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Hydraulic Structure I – CENG 3161

Chapter 3. Dam Outlet Works

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3.1 Spillways and Its Design Principles

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- 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 by pass flows exceeding those, which are turned into the diversion system.

Essential Requirements Of a Spillway

The essential requirements of a spillway are:

- The spillway must have sufficient capacity;
- It must be hydraulically and structurally adequate;

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

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

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

- The inflow flood;
- The available storage capacity;
- The discharge capacity of other outlet works;
- Whether the spillway is gated or un gated;
- The possible damages if a spillway of adequate capacity is not provide

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The following are the main components of a spillway:

- ✓ Control Structure
- ✓ Discharge channel (or waterway, or conveyance structure):
- ✓ Terminal structure or energy dissipater
- ✓ Entrance or approach channel and outlet channel:

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Types Of Spillway

1. According to their function

- Service (or main) spillways
- Auxiliary Spillways:
- Emergency spillways

2. According to Mode of Control as:

- Free (or uncontrolled) spillways,
- Gated (or controlled) spillways.

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3. Based on prominent features pertaining to the various components of the spillway (or according to hydraulic criteria) as:

- Free over fall or straight drop spillway,
- Overflow or ogee spillway,
- Chute or open channel or Trough spillway,
- Side channel spillway,
- Siphon spillway,
- Shaft or Morning Glory spillway,
- Conduit or tunnel spillway

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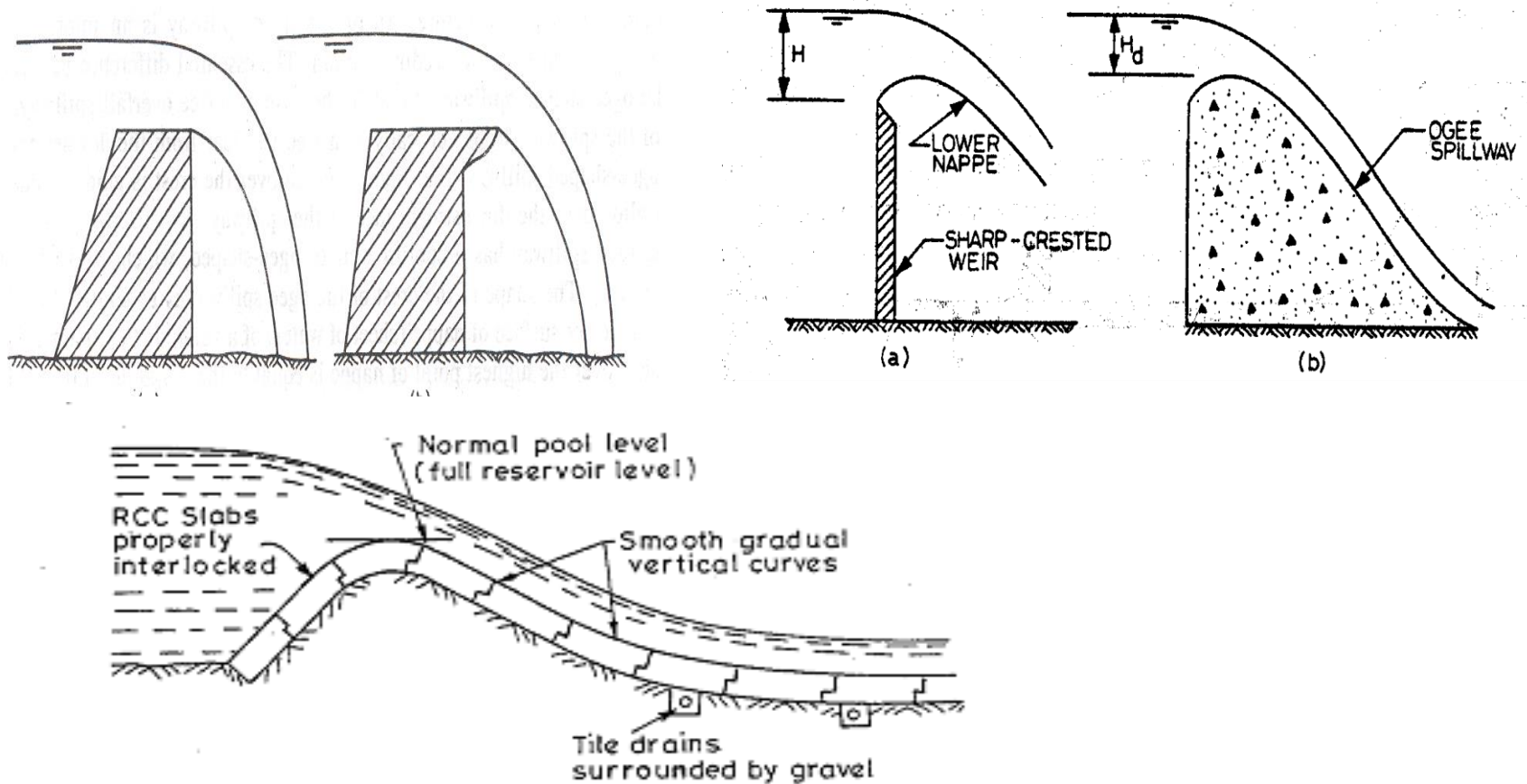


