Design Principles that are involved in the Design of Flow over an Ogee Crest Spillway

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Abstract: ***The ogee-crested spillway's ability to pass flows efficiently and safely, when properly designed and constructed, with relatively good flow measuring capabilities, has enabled engineers to use it in a wide variety of situations as a water discharge structure (USACE, 1988; USBR, 1973). The ogee-crested spillway's performance attributes are due to its shape being derived from the lower surface of an aerated nappe flowing over a sharp-crested weir. The ogee shape results in near-atmospheric pressure over the crest section for a design head. At heads lower than the design head, the discharge is less because of crest resistance. At higher heads, the discharge is greater than an aerated sharp-crested weir because the negative crest pressure suctions more flow. The spillway is among the most important structures of a dam project. It provides the project with the ability to release excess or flood water in a controlled or uncontrolled manner to ensure the safety of the project. It is of paramount importance for the spillway facilities to be designed with sufficient capacity to avoid overtopping of the dam, especially when an earth fill or rockfill type of dam is selected for the project. In cases where safety of the inhabitants downstream is a key consideration during development of the project, the spillway should be designed to accommodate the probable maximum flood. Many types of spillways can be considered with respect to cost, topographic conditions, dam height, foundation geology, and hydrology. In this study, the ogee-crested spillway's or overflow spillways discussed in this paper. A section on design of spillways that considers cavitation and aeration.***

Keywords: ogeecrested spillway, dam height, design head, discharge, spillway crest profile

1. Introduction

Spillway is a passage in a dam through which the design flood could be disposed off safely to the downstream. The ogee-crested spillway, because of its superb hydraulic characteristics, has been one of the most studied hydraulic structures. Its ability to passflows efficiently and safely, when properly designed, with relatively good flow measuring capabilities, has enabled engineers to use it in a wide variety of situations. Although much is understood about the general ogee shape and its flow characteristics, it is also understood that a deviation from the standard design parameters such as a change inupstream flow conditions, slightly modified crest shape, or construction variances canchange the flow properties. These small changes often require engineers to evaluatethe crest and determine whether or not the change or deviation will be detrimental to the spillway's performance. Such is the case when an updated probable maximum flood calculation requires a spillway to pass a larger flow than it was designed to handle. In general, spillways comprise five distinct components namely: (i) an entrance channel,(ii) a control structure, (iii) a discharge carrier, (iv) an energy dissipator, and (v) an outlet channel. The entrance channel transfers water from the reservoir to the control structure, which regulates the discharge from the reservoir. Water is then conveyed from reservoir to the low-level energy dissipator on the riverbed by the discharge conveyor. An energy dissipator is required to reduce the high velocity of the flow to an on scouring magnitude. Most common types of spillway-control system used are roller, tainter, vertical-lift, and drum gates. In view of the varying conditions, the choice of suitable gate is bound by thecost, the head on the crest, the height of dam, and the hydraulic behaviour of the gate.Piers are located on the spillway crest for the purpose of supporting the control gates,the gate-operating mechanisms or a roadway. Their size and shape will vary accordingly with their function. The piers should be streamlined both in the upstreamand the downstream sides to reduce contraction of the overflowing jet and to provide asmooth water surface.The element which introduces the energy-reducing action is generally known as "stilling basin." One of the most common methods out of several methods are dissipating theflow at the toe of a spillway, is the hydraulic jump. Other types used in conjunction with spillways are roller and trajectory buckets. Spillway outlets means the combination ofstructures and equipment required for the safe operation and control of the waterreleased for different purposes for which the dam is planned. These structures may beriver outlets, penstocks, canal outlets. The size and number of river outlets satisfy the discharge requirements at various stages of the reservoir. If the outlets are located inthe overflow portion, the conduits should be aligned downwards to minimise disturbanceto the flow over spillway. The discharge from an outlet, (gates, valves, or free-flow conduits) has a relatively high velocity. Flow must expend the energy in order to preventscour of the bed and banks of the river channel. This may be accomplished byconstructing a stilling basin immediately downstream from the outlet.The crest of the spillway is usually provided at F.R.L (Full Reservoir Level). However, inorder to control floods, the gates could be provided at the top and the water level could be increased up to maximum water level. The height between F.R.L and M.W.L is calledthe "Flood lift". Reservoir level should not cross MWL. Following are different types ofspillways usually adopted in practice.

1) Overflow spillway.
2) Side channel spillway.
3) Shaft spillway.
4) Siphon spillway.
5) Chute.
6) Breaching section (emerging spillway).

Major dam will be usually provided with an overflow spillway with crest gates. However,the type and location of spillway depends on the site conditions of topography.

1.1 Necessity of Spillways

The height of the dam is always fixed according to the maximum reservoir capacity. The normal pool level indicates the maximum capacity of the reservoir. The water is never stored in the reservoir above this level. The dam may fail by overturning so, for the safety of the dam the spillways are essential.The top of the dam is generally utilized by making road. The surplus water is not be allowed to over top the dam, so to stop the over topping by the surplus water, the spillways become extremely essential.To protect the downstream base and floor of the dam from the effect of scouring and erosion, the spillways are provided so that the excess water flows smoothly.

