II. SPILLWAYS

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

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

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

2.2. Essential Requirements Of a Spillway

The essential requirements of a spillway are:

- i) The spillway must have sufficient capacity;
- ii) It must be hydraulically and structurally adequate;
- iii) It must be so located that it provides safe disposal of water, i.e. spillway discharge will not erode or undermine the d/s of the dam;
- iv) The bounding surfaces of the spillway must be erosion resistant to withstand the high scouring velocities created by the drop from the reservoir surface to the tail water.
- v) Some device will be required for dissipation of energy on the d/s side of the spillway.

2.3. Spillway Capacity

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

- i) Inflow hydrograph (plot of rate of inflow Vs time)
- ii) Reservoir capacity curve (plot of reservoir storage Vs water surface elevation)
- iii) Discharge curve (plot of rate of outflow Vs reservoir water surface elevation).

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

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

- i) The inflow flood;
- ii) The available storage capacity;
- iii) The discharge capacity of other outlet works;
- iv) Whether the spillway is gated or ungated;
- v) The possible damages if a spillway of adequate capacity is not provided.

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

2.4. Components Of A Spillway

The following are the main components of a spillway:

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

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

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

iv) **Entrance or approach channel and outlet channel:** Entrance channels may be required to draw water from the reservoir and convey it to the control structure. Similarly outlet channels may be required to convey the spillway flow from the terminal structure to the river channel below the dam. The entrance and outlet channels are not required where a spillway draws water directly from the reservoir and delivers it directly back into the river; e.g. overflow spillway. However, in the case of spillways placed through abutments or through saddles or ridges, the entrance and outlet channels may be required.

2.5. Types Of Spillway

Spillways may be classified:

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

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

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

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

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

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

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

2.5.1. Free Overfall Or Straight Drop Spillway

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

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

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

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

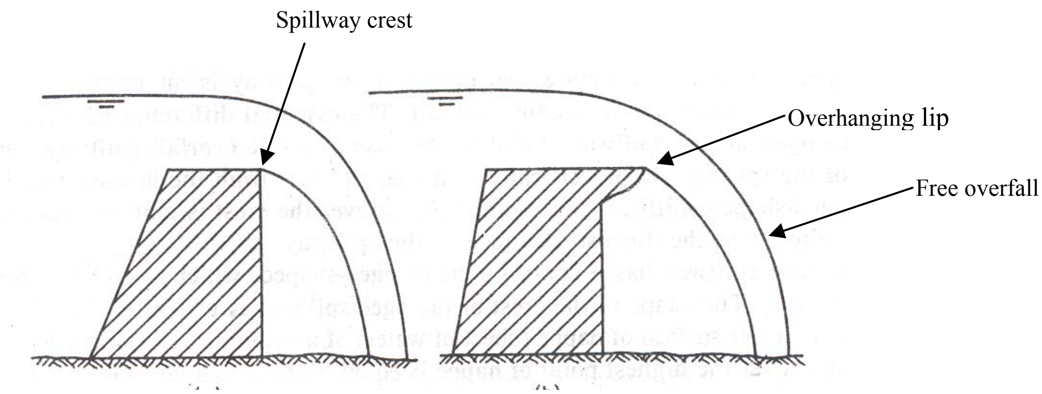

Fig 2.1 Straight drop spillway

2.5.2. Overflow (or Ogee) Spillways

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

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

2.5.2.1. Crest Shape Of Overflow Spillway

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

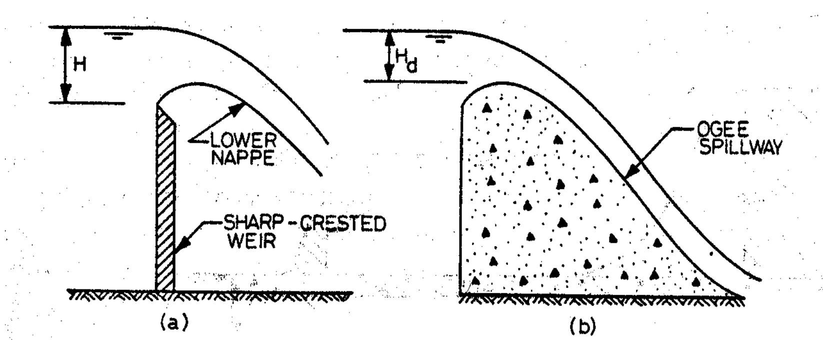

Fig 2.2 Crest shape of overflow spillway

At the design head ($H = H_d$) the water flowing over the crest of the spillway will remain in contact with the surface of the spillway as it glides over it and optimum discharge will occur.

In this case no pressure is exerted on the spillway by the flowing water, as there will be atmospheric pressure along the contact surface between the flowing water and the spillway.