Fig. 21.18. (d) Section through a Chute Spillway.

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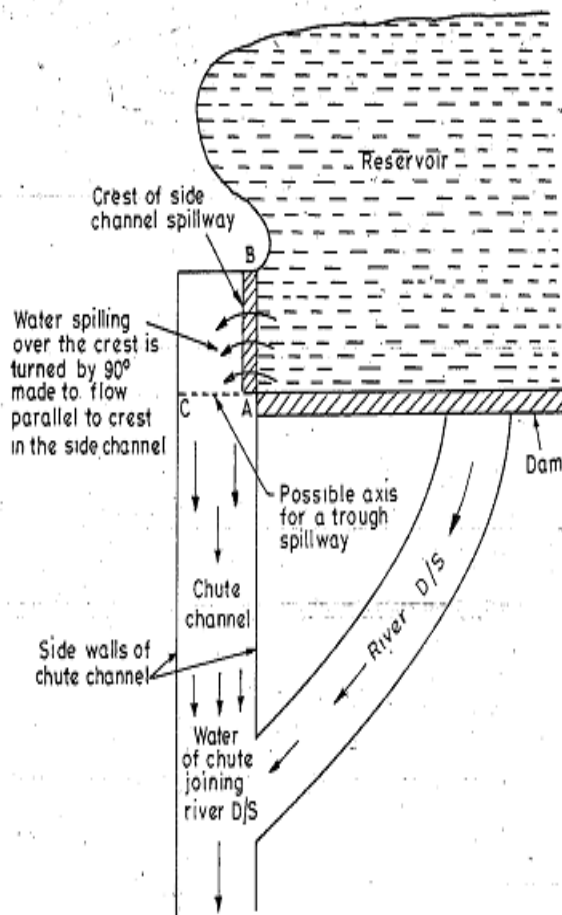


Fig. 21.22. (a) Simplified line sketch of a Side Channel Spillway.

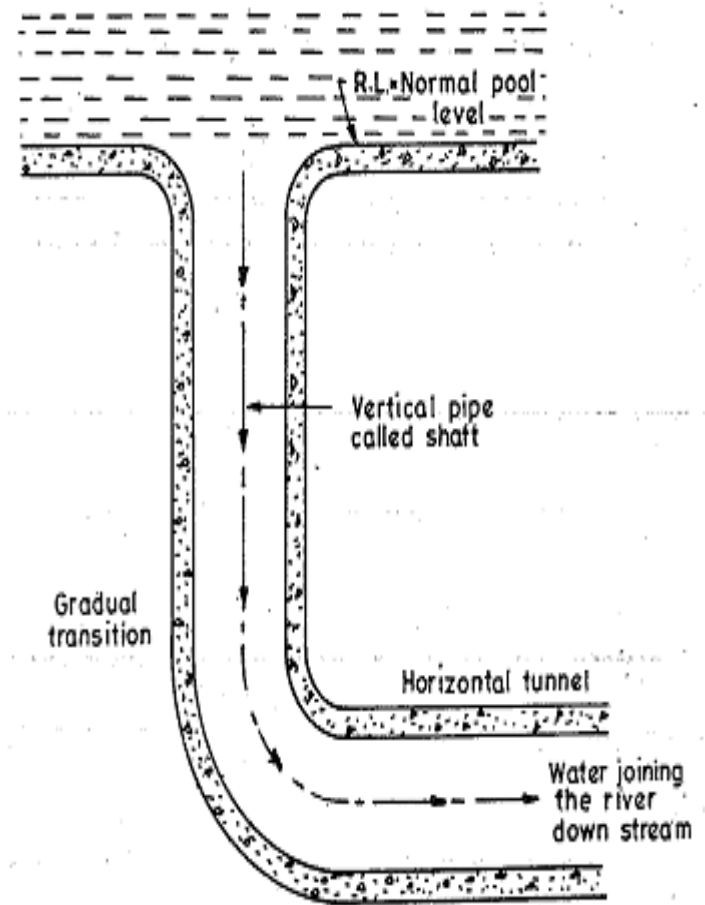


Fig. 21.23. Shaft Spillway.

