed on the basis of model studies, for evaluating the amount of air required for aeration, and may have to be used in large dam designs, but are beyond the scope of this book.

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

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

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

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

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

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

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

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

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

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

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

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

or $$y = \frac{x^{1.85}}{21.6} \qquad \text{...(21.7)}$$

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

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

The co-ordinates from $x = 0$ to $x = 22.4$ m are worked out in Table 21.5.

Table 21.5

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

The u/s profile. The u/s profile may be designed as per equation (21.3), as :

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

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

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

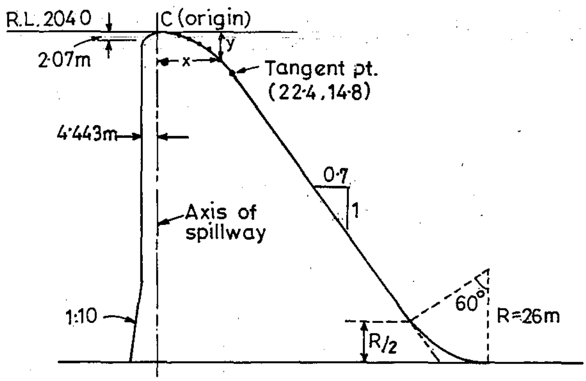


Fig. 21.17

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 pressures. The energy dissipation arrangements have not been shown. They should be designed depending upon the position of the jump height curve and tail water curve, as explained afterwards. A sky jump bucket or an apron may be provided as per the prevailing conditions.

21.7. Chute Spillway or the Trough Spillway

An ogee spillway is mostly suitable for concrete gravity dams especially when the spillway is located within the dam body in the same valley. But for earthen and rockfill dams, a separate spillway is generally constructed in a flank or a saddle, away from the main valley, as explained earlier. Sometimes, even for gravity dams, a separate spillway is required because of the narrowness of the main valley. In all such circumstances, a separate spillway may have to be provided. The *Trough Spillway* or *Chute Spillway* is the simplest type of a spillway which can be easily provided independently and at low costs. It is lighter and adaptable to any type of foundations ; and hence provided easily on earth and rockfill dams. A chute spillway is sometimes known as a **waste weir**. If it is constructed in continuation to the dam at one end, it may be called a **flank weir**. If it is constructed in a natural saddle in a bank of the river separated from the main dam by a high ridge, it is called a **saddle weir.**

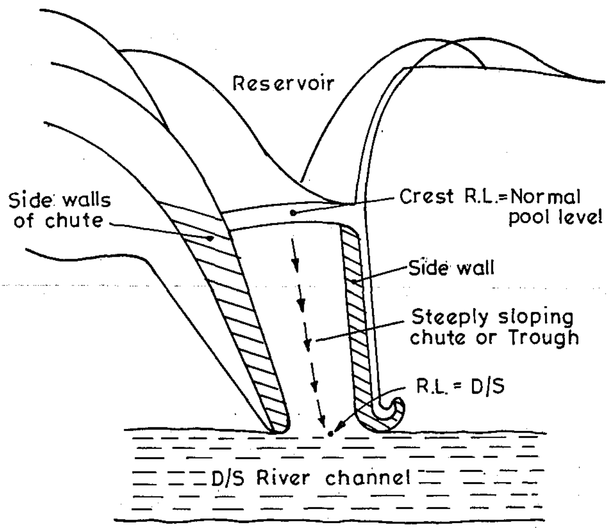


Fig. 21.18 (*a*). Simplified Line sketch of a Chute Spillway.

A chute spillway (Fig. 21.18) essentially consists of a steeply sloping open channel, placed along a dam abutment or through a flank or a saddle. It leads the water from the reservoir to the downstream channel below. The base for the channel is usually made of reinforced concrete slabs, 25 to 50 cm thick. Light reinforcement of about 0.25% of the concrete area is provided in the top of the slabs in both directions. The chute is sometimes 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 chute at intervals of about 10 metres in either direction. The expansion joints should be made watertight 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 under-drains may be in the form of perforated steel pipes, clay tiles or rock-filled trenches.

If the slope of the chute can confirm to available topography, the excavations shall be minimum. But the slope of the chute must be high enough, and should atleast be able to maintain super critical flow, to avoid unstable flow conditions.

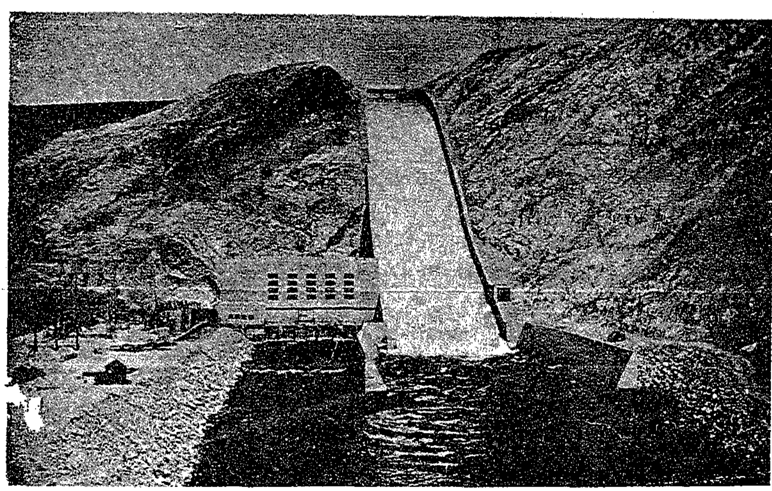


Fig. 21.8. (*b*) Photoview of the Chute Spillway of Anderson Ranch Dam, Idaho (USA).

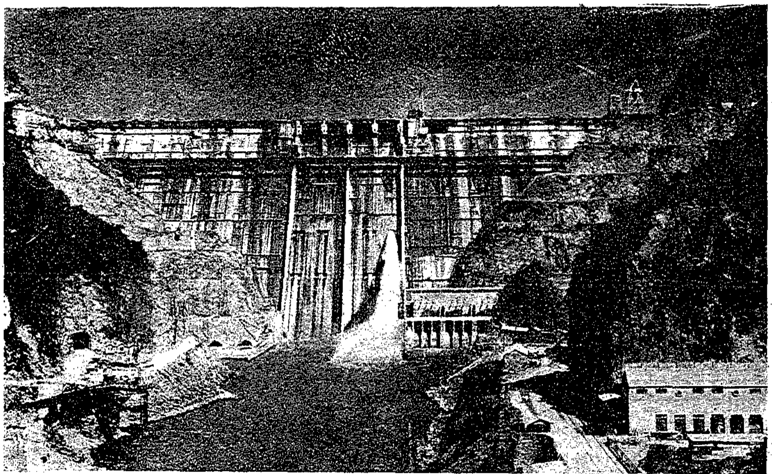


Fig. 21.18. (*c*) Photoview of two chutes provided at Bhakra Dam (India).

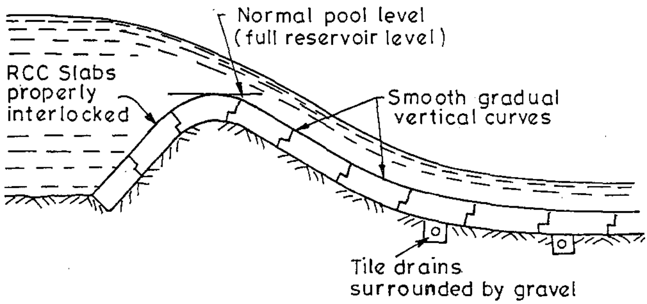


Fig. 21.18. (*d*) Section through a Chute Spillway.

When a vertical curve is provided at a point where the chute slope changes, it must be gradual and designed so as to avoid any separation of flow.

21.7.1. Control Structure or a Low Ogee Weir. Since the trough spillway is provided in a flank or a saddle, the height of spillway or ogee weir required to be constructed in that flank, will be small ; sometimes almost flat low weir shall be required depending on the natural levels of the bottom of the flank. If the flank bottom is at a level lower than the natural pool level, an ogee weir shall have to be constructed upto that level. If the flank bottom is at higher level than the normal pool level, excavations will have to be done upto that level. In such a case, the weir crest is normally left flat as it shall seldom be economical to excavate the rock just for the sake of constructing on 'ogee shape' for obtaining high coefficient of discharge.

21.7.2. Chute Slope. The water spilling over the control structure (*i.e.* ogee weir), then flows through the chute channel. The minimum slope of the chute is governed by the condition that supercritical flow must be maintained. The slope of the chute is kept just sufficient to meet this flow requirement from the crest for as long a distance as possible without any filling. After that, the slope is made as steep as possible without endangering the stability or without getting into heavy excavations.

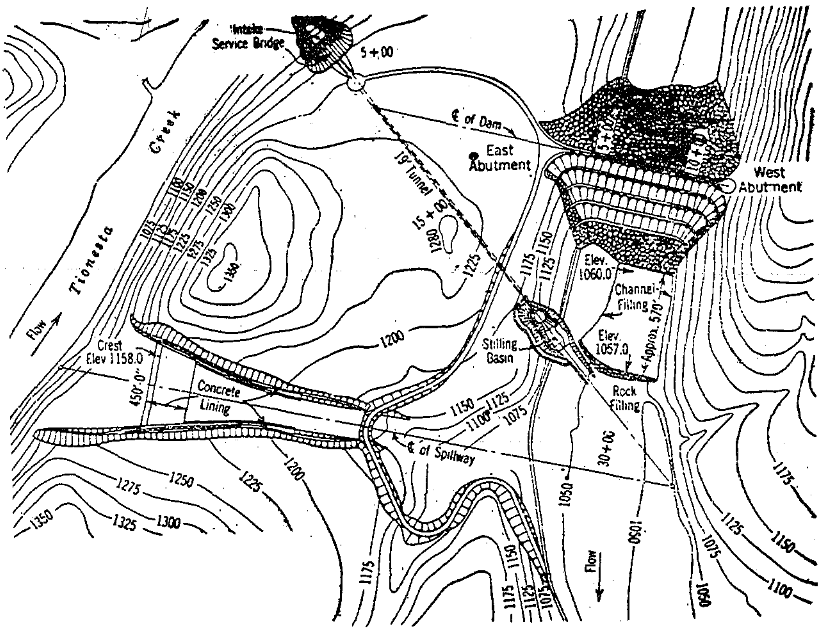


Fig. 21.18. (*e*) Layout Plan of Tionesta Chute Spillway.

21.7.3. Side Walls. The side walls (called training walls) of the chute should be of such a height that water does not spill over them. A sufficient freeboard must, therefore, be provided above the top water nappe, for obtaining the top levels of the side walls. The freeboard is generally given by equation,

$$\text{Freeboard} = 0.61 + 0.04\, V_m \cdot d_m^{1/3} \text{ (M.K.S. or S.I. units)} \qquad ...(21.9)$$

where V_m = mean velocity of water in the chute reach

d_m = mean depth of water in the chute reach under consideration.

The side walls of the chute may be kept vertical or sloping. But in the vicinity of gated ogee weirs, they will have to be vertical. Generally, a rectangular chute channel is designed.

21.7.4. Design of the Small Ogee Weir Required as a Control Structure for the Chute Spillway. The profile for low ogee weir as recommended by W.E.S. confirms to the following equations for different ratios of H_a/H_e, where H_a is the head due to velocity of approach and H_e is the design head over the crest including velocity head, *i.e.* $(H_e = H_d + H_a)$.

Table 21.7. Table showing equations for d/s profile of a low ogee weir

Values of $\frac{H_a}{H_e}$	$\frac{h}{H_e}$ *range*	*Equation for the d/s profile*
0.0	≥ 1.0	$x^{1.78} = 1.852\, H_e^{0.78} \cdot y$
0.08	$1.0 - 0.58$	$x^{1.75} = 1.869\, H_e^{0.75} \cdot y$
0.12	$0.58 - 0.30$	$x^{1.747} = 1.905\, H_e^{0.747} \cdot y$

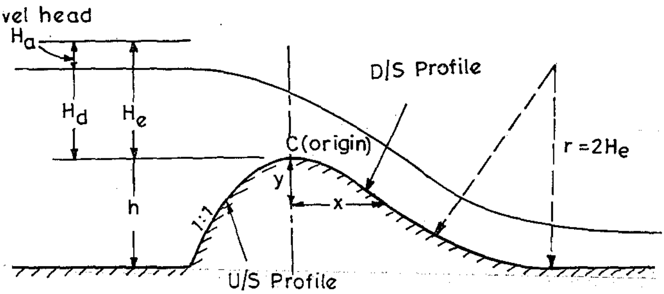


Fig. 21.19. Profile of the small ogee weir to be used as control structure for a chute spillway.

The co-ordinates of the u/s profile, which should merge in a slope of 45° (*i.e.* 1 : 1), as shown in Fig. 21.19, are given in Table 21.8.

Table 21.8. Co-ordinates of u/s profile for low ogee weir

$\frac{x}{H_e}$	$\frac{y}{H_e}$ for different values of $\frac{H_a}{H_e}$		
	$\frac{H_a}{H_e}=0.00$	$\frac{H_a}{H_e}=0.08$	$\frac{H_a}{H_e}=0.12$
– 0.000	0.0000	0.0000	0.0000
– 0.020	0.0004	0.0004	0.0004
– 0.060	0.0036	0.0035	0.0035
– 0.100	0.0103	0.0101	0.0099
– 0.120	0.0150	0.0150	0.0147
– 0.140	0.0207	0.0208	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	0.0420	—
– 0.200	0.0450	—	—

The reverse curve at the toe may have a radius equal to $2H_e$.

21.7.5. Design of Vertical Curves of the Chute. This is discussed below :

Concave Curve. Whenever the slope of chute changes from steeper to milder [Fig. 21.20 (*a*)], a concave vertical curve shall be provided. In no case, the radius of this curve should be less than 10 *d*, where *d* is the depth of water in metres.

Convex curve. Whenever the slope of the chute changes from milder to sleeper [Fig. 21.20 (*b*)], convex vertical curve shall have to be provided. The curvature should approximate to a parabolic shape given by the equation :

$$q = -x \tan\theta - \frac{x^2}{K\,[4\,(d+h_v)\cos^2\theta]} \qquad ...(21.10)$$

where θ is the slope angle of the floor upstream ;

$(d + h_v)$ is the specific energy of flow at the junction point ;

K is constant which is ≥ 1.5.

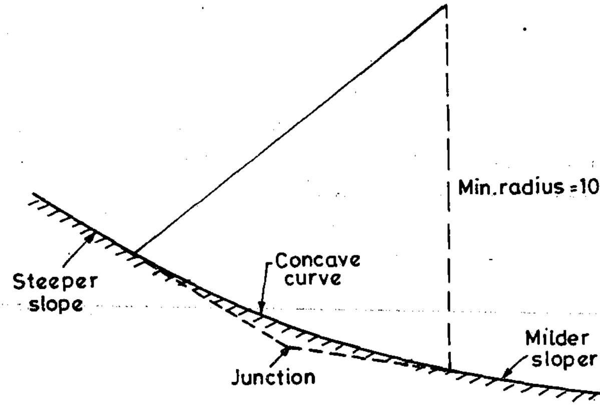


Fig. 21.20 (*a*). Concave Vertical Curve.