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

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

2.5.2.2. Design Of Crest Of Ogee Spillway

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

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

$$\mathbf{X^n = KH_d^{n-1} y} \qquad (2.1)$$

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

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

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

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

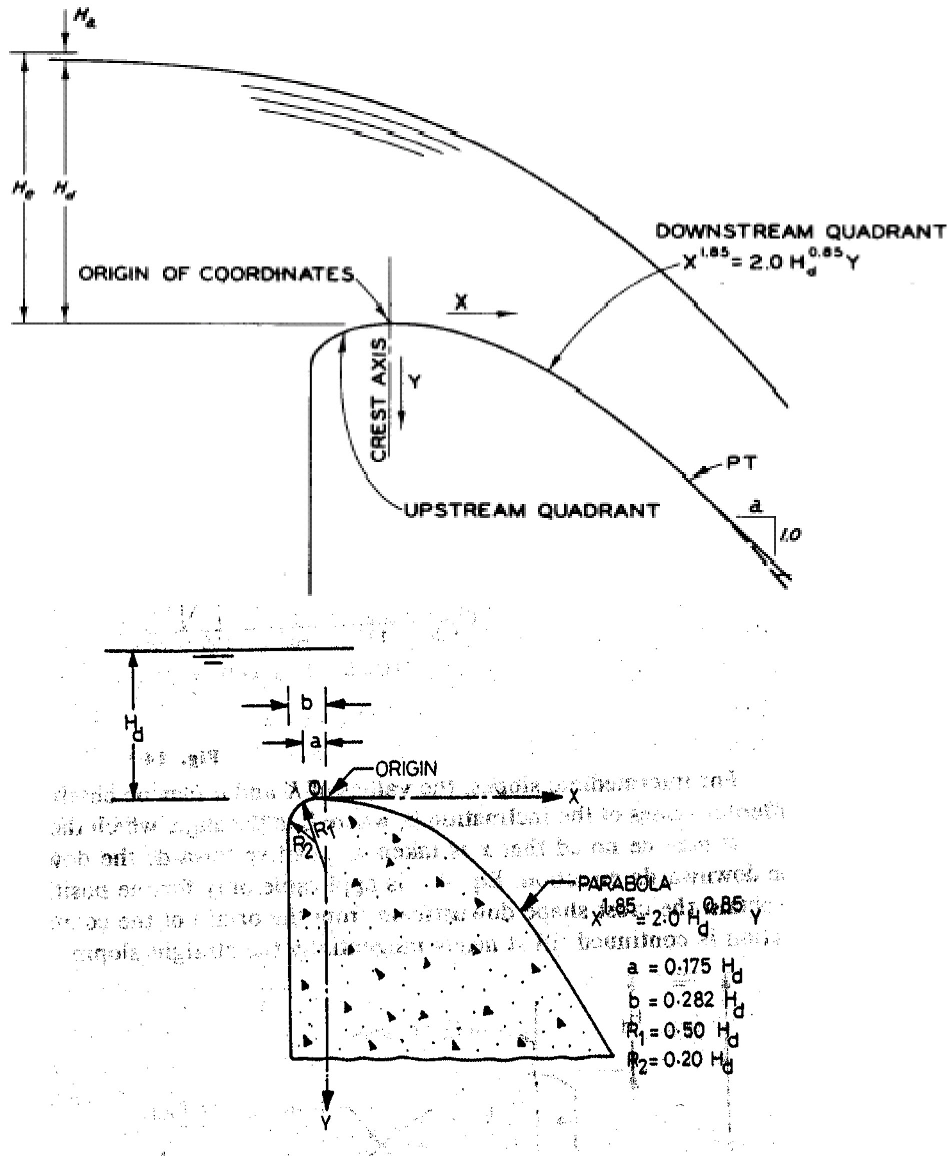

Fig 2.3 WES- standard spillway shape (vertical upstream face)

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

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

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

$$y = \frac{0.724(x + 0.27H_d)^{1.85}}{H_d^{0.85}} + 0.126H_d - 0.4315H_d^{0.375}(x + 0.27H_d)^{0.625} \qquad (2.2)$$

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

The corresponding y value is equal to 0.126 H_d.
The curved profile of the crest section is continued tangentially along the straight sloping surface, which forms the d/s face of the spillway. The location of the point of tangency (P.T) depends on the slope of the straight portion of the d/s face of the spillway, which in turn depends on the stability requirements and on the features of the stilling basin at toe of the spillway. The slope of the straight portion varies between 1V: 0.6H to 1V: 0.8H. At the end of the sloping surface a curved bucket is provided to create a smooth transition of flow from the spillway to the outlet channel or the river on the d/s side and prevent scoring.

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

$$\mathbf{R = 10^{(v+6.4H+4.88)/(3.6H+19.52)}} \qquad (2.3)$$

V= velocity of flow at to e of spillway [m/s
H = head excluding head due to velocity of approach (m)

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

$$\mathbf{V = \sqrt{2g(Z + Ha - y}} \qquad (2.4)$$

where Z = the fall, m
H_a = head due to velocity of approach, m
y = depth of flow at the toe, m

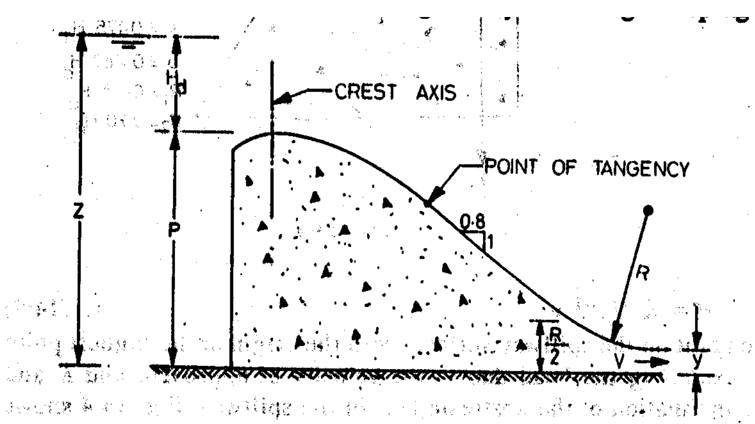

Figure 2.4 Profile of an ogee spillway

2.5.2.3. Discharge Of Overflow Spillway

The discharge over an overflow spillway is given by

$$Q = CLH_e^{3/2} \qquad (2.5)$$

Where Q = discharge, m^3/s

C = coefficient of discharge

L = effective length of crest of spillway, m

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

$H_e = H_d + H_a$

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

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

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

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

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

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

Fig 2.5 Plot of coefficient of discharge versus (P/H_e)

(b) Effect of heads differing from the design head: The plot of (C/C') versus (H/H_e) for a spillway of height P above stream bed greater than 1.33 H_d, where C is coefficient of discharge corresponding to the actual head of flow H and C' is the coefficient of discharge corresponding to the design head H_e. It may be observed from this plot that with increase in the value of (H/H_e) the value of (C/C') increases. In other words, with increase in the head H the coefficient of discharge increases. However, for $H< H_e$, $C < C'$; and for $H > H_e$, $C > C'$.

Fig 2.6 plot of C/C' Vs H/H_e

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

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

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

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

$$\mathbf{C = C'\left(\frac{H}{H_e}\right)^{0.12}} \qquad (2.6)$$

Where H_e = design head including velocity head and C' = 2.2

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

Figure 2.7 Effect of upstream face slope

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

Fig 2.8 Maximum TW depth for a non-submerged weir

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

$$Le = L - 2\,(NK_p + Ka)\,H_e \qquad (2.7)$$

Where L_e = effective length of crest

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

H_e= total head on crest including velocity head

N = number of Piers
K_P = Pier contraction coefficient
K_a = abutment contraction coefficient

The pier contraction coefficient, K_p depends on
i) Shape and location of pier nose;
ii) Thickness of pier;
iii) Velocity of approach; and
iv) Ratio of actual head to design head.

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

Pier coefficients, K_p:

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

The abutment contraction coefficient K_a depends on:
i) Shape of abutment;
ii) Angle between upstream approach wall and axis of flow;
iii) Approach velocity; and
iv) Ratio of actual head to design head

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

Abutment coefficients, K_a:

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

Where r = radius of abutment rounding
H_d = design head.

Figure 2.9 Abutment contraction & Pier nose shapes

2.5.3. Chute (or Open Channel or Trough) Spillway

For embankment dams it is not possible to provide overflow spillway. Further for concrete or masonry dams also, the narrowness of the valley, the erodible nature of the streambed, etc., overflow spillway may be impossible to be provided. In such cases, a chute spillway, which may be isolated from the main dam, must be provided.

The chute spillway is provided either in a saddle along the reservoir rim, or along abutment of the dam. It consists of a steep sloped open channel called a chute (hence this spillway derives its name from the carrier channel). Chute spillway is composed of the following components and arrangements.

2.5.3.1. Control Structure (or Low Ogee Weir)

Since the chute 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 saddle is at higher level than the full reservoir level, it is excavated to the latter level and left to serve as a flat crested weir. On the other hand, if the saddle is at a lower level than the full reservoir level then a weir has to be built up to that level having an ogee shape to obtain a high discharge coefficient.