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Free Over fall Or Straight Drop Spillway

- ✓ This is the simplest type of spillway, which is constructed in the form of low height weir having d/s face either vertical or nearly vertical.
- ✓ If no artificial protection is provided on the d/s side of the over-fall section, the falling jet usually causes the scouring of the stream bed and will form a deep plunge pool.
- ✓ The free over fall spillway is suitable for thin arch dams and for those dams with nearly vertical downstream face and would permit free fall of water.

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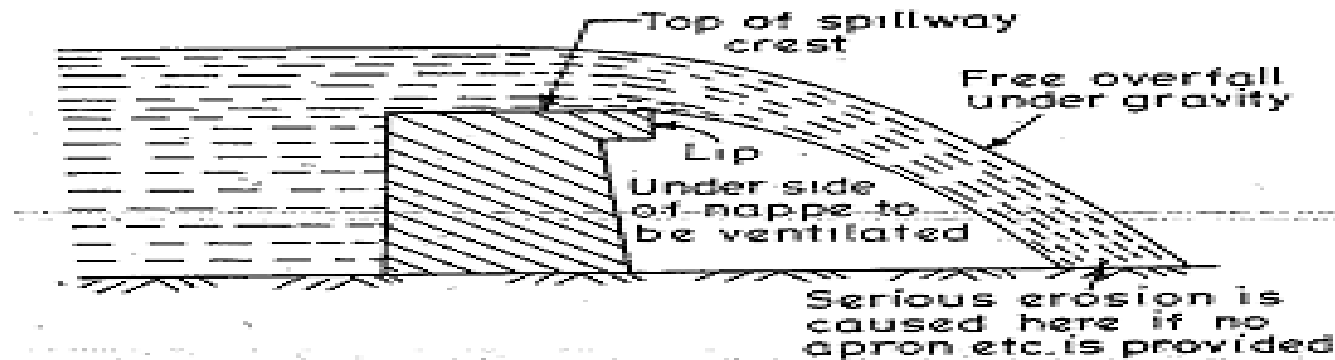


Fig. 21.3. (a) Straight drop spillway without d/s protection.

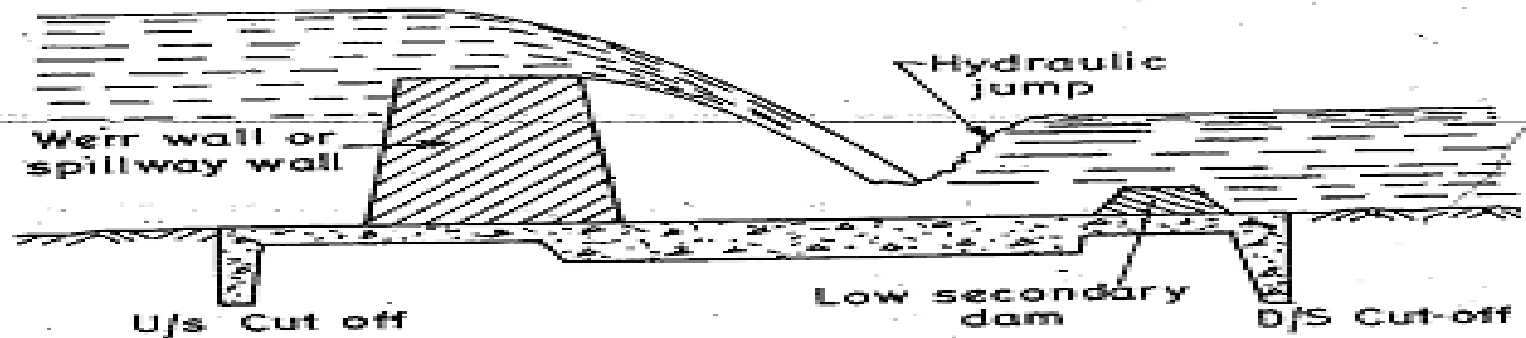


Fig. 21.3. (b) Straight drop spillway with d/s protection works.