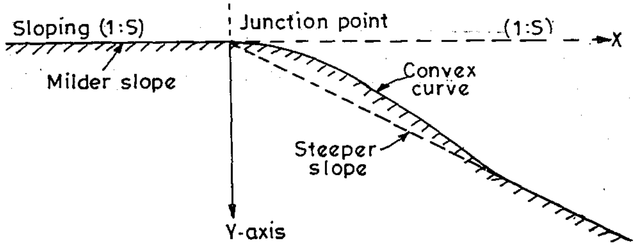


Fig. 21.20 (*b*). Convex Vertical Curve.

21.7.6. Approach Channel of Chute Spillway. An entrance channel called an approach channel, trapezoidal in shape with side slopes 1 : 1, may be constructed so as to lead the reservoir water up to the control structure (low ogee weir). If any curvature (in plan) is required, it is generally confined to the entrance channel, because the velocity of flow is low in this channel. The chute channel called the discharge channel or *discharge carrier* is generally kept straight in plan. If, however, any curvature is required to be provided, its floor should be super elevated to guide the high velocity flow around the bend, thus avoiding piling up of flow towards the outside of the chute. The friction head lost in the entrance channel upto the spillway crest can be calculated by Manning's formula, given as :

$$h_f = S_f \times L = \frac{n^2 \cdot V^2 \cdot L}{R^{4/3}} \qquad ...(21.11)$$

where n = Manning's coefficient of roughness
V = Velocity in channel
R = Hydraulic mean depth
L = Length of channel
S_f = Mean energy slope between two points.

The entire chute spillway can hence be divided into the following parts :

(*i*) Entrance channel

(*ii*) Control structure (Low Ogee weir)

(*iii*) Chute channel or Discharge carrier

(*iv*) Energy dissipation arrangements at the bottom in the form of a stilling basin.

The design principles for the various components have already been explained except those for energy dissipation. The energy dissipation arrangements required at the bottom of the spillway shall be explained a little later.

Example 21.2. *Design a suitable profile for a chute spillway with the following data :*

Spillway crest level = *200.0 m*

Level of bottom of flank at which the low Ogee weir is to be constructed = *192.0 m*

Design discharge = *5,000 cumecs.*

Downstream tail water level corresponding to 5,000 cumecs = *103.0 m.*

The spillway length consists of 5 spans of 10.0 m clear width each. The thickness of each spillway pier may be assumed to be 3 m. Assume any other data required.

Solution.

Design of approach channel :

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

Assume the coefficient of discharge as 2.18, and taking L_e as the clear width (approximately) $= 10 \times 5 = 50$ m :

and assuming $H_e \approx H$

we get $5{,}000 = 2.18\,(50)\,H^{3/2}$

or $$H^{3/2} = \left(\frac{100}{2.18}\right) = 45.8 \text{ m.}$$

or $$H = (45.8)^{0.667} = 12.8 \text{ m.}$$

Upstream water level = Crest level + $H = 200 + 12.8 = 212.8$ m.

Bed level of river in flank = 192.0

$\therefore$ Water depth $= 212.8 - 192.0 = \mathbf{20.8}$ **m.**

Assuming the trapezoidal approach channel with 1 : 1 side slopes,

The width of the channel = B

$= $ Total length of spillway $= 50 + 4 \times 3 = 62$ m.

Area of channel $= A = (B + y)\,y = (62 + 20.8)\,20.8 = 1{,}720$ sq. m.

Velocity of approach $= V_a = \dfrac{5{,}000}{1{,}720} = 2.9$ m/sec

Velocity Head $= H_a = \dfrac{V_a^2}{2g} = \dfrac{(2.9)^2}{2 \times 9.81} = \mathbf{0.43}$ **m.**

The wetted perimeter P of the channel

$= [B + 2\sqrt{2}$ (water depth)]

$= [62 + 2\sqrt{2} \times 20.8] = (62 + 58.8) = 120.8$ m.

$$R = \frac{A}{P} = \frac{1{,}720}{120.8} = \mathbf{14.2 \text{ m.}}$$

Assuming the length of the channel to be 160 m ; the head loss due to friction upto the spillway crest, is given by Manning's formula, as :

$$h_f = \frac{n^2 V^2 L}{R^{4/3}}$$ (assume n = Manning's rugosity Coefficient = 0.019)

$$\therefore \quad h_f = \frac{(0.019)^2 \times (2.9)^2 \times 160}{(14.2)^{1.33}} = \mathbf{0.016 \text{ m.}}$$

Level of u/s TEL = u/s water level + Velocity Head – Head lost up to spillway crest

$$= 212.8 + 0.43 - 0.016 = 213.214 \text{ ; say } \mathbf{213.21\ m.}$$

Hence, head over the crest including velocity of approach

$$= H_e = 213.21 - 200.0 = \mathbf{13.21\ m.}$$

and $$H_d = 13.21 - 0.43 = \mathbf{12.78\ m.}$$

Correct Coefficient of Discharge

(*i*) *Correction due to height of weir,*

$$\frac{h}{H_d} = \frac{8}{12.78} = 0.63$$

$$\frac{H_e}{H_d} = \frac{13.21}{12.78} = 1.03$$

From Fig. 21.12 ; $\frac{C}{C_d} = 0.98.$

(*ii*) *Correction due to u/s slope of 45° (i.e. 1 : 1 slope)*

From Fig. 21.13 ; Correction factor = 1.008.

(*iii*) Assuming that the downstream apron elevation is maintained such that it does not affect C_d, we have,

The correct value of coefficient of discharge

$$= C = 2.2 \times 0.98 \times 1.008 = 2.17$$

Effective length $= L_e = 50 - 2\,[N \cdot K_p + K_a]\,H_e$

assuming $K_p = 0.01$ and $K_a = 0.1$

$$L_e = 50 - 2\,[4 \times 0.01 + 0.1] \times 13.21 = 50 - 3.7 = \mathbf{46.3\ m.}$$

The correct head H_e will then be given by

$$5{,}000 = 2.17 \times 46.4 \times H_e^{3/2}$$

or $$H_e^{3/2} = \frac{5{,}000}{2.17 \times 46.3} = 49.7$$

or $$H_e = (49.7)^{0.667} = \mathbf{13.6\ m.}$$

Hence, corrected $H_d = 13.6 - 0.43 = 13.17$ m ; say 13.2 m.

Hence, the design shall be done for a design head equal to 13.2 m.

Design of Crest Profile

$$\frac{H_a}{H_e} = \frac{0.43}{13.2 + 0.43} = \frac{0.43}{13.63} = 0.0315$$

$$\frac{h}{H_e} = \frac{8}{13.6} = 0.58$$

Since $\frac{H_a}{H_e}$ lies between 0 and 0.08 and $\frac{h}{H_e} \approx 0.58$, we can use the equation $x^{1.75} = 1.869\, H_e^{0.75} \cdot y$ (from Table 21.7).

Hence, the d/s profile is given by the equation

$$x^{1.75} = 1.869\,(13.63)^{0.75} \cdot y$$

or $$x^{1.75} = 13.2\,y$$

or $$y = \frac{x^{1.75}}{13.2} \qquad \text{...(21.12)}$$

Position of the d/s apron of spillway

The apron or toe of spillway should be at such an elevation that it does not affect the coefficient of discharge.

or $$\frac{h_a + d}{H_e} \geq 1.7$$

or $$h_d + d \geq 1.7\,(13.63) = 23.2 \text{ m.}$$

Hence, maximum apron elevation

$$= \text{T.E.L.} - (h_d + d)$$

where T.E.L. $= 212.63 \times 0.016$

$= 213.614$ m ; say **213.61 m**

$\therefore$ Max. apron elevation $= 213.61 - 23.2 = 190.41$ m.

Hence, provide the toe of the spillway (*i.e.* apron level) at *RL* **190.4 m.**

Discharge intensity downstream of spillway piers

$$= q = \frac{5{,}000}{50 + 12} = \frac{5{,}000}{62} = 80.7 \text{ m}^2/\text{sec.}$$

If d is the width, then the velocity $= \dfrac{80.7}{d}$ m/s

$$\text{Specific energy} = d + \frac{V^2}{2g} = g + \frac{\left(\dfrac{80.7}{d}\right)^2}{2 \times 9.81}$$

But specific energy $= 213.614 - 190.4 = 23.214$

$$\therefore \qquad d + \frac{\left(\dfrac{80.7}{d}\right)^2}{2 \times 9.81} = 23.214$$

or $$d + \frac{(80.7)^2}{19.62\, d^2} = 23.214$$

or $$d^3 + 331 = 23.21\, d^2$$

$$d^3 - 23.21\, d^2 + 331 = 0$$

Solving by hit and trial, we get

$$\boxed{d = 4.2 \text{ m}}$$

The d/s profile is designed between *RL* 200.0 m (crest level) and *RL* = 190.4 (apron level)

Hence, maximum ordinate $y = 200.0 - 190.4 = $ **9.6 m.**

Corresponding value of x is obtained from eq. (21.12), as :

$$9.6 = \frac{x^{1.75}}{13.2}$$

or $$x^{1.75} = 126.7$$

or $$x = (126.7)^{0.57} = 15.08$$

The remaining co-ordinates of d/s profile between $x = 0$ and $x = 15.08$ m are worked out, as given in Table 21.9.

Table 21.9. Coordinates of d/s Profile

x in metres	$y = \frac{x^{1.75}}{13.2}$ *in metres*
0	0
0.5	0.023
1.0	0.077
2.0	0.25
4.0	0.86
6.0	1.74
8.0	2.88
10.0	4.03
12.0	5.84
14.0	7.9
15.08	9.6

The co-ordinates of the u/s profile are calculated in Table 21.10.

Table 21.10. Co-ordinates of u/s Profile
H_e = 13.61 m

$\frac{x}{H_e}$	$\frac{y}{H_e}$	x	y
– 0.000	0.0000	0	0
– 0.020	0.0004	– 0.272	0.0054
– 0.060	0.0035	– 0.817	0.0476
– 0.100	0.0101	– 1.361	0.1375
– 0.120	0.0150	– 1.633	0.2042
– 0.140	0.0208	– 1.905	0.2831
– 0.150	0.0235	– 2.042	0.3198
– 0.160	0.0270	– 2.178	0.3675
– 0.175	0.0328	– 2.382	0.4464
– 0.190	0.0395	– 2.586	0.5375
– 0.200	0.0420	– 2.722	0.5716

The u/s crest is joined at an angle of 45° to the bottom, as shown in Fig. 21.21.

Design of the Chute or Discharge Carrier

The critical depth $y_c = \sqrt[3]{\frac{q^2}{g}} = \sqrt[3]{\frac{(80.7)^2}{9.81}} = 8.77$ m.

The depth at the top of spillway (d) was calculated to be 4.2 m, which is less than y_c. Hence, the flow at the top of the spillway is supercritical.

The chute channel or the discharge carrier should now be given a milder slope for a little distance from toe, but in no case less than the critical slope, so that the flow remains super-critical.

Critical velocity $= V_c = \frac{q}{y_c} = \frac{80.7}{8.77} = 9.18$ m/sec.

But $$V = \frac{1}{n} \cdot R^{2/3} \cdot S^{1/2}$$

A rectangular channel with bottom width 62 m should be provided as the chute channel (discharge carrier) for the chute spillway.

At critical flow, $A = 62 \times 8.77$

$$P = 62 + 8.77 \times 2$$

$$\therefore \quad R = \frac{A}{P} = \frac{62 \times 8.77}{(62 + 8.77 \times 2)} = 6.82$$

Critical slope S_c is, therefore, given as :

$$9.18 = \frac{1}{0.019} (6.82)^{\frac{2}{3}} \cdot S_c^{1/2}$$

or $$S_c^{1/2} = \frac{9.18 \times 0.019}{(6.82)^{2/3}} = \frac{9.18 \times 0.019}{3.61} = \frac{1}{20.7}$$

or $$S_c = \frac{1}{428}$$

A slope of say 1/250 is provided in 25 m distance from the top of the spillway.

Bed level at the end of $\frac{1}{250}$ slope $= 190.4 - \frac{25}{250} = 190.3$ m.

The reverse curve at toe may have a radius equal to

$$2H_e = 2 \times 13.63 = 27.22 \text{ m} \text{ ; say } 27.5 \text{ m.}$$

From this point onwards, *i.e.* $RL = 190.3$ m and upto RL 103.0 m, the slope of the discharge carrier may be steepened depending upon the site contours. A slope such as 8 : 1 to 4 : 1 may be given in the initial stage and then a steeper slope of say 2 : 1 or 3 : 1 may be given. The entire design is now like that of a rectangular channel. Wherever, the slope is changed, transition curve (convex or concave) may be designed as explained earlier.

In the given question, since the site contours are not known, we shall assume that a slope of 6 : 1 is given for the first 21 m fall (*i.e.* up a RL 169.3) and then a slope of say 2 : 1 is given for the rest of the reach. The depth, velocity, etc. at the end point of 1 in 250 slope may be taken to be the same, as they were at the toe of the spillway, because the small length of 25 m shall not produce much difference. With this assumption, the TEL at the starting point of new slope 6 : 1 ($RL = 190.3$) is equal to $213.21 - 0.1 = 213.11$ m.

The calculations of water depth, velocity, etc. can now be carried out for the entire reach ($RL = 190.3$ to $RL = 103.0$ m) of the discharge carrier by dividing the channel length into small reaches ; say of length 42 m each, as shown in Table 21.11. The depth (d) at the point is assumed in Column (6). Specific energy is calculated in Col. (9). The TEL at the end is calculated in Col. (10). The drop in energy line (h_f) in the interval is calculated in Col. (17). The drop (h_f) at the end of the interval is subtracted from the final calculated TEL at the beginning of interval in Col. (18). The Col. (18), is then compared with Col. (10). They should be almost equal. If difference is large, the assumed depth is changed till equivalence is obtained. The table is otherwise self explanatory.