The downstream profile of the low ogee weir is represented by

$$\mathbf{x = kH_d^{n-1}y} \tag{2.8}$$

The values of k and n for low ogee weir depend on the ratios (H_a/H_d) and (P/H_d). U.S. W.E.S. recommends the following values for k and n.

Values of (H_a/H_d)	**Applicable (P/H_d) range**	**Values of k**	**Values of n**
0.00	≥ 1.0	1.852	1.780
0.08	1.0 – 0.57	1.896	1.750
0.12	0.57 – 0.30	1.905	1.747

The toe curve radius equal to $2H_d$ may be provided.

2.5.3.2. General Arrangement

The trough (or chute) is usually widest at the crest and then narrows to a width, which is determined by the most economical shape of the trough. At the extreme lower end the trough is again widened to reduce the exit velocity.

2.5.3.3. Longitudinal Chute Slope

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 bed filling. After that, the slope is made as steep as possible without endangering the stability or without getting into heavy excavation.

2.5.3.4. Side Walls

The sidewalls 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 surface. The minimum freeboard to be provided may be computed from the following empirical relation:

$$\text{Freeboard (m)} = \left(0.61 + 0.04Vd^{1/3}\right) \qquad (2.9)$$

where V = mean velocity of flow in the chute reach (m/s)
d = depth of flow in the chute reach under consideration (m)

The sidewalls 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.

2.5.3.5. Design of Vertical Curves

This is needed where the bed slope changes.

(a) Concave Curve: Whenever the slope of the chute changes from steeper to milder, a concave vertical curve will be provided. In no case should the radius of this curve be less than 10d, where d is the depth of water in meters. It is generally given by

$$R = \frac{\gamma}{g}\frac{dV^2}{p} = \frac{\gamma}{g}\frac{qV}{p} \qquad (2.10)$$

where d = depth of flow
V = velocity of flow
q = discharge per unit width (dxV)
p = normal dynamic pressure exerted on the floor.

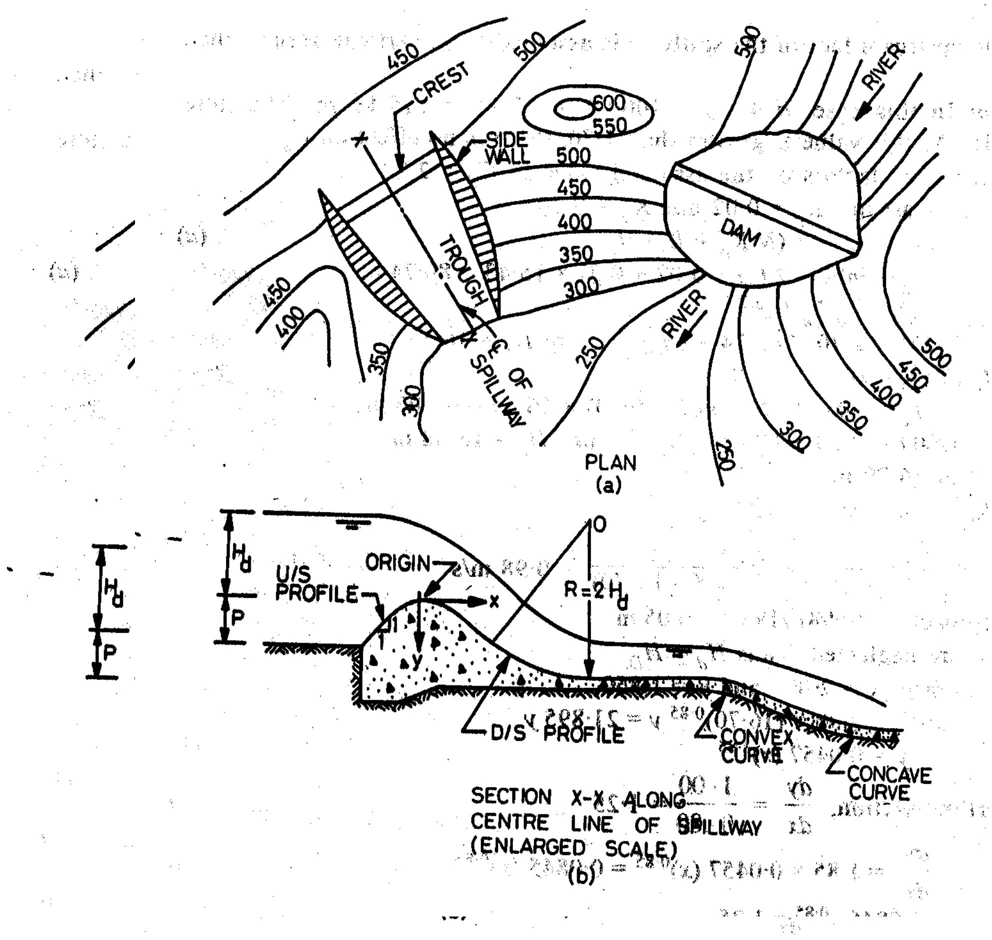

Figure 2.10 Chute spillway

The concave curve should have a sufficiently long radius of curvature to minimize the dynamic pressure exerted on the floor by the centrifugal force that results from a change in the direction of flow.

(b) Convex Curve: Provided when the slope of the chute changes from milder to steeper slopes. It should be flat enough to maintain positive pressures and thus avoid the tendency for separation of the flow from the floor. It should conform to the shape defined by:

$$y = -x\tan\theta - \frac{x^2}{k\left(4\left(d + h_v\right)\cos^2\theta\right)} \qquad (2.11)$$

where:
x, y = coordinates of the convex curve with origin at the end of the upstream sloping floor,
θ = slope of the upstream sloping floor,
d = depth of flow, h_v = velocity head (at the end of upstream sloping floor),
k = a factor (to ensure positive pressure along the entire contact surface of the curve, k = 1.5).

2.5.3.6. Approach Channel (or Entrance Channel) of a Chute Spillway

A trapezoidal channel with side slopes 1:1 may be constructed to lead the reservoir water up to the 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 the channel. The chute channel called discharge channel or discharge carrier is generally kept straight in plan. If, however, any curvature is required to be provided, its floor should be supper elevated to guide the high velocity flow around the bend thus avoiding piling up of flow towards the outside of the chute.

2.5. 4. Side Channel Spillways

The distinguishing characteristics of side channel spillways is that the flow, after passing over the ogee crest, is carried away by a channel running essentially parallel to the crest. This type of spillway is suitable for earth or rockfill dams in narrow canyons, and for other situations where direct overflow is not permissible and where the space required for a chute spillway of adequate crest length is not available. Flows from the side channel can be directed into an open discharge channel or into a closed conduit or inclined tunnel.
The spillway proper is usually designed as a normal overflow spillway. The depth, width, and bed slope of the flume must be designed in such a way that even the maximum flood discharge passes with a free overflow over the entire horizontal spillway crest, so that the reservoir level is not influenced by the flow in the channel (flume). The width of the flume may therefore increase in the direction of flow (Ref. Fig. 2.11a).