1.2 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. 1), is available, the spillway can be best built independently of the dam.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 (such as shown in Fig. 2), while the flank or flanks are closed by earthen dikes or embankments.

Fig 1. Photo view of location of spillway of Polavaram dam under construction across river Godavari in Andhra Pradesh in India

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 dueto failure of gates of main spillway, etc., the water rises abovethe 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 darn, or at one end of it, or independently in a saddle away from the main dam.A separate independent spillway is generally preferred for earth dams, although due to non-availability of sites, a concrete spillway is sometimes constructed within or atone of the ends of anearth dam. If the main spillway issituated in a flank, a secondary emergency spillway may be situated in another flank.

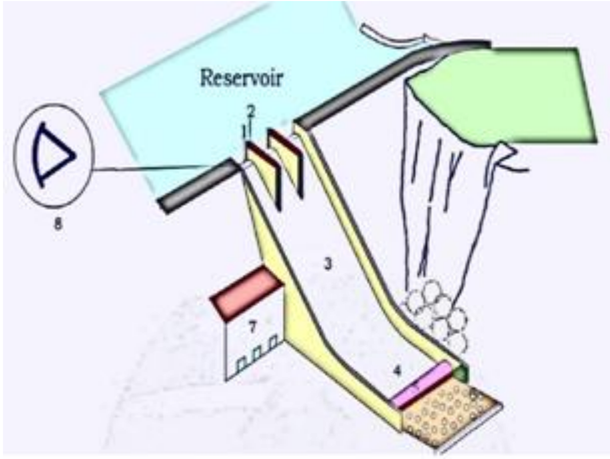

Fig 2. Spillway layout

Schematic representation of typical spillway
1. Spillway crest / bay
2. Pier
3. Spillway face
4. Energy dissipator - stilling basin
5. End sill
6. Armoured scour preventing bed
7. Power house
8. Sector gate

Major dam will be usually provided with an overflow spillway with crest gates. However,the type and location of spillway depends on the site conditions of topography. Ordinarily, the excess flow is drawn from the top of the pool created by the damand conveyed through an artificial waterway i.e. spillway, back into the same river orto some other drainage Channel. Fig. 3 show ogee type spillways in Amaravati Dam.The Amaravati river is one of the longest rivers (282 km) in Tamil Nadu, which join with the river Cauvery, near Karur.

The Amaravati River is the longest tributary of Kaveri River in fertile the districts of Karur and Tirupur, Tamil Nadu state, South India.The 282km (175 miles) long Amaravati River begins at the Kerala/Tamil Nadu border at the bottom of Manjampatti Valley between the Annamalai Hills and the Palni Hills in Indira Gandhi Wildlife Sanctuary and National Park in Tirupur district. It descends in a northerly direction through Amaravati Reservoir and Amaravati Dam at Amaravathi nagar. It is joined by the Kallapuram River at the mouth of the Ajanda valley in Udumalaipettai. Through Dharapuram and Aravakurichi it joins with the Kaveri at Thirumukkudal, about 10 kilometres (6 miles) from Karur.Nanganji, Kudavanar, Shanmuganadhi, Uppar, Kudumiar, Thenar and so many tributaries are joint with the Amravati river. It has the Tributary of the Pambar and Chinnar rivers from Kerala also.This river irrigates over 60,000 acres (240 Sq.km) of agricultural lands in Tiruppur and Karur districts. The Amaravati Dam has 4 megawatts of electricity generating capacity installed. The Amaravati River and its basin, especially in the vicinity of Karur, are heavily used for industrial processing water and waste disposal and as a result are severely polluted due to large amount of textile dyeing and bleaching units. But nowadays in karur, the changes are vicinity by seeing Amaravati river on its clean surface because of pollution controlled by government.

Fig 3. Photo view of an ogee spillway of Amaravati dam constructed across the River Amaravati in Tirupur district in the Indian state of Tamil Nadu

1.3 Design Considerations for the Main Spillway

The main spillway, often called the spillway, is properly designed so as to dispose of theexcess waterwithout causing anydamage 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, be 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 protectthe 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.

1.4 Classification of Spillways

1) According to the most prominent feature

a) Ogee spillway
b) Chute spillway
c) Side channel spillway
d) Shaft spillway
e) Siphon spillway
f) Straight drop or overfallspillway
g) Tunnel spillway/Culvertspillway

2) According to Function

a) Service spillway
b) Auxiliary spillway
c) Fuse plug or emergencyspillway

3) According to Control Structure

a) Gated spillway
b) Ungated spillway
c) Orifice of sluice spillway

1.5 Spillway Design

(Ogee or Overflow Spillways)

The following aspects are involved in the design of spillways:

1) Hydrology

A. Estimation of inflow design flood
B. Selection of spillway design flood
C. Determination of spillway outflow discharge
D. Determination of frequency of spillway use

2) Topography and geology

A. Type and location of spillway

3) Utility and operational aspects

A. Serviceability

4) Constructional and structural aspects

A. Cost-effectiveness

1.6 Spillway Design Flood

1) Probable Maximum Flood (PMF)

This is the flood that may be expected from the most severe combination of critical meteorological and hydrological conditions thatare reasonably possible in the region. This is computed by using theProbable Maximum Storm.