Table 21.11. Calculations of water depth on chute channel (*i.e.* Discharge Carrier)

S.N.	Distance from start of 6 : 1 slope	Length L	Drop in bed	Bed level	Depth (d) assumed)	Velocity $v = q/d = 80.7/d$	Vel. head $= v^2/2g$	Sp. energy $= d + v^2/2g$	Bed level + Sp. energy = TEL calculated at the end	$A = Bd = 62d$ (Area)	$P = B + 2d = 62 + 2d$ wetted perimeter	$R = A/P$	$R^{4/3}$	$S_f = (n^2 \cdot v^2)/R^{4/3}$; $n = 0.019$	Average S_f	$L \cdot S_f = h_f$	Actual TEL	Froude number $F = V/\sqrt{gd}$
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)
1	0	0		190.3	4.2	19.2	18.9	23.1	213.4	260	70.4	3.7	5.72	0.0232			213.53	2.97
2	42	42	7	183.3	3.7 3.7 3.62	21.4 22.4 22.3	24.2 25.7 25.3	27.9 29.3 28.92	211.2 212.6 212.22	229 223 224	69.4 69.2 69.24	3.31 3.23 3.24	4.94 4.78 4.8	0.0378 0.0378 0.0374	0.029 0.0305 0.0303	1.22 1.28 1.27	212.18 212.12 212.13	3.74
3	84	42	7	176.3	3.30 3.28	24.45 24.60	30.4 30.8	33.7 34.08	210.0 210.38	205 205	68.6 68.56	2.98 2.98	4.28 4.28	0.0504 0.0505	0.0439 0.0439	1.84 1.84	210.38 210.38	4.33
4	126	42	7	169.3	3.05	26.4	35.7	38.75	208.05	188	68.08	2.77	3.88	0.0652	0.0578	2.43	207.95	4.95
Start of Slope 2 : 1																		
5	168	45	21	148.3	2.5	32.3	53.2	55.7	204.0	155	67	2.32	3.07	0.1224	0.0938	3.94	204.11	6.46
6	210	42	21	127.3	2.21	36.5	67.9	70.11	197.41	136	66.4	2.05	2.56	0.1895	0.1559	6.54	197.46	7.84
7	258.6	48.6	24.3	103.0	1.98 2.015	40.7 40.0	84.7 81.85	86.68 83.865	189.98 186.865	122.6 125.0	66 66.04	1.86 1.895	2.29 2.35	0.259 0.245	0.2242 0.2172	10.9 10.54	186.51 186.87	9.00

Design of Curve No. 1. (At the junction of 250 : 1 and 6 : 1 slopes, a convex curve shall be provided).

The convex curve No. 1 can be designed as per Equation (21.10), as :

$$y = -x \tan\theta - \frac{x^2}{1.5\,[4\,(d+h_v)\cos^2\theta]}$$

$(d+h_v)$ is the sp. energy at the junction point

$$= 4.2 + \frac{\left(\frac{80.7}{4.2}\right)^2}{2 \times 9.81} = 23.21 \text{ m.}$$

$\tan\theta$ = slope of the angle of the floor u/s of the junction point $= \frac{1}{250}$

Since $\tan\theta$ is small, $\cos^2\theta$ will be approximately unity.

$$\therefore \quad y = -\frac{x}{250} - \frac{x}{6 \times 23.21} = \frac{-x}{250} - \frac{x^2}{139.3}$$

Differentiating w.r. to x, we get

$$\frac{dy}{dx} = \frac{1}{250} - \frac{x}{69.7}$$

The curve meets the downstream slope where

$$\frac{dy}{dx} = -\frac{1}{6} \qquad \text{(–ve sign shows that as } x \text{ increases, } y \text{ decreases).}$$

Equating, we get $\frac{1}{250} + \frac{x}{69.7} = \frac{1}{6}$

or $$x = 69.7\left(\frac{1}{6} - \frac{1}{250}\right) = 69.7\left[\frac{244}{1{,}500}\right]$$

or $\mathbf{x = 11.3\ m.}$

Other co-ordinates of this curve for values of x between $x = 0$ and $x = 11.3$ m, determined from the equation $y = \frac{x}{250} + \frac{x^2}{139.3}$ (y is negative of course) are given in table 21.12.

Table 21.12

x in metres	$y = \frac{x}{250} + \frac{x^2}{139.3}$ *in metres*
0	0
1	0.011
3	0.077
5	0.20
7	0.371
9	0.619
10.0	0.78
11.3	0.97

Design of curve No. II (convex). *At the junction of 6 : 1 slope and 2 : 1 slope.* From Table 21.11, the specific energy at this point is found to be

$$= d + h_v = 3.05 + 35.7 = 38.75 \text{ m}$$

$$\therefore \quad y = -\left\{x \tan\theta + \frac{x^2}{1.5[4(d+h_v)\cos^2\theta]}\right\}$$

$$\tan\theta = \frac{1}{6}; \cos\theta \approx 1$$

$$\therefore \quad y = -\left[\frac{x}{6} + \frac{x^2}{6 \times 38.75 \times 1}\right]$$

or
$$y = -\left[\frac{x}{6} + \frac{x^2}{232.5}\right]$$

$$\therefore \quad \frac{dy}{dx} = -\left[\frac{1}{6} + \frac{x}{116.25}\right]$$

It meets the downstream slope of 2 : 1 ; $\therefore \frac{dy}{dx} = -\frac{1}{2}$ (–ve sign shows that as x increases, y decreases.)

$$\therefore \quad \frac{1}{6} + \frac{x}{116.25} = \frac{1}{2}$$

or
$$\frac{x}{116.25} = \frac{1}{2} - \frac{1}{6} = \frac{1}{3}$$

or
$$x = \frac{116.25}{3} = 38.75 \text{ m}$$

are
$$x = \mathbf{38.75\ m.}$$

It means that this convex curve shall become tangential to the slope of 2 : 1 after traversing a distance of 38.75 m. The co- ordinates of this curve can be found (between $x = 0$ and $x = 38.75$ m), using the equation $y = -\left[\frac{x}{6} + \frac{x^2}{232.5}\right]$ as shown in table 21.13.

Table 21.13

x in metres	$y = \frac{x}{6} + \frac{x^2}{232.5}$ *in metres (y is downward)*
0	0
1	0.17
5	0.95
10	2.10
15	3.47
20	5.05
25	6.85
30	8.87
35	11.10
38.75	12.93

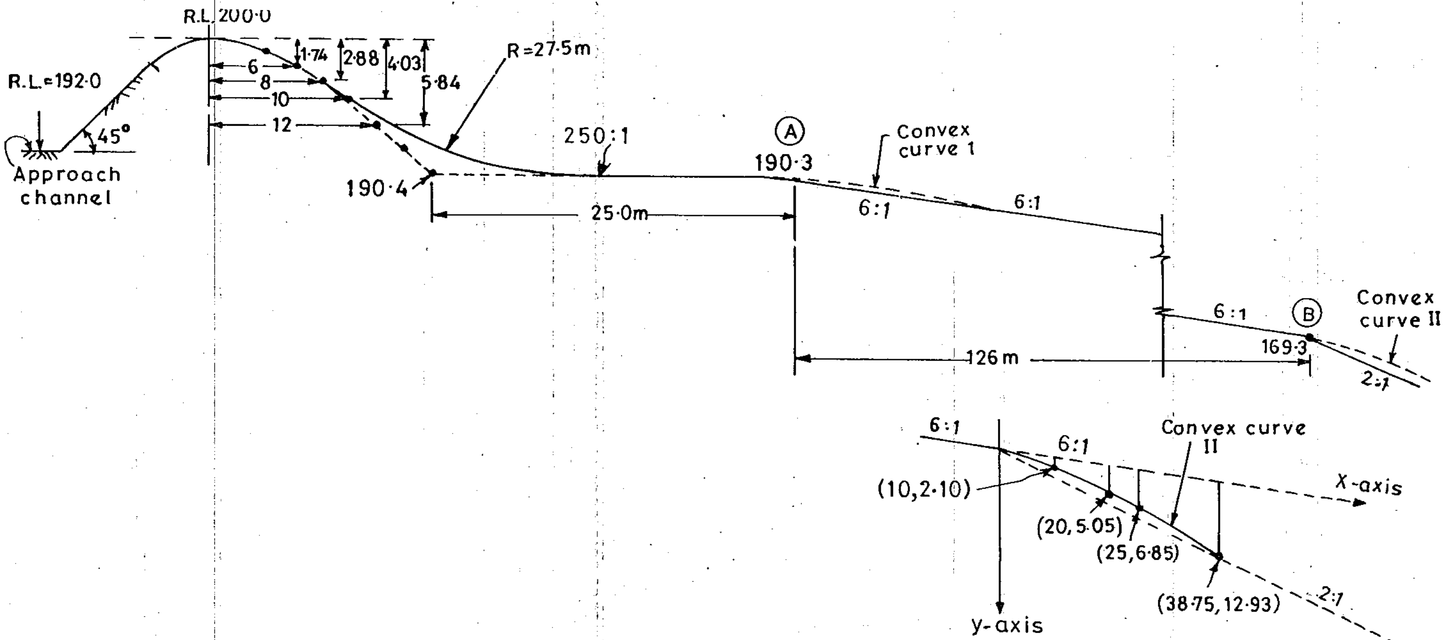


Fig. 21.21. Section of Chute Spillway.

Position of Stilling Basin for Energy Dissipation by Hydraulic Jump (Explained later)*

From Table 21.11, the depth at tail water level

$$d = 2.015 \text{ m}$$
$$F = 9.00$$
$$V = 40 \text{ m/sec.}$$
$$\therefore \quad y_1 = 2.015 \text{ m}$$
$$V_1 = 40 \text{ m/sec.}$$
$$F_1 = 9.0.$$

Hence
$$y_2 = \frac{y_1}{2}\left[\sqrt{1+8F_1^2} - 1\right] = \frac{2.015}{2}\left[\sqrt{1+648} - 1\right]$$
$$= 1.0075\,[\sqrt{649} - 1] = \mathbf{24.7\ m.}$$

Provide the basin tail water depth, 50% more than the conjugate depth, as the Froude number is large, or at a depth

$$= 1.05 \times y_2 = 1.05 \times 24.7$$
$$= 25.9 \text{ m say ; } \mathbf{26\ m.}$$

Hence, floor level of jump basin

$$= 103 - 26 = \mathbf{77.0\ m.}$$

U.S.B.R. stilling basin II can be *provided.* This is described in this chapter in article 21.15.

21.8. Side Channel Spillway

The side channel spillway (Fig. 21.22) differs from the chute spillway in the sense that while in a chute spillway, the water flows at right angles to the weir crest after spilling over it, whereas in a side channel spillway the flow of water after spilling over the crest, is turned by 90° such that it flows parallel to the weir crest (AB), as shown in Fig. 21.22 (*a*).

This type of spillway is provided in narrow valleys where no side flanks of sufficient width to accommodate a chute spillway are available. If a crest length equal to AB is provided along AC (*i.e.*, along axis of a chute spillway), heavy cutting shall be required. In such topographies, a chute spillway may be replaced by a side channel spillway.

The design of side channel, required for diverting the flow, is beyond the scope of this book. However, it may be mentioned that the analysis of flow in the side channel, is made by the application of the momentum principle in the direction of flow. The water entering the side channel has no momentum in the direction in which it has to move. The slope of the side channel should, therefore, be sufficient to overcome friction losses as well as to provide acceleration in the direction of flow against the mass of incoming water.

After the end of the crest A, the water is taken away as in an ordinary chute channel, till it joins the river downstream.

* Readers are advised to go through this article, only after going through its theory as given in article 21.11.

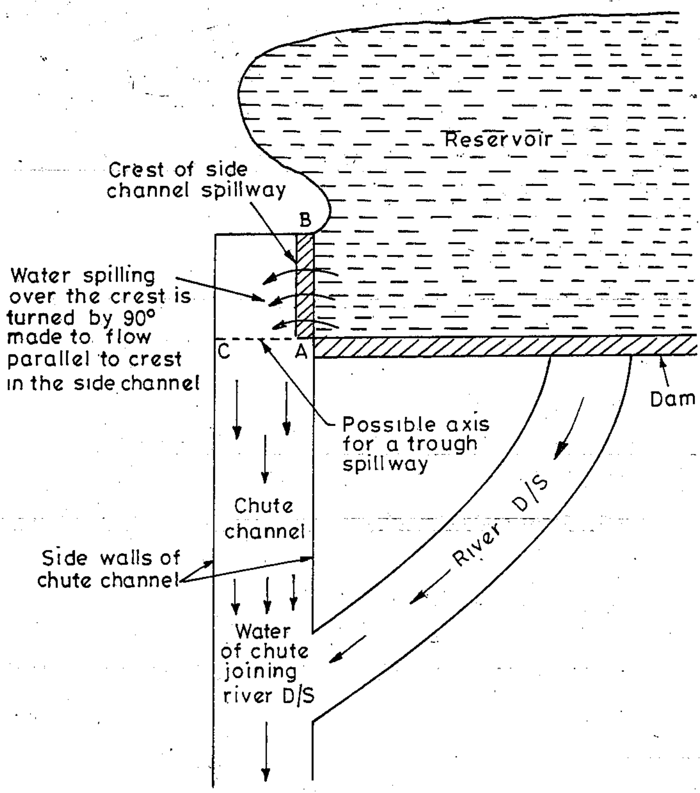


Fig. 21.22. (*a*) Simplified line sketch of a Side Channel Spillway.

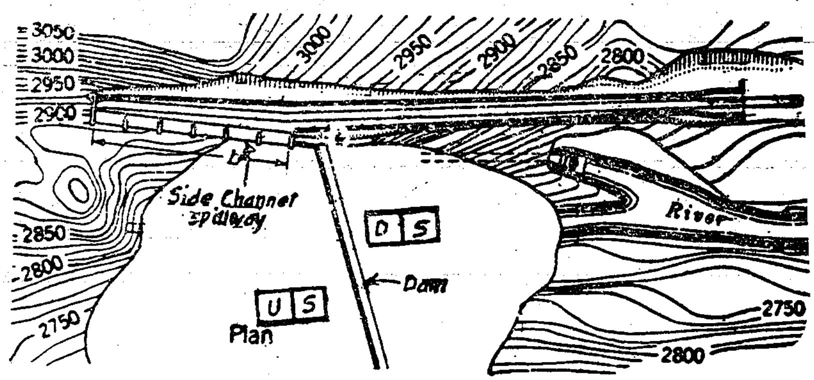


Fig. 21.22. (*b*) Layout plan of a Side Channel Spillway.

Many other spillways may be constructed somewhere in between the chute spillway and the side channel spillway. In such cases, the direction of water after passing over the crest is changed somewhere between 0° and 90°.

21.9. Shaft Spillway

In a shaft spillway (Fig. 21.23), the water from the reservoir enters into a vertical shaft which conveys this water into a horizontal tunnel which finally discharges the water into the river downstream. Sometimes, the vertical shaft may be excavated through some natural rocky island or rocky spur existing on the u/s of the river near the dam. Sometimes, artificial shafts may be constructed. For small heights, the shafts may be constructed entirely of metal or concrete, or clay tiles. But for larger heights, reinforced cement concrete may be used. For smaller heights, no special inlet design is necessary, but on large projects, a flared inlet called **morning glory** is often used.