Figure 2.11 Side channel flow characteristics and cross-sections

The theory of flow in a side channel spillway is based principally on the law of conservation of linear momentum, assuming that the only force producing motion in the channel result from the fall in the water surface in the direction of the axis. This premise assumes that the entire energy of the flow over the crest is dissipated through its intermingling with the channel flow and is therefore of no assistance in moving the water along the channel. Axial velocity is produced only after the incoming water particles join the channel stream.

For any short reach of side channel, the momentum at the beginning of the reach plus any increase in momentum due to external forces must equal the momentum at the end of the reach.

If Q_1 and V_1 are values at the beginning of the reach and Q_2 and V_2 are the values at the end of the reach, the equation for the change in water surface elevation, can be written as:

$$\Delta y = \frac{Q_1}{g}\frac{(V_1 + V_2)}{(Q_1 + Q_2)}\left[(V_2 - V_1) + \frac{V_2(Q_2 - Q_1)}{Q_1}\right] \qquad (2.12)$$

By use of equation (2.12), the water surface profile can be determined for any particular side channel by assuming successive short reaches of channel once a starting point is found. The solution of equation (2.12) is obtained by trial-and-error procedure. For a reach of length Δx in a specific location, Q_1 and Q_2 will be known. If the depth at one end of the reach has been established, a trial depth at the other end of the reach can be found which will satisfy the indicated computed values of Δy.

In the determination of the water surface profile, the depth of flow and the hydraulic characteristics of the flow will be affected by backwater influences from some control point, or by critical conditions along the reach of the channel under consideration.

If the slope of the bottom is greater than critical and a control section is not established downstream of the side channel trough, supercritical flow will prevail throughout the length of the channel. For this stage, velocities will be high and water depths will be shallow, resulting in relatively high fall from the reservoir water level to the water surface in the trough. This flow condition is illustrated by profile B in Figure (2.11). If control section is established downstream from the side channel trough to increase the upstream depths, the channel can be made to flow at the subcritical stage. Velocities will be less than critical and the greater depths will result in a smaller drop from the reservoir water surface to the side channel water surface profile (Ref. Profile A in Fig. 2.11).

For subcritical stage, the incoming flow will not develop high transverse velocities because of the low drop before it meets the channel flow, thus effecting a good diffusion with the water bulk in the trough. Since both the incoming velocities and the channel velocities will be relatively slow, a fairly complete intermingling of the flows will take place, thereby producing a comparatively smooth flow in the side channel.

For supercritical stage, the channel velocities will be high, and the intermixing of the high-energy transverse flow with the channel stream will be rough and turbulent. The transverse flows will tend to sweep the channel flow to the far side of the channel, producing violent water action with resulting vibrations.

Thus, flows should be maintained at subcritical stage for good hydraulic performance. This is achieved by establishing a control section downstream from the side channel trough. A control section downstream from the side channel trough is achieved by constricting the channel sides or elevating the channel bottom to produce a point of critical flow.

2.5. 4.1. Cross Sectional Shape of Side Channel Trough

The cross sectional shape of the side channel trough is influenced by
a) The overflow crest, and
b) The bank conditions on the opposite side

Because of turbulences and vibrations inherent in side channel flow, a side channel design is considered only where a competent foundation such as rock exists. The channel sides will usually be a concrete lining placed on a slope and anchored directly to the rock.

A trapezoidal cross section is the one most often employed for side channel trough. If the width to depth ratio is large, the depth of flow in the channel will be shallow and poor diffusion of the incoming flow with the channel flow will result. A cross section with a minimum width-depth ratio will provide the best hydraulic performance. Therefore, a minimum bottom width must be selected which is commensurate with both the practical and structural aspects of the problem.

2.5.5. Siphon Spillways

A siphon spillway is a closed conduit system formed in the shape of an inverted U, positioned so that the inside of the bend of the upper passageway is at normal reservoir storage level. Such a spillway occupies less space and regulates the reservoir level within narrow limits. The initial discharges of the spillway, as the reservoir level rises above normal, are similar to the flow over a weir. Siphonic action takes place after the air in the bend over the crest has been exhausted. Continuous flow is maintained by the suction effect due to the gravity pull of the water in the lower leg of the siphon.

Most siphon spillways are composed of five component parts as shown in Figure (2.12). These include an ***inlet, an upper leg, a throat or control section, a lower leg, and an outlet***. The throat or control section is generally rectangular in cross section and is located at the crest of the upper bend of the siphon.

Figure 2.12 Component parts of a siphon spillway

The entrance and exit lips of the hood are so shaped that the siphon duct has bell mouth entry and exit. The inlet of the siphon duct is kept submerged well below the full reservoir level so that floating debris, etc., do not enter the siphon duct and also the formation of vortices and drawdown that might break the siphonic action is avoided.

The outlet is kept submerged in cup-like basin that forms a water seal so that air cannot enter the siphon duct from this end. This can be replaced by creating a cistern by constructing a low weir a little away from outlet.

At full reservoir, water stands up to the crest of the spillway and hence no flow. When water rises above this level, i.e., above crest of spillway, water starts flowing over the crest and the inlet of the deprimer hood gets submerged with the result that entry for air into the deprimer hood and the main hood is sealed. Thus air cannot enter from both the inlet and the outlet of the siphon duct and the air remaining in the top portion of the siphon duct above the sheet of water flowing over the crest is gradually sucked by the flowing water. As the air is sucked the pressure drops to less than atmospheric in the top portion of the siphon duct, which was having atmospheric pressure at the starting of the flow. Thus a difference in pressure develops between the outside atmosphere and the inside of the siphon duct which creates a suction pull and draws in more water over the crest. This sucking action, which increases progressively and gradually creates the necessary pull for the commencement of the siphonic action and the siphon duct starts flowing full. The action of siphon spillway from the moment the water just begins to flow over the crest to the instant when the siphon duct starts running full is known as ***priming.***

The siphonic action once initiated will continue as long as reservoir water level is above full reservoir level. The operating head for the spillway is then equal to the difference between the water levels on the upstream and downstream sides of the spillway. When the water level in the reservoir drops to such a level that the inlet of the deprimer hood gets exposed, air enters the siphon duct and breaks the siphonic action thus stopping the flow. This action is called ***depriming*** of the siphon spillway.

Siphonic action would normally continue to take place until the reservoir level comes down below the inlet or mouth level, which means loss of valuable storage. To control this effect, an air vent is provided at the crest of the siphon (Ref. Fig. 2.12). As soon as water level drops down, the moth of the vent gets exposed to air; the air rushes into the siphon hood and siphonic action stops. On the other hand, as the water level rises, the air vent goes out of operation due to its open end being sealed by the water surface.