2) Standard Project Flood (SPF)

This is the flood that may be expected from the most severe combination of hydrological and meteorological factors that areconsidered reasonably characteristic of the region and is computed byusing the Standard Project Storm (SPS).In US, generally, large dams are designed for PMF, intermediate forSPF/PMF, and small dams for floods of return period of 100 years toSPF.The estimation of spillway design flood or the inflow design flood is anexercise involving diverse disciplines of hydrology, meteorology,statistics and probability.There is a great variety of methods used around the world to determineexceptional floods and their characteristics. ICOLD (1992) groups allthese methods under the two main categories:

a) Methods based mainly on flow data.
b) Methods based mainly on rainfall data.

The ogee or overflow spillway is the most common type of spillway. It has a control weir that is Ogee or S-shaped. It is a gravity structure requiring sound foundation and is preferably located in the main riverchannel.

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1.7 Spillway Components

Spillways generally are made up of four components: a controlstructure, discharge channel, terminal structure, andentrance/outlet channels. control structures regulate the flows from the reservoir into thespillway, ensuring that flow will not enter the spillway until thewater in the reservoir reaches the designed level, and moderatingflow into the spillway once the design level has been reached.Control structures can be sills, weirs, orifices, or pipes.

Discharge channels, also known as waterways, convey flow thatpasses through the control structure down to the streambed below the dam. Note that conveyancestructures are not always presentin a spillway design; at times discharge may fall freely afterpassing through the control structure.

Terminal structures ensure that the flow, which oftentimes acquires a high velocity whiletraveling down a spillway, will not cause excessive erosion to the toe of the dam, or any other nearby structures. Plunge basins, flip buckets, and deflectors are all examples of terminalstructures.

Entrance channels convey water from a reservoir to the control structure. Outlet channels conveyflow that has reached the terminal structure to the river channel that resides below the dam. Entrance and outlet channels are not necessarily a component of all spillways; it is possible forthe spillway to transport flow directly from the reservoir to the river channel.

1.8 Ogee Spillway Crest Profile

This type of spillway is the most common type adopted in the field. It divides naturallyinto three zones. Crest, spillway face and the toe. The concept evolves from replacingthe lower nappe of the flow over thin plate weir by solid boundary.

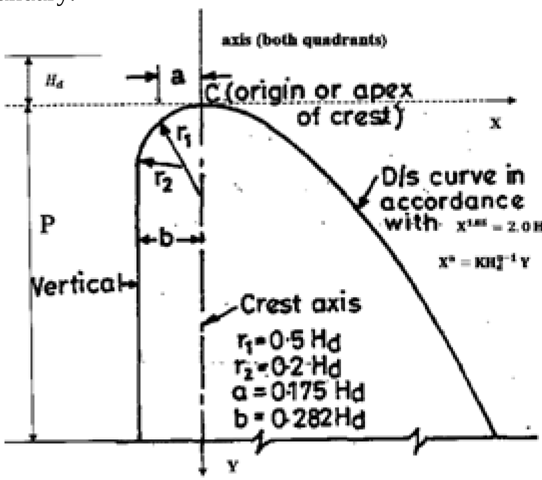

Fig 4. WES profile for Ogee Spillway for vertical u/s slope

The ogee spillways were being designed in the earlier periods, in accordance with the theoretical profile obtained for the lower nape of, a free-falling jet. The profile was known, as Bazin's profilhe. Theoretically, the adoption of such a profile, "should cause no negative pressures on thecrest 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 seemsinevitable. The presence of negative pressure causes the danger of cavitation and sometimesfluctuations and pulsations of the nappe. Hence, while adopting a profile for thespillway crest, the avoidance of negative pressures must be an objective along withconsideration of other factors such as practicability, hydraulic efficiency, stability andeconomy. Depending upon research work based on these objectives, various modified·profiles have been proposed these days.

Crest shapes have been studied extensively in the USBR hydraulic laboratorieswith various approach depths. The upper and lower nappe surfaces werecarefully measured for various discharges and velocities of approach. On the basisof experimental data including Bazin's, the Bureau has developed coordinatesof nappe surfaces for vertical and upstream sloped weirs. The results are welldocumented in the USBR publication 'Design of Small Dams' (1960). The profilesare defined as they relate to the coordinate axes at the apex of the crest.The portion upstream of the origin is defined as a compound circular arc. Theportion downstream is defined by the equation

$$\left(\frac{Y}{H_d}\right) = -K\left(\frac{X}{H_d}\right)^n \qquad (1)$$

On the basis of the USBR data, the US Army Corps of Engineers, Waterways Experimental Stationin Vicksburg, (WES) 1952 has developed several standard shapes, designated as WES standard spillwayshapes, represented on the downstream of the crest axis by the equation

$$X^n = K H_d^{n-1} Y \qquad (2)$$

Where
X and Y are coordinates of crest profile with origin at the highest point C of the crest, called the apex.

H_d design head including velocity head of the approach flow.K and n are parameters depending on the slope of the upstream face.The values ofK and n are tabulated in Table 1.0.