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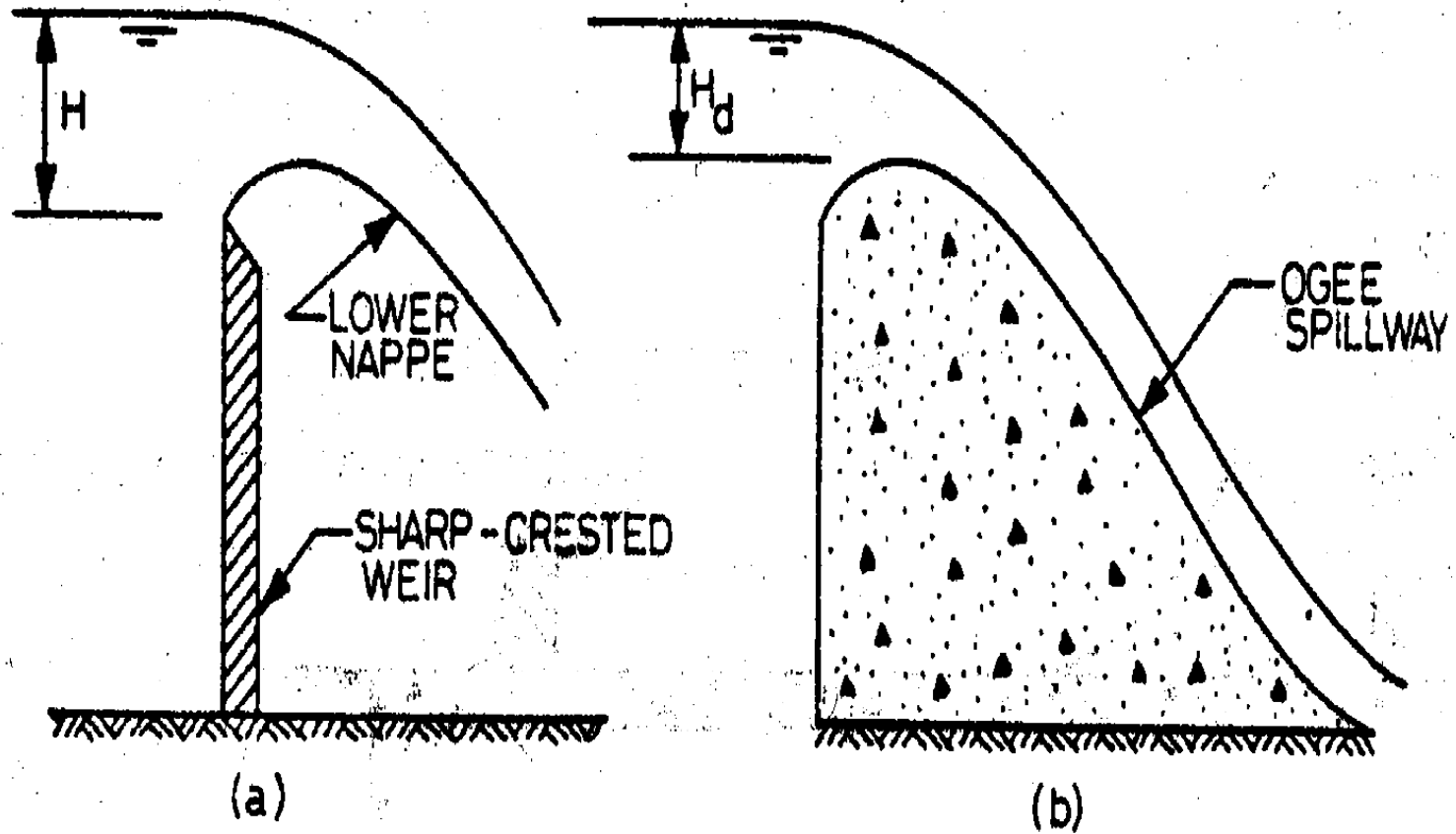
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Overflow (or Ogee) Spillways

- Overflow spillways are by far the most widely adopted.
- They are mainly used on masonry or concrete dams, and if used with earth fill and need a separate concrete structure.
- An overflow spillway is an improvement upon the free over fall spillway.