2.5.5.1. Pressure in Siphon Spillway

After the siphon spillway gets primed and is running full, consider point (1) on the water surface in the reservoir, point (2) at the throat section (at the same elevation as point 1) and point (3) at the exit end of the siphon duct if it is discharging freely or on the tailwater surface if the exit end of the siphon duct is held submerged in the tailwater (see Fig. 2.13).

Figure 2.13 Pressure in siphon spillway

Let H = operating head for siphon spillway = difference in elevation of points 1 or 2 and 3

V_1, V_2 = mean velocity of flow at the throat (point 2) and at the exit of siphon duct, respectively

P_2 = pressure at throat section of the spillway (point 2)

P_a = atmospheric pressure

H_L = head loss in the entire siphon duct (i.e., between points 1 and 3)

h_L = head loss between the throat and the exit end of the siphon duct (i.e., between points 2 and 3)

h'_L = head loss between the inlet and the throat section (i.e., between points 1 and 2)

Writing energy equation between points (1) and (3), we get

$$H = H_L + \frac{V_3^2}{2g} \Rightarrow V_3 = \sqrt{2g(H - H_L)} \qquad (2.13a)$$

where

$$H_L = h_f + h_m = f\frac{LV^2}{2gD} + \sum k\frac{V^2}{2g} \qquad (2.13b)$$

Similarly, applying energy equation between points (2) and (3), we get

$$\frac{P_2}{\gamma} + \frac{V_2^2}{2g} + H = \frac{P_a}{\gamma} + \frac{V_3^2}{2g} + h_L \Rightarrow \frac{P_2}{\gamma} = \frac{P_a}{\gamma} + \left(\frac{V_3^2 - V_2^2}{2g}\right) - H + h_L \qquad (2.14a)$$

If throat and exit cross sectional areas are equal, $V_2 = V_3$, then

$$\frac{P_2}{\gamma} = \frac{P_a}{\gamma} - H + h_L \qquad (2.14b)$$

Equation (2.14b) shows that there is negative pressure at the throat section of a siphon spillway as $h_L < H$. However, due to the loss of head h_L, the negative pressure is less than H. If h_L were negligible, the average absolute pressure at the throat would fall to zero when the exit end of the siphon duct or the tailwater level were lower by $H = P_a/\gamma = 10.3$ m of water below the water surface in the reservoir, i.e., if H = 10.3 m, then $P_2/\gamma = (P_a/\gamma) - H = 10.3 - 10.3 = 0$. This is the theoretical limit of the value of H. The practical limit of H is governed by the limit up to which negative pressure may be allowed to avoid cavitation at the throat section. Cavitation may result when the pressure P_2 falls below the vapor pressure of water. Hence, limiting value of H is

$$H = \frac{P_a}{\gamma} - \frac{P_2}{\gamma} + h_L \qquad (2.15a)$$

If $P_2 = P_v$ (vapor pressure of water), we have

$$H = \frac{P_a}{\gamma} - \frac{P_v}{\gamma} + h_L \qquad (2.15b)$$

In order to avoid cavitation at the throat section,

$$H \le \left(\frac{P_a}{\gamma} - \frac{P_v}{\gamma}\right) + h_L \qquad (2.15c)$$

Similarly, from equation (2.14a) the limiting value of H is obtained as

$$H = \left(\frac{P_a}{\gamma} - \frac{P_v}{\gamma}\right) + \left(\frac{V_3^2 - V_2^2}{2g}\right) + h_L \qquad (2.16a)$$

In order to avoid cavitation at the throat section,

$$H \le \left(\frac{P_a}{\gamma} - \frac{P_v}{\gamma}\right) + \left(\frac{V_3^2 - V_2^2}{2g}\right) + h_L \qquad (2.16b)$$

Equation (2.16a) indicates that the limiting value of H is increased by losses and also by reducing the area at the exit end of the siphon duct from that of the throat section so that $V_3 > V_2$.

The pressure at any point in the flow at summit of the bend can be obtained as follows by writing energy equation between points (1) and (2).

Figure 2.14 Pressure distribution at siphon crest

$$\frac{P_a}{\gamma}+\frac{V_1^2}{2g}+h=\frac{P_2}{\gamma}+Z+\frac{V_2^2}{2g}+h'_L \Rightarrow \frac{P_2}{\gamma}=\frac{P_a}{\gamma}+h-Z-\frac{V_2^2}{2g}-h'_L \qquad (2.17a)$$

or in terms of absolute pressure

$$\frac{P_s}{\gamma}=\frac{P_a}{\gamma}+h-Z-\frac{V_s^2}{2g}-h'_L \qquad (2.17b)$$

In terms of gauge pressure, it will look like

$$\frac{P_s}{\gamma}=h-Z-\frac{V_s^2}{2g}-h'_L \qquad (2.17c)$$

2.5.5.2. Discharge Through Siphon Spillways

The discharge through siphon spillway is given by

$$Q=CA\sqrt{2gH} \qquad (2.18)$$

where C = coefficient of discharge ≈ 0.65

A = cross sectional area of throat = L x b

L = length of the spillway; b = height of the throat

However, another expression for discharge through siphon spillway may be obtained on the assumption that flow around the upper bend of the flow is similar to free vortex flow in which the water stream rotates around a central axis, and the velocities of the water streams vary inversely with the distance from the center of rotation. That is

$$V\ \alpha\ \frac{1}{r}\Rightarrow V\,r=\text{constant}=V_0r_0=V_ir_i \quad \text{or} \quad V=\frac{\text{constant}}{r}=\frac{k}{r}$$

The constant can be obtained by integration

$$dQ=VLdr=\frac{k}{r}Ldr=kL\frac{dr}{r}\text{, from which } Q=\int_{r_i}^{r_0}kL\frac{dr}{r}=kL\ln r=kL\ln(r_0/r_i)$$

Therefore,

$$k=\frac{Q}{L\ln(r_0/r_i)} \qquad (2.19)$$

Since, $k = V_i r_i = V_0 r_0$, we have $V_i=\frac{k}{r_i}=\frac{Q}{r_iL\ln(r_0/r_i)}$ and $V_0=\frac{Q}{r_0L\ln(r_0/r_i)}$

From the relation of equation (2.17), when Z = 0 and h is neglected (to be on safe side), we have $\frac{V_s^2}{2g}=\frac{-P_s}{\gamma}-h'_L$

$$V_i=\sqrt{2g\left(\frac{-P_s}{\gamma}-h'_L\right)}=\frac{Q}{r_iL\ln(r_0/r_i)}\Rightarrow Q_1=r_iL\ln(r_0/r_i)\sqrt{2g\left(\frac{-P_s}{\gamma}-h'_L\right)} \qquad (2.20)$$