Thus, for a spillway having a verticalu/s face, the D/S crest is given bythe equation

$$X^{1.85} = 2.0 H_d^{0.85} Y \qquad (3)$$

According to the latest studies of the U.S. Army Corps of Engineersat their' Waterways Experimental Station (WES),the U/s curve of theogee spillway having a vertical U/s face, should have the following equation:

$$Y = \frac{0.724(X + 0.27H_d)^{1.85}}{H_d^{0.85}} + 0.126H_d - 0.4315H_d^{0.375}(X + 0.27H_d)^{0.625} \qquad (4)$$

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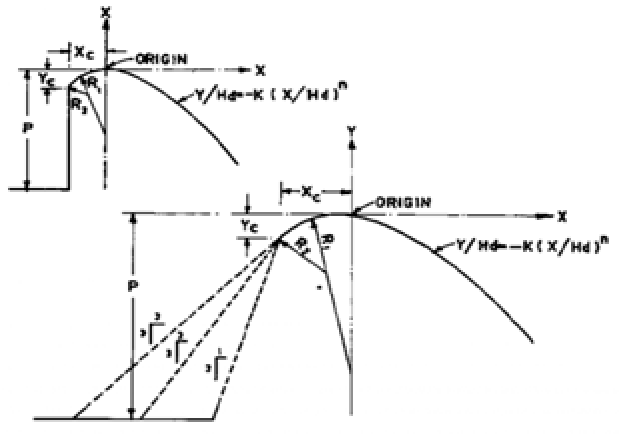

Fig 5. Typical USBR crest profile

The u/s profile extends up to

$$X = -0.27H_d$$

Co-ordinates for the upper nappefor various WES shapes of ogeespillway are also available and can beutilised in the design of training wallsand spillway bridge etc.

Table 1.0 Typical values

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

1.9 Spillway Discharge Characteristics

Similar to the crest profile, the discharge characteristics of the standard spillway can also be derived from the characteristics of the sharp crested weir. The general equation for discharge is given by

$$Q = \frac{2}{3} C\sqrt{2g}L_e H_e^{3/2} \qquad (5)$$

(As per clause 4.2.1 of IS 6934: 1998).
Where
Q = total discharge (m³/sec);
L = Effective length of the spillway crest (m);
H_e = total head upstream from the crest;
g = gravitational constant (m/sec); and
C = constant.

The non-dimensional coefficient of dischargehas a theoretical minimum value of $\frac{\pi}{(\pi+2)} = 0.611$and a practical upper limit of about 0.75. Theparameter $\frac{2}{3}\ C\ \sqrt{2g}$ is often called C_d, which,however, is a dimensional quantity. The value ofC_d, generally varies from 1.80 to 2.21 (SI units). As per clause 4.2.2 of IS 6934: 1998.

2. Ogee spillway discharge equation

Ogee spillways are spillways that generally have an S-shaped profile. Thisprofile is advantageous because it can approximate the shape of the jet that travels along itssurface, consequently making it easier to obtain ideal discharges.The discharge passing overthe ogee spillway is given by the equation:

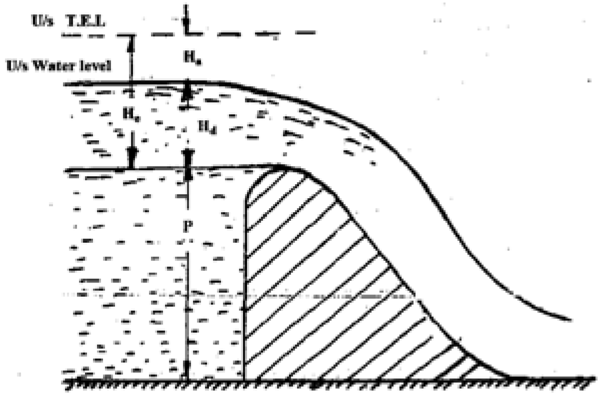

Fig 6. Profile of ogee spillway

$$Q = C_d\ L_e H_e^{3/2} \qquad (6)$$

The value of the coefficient of dischargeC_d dependson the following:
a) shape of the crest,
b) depth of overflow in relation to design head,
c) depth of approach,
d) extent of submergence due to tail water, and
e) inclination of the upstream face.
(As per clause 4.2.3 of IS 6934: 1998)
H_e = Total head over the crest including thevelocity head.
If the discharge Q is used as the design discharge in Equation (5), then the term $H_e$will be the corresponding design head
(H_d) plus, the velocity head (Ha). In such a case,$H_e = H_d + H_a$. For high ogee spillways, the velocity head is very small, and $H_e \cong H_d$

It may be noted that H_e, the total head, includes the velocity head. Generally, this requires aniterative solution technique as the velocity head is unknown, as it depends on flow rate which is to be calculated. However, as the velocity head is generally small, theequation converges quickly.

The coefficient of discharge depends upon the height of theogee weir (h) to the design headover the weir (H_d). If the height of the weir is more than 1.33times the design head, the velocity of approach has beenfound to have a negligible effect upon discharge, and as such $H_d$becomes equal to $H_e$or $\frac{H_e}{H_d} = 0.10$. In such a case, thecoefficient of'discharge, say$C = C_d$, has been found to be 2.2in M.K.S. or S.I. Units.