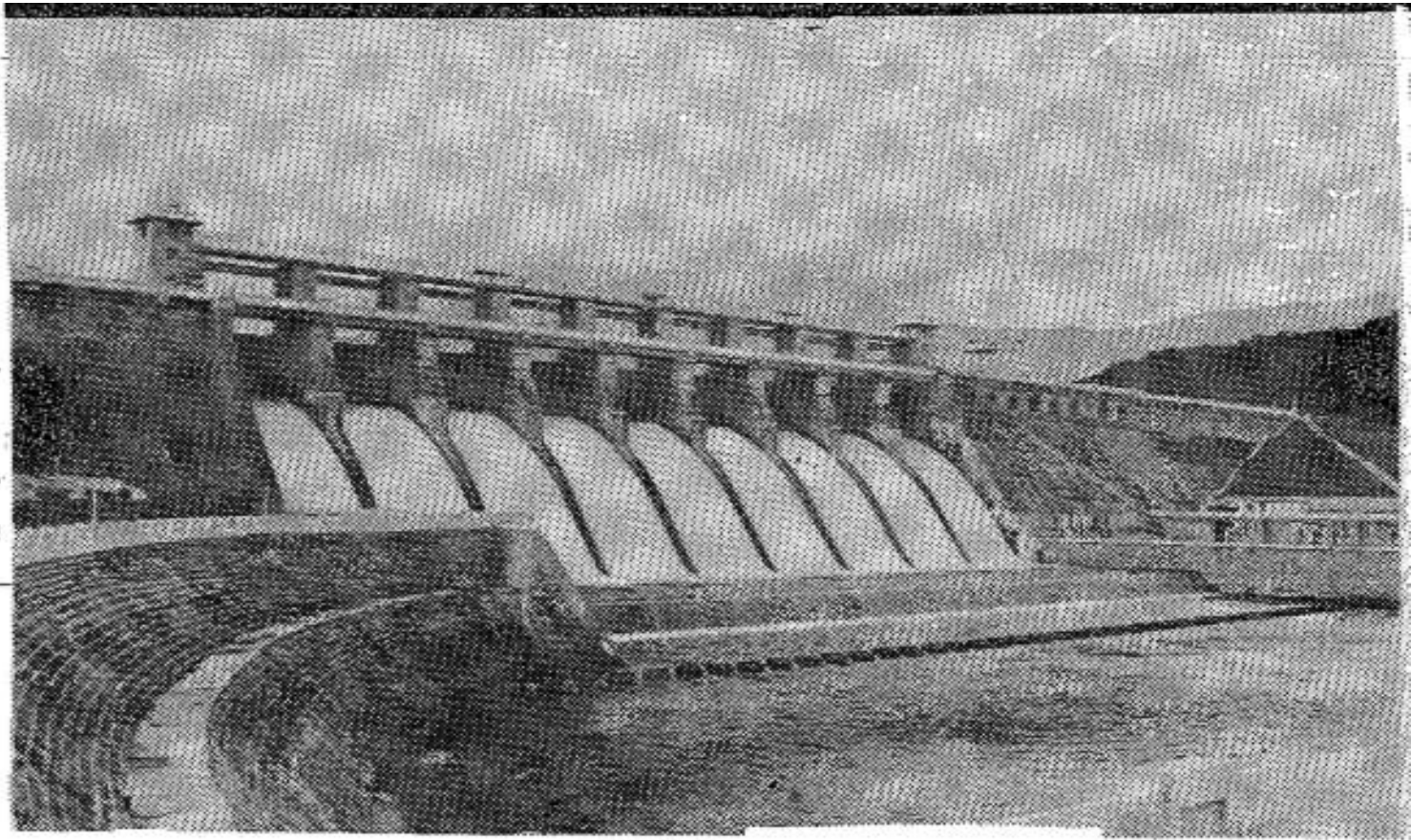
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**Photoview of an Ogee Spillway of Amaravathi dam (earthen cum masonry dam)
located on river Amaravathi (a tributary of river Cauvery)
in Coimbatore District in Tamil Nadu state.**

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- 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
- 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.

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- At head less than the design head ($H < H_d$) the overflowing water will remain in contact with the surface.
- At a head greater than the design head ($H > H_d$), the nape 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. This may result in **cavitations**.

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Design Of Crest Of Ogee Spillway

- The shape of the Ogee 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.