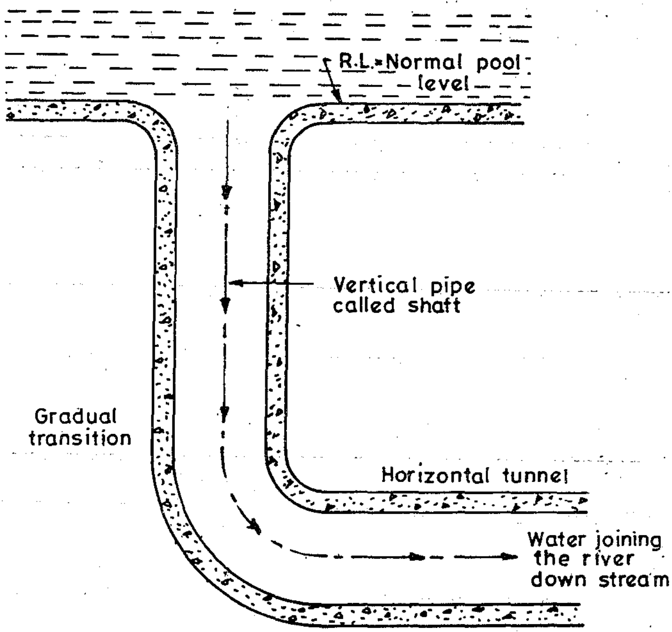


Fig. 21.23. Shaft Spillway.

The horizontal tunnel or the conduit may be taken either through the body of the dam (as may be done in concrete gravity dams) or below the foundations (as may be done in earthen dams). The diversion tunnels constructed for diversion of the river, may sometimes be planned and used for shaft spillways, as shown in Fig. 21.24.

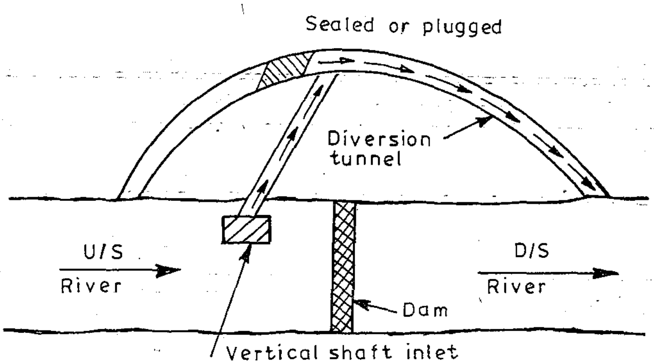


Fig. 21.24. Diversion Tunnel being used as a conduit for the shaft spillway.

A shaft spillway may be adopted when the possibility of an over-flow spillway and a trough spillway has been ruled out because of non-availability of space due to topography. The choice may then lie between a side channel spillway and a shaft spillway. If some suitable rock spur, etc. is available near the reservoir on the upstream, naturally, shaft spillway may become economical and consequently the first choice.

The discharge through the shaft spillway does not increase at such a high rate as it increases in weir type spills. Hence, if the unestimated high floods occur, the shaft spillway may not prove as useful as a weir type spillway (such as overflow, chute or side channel) would have been. Hence, a shaft spillway design should be much more conservative than that of other weir type spillways.

A gradual transition must be provided between the vertical shaft and the horizontal conduit, in order to avoid danger of cavitation. Hydraulic analysis of shaft spillways is difficult and their functioning is generally tested in models. The entry of debris and other materials floating in the reservoir must be prevented from entering the shaft, otherwise, they will clog the shaft or the conduit, especially at the junction point. Properly designed or tested trash racks or floating booms may be provided for this purpose. Since, a shaft spillway is surrounded by water on all sides, it must be connected to the dam or the hill side by a bridge.

21.10. Syphon Spillway

A siphon spillway essentially consists of a siphon pipe, one end of which is kept on the upstream side and is in contact with the reservoir, while the other end discharges water on the downstream side. Two typical installations of siphon pipes are shown in Figs. 21.25 and 21.26.

21.10.1. Tilted Outlet Type of a Syphon Spillway. The siphon pipe in Fig. 21.25 has been installed within the body of the dam. When the valley is very narrow and no space is available for constructing a separate spillway, the siphon pipes can be installed within the dam body, as shown in Fig. 21.25. An air vent may be connected with the siphon pipe. The

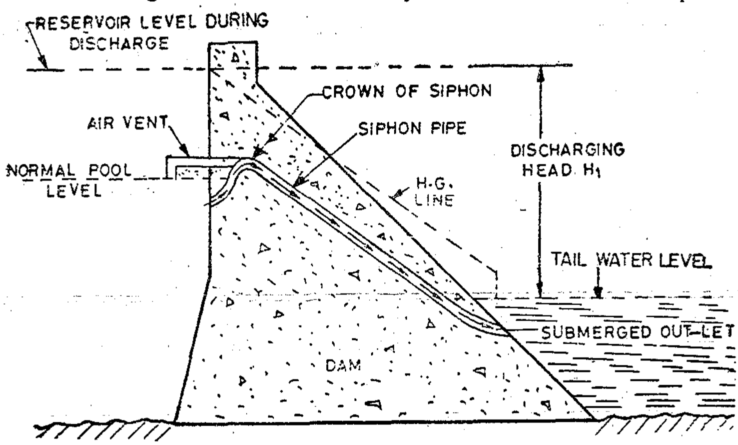


Fig. 21.25. Siphon pipe installed within the gravity dam.

level of the air vent may be kept at normal pool level, while the entry point of the siphon pipe may be kept still lower so as to prevent the entry of debris, etc. in the siphon. The outlet of the siphon may be submerged so as to prevent the entry of the air in the siphon from its d/s end.

When the water in the reservoir is upto or below the normal pool level, air enters the siphon through the vent and siphonic action cannot take place. When once the water level in the reservoir goes above the normal pool level, and if once the siphon is filled with water (*i.e.*, it is primed) ; the water will start flowing through the siphon by siphonic action. The outflow will continue till the water level in the reservoir falls back to normal pool level. As soon as it happens, the air will enter the siphon through the new exposed air vent and the flow will stop.

Discharge formula. The velocity of flow at the outlet of the siphon can be obtained by equating the effective head H_1 (*i.e.*, the difference of water level in the reservoir and the tail water level, for submerged outlet) to the velocity head.

$$\therefore \qquad \frac{V^2}{2g} = H_1$$

or
$$V = \sqrt{2gH_1}$$

$$\therefore \qquad Q = \text{Discharge} = C_d \cdot A\sqrt{2gH_1}.$$

Hence, the discharge through the siphon is given by the equation,

$$Q = C_d A \cdot \sqrt{2gH_1} \qquad \text{...(21.13)}$$

where C_d is the co-efficient of discharge, which is usually about 0.9.

The above equation clearly shows that the discharge, through a siphon spillway is sufficiently independent of the water surface elevation of the reservoir. If the water surface in the reservoir rises, the discharge through the spillway is affected to a less extent because change in H_1 is small as compared to the corresponding change in head over an ogee type spillway. Hence, the discharge through the siphon is nearly always at capacity, when once the water level has risen above the normal pool level. This makes the siphon spillway particularly advantageous in disposing of sudden surges of water, such as may occur in canals and forebays when the outlet gates are closed rapidly. As the rise of water above the crest level is smaller (because of higher discharging capacity even at the start of inflow flood above normal pool level), the height of the non-overflow section of the dam can be kept smaller for the same height of overflow section.

Sometimes, it may not be possible to keep the outlet submerged in tail water because of limitations of negative pressure at the crown. In such a case, the outlet can be left open above the water level, but air entry through the outlet will have to be controlled by priming devices. The effective head H_1, in such a case, will be the difference of reservoir water level and outlet level of siphon.

21.10.2. Hooded Type of a Syphon Spillway. The construction of a Hooded type of siphon spillway [Fig. 21.26 (*a*)] is more commonly adopted. In this case, a reinforced concrete hood is constructed over an ordinary overflow section of a gravity dam. The inlet of this hood is kept submerged so as to prevent the entry of debris, ice, etc. A small depriming hood is kept above the main hood and both these hoods are connected by an

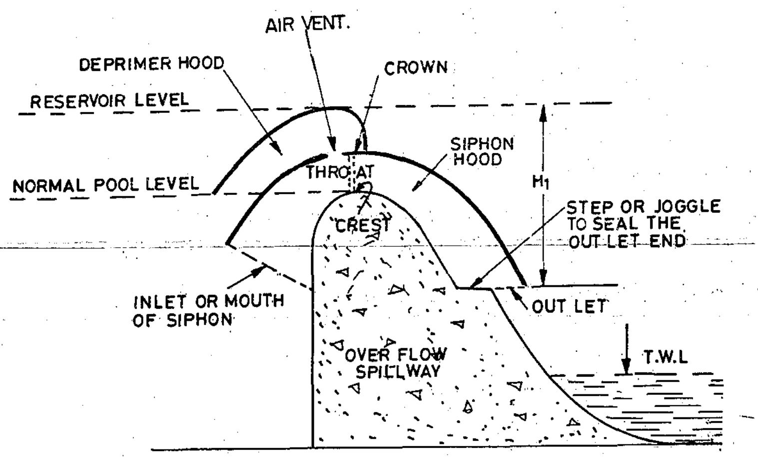


Fig. 21.26. (*a*) Siphon installed over the overflow spillway to increase its effectiveness and discharging capacity.

air vent. The inlet of the deprimer hood is kept at normal pool level. The principle of functioning of both types of installations is essentially the same, except for the initial filling up of the siphon or 'priming'. At normal pool level (*i.e.* full reservoir level), the water stands up to the crest of the spillway. If now a flood enters the reservoir, the water level would start rising and a sheet of water would start flowing over the spillway crest. Since the water level is above the deprimer hood, the air entry at the inlet is sealed. The air entry at the outlet is also sealed by tail water, etc. Hence, the water spilling over the crest, sucks all the remaining air from the hood within minutes. Siphonic action gets established after the air in the bend over the crest have been completely exhausted. A photoview of this type of siphon spillway, called Sarala Sagar spillway, flanked by earth dam constructed on China Vagu stream (a tributary of Krishna river) is shown in Fig. 21.26 (*b*).

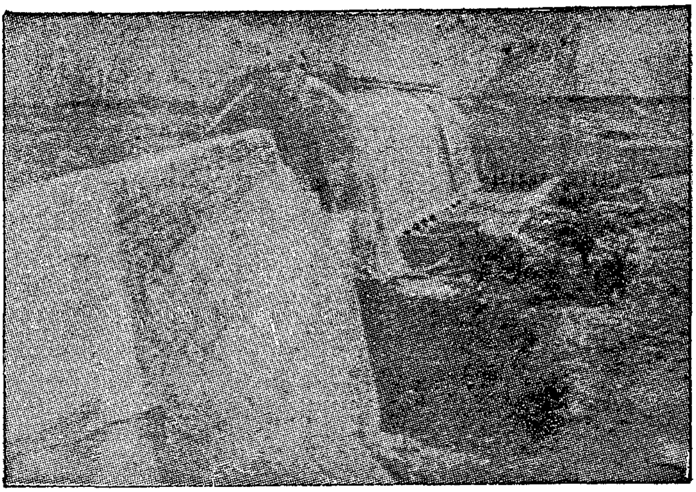


Fig. 21.26 (*b*). Photoview of a Siphon Spillway (Hooded Type).

21.10.3. Priming Devices. When the outlet cannot be sealed by tail water, some other devices, called *priming devices*, are used which lead to an automatic priming of the siphon at a certain rise of water level above the crest. The maximum rise of water level is called *priming depth.* In Fig. 21.26 (*a*), the priming is accomplished by means of a step called *Joggle* which deflects the sheet of water to strike against the lower end of the cover or hood, thus sealing the lower end from the atmosphere. Sometimes, an additional *baby siphon* may be installed as a priming device, as shown in Fig. 21.27.

When the water level reaches slightly above the crest, the baby siphon, which is an additional siphon, starts running full. The sheet of water issuing from it, is arranged to shoot across the lower end of the main siphon, so as to seal it from the atmosphere.

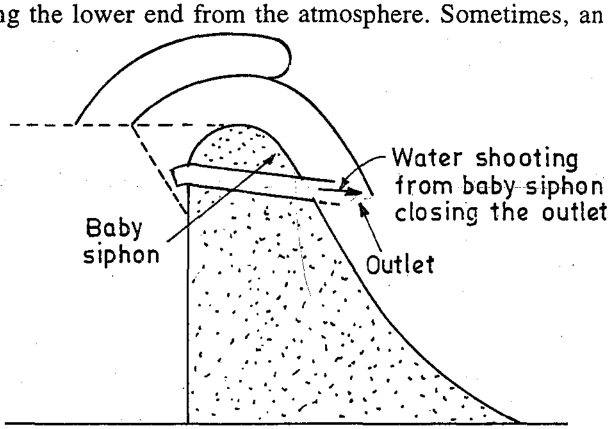


Fig. 21.27. Baby Siphon installed as a priming device.

21.10.4. Negative Pressures. As soon as the siphon is primed, a vacuum is formed at the throat. The negative pressure developed should be limited to such a magnitude that the absolute pressure of water does not exceed the vapour pressure of water at the temperature. This is necessary to avoid cavitation (explained earlier) and its ill effects. Hence, on an average, a maximum negative head equal to 7.5 m or so can be allowed. In other words, the vertical distance from the crown of the siphon (Top point) down to the discharging point (more precisely it is down to the hydraulic gradient line) should not exceed a value of about 7.5 m or so, under average conditions [∵ atmospheric pressure − vapour pressure = limiting vacuum pressure ; or $10 - 2.5 = 7.5$ m of water'. At high altitudes or in hotter regions, this limit may still go down. This fixes a limit on H_1 and on the discharging capacity of a siphon spillway.

Both types of syphon spillways described above are known as **Saddle siphon spillways**. A special type of siphon spillway called *Volute Siphon Spillway* has been designed in India by Ganesh Iyer and is shown in Fig. 21.28.

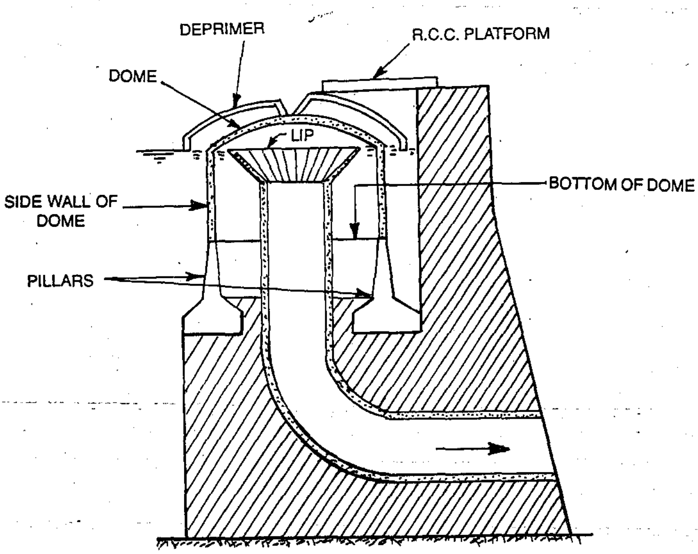


Fig. 21.28. Volute Siphon Spillway.