2.1 Ogee or Overflow Spillways

Murphy (1973), of WES, observed that there was no simple universalprocedure for design of overflow spillway crests and that the designers followedone set of rules if the approach depth was deep and another if the approachdepth was shallow. Also, there were different sets of rules depending on theinclination of the upstream face of the spillway. Murphy felt the need ofeliminating the discontinuity at the intersection of the spillway crest and theupstream face of the dam. In the revised procedure developed by Murphy,using the same basic data of USBR, the upstream quadrant was shaped asan ellipse with the equation

$$\frac{X^2}{A^2} + \frac{Y^2}{B^2} = 1 \qquad (7)$$

where
A = Semi-major axis (functions of the ratio of approach depth to design head)
B = Semi-minor axis (functions of the ratio of approach depth to design head)and the downstream profile conformed to the equation.

$$X^{1.85} = K\ H_d^{0.85}\ Y \quad \textit{For vertical U/s face} \quad (8)$$

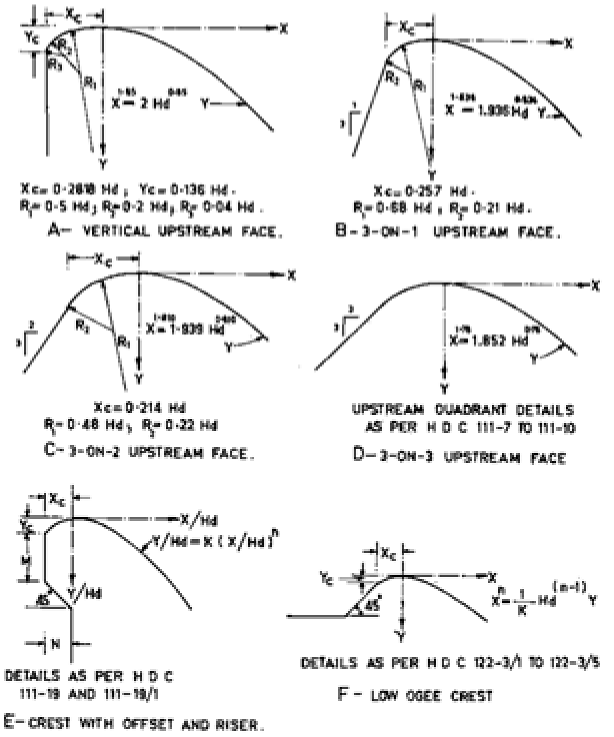

Fig 7. Typical WES crest profiles

Where
K is a parameter depending on the ratio approach depth and design head.

The design curves suggested by Murphy are reproduced in Figure 7.It would be seen that these curves cover both the governing parametersnamely design head and approach velocity head while also facilitating any desiredupstream face slope (including vertical) without introducing discontinuity at theintersection of the crest with upstream slope.For $\frac{P}{H_d} \geq 2$,

A and B become constant with values of 0.28 H_d and 0.164 H_d respectively.($\frac{P}{H_d} = 0.5$) and a high overflow spillway ($\frac{P}{H_d} = 4$) for the same depth ofoverflow of 10 m and with appropriate heads due to the velocity of approach.

Although all three procedures for defining crest shapes described earlierare based on the same set of data (viz. USBR), there is some dissimilarity. Whilethe profiles as per the USBR and WES (original) procedures show insignificant differences, the WES (Murphy) profile appears to exhibit a somewhat largerupstream quadrant than that given by the other two profiles, especially for slopingupstream faces. This is mainly because of the elaborate transition from an elliptical profile to a sloping upstream face.

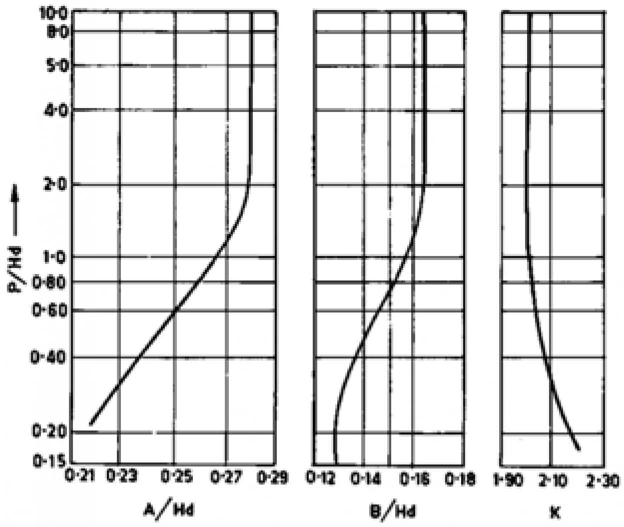

Fig 8. Coordinate coefficients for spillway crest (USACE,1986)