In the same manner, using V_0, $Z = b$, $r = r_0$

$$V_0 = \sqrt{2g\left(\frac{-P_s}{\gamma} - b - h'_L\right)} = \frac{Q}{r_0 L \ln(r_0/r_i)} \Rightarrow Q_2 = r_0 L \ln(r_0/r_i)\sqrt{2g\left(\frac{-P_s}{\gamma} - b - h'_L\right)} \qquad (2.21)$$

where $b = Z = r_0 - r_i$

The capacity of the siphon is the minimum of Q_1 and Q_2. The head loss in the siphon is due to inlet, the bend, outlet and friction. Loss coefficients for typical rectangular siphon of constant cross section are:

Entrance loss: $h_e = 0.2\dfrac{\overline{V}^2}{2g}$ (2.22a)

Friction loss: $h_f = 0.25\dfrac{\overline{V}^2}{2g}$ (2.22b)

Bend losses, where centerline radius is ≥ 2.5 b: $h_b = 0.42\dfrac{\overline{V}^2}{2g}$ (2.22c)

Transition losses: Diverging outlet, $h_{ex} = 0.2\dfrac{\overline{V}^2 - \overline{V}_0^2}{2g}$ (2.22d)

Converging outlet, $h_{ex} = 0.1\dfrac{\overline{V}^2 - \overline{V}_0^2}{2g}$ (2.22e)

Exit losses: $h_v = \dfrac{V^2}{2g}$ (2.22f)

2.5.6. Shaft (or Morning Glory) Spillway

Shaft spillway is one in which the water enters over a horizontally positioned lip, drops through a vertical shaft, and then flows to the downstream river channel through a horizontal or near horizontal conduit or tunnel. The structure is made up of three elements; namely an ***overflow control weir, a vertical transition, and a closed discharge channel***.

This type of spillway is suitable for narrow gorges where other types of spillways do not find adequate space. In earthen dams where even a side channel or chute spillway is unsuitable due to space constraints, a shaft spillway may be excavated through the foundation or flanks of the river valley.

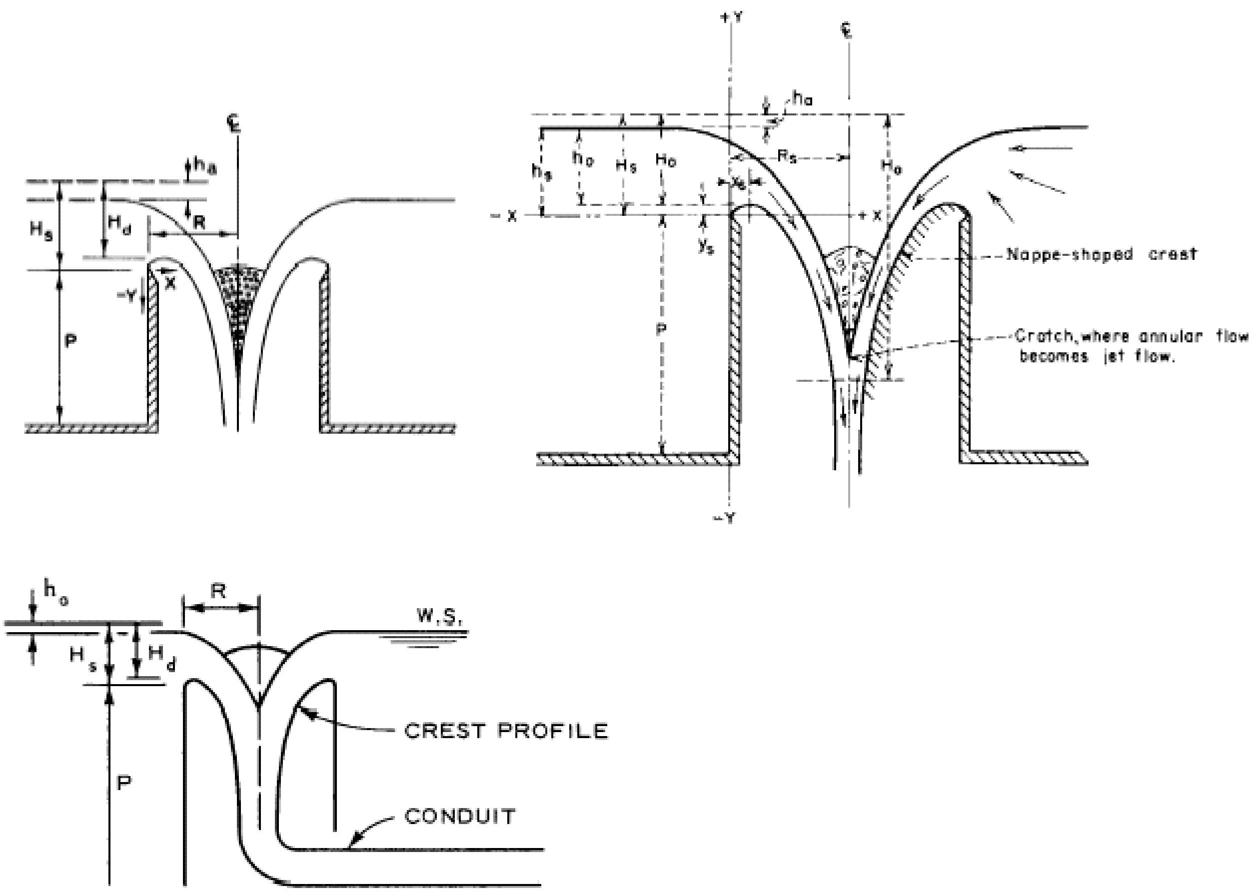

Figure 2.15a Morning Glory Spillway

2.5.6.1. Flow Characteristics of Shaft Spillways

The discharge control may be at one of the three points depending on the head. From the discharge curve (Fig. 2.15b), it can be seen that crest control (condition 1) will prevail for heads between the ordinates of a and g; orifice or tube control (condition 2) will govern for heads between the ordinates of g and h, and the spillway conduit will flow full for heads above the ordinate of h (condition 3).