ENERGY DISSIPATORS

21.11. Energy Dissipation below Overflow Spillways

The water flowing over the spillway acquires a lot of kinetic energy by the time it reaches near the toe of the spillway (because of conversion of potential energy into kinetic energy). If arrangements are not made to dissipate this huge kinetic energy of water, and if the velocity of water is not reduced, large scale scour can take place on the downstream side near the toe of the dam and away from it. These arrangements are known as energy dissipation arrangements or *energy dissipators*.

In general, the kinetic energy of this super-critical flow can be dissipated in two ways :

(*i*) By converting the super critical flow into sub-critical flow by *hydraulic jump.*

(*ii*) By directing the flow of water into air and then making it fall away from the toe of the structure. The energy is dissipated by the *aeration of jet and impact* of water on the river bed. Though some scour will take place, but it is too small or too far away from the dam to endanger it. Bucket type energy dissipators work on this principle.

21.11.1. Hydraulic Jump Formation. The phenomenon of hydraulic jump has already been explained in details in Chapter 10. It was mentioned therein, that a hydraulic jump can form in a horizontal rectangular channel, when the following relation is satisfied between the pre-jump depth (y_1) and post-jump depth (y_2).

$$y_2 = -\frac{y_1}{2} + \sqrt{\frac{y_1^2}{4} + \frac{2q^2}{gy_1}} \quad \textit{i.e. } \text{Eq. (10.4)}$$

where q is the discharge intensity.

For a given discharge intensity over a spillway, the depth y_1 is equal to q/V_1 ; and V_1 is determined by the drop H_1, being equal to $\sqrt{2gH_1}$.

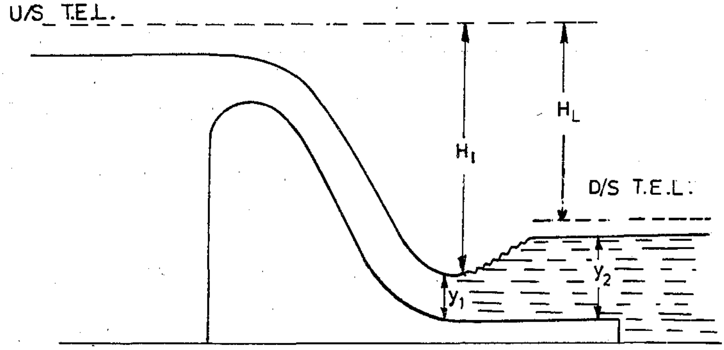
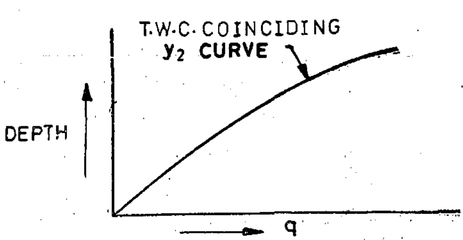


Fig. 21.29

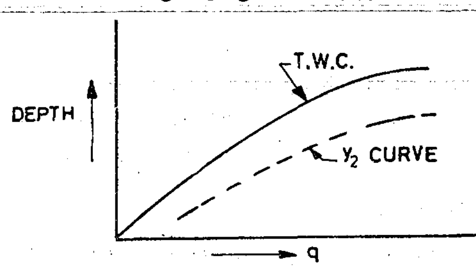
Hence, for a given discharge intensity and given height of spillway, y_1 is fixed and thus y_2 (*i.e.* the depth required for the formation of hydraulic jump) is also fixed. But the availability of a depth equal to y_2 in the channel on the d/s cannot be guaranteed as it depends upon the tail water level, which depends upon the hydraulic dimensions and slope of the river channel below. The problem should, therefore, be analysed before any solution can be found. Hence, for different discharges, the tail water depth is found by actual gauge discharge observations and by hydraulic computations. The post jump depths (y_2) for all those discharges, are also computed from equation (10.4). If a graph is now plotted between q and tail water depth, the curve obtained is known as the *Tail Water Curve (T.W.C.).* Similarly, if a curve is plotted on the same graph, between q and y_2, the curve obtained is known as the *Jump Height Curve (J.H.C.)* or y_2 *curve.*

Now there are five possibilities

(*a*) T.W.C. coinciding with y_2 curve at all discharges [Fig. 21.30 (*a*)].



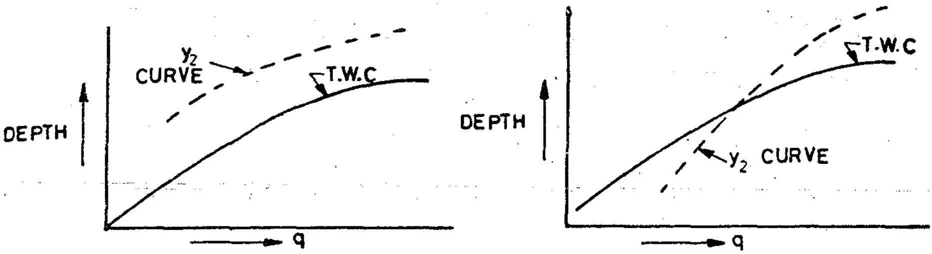
(*a*)



(*b*)

(*b*) T.W.C. lying above the y_2 curve at all discharges [Fig. 21.30 (*b*)].

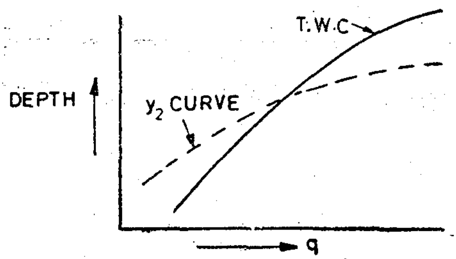
(*c*) T.W.C. lying below the y_2 curve at all discharges [Fig. 21.30 (*c*)].



(*c*) (*d*)

(*d*) T.W.C. lying above the y_2 curve at smaller discharges and lying below the y_2 curve at larger discharges [Fig. 21.30 (*d*)].

(*e*) T.W.C. lying below the y_2 curve at smaller discharges and lying above the y_2 curve at larger discharges [Fig. 21.30 (*e*)].



(*e*)
Fig. 21.30

Depending upon the relative positions of T.W.C. and y_2 curve, the energy dissipation arrangements can be provided below the spillway, as explained below for all these five cases.

21.11.1.1. Energy dissipators for case (*a*) : *When T.W.C. coincides with y_2 curve at all discharges.* This is the most ideal condition for jump formation. The hydraulic jump will form at the toe of the spillway at all discharges. In such a case, a simple concrete apron of length $5\,(y_2 - y_1)$ is generally sufficient to provide protection in the region of hydraulic jump, as shown in Fig. 21.31 (*a*).

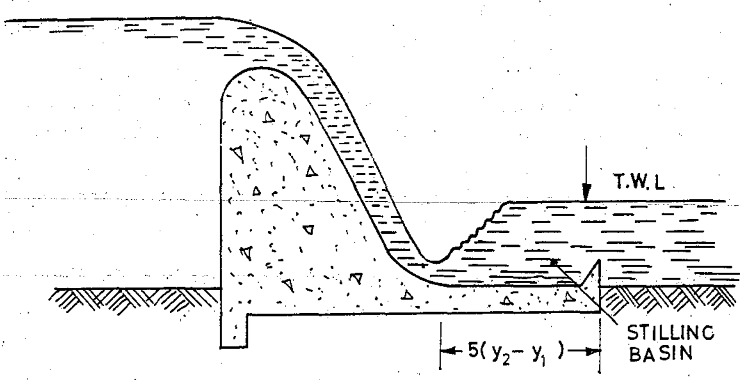


Fig. 21.31. (*a*) Simple horizontal apron.

21.11.1.2. Energy dissipators for case (*b*) : *When T.W.C. is lying above the* y_2 *curve at all discharges.* In this case, when y_2 is always below the tail water, the jump forming at toe will be drowned out by the tail water, and little energy will be dissipated. Water may continue to flow at high velocity along the channel bottom for a considerable distance.

The problem can be solved :

(*i*) by constructing a **sloping apron above the river bed** level as shown in Fig. 21.31 (b_1). The jump will form on the sloping apron where depth equal to y_2 (lesser than the tail water depth at toe) is available. The slope of the apron is made in such a way that proper conditions for a jump will occur somewhere on the apron at all discharges. A lot of extra concreting is required to be done, as shown.

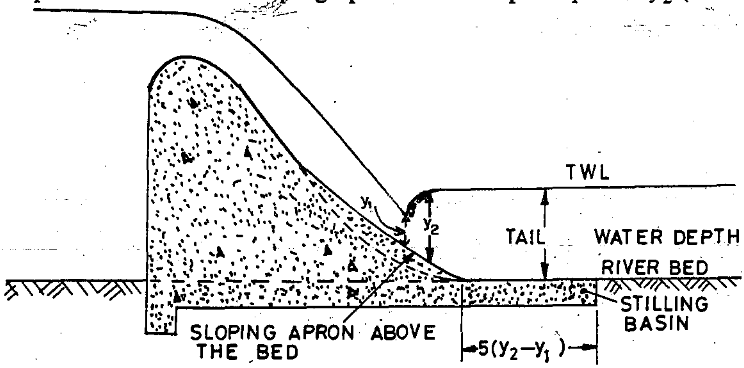


Fig. 21.31. (b_1) Sloping apron above the bed.

(*ii*) A second solution of this problem can be in the form of providing a **roller bucket** type of energy dissipator. It consists of an apron, which is upturned sharply at ends, as shown as in Fig. 21.31 (b_2).

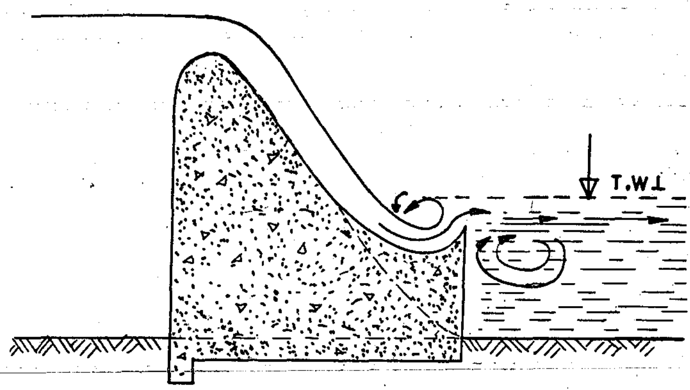


Fig. 21.31. (b_2) Roller Bucket.

Two main rollers are formed which dissipate the energy due to internal turbulence.

The roller which is formed downstream of the bucket, tends to move the scoured bed material towards the dam, thus, preventing serious scour at toe of the dam. Sometimes, the scoured material may enter the bucket under the action of u/s roller, and may cause severe abrasion. A *dentated bucket lip* may, therefore, have to be provided, so as to permit removal of material caught in the bucket.

21.11.1.3. Energy dissipators for case (*c*) : *When T.W.C. lies below the y_2 curve at all dischages.* (*i*) If the tail water is very low, the water may shoot up out of the above bucket, and fall harmlessly into the river at some, distance downstream of the bucket. *This bucket is then known as* **ski jump bucket** and can be used for energy dissipation in case (*c*) : *i.e.* when the tail water depth is insufficient or low at all discharges. The ski jump bucket type of an energy dissipator requires sound and rocky river bed, because a part of the energy dissipation takes place by impact, although some of the energy is dissipated in air by diffusion and aeration.

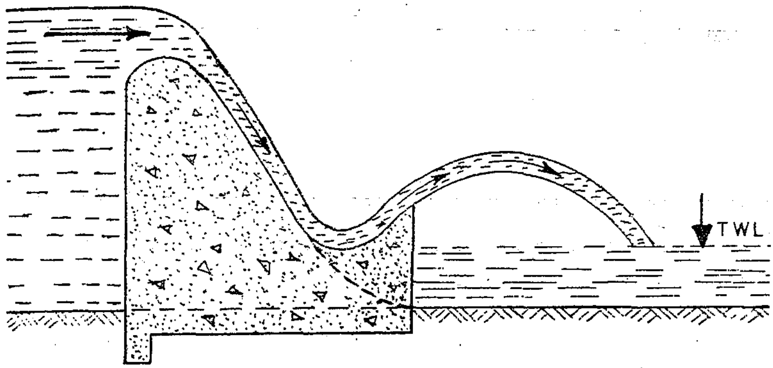


Fig. 21.31. (c_1) Ski jump bucket.

(*ii*) The second solution to the problem can be the *provision of a* **sloping apron** as *in case (b) but* **below the river bed,** as shown in Fig. 21.31 (c_2). The required depth y_2

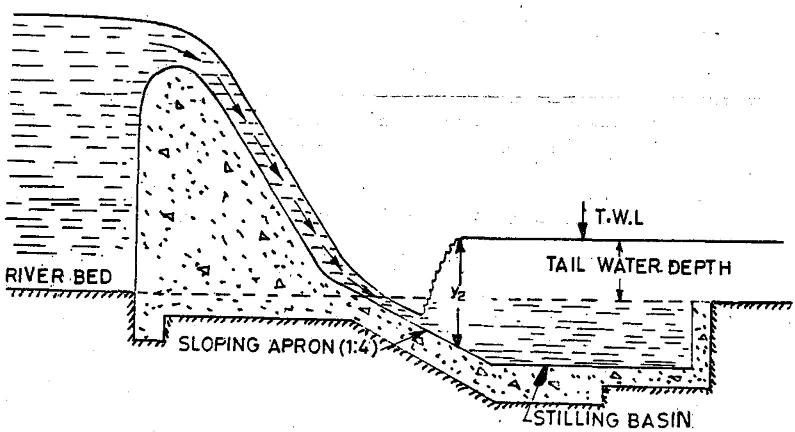


Fig. 21.31. (c_2) Sloping apron below the bed.

which is greater than T.W. depth, can thus be made available by letting the jump form on this sloping apron as shown. This sloping apron and the horizontal cistern of length 5 $(y_2 - y_1)$ shall be entirely in cutting and may be expensive, though otherwise quite satiactory.

(*iii*) The third solution to this problem may be the construction of a **subsidiary dam** below the main dam, so as to increase the tail water depth and cause a jump to form at the toe of the main dam, as shown in Fig. 21.31 (c_3).

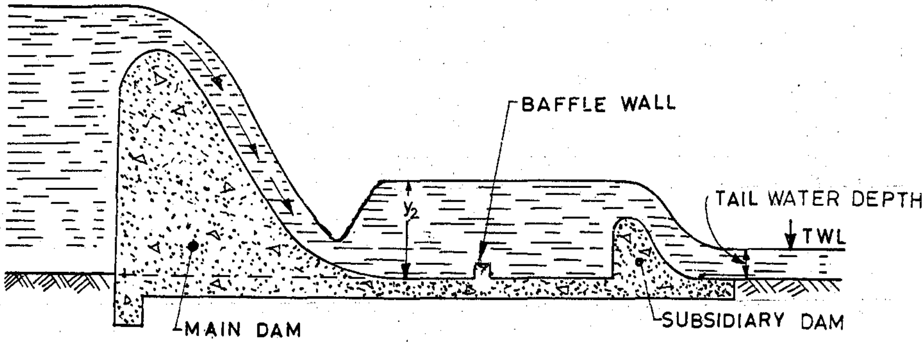


Fig. 21.31. (c_3) Subsidiary dam construction.