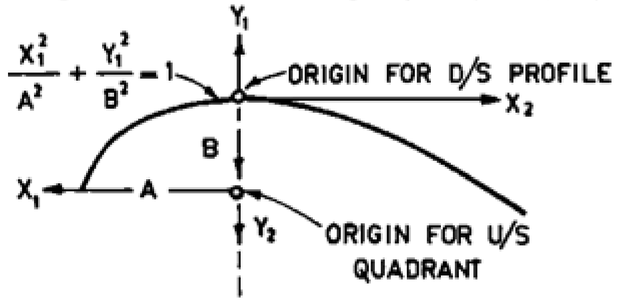

Fig 9. Typical USBR crest profiles

2.2The effective length (L_e) of Ogee spillway crest

Crest piers and abutments cause contraction of the flow, reduction inthe effective length of the crest, and cause reduction in the dischargeas compared to that of an otherwise uncontrolled crest. The followingrelationship applies:

$$L_e = L - 2(NK_p + K_a)\,H_e \qquad (9)$$

where
L_e = Effective length of crest for calculating discharge
L = Net clear length of the spillway crest
N = Number of piers
K_p = Pier contraction coefficient
K_a = Abutment contraction coefficient
H_e = Total design head on the crest including velocity head

The values of K_p andK_a depend mainly upon the shape of the piersand that of the abutments. As per clause 4.3.1of IS 6934: 1998.

The pier contraction coefficient, K_p is affectedby the shape and location of the pier nose &, thicknessof the pier, the head in relation to the design headand the approach velocity. Average pier contractioncoefficients may be taken as follows:(As per clause 4.3.2of IS 6934: 1998).

Table 1.1 Pier contraction coefficients

Sl No	Type	K_p
1	For square-nosed piers with rounded corners on a radius of about 0.1 times the pier thickness	0.02
2	For round-nosed piers	0.01
3	For pointed-nosed piers	0

The abutment contraction coefficient is affectedby the shape of the abutment, the angle between theupstream approach wall and the axis of flow, the headin relation to design head and the approach velocity. Average abutment contraction coefficient may betaken as follows: (As per clause 4.3.3of IS 6934: 1998).

Table 1.2 Abutment contraction coefficients

Sl No	Type	K_a
1	For square abutments with head wall at 90' to direction of flow	0.20
2	For rounded abutments with head wall at 90° to direction of flow, when $0.5\,H_d > R > 0.15\,H_d$	0.10
3	For rounded abutments where $R > 0.5\,H_d$ and head wall is placed not more than 45° to the direction of flow	0

2.3 Determination of Design Head

Designing the crest profile for a particular head H_d,results in a profile conforming to the lower nappeof a fully ventilated sharp crested weir and hencethe pressures on the profile for the head H_d, areatmospheric. Operating the spillway for heads lower than H_d, wouldgive pressures higher than atmospheric and for heads higher than H_d,the pressure would besub-atmospheric. At the same time the coefficientof discharge would be reduced or increased (relativeto that for the design head) for the heads lower orhigher than the design head. Generally, designing theprofile for a head lower than the highest anticipatedhead results in a steeper profile provided the sub atmospheric pressures could be kept within acceptablelimits so as not to induce cavitation. The ratio of actual head to design head $(\frac{H}{H_d})$ for ensuring cavitation free performance of the spillway crest is a function of design head H_d. The extent of sub atmospheric pressure for an under designed spillway profile shall be ascertained from hydraulic modelstudies for the specific case. Generally, design headis kept as 80 to 90 percent of the maximum head.As per clause 4.4 of IS 6934: 1998.

When the actual operating head passing over the spillway is 'lessthan the designed head, the prevailing coefficient of discharge (C_d) tends to reduce, andis given by the equation

$$C_d = \left(\frac{H}{H_e}\right)^{0.12} \qquad (10)$$

where H_e is the designed head including velocityhead.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 90% head will then be

$$C_d = 2.2\left(\frac{0.9\,H_e}{H_e}\right)^{0.12} = 2.2 \times 0.987 = 2.17 \cong 2.2$$

Similarly, for still lower heads, the coefficient of discharge goes on reducing and tend~ 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).

2.4Methodology and Design of Ogee crest Spillway

The Technical details of the Spillway

(1) Discharge capacity = 21530 m³/sec
(2) Total length = 183.46 m
(3) Total number of bays = 10 Nos
(4) Width of pier = 2.44 m
(5) Width of bay = 16.15 m
(f) H.F.L = + 206.80 m
(g) Crest level = + 197.53 m
(h) River bed level = + 94.49 m
(i) D/s slope of the spillway= 0. 75H: 1 V
(j) Elevation of basin floor level = 90.22 m
(k) Type of energy dissipator stilling basin
(l) Design Length of stilling basin

Since the given spillway looks like a high weir, the coefficient of dischargemay be assumed to be 2.2.
Now

$$Q = C_d\, L_e H_e^{3/2} \qquad (6)$$

$$L_e = L - 2\left(NK_p + K_a\right) H_e \qquad (9)$$

Where
Let us first work out the approximate value of H_e for a value of effective length
$L_e \approx L$= clear waterway= 10 x 16.15 = 161.50 m

$$\therefore\ 21530 = 2.2 \times 161.50\ H_e^{3/2}$$

$$\text{or}\quad H_e^{\frac{3}{2}}\left(\frac{21530}{2.2 \times 161.50}\right) = 60.60$$

$$\text{or}\quad H_e = (60.6)^{2/3} = 15.43\ \text{m}$$

The height of the spillway above the river bed
P = 197.53-94.49 = 103.04 m
Since

$$\frac{\text{P}}{\text{H}_e}, i.e. \frac{103.04}{15.43} = 6.68 > 1.33\ \ ok$$

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

$$\frac{H_e + P}{H_e} = \frac{(15.43 + 103.04)}{15.43} = 7.68 > 1.7\ ok$$

the discharge coefficient is not affected by fail water. conditions, and the spillway remainsa 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 inthe lower part. This batter is small and will not have any effect on the coefficient ofdischarge.