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1. Downstream profile

➤ The downstream profile can be represented by

$$X^n = KH_d^{n-1}y$$

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

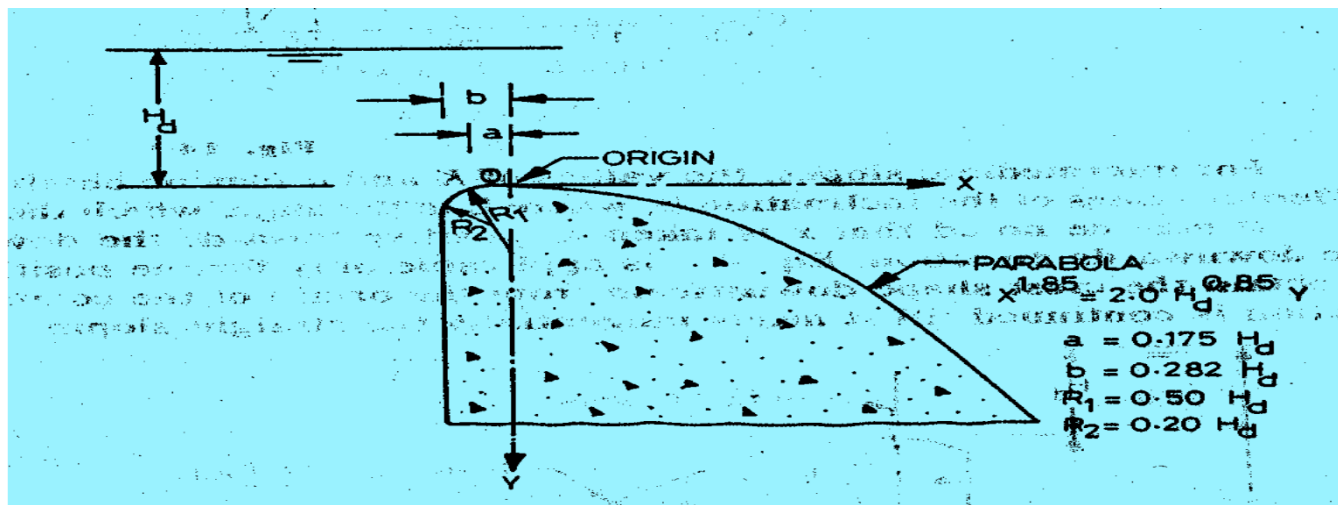
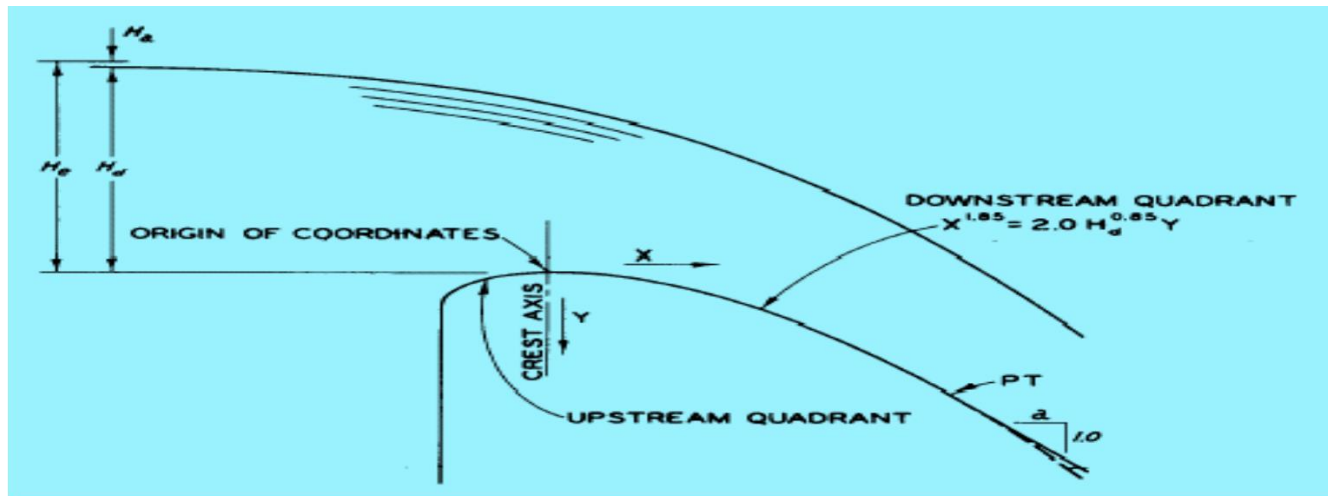
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U/s Face slope	K	n	$\frac{a}{H_d}$	$\frac{b}{H_d}$	$\frac{R_1}{H_d}$	$\frac{R_2}{H_d}$	Crest Equation
Vertical	2.000	1.850	0.175	0.282	0.200	0.50	$x^{1.85} = 2 H_d^{0.85} y$
1H: 3v	1.936	1.836	0.139	0.237	0.210	0.68	$x^{1.836} = 1.936 H_d^{0.836} y$
2H: 3v	1.936	1.810	0.115	0.214	0.220	0.48	$x^{1.810} = 1.939 H_d^{0.810} y$
3H: 3v	1.873	1.776	0.000	0.119	∞ (Straight line)	0.45	$x^{1.776} = 1.873 H_d^{0.776} y$