If the tail water defficiency is small, a **baffle wall** or a **row of friction blocks** may be provided so as to dissipate the residual energy. The baffles, generally give way, under high velocity jets due to their cavitation effects, and hence, are suitable only for low spillways or weirs. They should be sufficiently strong to withstand impact from ice and floating debris. The location, shape, size and spacing of these baffles can be best determined by model studies. Their use for weirs and canal falls have already been explained in the earlier chapters.

21.11.1.4. Energy dissipators for case (*d*). *When T.W.C. lies above the y_2 curve at low discharges and lies below the y_2 curve at high discharges.* In this case, at low discharges, the jump will be drowned and at high discharges, tail water depth is insufficient. The solution to the problem lies in providing a **sloping apron partly above and partly below the river bed** as shown in Fig. 21.31 (*d*). The horizontal apron and end sill should also be provided.

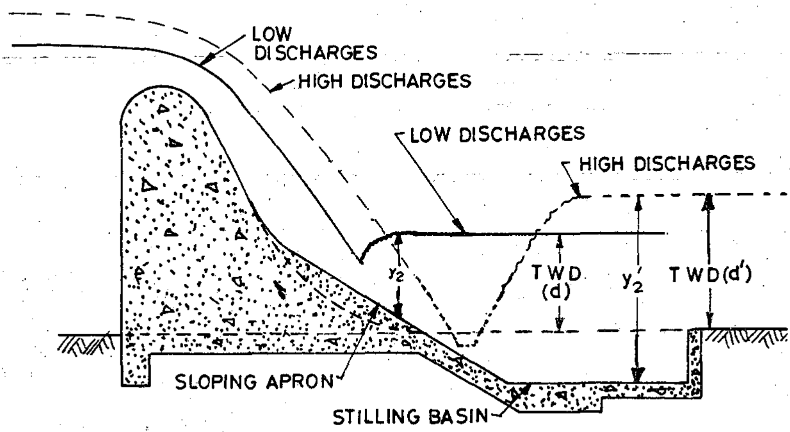


Fig. 21.31. (*d*) Sloping apron partly above and partly below the ground level.

At low discharges, the jump will form on the apron above the river bed, where the available depth is equal to the required depth and less than the T.W. depth. Similarly, at high discharges, the jump will form on the apron below the river bed, where the available depth is more than the T.W. depth and equal to the depth required for jump formation.

21.11.1.5. Energy dissipators for case (*e*). *When tail water depth is insufficient at low discharges and is greater at high discharges.* This case is just the reverse of case (*d*) and the same arrangement which was made in case (*d*) will serve the purpose. The only difference will be that at low discharges, the jump will form on the apron below the bed; and at high discharges, the jump will form on the apron at a point above the bed.

21.12. Energy Dissipation Below Other Types of Spillways

The five hydraulic conditions between tail water depth and post jump depth (y_2) discussed in the previous article, can occur below any type of spillway or even below the sluiceways or dam outlets, and erosion control can be obtained by the methods described. A chute or a shaft or a side channel spillway generally discharges water at a point far away from the dam. Hence, the protection is required only for the spillway, as the danger to the main dam is not there. Due to this reason, a hydraulic stilling basin is generally sufficient, and may be provided at the discharging point of the spillway. If sound rocks are available, a Ski jump bucket may be provided at low cost.

21.13. Energy Dissipation Below Sluiceways or Dam Outlets

Discharge through the sluiceways is generally small, and less protection is required below them. The water from the sluiceway is often allowed to fall directly into the stilling basin. Sometimes, the jet may be spread by a *jet deflector* at the outlet. Sometimes the energy dissipation arrangement of the main spillway may be utilised for sluiceways also.

21.14. Use of Hydraulic Jump as Energy Dissipator and Design of Stilling Basins

It was stated in chapter 10, that the hydraulic jump formation depends considerably upon the Froude number of the incoming flow (F_1). The pre-jump depth (y_1) and post-jump depth (y_2) are also governed by the equation (10.6) [derived in chapter 10] as:

$$\frac{y_2}{y_1} = \frac{1}{2}\left[\sqrt{1+8F_1^2} - 1\right] \qquad \text{...(10.6)}$$

$$\text{where } F_1 = \frac{V_1}{\sqrt{g \cdot y_1}}$$

It was also stated in the same chapter, that the energy dissipation in the jump, depends upon the Froude number. Different types of jump were described for different values of Froude numbers. If this incoming Froude number F_1 is higher, the greater energy dissipation can take place. The approximate percentage loss of energy for various values of F_1 are given in Table 21.14.

Table 21.14

F_1	*% loss of energy*
2.5	17
4.5	45
9.0	70
14.0	80
20.0	85

However, the real problem in the design of stilling basins, is not the absolute dissipation of energy, but is the dissipation of this energy in as short a length as possible.

Hence, the effectiveness of a hydraulic jump has to be viewed in this light. The types of jumps for different ranges of Froude numbers, given in chapter 10, are again summarised below :

(*i*) When $F_1 < 2.5$, the jump is weak and energy loss is low. No blocks or other devices are provided in this range.

(*ii*) When F_1 lies between 2.5 to 4.5, as in case of weirs and barrages, the jump is trouble-some and oscillating, which gives rise to heavy waves on the surface. *Wave Suppressors* may be needed in this range. The length of jump may be taken as $5\,(y_2 - y_1)$.

(*iii*) *When F_1 lies between 4.5 and 9.0 ; as is generally the case for dam spillways,* the jump performance is at its best. The jump is then called a steady jump. The length of the jump is almost constant and equal to $6y_2$ in this range. Hence for large spillways, where y_2 may be quite high, very long and expensive stilling basins may be required. Some auxiliary devices may also be introduced for further stabilising the flow and to reduce the length of the basin.

(*iv*) When $F_1 = 9.0$ and larger ; it is a strong jump. But the jump is likely to be rough and choppy. Hence, a bucket type of energy dissipator is preferred to a stilling basin of hydraulic jump type.

21.15. Standard Stilling Basins

Various types of stilling basins have been generalised for use on different types of works, by various agencies. The designs of these basins have been developed on the basis of long experience and on model studies, keeping in view the protection obtained consistent with economy. These basins are not simple concrete aprons but are generally provided with **auxiliary devices** such as chute blocks, sills, baffle walls, etc. These devices can help in dissipating the energy of flow by offering resistance to flow and may stabilise the flow in a shorter length of the basin, thus affecting economy.

In general, a stilling basin may be defined, as a structure in which the energy dissipating action is confined. If the phenomenon of hydraulic jump is basically used for dissipating this energy, it may be called a hydraulic jump type of stilling basin. The auxiliary devices may be used as additional measures for controlling the jump, etc.

Before we reproduce a few standard stilling basins, let us first describe, in brief, the effects produced by auxiliary devices.

Chute Blocks. Chute blocks are a kind of serrated device (*i.e.* row of small projections like teeth of saw) and provided at the entrance of the stilling basin. The incoming jet of water is furrowed and partly lifted from the floor, producing a shorter length of jump than what would have been without them. They also help in stabilising the flow and thus improve the jump performance (Fig. 21.32).

Sills and Dentated Sills. Sill or more preferably dentated sill is generally provided at the end of the stilling basin. The dentated sill diffuses the residual portion of high velocity jet reaching the end of the basin. They, therefore, help in dissipating residual energy and to reduce the length of the jump or the basin (Refer Fig. 21.32).

Baffle Piers. They are the blocks placed within the basin, across the basin floor. They help in breaking the flow and dissipate energy mostly by impact. These baffle piers, sometimes called **friction blocks,** are very useful in small structures, such as low

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spillways and weirs, etc. They, however, give way due to cavitation, under the influence of high velocity jets, and hence are unsuitable for large works.

21.15.1. U.S.B.R. Basins. U.S.B.R. has standardised stilling basins for different ranges of Froude numbers. The important of these basins, are :

(1) **U.S.B.R. stilling basin II.** This is recommended for use on large structures, such as dam spillways, large canal structures, etc., when the incoming *Froude number* (F_1) *is more than 4.5.* The dimensions of the chute blocks, dentated sill, etc. are shown in Fig. 21.32. The length of the basin is related to the Froude number (F_1) as given in Table 21.15.

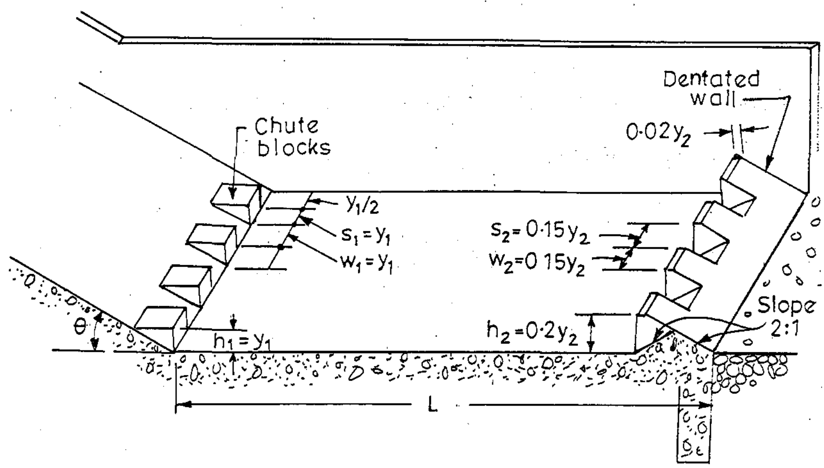


Fig. 21.32. U.S.B.R. Stilling basin II ($F_1 > 4.5$).

Table 21.15

F_1	*Length of the basin*
4	3.6 y_2
6	4 y_2
8	4.2 y_2
10 or more	4.3 y_2

An economy in the length of the basin up to about 35% (4.3 y_2 in place of $6y_2$) is thus obtained with auxiliary devices. The floor of the basin should be set at such a level as to provide 5% more water depth than y_2.

(2) **U.B.S.R. stilling basin IV.** This type of stilling basin is shown in Fig. 21.33. It is used for Froude number varying between 2.5 and 4.5, which generally occurs in canal weirs, canal falls, diversion dams, etc. This basin is applicable only to rectangular cross sections. Since oscillating waves are generated in this range of Froude number, they are tried to be controlled at source by providing large chute blocks. The spacing between the blocks may be further reduced than what is shown in Fig. 21.33 (up to say 0.75 W) for better hydraulic performance. The floor of the basin may be set at such a level as to

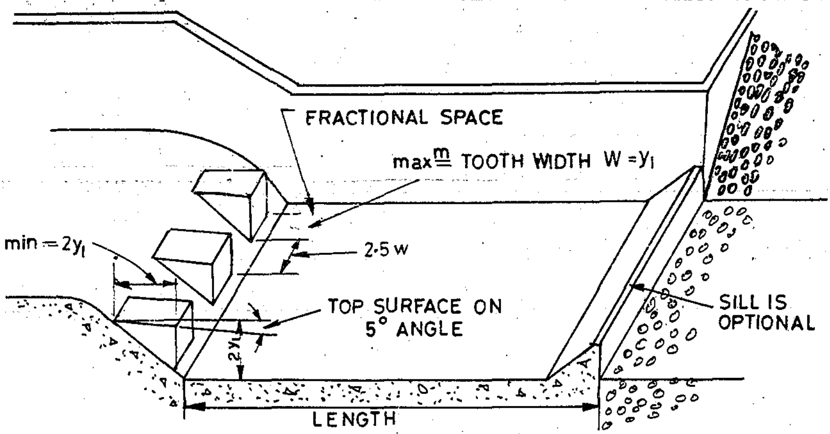


Fig. 21.33. U.S.B.R. Stilling basin IV (F_1 lies between 2.5 and 4.5).

provide 5 to 10% more water depth than the theoretical y_1. The length of the basin is generally kept equal to 5 $(y_2 - y_1)$.

21.15.2. I.S.I. Standardised Basins. Indian Standard Institution has also standardised certain stilling basins for uses under different conditions. A full description of such basins is available in IS : 4997 – 1968. Stilling basin I and II (for $F_1 < 4.5$ and $F_1 > 4.5$ respectively) with horizontal aprons and stilling basins III and IV with sloping aprons, have been standardised and described in the above reference. Students or field staff may refer to this I.S.I. publication in their particular needs.

21.16. Dynamic Force on Spillway Bucket and Spillway Bottom

When water flows over the curved bottom portion of an ogee spillway, there is a continuous change of velocity. Due to this change in velocity, there is a change in momentum (momentum = mass × velocity). Hence, a force is exerted on the spillway in accordance with Newton's second law of motion, *i.e.* rate of change of momentum is equal to this impressed force. This force is known as *dynamic force.* The spillway bottom where this curvature is provided is subjected to this force. The evaluation of this force is, therefore, necessary in the design of the bottom portion, or the design of the bucket, if it is provided as energy dissipator in an overflow spillway.

The magnitude of this force can be easily determined by using Newton's second law and by resolving all the forces acting on the curved element of water AB [Fig. 21.34 (*a*)] in horizontal and vertical directions. Mass of water when multiplied by net change in velocity in one direction will give the net component of this dynamic force in that direction. If F is this dynamic force, then

$$F_{(H)} = [V_{2(H)} - V_{1(H)}] \times \text{mass of water in the element}$$

$$F_{(V)} = [V_{2(V)} - V_{1(V)}] \times \text{mass of water in the element}$$

$$F = \sqrt{F_{(H)}^2 + F_{(V)}^2}$$

where V_1 and V_2 are the velocities at the entrance and exit of the curved element. $V_{1(H)}$ and $V_{2(H)}$ are their horizontal components, and $V_{1(V)}$ and $V_{2(V)}$ are their vertical components. An illustrative example has been solved below :

Example 21.3. *An overflow ogee spillway of height 15 m is discharging water with a head of 2 m over the crest. A reverse curvature of radius 4 m, subtending an angle of 60° at the centre, is provided at the spillway bottom as shown in Fig. 21.34 (a). Assuming the discharge coefficient for the spillway as 2.2, determine the magnitude of the dynamic force on the curve portion AB of the spillway.*

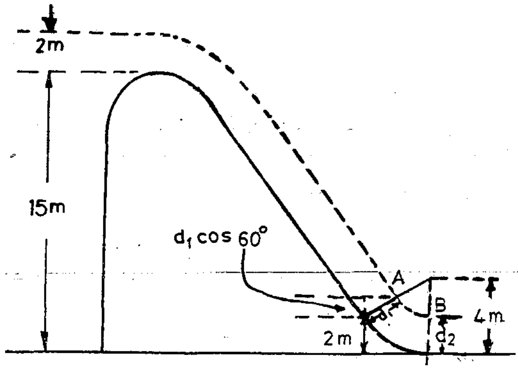


Fig. 21.34 (*a*)

Solution. The discharge over the spillway is given as :

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

Discharge per unit length,

$$q = C \cdot H^{3/2}$$

$$= 2.2 \times (2)^{3/2} = 6.23 \text{ cumecs/m}$$

Let d_1 and d_2 be the water depths, V_1 and V_2 be the flow velocities at A and B respectively.