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

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

Assuming that 90° cut water nose piers and rounded abutments shall be provided,
we have
K_p = 0.01 (Table1.1) and K_a = 0.10 (Table 1.2)
No. of piers = N = 9
Also, assuming that the actual value of H_e is slightly more than the approximatevalue worked out (i.e. 15.43m), say, let it be 15.75 m, we have

$$\therefore\ L_e = 161.50 - 2[9x\ 0.01 + 0.10]15.75 = 155.52\ m$$

Table 1.3. D/s profile values of X, Y

X (in metres)	$Y = \frac{X^{1.85}}{20.91}$ (in metres)
1	0.048
2	0.172
3	0.365
4	0.622
5	0.939
6	1.316
7	1.750
8	2.241
9	2.786
10	3.386
11	4.039
12	4.744
13	5.501
14	6.309
15	7.168
16	8.077
17	9.036
18	10.044
19	11.100
20	12.205
21	13.358
22	14.559
23	15.807
24	17.102
24.32	17.53

Hence

$$Q = 2.2x\ 155.44\ H_e^{3/2}$$

$$H_e^{3/2} = \left(\frac{21530}{2.2x155.52}\right) = 62.93$$

or

$H_e = (62.93)^{3/2} = \mathbf{15.82\ m} \cong \mathbf{15.75\ m}\ (assumed)$

Hence, the assumed H_e for calculating. L_e is all right. The crest profile will bedesigned for H_d= 15.82m (neglecting velocity head).

Note: The velocity head H_a can also be calculated as follows:

Velocity of approach

$$V_a = \left(\frac{21530}{(155.52 + 9x2.44)(103.04 + 15.82)}\right)$$

$$= \frac{21530}{177.48x\ 118.86}$$

$$= 1.02\ \mathrm{m/sec}$$

$$H_a = \text{Velocity Head} = \frac{V_a^2}{2g}$$

$$= \frac{(1.02)^2}{2x\ 9.81} = 0.053\ m$$

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 byequation (3) as:

$$X^{1.85} = 2.0\ H_d^{0.85}\ Y$$

$$Y = \frac{X^{1.85}}{2.0\ H_d^{0.85}} = \frac{X^{1.85}}{2.0\ x(15.82)^{0.85}}$$

$$Y = \frac{X^{1.85}}{20.91} \qquad (11)$$

Before we determine the various coordinates of the D/s profile, we shall firstdetermine the tangent point.

The D/s slope of the dam is given to be 0. 7.5H: 1 V.

Hence

$$\frac{d_y}{d_x} = \frac{1}{7.5}$$

Differentiating the equation (10) of the D/s profile w.r. to x, we get

$$\frac{d_y}{d_x} = \frac{1.85\ X^{(1.85-1)}}{20.91} = \frac{1}{0.75}$$

$$X^{0.85} = \frac{20.91}{(1.85x0.75)} \qquad (12)$$

$$X = 24.32\ \mathrm{m}$$

$$\therefore\ Y = \frac{X^{1.85}}{20.91} = \frac{24.32^{1.85}}{20.91} = 17.53\ \mathrm{m}$$

The coordinates from x = 0 to x = 24.32 mare worked out in Table 1.3.

The U/s profile: The U/s profile may be designed as per equation (4), as:

$$Y = \frac{0.724(X + 0.27H_d)^{1.85}}{H_d^{0.85}} + 0.126H_d$$

$$-0.4315\ H_d^{0.375}\ (X + 0.27H_d)^{0.625} \qquad (4)$$

Using $H_d = 15.82$ m

$$Y = \frac{0.724(X + 0.27x15.82)^{1.85}}{(15.82)^{0.85}} + 0.126x15.82$$

$$-0.4315x15.82^{0.375}\ (X + 0.27x15.82)^{0.625}$$

$$Y = 0.069(X + 4.27)^{1.85} + 1.99 - 1.215(X + 4.27)^{0.625} \quad (11)$$

This curve should go upto X = - 0.27 H_d

or

X = - 0.27 x 15.82 = - 4.27 m.

Various values of x such as, X=0.5 to X= - 4.27 are substituted in equation (11) arid corresponding values of Y are workedout, as given below in Table 1.4.

Table 1.4. U/s profile values of X, Y

X (in metres)	$Y = 0.069(X + 4.27)^{1.85} + 1.99 - 1.215(X + 4.27)^{0.625}$ (in metres)
-0.5	0.009
-1.0	0.060
-1.5	0.123
-2.0	0.276
-2.5	0.452
-3.0	0.687
-3.5	1.001
-4.0	1.460
-4.27	1.990

The profile of the spillway has been determined and plotted in Fig 10. A reversecurve at the toe with a radius equal to$\frac{P}{4} = \frac{103.04}{4} = 25.76 \approx 26$m can be drawn at angle 60°, asshown in Fig 10.