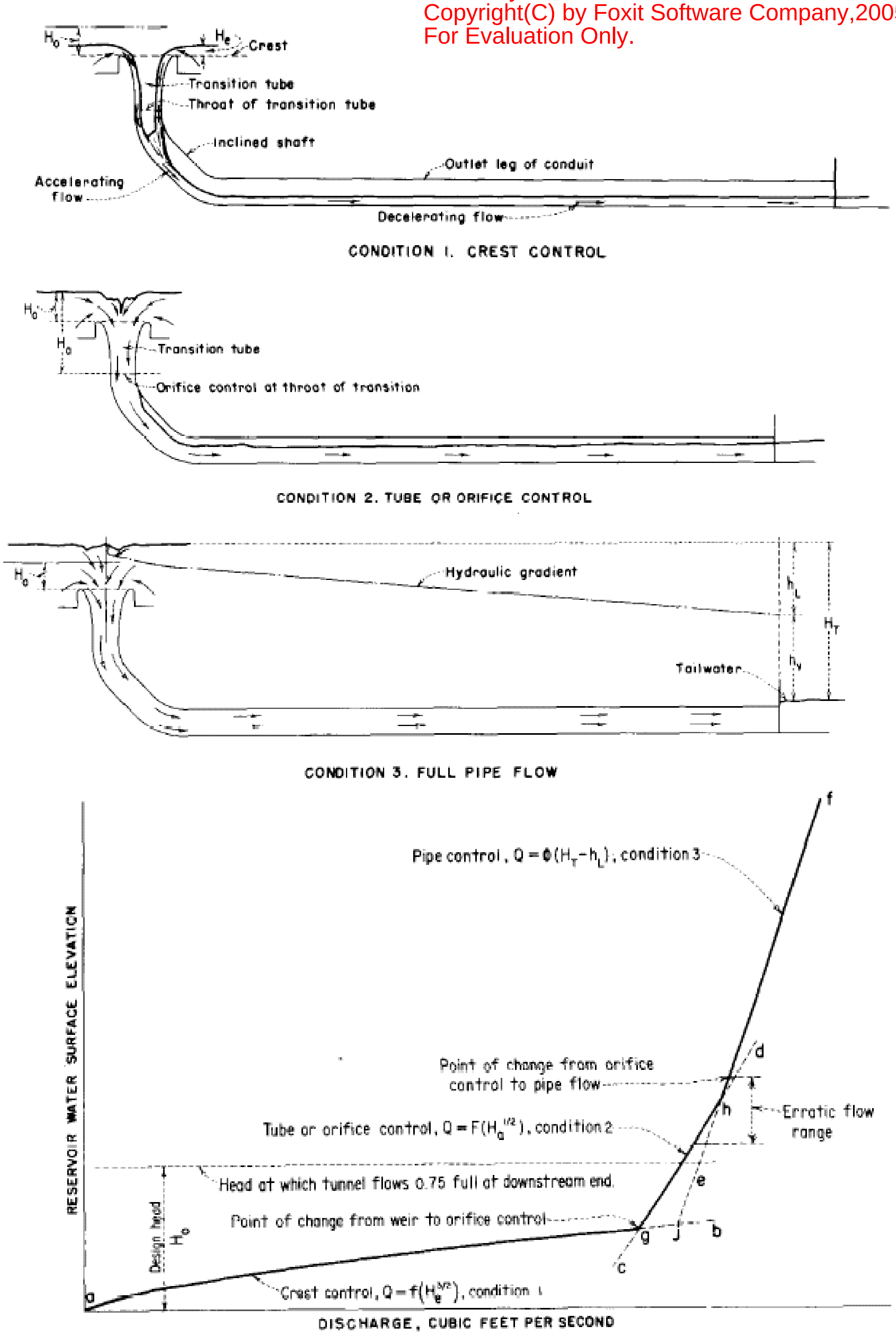

Figure 2.15 b Flow characteristics of shaft (morning glory) spillway

Condition 1 - Crest discharge: For small heads, the flow is governed by the characteristics of the crest discharge. The vertical transition beyond the crest will flow partly full and the flow will cling to the sides of the shaft. The outlet shaft will flow only partially full and is therefore, in effect, an open channel. As the discharge over the crest increases, the overflowing annular nappe becomes thicker, and eventually the nappe flow will converge into a solid vertical jet. The point where the annular nappe joins the solid jet is called the crotch.

If the crest profile and transition conform to the shape of the lower nappe of a jet flowing over a sharp-crested circular weir, the discharge characteristics for flow over the crest can be expressed as

$$Q = CLH^{3/2} \tag{2.23}$$

where C = circular crest discharge coefficient (0.6 – 2.2)

L = arc length of the crest,

H = head measured either to the apex of the under nappe of the overflow, to the spring point of the circular sharp-crested weir, or to some other established point on the overflow.

The choice of L is related to some specific point of measurement such as the length of the circle at the apex, along the periphery at the upstream face of the crest, or along some other chosen reference line. The value of C will change with different definitions of L and H. The coefficient of discharge for a circular crest differs from that for a straight crest because of the effects of submergence and backpressure incident to the joining of converging flows.

When the crest outline and transition shape conform to the profile of the nappe shape for a head H_0 over the crest, free flow prevails for H_0/R_s ratio up to approximately 0.45, and weir control governs. As H_0/R_s ratio increases above 0.45, the weir partly submerges and the flow showing characteristics of a submerged weir is the controlling condition.

b) Condition 2 – Orifice Control: As the head increases, so the annular nappe must increase in thickness. Eventually, the nappe expands to fill the section at the entry to the drop shaft. The discharge is now being controlled from this section, and this is referred to as 'Orifice Control'. When H_0/R_s ratio approaches 1.0, the water surface above the weir is completely submerged. For this and higher stages of H_0/R_s, the flow phenomenon is that of orifice flow. The outlet tunnel is not designed to run full at this discharge.

c) Condition 3 – Full Pipe Flow: Further increase in head will induce blackwater flow throughout the drop and outlet shafts. The Q – H relationship must now conform to that for full pipe flow, the weir will in effect be submerged. The head over the weir rises rapidly for a given increase in discharge, with a consequent danger of overtopping the dam.

The capacity of shaft spillway should be quite adequate to insure free gravity flow in the shaft; otherwise, if the water level in the shaft rises near the crest, the efficiency of the spillway is considerably reduced. Therefore, the design head is usually less than the head required for blackwater flow. This is done to leave a margin of safety for exceptional floods. Even so, the discharge that can be passed by a shaft spillway is limited, so great care must be exercised at the design stage. An auxiliary spillway may be necessary. It is also worth noting that as the flow enters the transition it tends to form a spiral vortex. The vortex pattern must be minimized in order to maintain a smoothly converging flow, so anti-vortex baffles or piers are often positioned around the crest.

It may be undesirable for sub-atmospheric pressure to occur anywhere in the system, since this can lead to cavitation problems. To avoid such problems the system may

a) incorporate vents,
b) be designed with an outlet shaft which is large enough to ensure that the outlet end never flows full, or
c) have an outlet shaft with a slight negative slope, sufficient to ensure that the outlet does not flow full (and can therefore admit air).

2.5.1. Conduit or Tunnel Spillway

It is one in which a closed channel is used to convey the discharge around or under a dam. The closed channel may be in the form of a vertical or inclined shaft and horizontal tunnel or conduit. The control structure for this spillway may be in the form of an overflow crest, vertical or inclined orifice entrance and side channel crest. The conduit or tunnel is designed to flow partly full and it is not allowed to flow full as siphonic action may develop due to negative pressure being developed in the conduit. In order to ensure free flow in the conduit ratio of flow area to total area of the conduit should be limited to 75%. Air vents are also provided at critical points along the conduit to ensure an adequate air supply which will avoid unsteady flow through the spillway.

2.6. Cavitation and Aeration in Spillways

Cavitation occurs whenever the pressure in the flow of water drops to the value of the pressure of the saturated water vapor, P_v (at the prevailing temperature), cavities filled by vapor, and partly by gases excluded from the water as a result of the low pressure, are formed.