Neglecting any loss of energy and the head due to velocity of approach and applying Bernoulli's theorem between u/s water surface and at sections A and B, respectively we, get

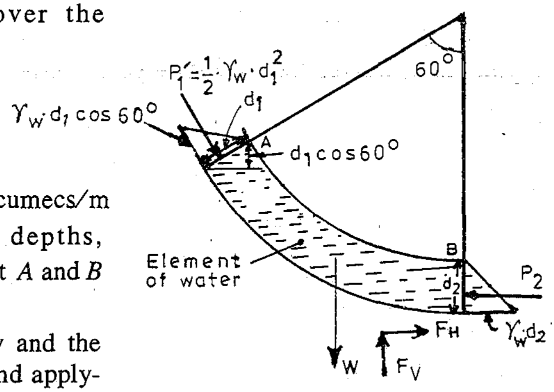


Fig. 21.34 (*b*)

$$15 + 2 = 2 + d_1 \cos 60° + \frac{V_1^2}{2g} = d_2 + \frac{V_2^2}{2g}$$

But $$V_1 = \frac{q}{d_1} \text{ ; and } V_2 = \frac{q}{d_2}$$

$\therefore$ $$17 = 2 + d_1 \cos 60° + \frac{q^2}{2gd_1^2} = d_2 + \frac{q^2}{2gd_2^2}$$

or $$17 = 2 + \frac{d_1}{2} + \frac{(6.23)^2}{2 \times 9.81\, d_1^2} = d_2 + \frac{(6.23)^2}{2 \times 9.81\, d_2^2}$$

or $$17 = 2 + \frac{d_1}{2} + \frac{1.97}{d_1^2} = d_2 + \frac{1.97}{d_2^2}$$

or $$17 = 2 + \frac{d_1}{2} + \frac{1.97}{d_1^2} \qquad \text{...(i)}$$

and $$17 = d_2 + \frac{1.97}{d_2^2} \qquad \text{...(ii)}$$

∴ (*i*) yields :

$$15 = \frac{d_1}{2} + \frac{1.97}{d_1^2}$$

or $$30d_1^2 = d_1^3 + 3.94$$

Solving by hit and trial, we get

$$\boxed{d_1 = 0.365 \text{ m}}$$

Similarly, solving equation (*ii*), we get

$$17 = d_2 + \frac{1.97}{d_2^2}$$

or $$17d_2^2 = d_2^3 + 1.97$$

by hit and trial, we get $\boxed{d_2 = 0.344 \text{ m}}$

Now the hydrostatic forces P_1 and P_2 per unit length are given as :

$$P_1 = \tfrac{1}{2} \cdot \rho_w \cdot d_1^2 \cos 60° = \tfrac{1}{2} \times 9.81 \times (0.365)^2 \times \tfrac{1}{2} = 0.327 \text{ kN/m run}$$

$$P_2 = \tfrac{1}{2} \cdot \rho_w \cdot d_2^2 = \tfrac{1}{2} \times 9.81 \times (0.344)^2 = 0.58 \text{ kN/m run}$$

Resolving all the Forces in the Horizontal Direction

Algebraic sum of all the forces in the horizontal direction

$$= P_1 \cos 60° + F_H - P_2 \qquad \text{...(iii)}$$

The rate of change of momentum per unit length in the horizontal direction

$$= \frac{\rho_w \cdot q}{g} [V_2 - V_1 \cos 60°]$$

$$= \frac{\rho_w \cdot q}{g} \left[V_2 - \frac{V_1}{2} \right] \qquad \text{...(iv)}$$

But $$V_1 = \frac{q}{d_1} = \frac{6.23}{0.365} = 17.06 \text{ m/sec.} \text{ ; and } V_2 = \frac{q}{d_2} = \frac{6.23}{0.344} = 18.12 \text{ m/sec.}$$

Equating (*iii*) and (*iv*), we get

$$P_1 \cos 60° + F_H - P_2 = \frac{\rho_w q}{g} \left[V_2 - \frac{V_1}{2} \right]$$

or $$0.327 \times \tfrac{1}{2} + F_H - 0.58 = \frac{9.81 \times 6.23}{9.81} \left[18.12 - \frac{17.06}{2} \right] = 59.75$$

or $$\mathbf{F_H = 60.16 \text{ kN}}$$

Absence of –ve sign indicates that the assumed direction of F_H is correct.

Resolving all the Forces in the Vertical Direction

The weight W of the water in the curved element AB

$$= \left[\frac{2\pi \cdot R}{360°} \times \theta° \right] \cdot y_{mean}(\rho_w)$$

$$= \left[\frac{2\pi \times 4}{360^\circ} \times 60^\circ\right] + \left[\frac{0.365 + 0.344}{2}\right] \times 9.81 = 14.57 \text{ kN.}$$

Resolving all the forces in the vertical direction, we get

$$W + P_1 \sin 60^\circ - F_V = \frac{\gamma_w \cdot q}{g} [0 - V_1 \sin 60^\circ]$$

or
$$14.57 + 0.327 \times \frac{\sqrt{3}}{2} - F_V = \frac{9.81 \times 6.23}{9.81} \left[- 17.06 \sin 60^\circ \right]$$

$$14.85 - F_V = -92.04$$

or
$$F_V = \mathbf{106.89 \text{ kN}}$$

Absence of –ve sign indicates that the assumed direction of F_V is correct and is vertically upward.

Resultant force $= \sqrt{F_H^2 + F_V^2} = \sqrt{(60.16)^2 + (106.89)^2}$
$= \mathbf{122.64 \text{ kN.}}$

Hence, the resultant dynamic force is equal to 122.64 kN and acts at an angle $= \tan^{-1}\left(\frac{106.89}{60.16}\right) = 60.6^\circ$ with the horizontal in the upward direction, as shown in Fig. 21.34 (*c*).

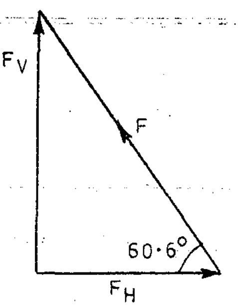


Fig. 21.34 (*c*)

SPILLWAY CREST GATES

If a temporary barrier can be installed over the permanent raised crest of a spillway, additional water can be stored between the spillway crest and the top of the barrier during the fag-end of the rainy season. The small flows in excess of the barrier top level, may be permitted to pass over the barrier. If, however, large flood occurs, the barrier may be removed and full spillway capacity made available for the outflow.

Sometimes on large dams, regular gates may be installed over the permanent crest, so as to function like a movable additional crest. In such a case, the height of the permanent raised crest can be reduced and the balance provided by the movable crest (*i.e.* gate). If there is a permanent raised crest up to the gate top, the storage, of course would be equal to that of a gated crest ; but in times of serious floods, the rise in flood level would be much more as compared to what would have been in a gated crest. This is because, the gates would be opened during serious floods so as to provide more head and hence larger discharge and consequent lesser rise in flood levels. Hence, the top level of the non-overflow section and the value of land acquisition for the reservoir which has to be determined by the maximum rise of flood above the spillway crest, can be reduced by providing gated crest or controlled crests. In other words, the dam height can be reduced for the same useful storage, or more useful storage can be obtained for the same height, provided the dam spillway is controlled by gates, etc.

This saving in the dam height and land acquisition will be more, if more height of the gates is provided. This saving is, however, counter-balanced by the cost of the gates. The cost of the gates would be more if their height is increased. (The gate cost include the principal cost and OMR, *i.e.* operational, maintenance and repair costs.) An economic balance between these two factors must be worked out and the cheapest combination found before deciding the height of the permanent crest and the height of the temporary crest (*i.e.* gates). This is also governed by the limitations of maximum available gate heights.

Gates can be provided on all types of spillways except siphon spillways. In siphon spillways, the gates are not required as the rise in flood level is already small compared to other types of spillways. The gates for earthen dams should be provided with caution, since the faulty operation or failure of their operation may lead to serious rise in flood levels, causing overtopping and failure of dam.

21.17. Types of Spillway Gates

The various types of spillway gates are described below in brief :

21.17.1. Dropping Shutters or Permanent Flash Boards. They consist of wooden panels usually 1.0 to 1.25 m high. They are hinged at the bottom and are supported against the water pressure by struts (Fig. 21.35). The shutters fall flat on the crest when the downstream supporting struts are tripped. Hence they are not suitable for curved crests.

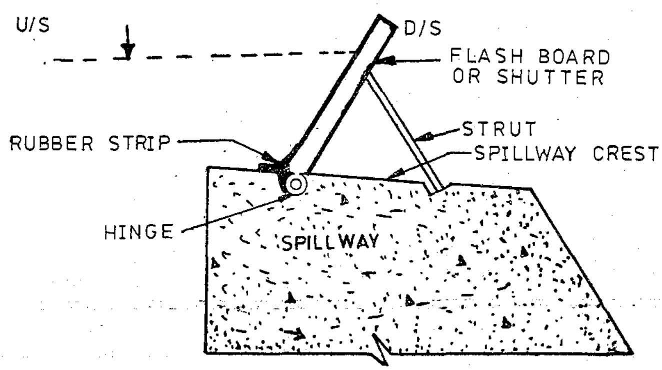


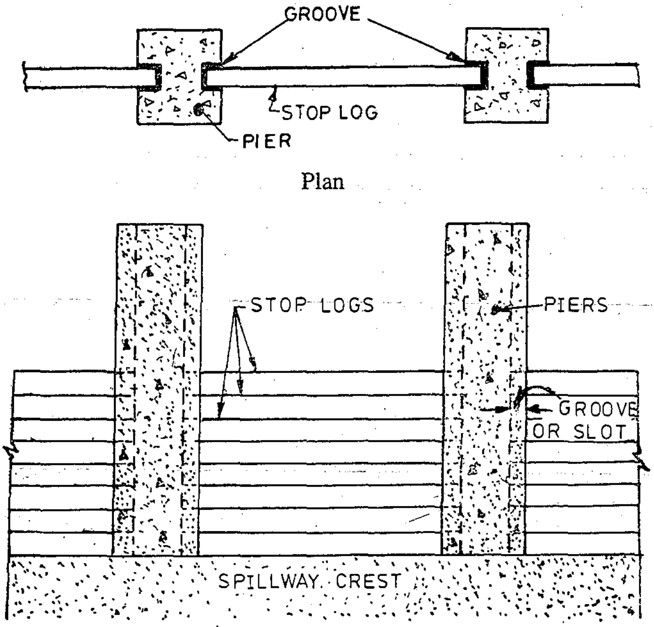
Fig. 21.35. Flash board or Dropping shutter.

These shutters can be raised or lowered from an overhead cableway or a bridge. Various types of shutters which drop and hoist themselves automatically, have been designed these days. These automatic shutters work on the principle of counter weights acting against the water pressure. Automatic shutters do not function well when interferred by floating debris, ice, etc.

Sometimes temporary flash boards, which shall fall as soon as overtopped by water, may be used for very minor works.

All kinds of flash-boards do have some disadvantages and hence used only on small spillways of minor importance.

21.17.2. Stop Logs and Needles. Stop logs consist of wooden beams or planks placed one upon the other and spanning in the grooves between the spillway piers (Fig. 21.36). They can be placed and removed either by hand or with hoisting mechanism. Considerable time may get wasted in removing them, if they become jammed in the slots. Leakage between the logs is also a big problem. They are, therefore, used on very minor works.



(Elevation)
Fig. 21.36. Stop logs.

Needles. Needles are wooden logs kept side by side with their lower ends resting in a keyway on the spillway and upper ends supported by a bridge (Fig. 21.37). It is very difficult to handle these needles at the time of flow and hence they are not used on any major works. They are sometimes used for emergency bulk heads, where they need not be replaced until the flow has stopped.

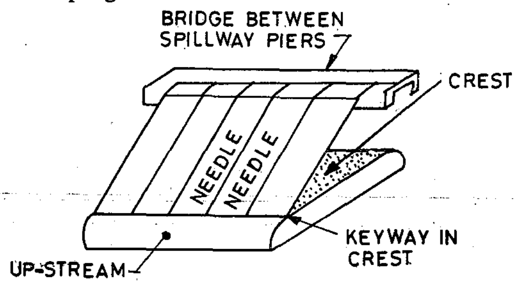


Fig. 21.37. Needles.

21.17.3. Vertical Lift Gates or Rectangular Gates. These are rectangular gates spanning horizontally between the grooves made in the supporting spillway piers (Fig.21.38). The grooves are generally lined with rolled steel channel sections of appropriate size, so as to provide a smooth bearing surface having sufficient bearing strength and are known as *grove guides.* These rectangular gates move between the groove guides, and can be raised or lowered by a hoisting mechanism at the top.

The gates are often made of steel, although they may be made of concrete or wood. They are generally placed vertical, although they may be kept slightly inclined downstream.

Because of the hydrostatic force caused by the upstream water standing against the gate, large friction is developed between the gate and the downstream groove guides. Hence, if the gate is in direct contact with the guides, as is there in a *sliding gate*, large friction will be developed, and it will be very difficult to move the gate. Hence, in a *sliding gate* relatively larger hoisting capacity is required to operate the gate because of the sliding friction that has to be overcome. The sliding gates are, therefore, seldom used.