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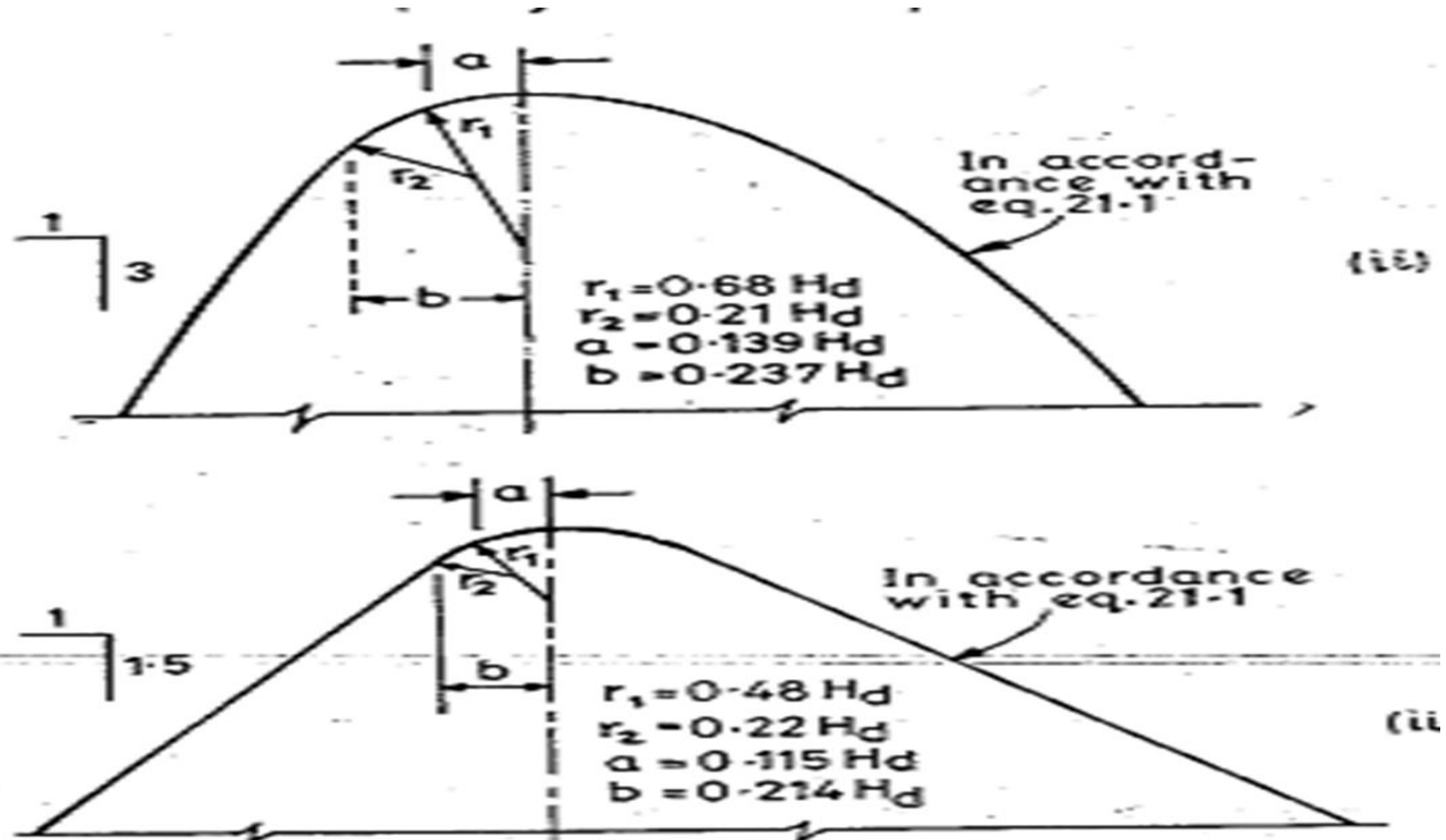


Fig. 21.6. WES profiles for Ogee Spillways for different u/s slopes.