Low pressures – well below atmospheric pressure – will occur at points of separation of water flowing alongside fixed boundaries, particularly if the flow velocity is high. Thus, there are two factors, pressure, P, and velocity, V, which influence the onset of cavitation. They are combined with density of water ρ in the cavitation number, σ_c, which is a form of the Euler number.

$$\sigma_c = \frac{2(P - P_v)}{\rho V^2} \quad \text{or} \quad \sigma_c = \frac{H - H_v}{V^2/2g} \tag{2.24}$$

Cavitation occurs if the cavitation number falls below a critical value σ_c, which is a function of the geometry.

In spillway design we should be very wary of cavitation problems at velocities exceeding 35 m/s (Novak, 1996), even if the spillway surface is 'smooth' and well constructed. The value of P_v is a function of atmospheric pressure and temperature (P_v = 10 m H_2O = P_0 for 100°C, P_v = 6.5 m for 90°C and 0.5 m for 30°C). Although it is generally assumed that the onset of cavitation occurs when $P \approx Pv$ (≈ 0, for normal water temperature, i.e., at 10 m below P_0/γ), the presence of dissolved gases and/or particles in suspension can cause cavitation at higher pressures; thus it is advisable to avoid pressures below about 7 m vacuum (3 m absolute) in hydraulic engineering design. It has to be stressed, however, that in turbulent flows the mean pressure may be well above the danger limit but cavitation can still occur owing to the fluctuating instantaneous pressures that fall below the limit (e.g. pressure fluctuations in hydraulic jump).

2.7.1. Cavitation Risks

In hydraulic structures under high velocity flows, pressure reduction is mostly related to changes in local velocity caused by boundary irregularities. As the local velocity increases, the pressure decreases proportionally, and it may reach a critical value at which the vapor and/or vapor and gas nuclei become unstable. As the bubbles or cavities move into higher pressure zone, they collapse – resulting in sudden intense pressure shock waves – which are responsible for the noise and associated damage. The damage to hydraulic structures is mostly associated with separate cavity flows in which a preferential zone of collapse results at the boundary. In such cases, the boundary layer is separated prior to the onset of cavitation.

Figure 2.16
The collapse of individual vapor bubbles dragged by the water is concentrated in the higher pressure zone around the stagnation point downstream from the large vapor cavity. Damage will be more likely in that region.

Ball has classified the main types of irregularities to be expected in concrete surfaces, indicating the corresponding damage zones as shown in Figure 2.15.
Figure 2.17

Offsets into the flow are more critical, than offsets away from the flow as sources of cavitation damage. Beveling of the irregularities has an important effect on reducing the risk of cavitation.

2.7.2. Self-aeration

It is the most important feature of supercritical flow. Although beneficial for cavitation protection and energy dissipation, it requires increase of the chute sidewalls due to the bulking (an increase in depth of flow) of the flow.

Air can be supplied to spillway surface automatically (self-) or artificially. Figure 2.14 shows the transition from critical flow at crest, through supercritical non-uniform non-aerated flow to no-uniform partially and fully aerated flow to, finally, uniform aerated flow.

Location of point of inception is the length required for growth of boundary layer.

$$L_i = 14.7\, q^{0.53} \approx 15\sqrt{q} \qquad (2.25)$$

Water levels in non-uniform non-aerated flow can be determined by standard non-uniform flow calculations.

Note: *Boundary layer thickness δ is the magnitude of normal distance from the boundary surface at which the velocity V_1 is equal to 99% of the limiting velocity, V_0, which the* *velocity distribution curve in the boundary layer approaches asymptotically. The effect of boundary layer on the flow is equivalent to a fictitious upward displacement of the channel bottom to a virtual position equivalent to the so-called displacement thickness, δ*. (See also figure)*

Some protection against cavitation starts at beginning of fully aerated flow region. In Figure 2.14, assuming $\theta_1 = \theta_2$, interface 2-3 is at 2 L_i for preliminary design. However, amount may not be sufficient to protect cavitation (about 7% in contact is needed, for which 30 – 35% concentration of air is required).

The depth of the uniform aerated flow, y_a, is estimated as follows:
Average air concentration,

$$C = \frac{Q_a}{(Q_a + Q)} \quad (2.26a)$$

Where Q_a = air discharge; Q = water discharge
Ratio of water discharge Q to the total discharge of the mixture of air and water is given by

$$\rho_1 = \frac{Q}{(Q_a + Q)} \quad (2.26b)$$

and ratio of air to water discharge, is given by

$$\beta = \frac{Q_a}{Q} \quad (2.26c)$$

Therefore, $C = 1 - \rho_1 = \dfrac{\beta}{\beta + 1}$

For a rectangular chute,

$$\rho_1 = \frac{Q}{Q + Q_a} = \frac{y_0}{y_a} < 1 \qquad \text{(Assuming } V_a = V_{water}\text{)} \quad (2.26d)$$

where y_0 = depth of non-aerated (uniform) flow, and y_a = depth of uniform aerated flow.

For a quick assessment of y_a and C, the following equation may be used:

$$y_a = C_1 y_c \cong C_1 \sqrt[3]{q^2/g}\,; \quad \text{for} \quad 0.32 < C_1 < 0.37 \quad (2.26e)$$

$C_1 \approx 0.35$ for concrete, and q = discharge per unit width

$$\frac{y_a - y_0}{y_0} = 0.1\sqrt{0.2F_r^2 - 1} \quad (2.26f)$$

It may be appreciated that uniform aerated flow is reached only at very considerable distance from the spillway crest, and in many chute spillways may never be reached at all, particularly if the specific discharge q is large (> 50 m²/s).

2.7.3. Artificial Aerators

In order to provide cavitation protection in cases where there is no air in contact with the spillway, or the air concentration is insufficient and the velocities are high enough to make cavitation damage, a real possibility is artificial aerators.

These aerators have the form of deflectors (ramps), offsets, or grooves, or a combination of two or all three of them. Air is supplied to the spillway surface automatically through air ducts as the flow separation causes the pressure downstream of the aerator to drop below atmospheric.

As air bubbles will not stay in contact with the spillway but rise to the water surface, a series of aerators is necessary to achieve the minimum required air concentration at the spillway surface. The higher the flow velocity, the larger will be the distance between the aerators; roughly, the distance (m) should be 1 – 2 times the mean flow velocity (m/s).

Pinto (1991) recommends the following formula for the computation of air discharge through an aerator.

$$\beta = q_a / q = 0.29 (F_r - 1)^{0.62} (D/y)^{0.59} \tag{2.27}$$

F_r is the Froude number just upstream of the aerator, and $D = CA/b$; A = aerator control orifice area; C = discharge coefficient; and b = chute width

The total airflow for an average negative pressure under the nappe at the aerator ΔP, is then

$$Q_a = q_a b = CA(2\Delta P / P_a)^{1/2} \tag{2.28}$$