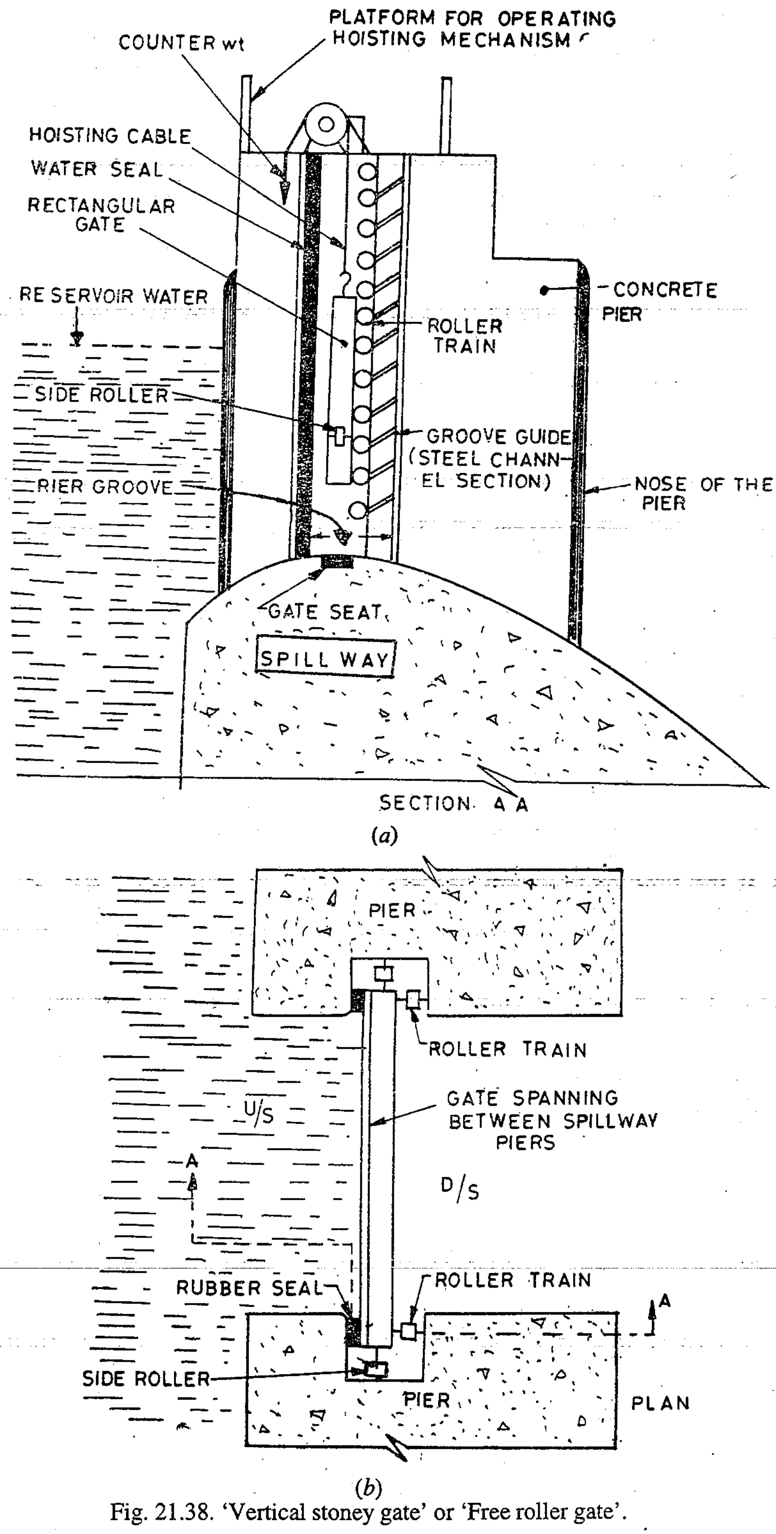


Fig. 21.38. 'Vertical stoney gate' or 'Free roller gate'.

This friction problem can be solved by placing cylindrical rollers between the bearing surfaces of the gate and the guide grooves. A train of rollers or wheels is, therefore, generally placed between the gate and the d/s guide, so that the sliding friction is much smaller. These rollers may be placed independent of the gate and the guide, thus eliminating axle friction, as shown in Fig. 21.38. Such an arrangement, when the rollers are neither attached to the gate nor to the guide grooves but rolls vertically between the two when the gate is moved, is known as a *Stoney gate* or a *Free Roller gate.* The design and construction of such a gate is difficult and rollers are, therefore, generally attached to the gate. Such an arrangement in which the rollers or wheels are attached to the gate and ride in tracks on the downstream side of the groove guide, is known as a *Fixed wheels gate* or *Fixed roller type gate.* Rubber seals are used to seal the openings between the upstream leaf plate and the sides of the pier grooves, as shown.

Large vertical lift gates may be counter balanced by a counter weight beam, which is loaded to balance the self-weight of the gate. Hence, hoisting force is required only to balance the frictional resistance.

Vertical lift gates have been used in size 15 m × 15 m (height and span). If the gate height is larger, head room is required for lifting the gate clear of the maximum reservoir level ; thus increasing the height of the operating platform. To reduce the height of the operating platform, high gates may be broken up into two horizontal sections, so that the upper portion may be lifted and removed from the guides before the lower portion is moved. This also reduces the load on the hoisting mechanism.

The discharge through a partially raised vertical gate, takes place by an undershot orifice flow, and the discharge formula is given by $C_d A \sqrt{2g \cdot H_1}$, where A is the area of the opening and H_1 is the water head above the centre line of the opening.

21.17.4. Radial Gates or Tainter Gates. A radial gate has a curved water supporting face made of steel. The curved water face which is in the shape of a sector of a circle is properly braced by steel frame work which is pivoted on horizontal shafts called *trunnions* or *pins* (Fig. 21.39). The pins are anchored in the downstream portion of the spillway piers. The gate can

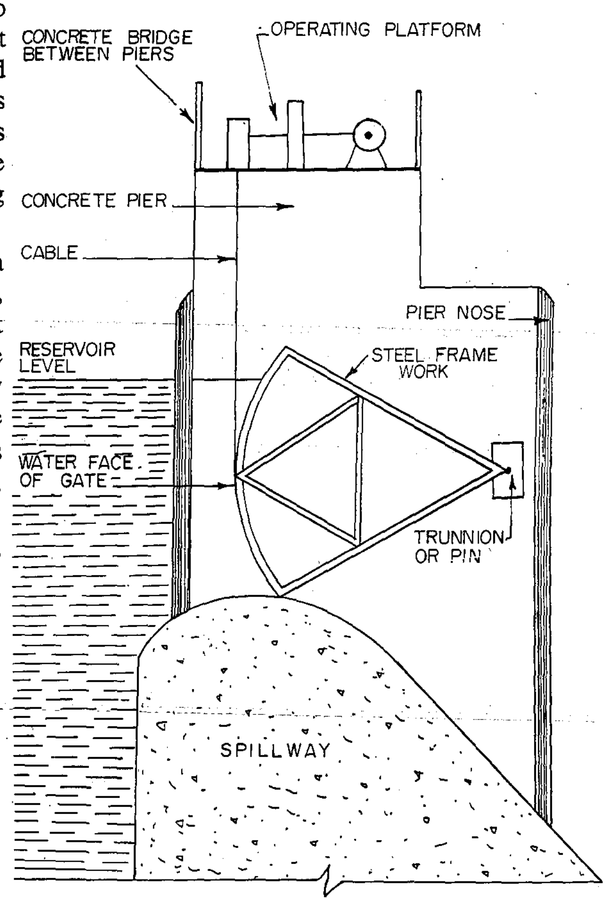


Fig. 21.39. Radial gate or Tainter gate.

thus rotate about the fixed horizontal axis. Hoisting cables are attached to the gate and lead to winches on the hoisting platform. The winches are usually motor driven, although hand driving is possible for smaller works or at times of power failures.

The water-face segment is made concentric to the supporting pins so that the entire water thrust passes through the pins, thus creating no moment against the lifting of the gate. Hence, the lifting force is required only against the weight of the gate, the friction between the seals and the piers, and the frictional resistance at the pins. Counter weights, in order to counter balance the self weight, may also be used, which further reduces the lifting force. Moreover, the hoisting load is nearly constant for all gate openings. Hence, radial gates can be used with smaller lifting force for all heads, and hand operating hoisting mechanism may suffice for smaller works ; whereas in the vertical lift gates of the same size, power mechanism might be needed.

21.17.5. Drum Gates. Drum gates are useful for longer spans of the order of 40 m or so and medium heights say 10 m or so. The drum gate consists of a segment of a cylinder which may be raised above the spillway crest or may be lowered into the *recess* made into the top of spillway.

The U.S.B.R. drum gate (Fig. 21.40) is completely enclosed and is hinged at the upstream end. The buoyant forces due to head water pressure underneath the drum, aid

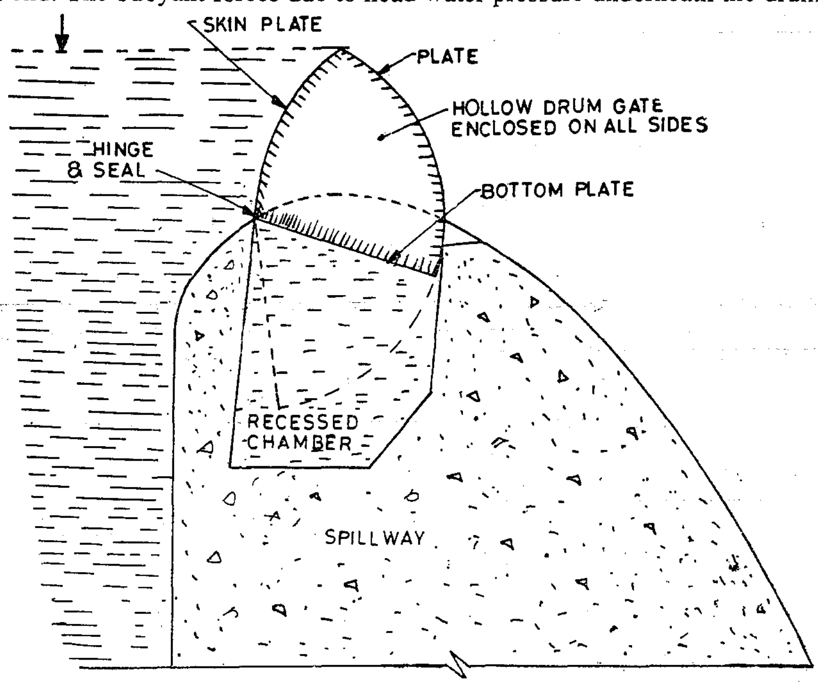


Fig. 21.40. Drum Gate (U.S.B.R. Type).

in its lifting. In this type of dam gate, the drum is enclosed on all the three sides as well as on the ends, thus forming a water tight vessel. When the drum is lowered, it fits into the recess in such a way that the surface becomes coincident with the designed ogee shape of the crest.

The other type of drum gate may have no bottom plate and shall be raised only by the buoyant action of water entering the recess, underneath the skin plate of drum.

The drum gates require large recess and hence, are not suitable for smaller spillways.

PROBLEMS

1. (*a*) "A spillway is a safety valve in a dam". Discuss the statement.

(*b*) Discuss the location of the 'main spillway' and 'subsidiary spillway' in gravity dams as well as in earthen dams.

(*c*) Briefly describe an 'ogee spillway'.

2. (*a*) What are spillways and where are they provided ?

(*b*) Enumerate the various types of spillways, and describe in details the most widely used type.

3. (*a*) What are the different kinds of spillways and how are they selected for individual conditions ?

(*Madras University, 1975*)

(*b*) Sketch an ogee profile and mark in it the different zones. (*Madras University, 1976*)

4. (*a*) What are spillways and what is their necessity ?

(*b*) Enumerate the different type of spillways which are used in dam construction.

(*c*) Discuss briefly the design principles that are involved in the design of an ogee spillway ; and a chute spillway.

5. (*a*) What is an ogee spillway and how is it designed ?

(*b*) Discuss the necessity of the aeration arrangements that are required in gated ogee spillways.

6. Write down an equation for calculating the discharging capacity of an ogee spillway. How does 'coefficient of discharge' in this equation varies with the :

(*i*) Depth of approach ;

(*ii*) Slope of upstream face of the ogee spillway ;

(*iii*) Submergence of the spillway by the tail water ;

(*iv*) The ratio of design head to actual head over the spillway.

7. (*a*) How would your compute the discharge passing over an ogee spillway. Discuss the various factors affecting the coefficient of discharge in the discharge equation.

(*b*) Compute the discharge over an ogee spillway with coefficient of discharge $c = 2.3$ at a head of 3.8 m. The effective length of the spillway is 110 m. Neglect the velocity of approach.

[**Ans.** 1872 cumecs]

8. Design a suitable section for the overflow section of a concrete gravity dam having the d/s face sloping at a slope of 0.7 H : 1 V. The design discharge for the spillway is 6000 cumecs. The height of the spillway above the river bed is 60 m. The effective length of spillway may be taken as 50 m.

[**Hint.** Follow example 21.1].

9. (*a*) What is meant by a 'spillway' and what is its necessity in dam construction ?

(*b*) Enumerate the different types of spillways, and draw neat sketches for all the types, showing the different parts of each.

10. (*a*) What is a siphon spillway ? Enumerate the two types of saddle syphon spillways, and describe with a neat sketch the component parts and functioning of one of these two types :

(*b*) A siphon spillway has the following cross-section at its throat :

Height of the throat = 1.5 m

Width of the throat = 4 m.

At the design flow, the tail water elevation in 7 m below the summit of the siphon, and the head water elevation is 2 m above the summit, (*i*) Taking a coefficient of discharge as 0.6 ; determine the capacity of the syphon (*ii*) Determine the head that would be required on an ogee spillway 3.8 m long to discharge this flow, if coefficient of discharge is 2.25 ; (*iii*) What length of this ogee weir would be required to discharge the same flow with a head of 2.2 m on the crest.

[**Solution.** (*a*) Discharge through siphon spillway (*i.e.* capacity)

$$= 0.6\,(1.5 \times 4)\sqrt{2 \times 9.81 \times (7+2)}$$

$$= 47.8 \text{ cumecs. } \textbf{Ans.}$$

(*b*)
$$47.8 = 2.25 \times 3.8\, H^{3/2}$$

or
$$H = 3.15 \text{ m. } \textbf{Ans.}$$

(c) $4.78 = 2.25\,L\,(2.2)^{3/2}$

or $L = 6.5$ m. **Ans.]**

11. (a) Describe briefly the construction and functioning of a 'Hooded type' of siphon spillway. What is its discharge equation ?

(b) What is meant by priming ? Discuss the priming arrangements used in saddle siphon spillways.

12. (a) What is a 'chute spillway' ? Where is it preferred to ogee and other types of spillways ?

(b) Discuss briefly the component parts and their design for a chute spillway.

13. Discuss the design of the following components of a chute spillway :

(i) Approach channel

(ii) Control structure

(iii) Discharge carrier

(iv) Energy dissipation arrangement at the bottom.

14. Write short notes on :

(i) Volute siphon

(ii) Side channel spillway or Chute spillway or Shaft spillway

(iii) Ogee spillway

(iv) Priming devices for siphon spillways.

15. What is meant by an 'energy dissipator' ? Discuss the various methods used for energy dissipation below spillways.

16. Discuss briefly the various types of energy dissipators that are used for energy dissipation below overflow spillways, under different relative positions of T.W.C. and J.H.C.

17. (a) What is a 'spillway gate' and what are the merits and demerits of installing such gates ?

(b) Enumerate the important types of spillway gates. Describe with a neat sketch the construction and working of 'Tainter gate'.

18. (a) Discuss the merits and demerits of different types of spillway gates.

(b) Discuss with a neat sketch the construction, and functioning of a 'Stoney gate' as well as a 'Drum gate'.

19. Write detailed notes on the following :

(i) Energy dissipation below spillways

or

Spillway buckets and stilling basins.

(ii) Spillway gates.