2.5 Length of stilling basin

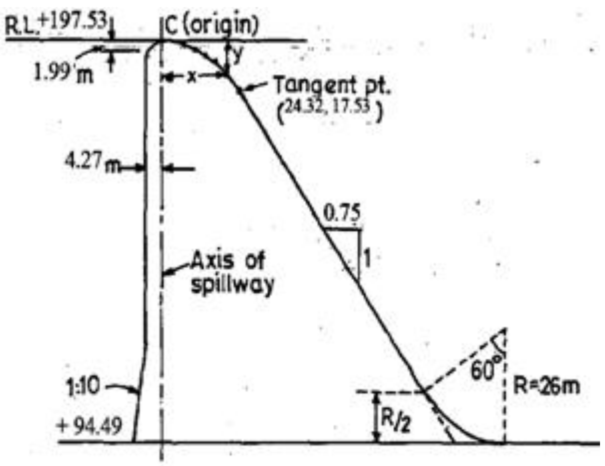

Fig 10. Ogee spillway profile

Discharge intensity per metre width

$$q = \frac{Q}{B} \mathrm{m}^3/\mathrm{sec/m} \quad (13)$$

$$q = \frac{21530}{161.50} = 133.31\ \mathrm{m}^3/\mathrm{sec/m}$$

Velocity of flow below jump

$$V_1 = \frac{q}{y_1} \quad (14)$$

Assume $y_1 = 2.8$ m

$$V_1 = \frac{133.31}{2.8} = 47.61\ \mathrm{m/sec}$$

$$F_1 = \frac{V_1}{\sqrt{g y_1}} \quad (15)$$

$$\frac{47.61}{\sqrt{9.81x2.8}} = 9.08 \approx 9.0 \text{ (range 4.5 to 9.0)}$$

$$y_2 = \frac{y_1}{2}\left[\sqrt{1 + 8xF_1{}^2} - 1\right] \quad (16)$$

$$= \frac{2.8}{2}\left[\sqrt{1 + 8x9^2} - 1\right]$$

$y_2 = 34.27$ m

Stilling basin length

$=4.2xy_2= 4.2x34.27= 143.93\text{m} \approx 144.00$ m

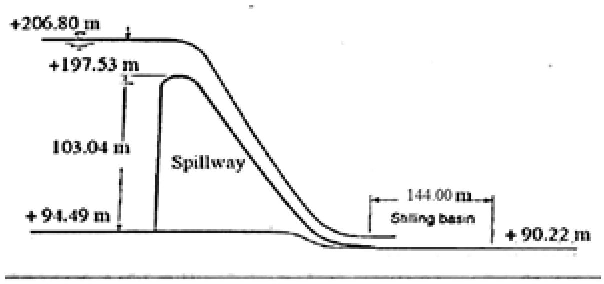

Fig 11. Stilling basin

I.S.I. Standardised Basins. Indian Standard Institution has also standardisedcertain stilling basins for uses under different conditions. A full description ofsuch basins is available in IS: 4997- 1968. Stilling basin, I and II (forF1 <4.5 and F1 > 4.5 respectively) with horizontal aprons and stilling basins III and IVwith sloping aprons, have been standardised and described in the above reference.

3. Conclusion

Hydroelectric developments include flood controlstructures designed to let excesswater escape safely from the reservoir.This "safety valve" prevents water fromspilling over the dam crest. It takes theform of a spillway, a weir or sometimes acombination of both.An ogee crest is a common control structure shape for service spillways, including morningglory inlets, side channel inlets, and controlled and uncontrolled overfall chutes. Consequently,the ogee crest has received much attention by researchers and its hydraulic characteristics arewell understood.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 bedesigned depending upon the position of the jump height curve and tail water curve. A sky jump bucket or an apron may be provided as per the prevailingconditions.The discharge over an uncontrolled ogee crest is influenced by a number of factors:Actual crest shape with respect to ideal nappe shapeRatio of actual head to design head. Height of crest apex above the entrance channel invertApproaching flow velocity. Downstream apron interference or tailwater submergenceUpstream face slope.Thorough discussions of ogee crest design can be found in design manuals prepared byReclamation (1987) and COE (Corps of Engineers) (1952). The spillway crest can be set at a higher elevation and still provide discharge capacityto maintain the original maximum pool elevation for a given inflow hydrograph.An engineer can choose which minimum pressure is acceptable and design the ogee crestaccordingly.An engineer can determine the discharge capacity of an existing ogee crest for heads upto 5 times the design head.An engineer can estimate the cavitation potential of an existing ogee crest for heads upto 5 times the design head. TheU.S. Bureau of Reclamation (1988) recommendation has been thatHe/HD should not exceed 1.33.On the basis of the USBRdata, the US Army Corps of Engineers, WES (1952) has developedseveral standard shapes, designated as WES standard spillwayshapes

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