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- 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 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.

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

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

V = velocity of flow at toe 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

$$V = \sqrt{2g(Z + H_a - y)}$$

Where Z = the fall, m

H_a = head due to velocity of approach, m

y = depth of flow at the toe, m

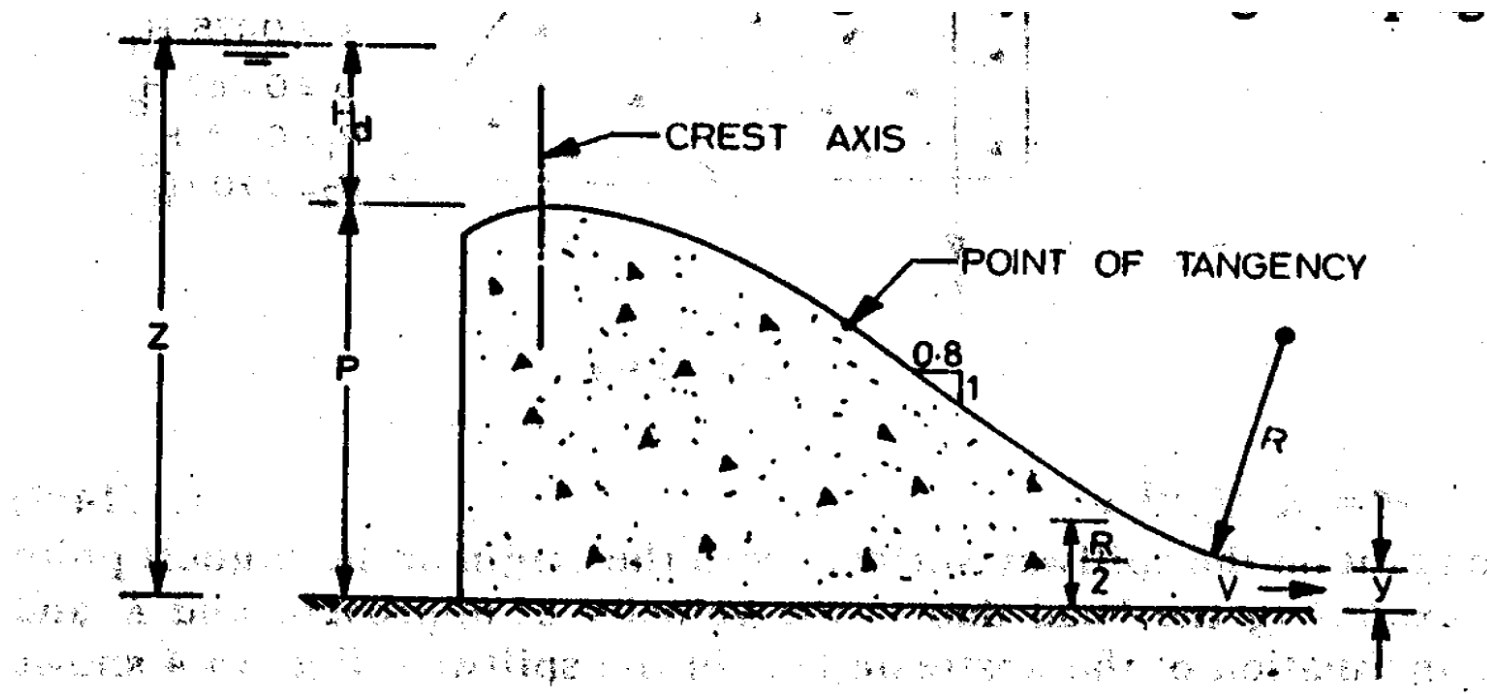
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- Generally, a radius of about one-fourth of the spillway height is found to be satisfactory. Thus

$$R = p/4$$

Where p is the height of spillway crest above the bed



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2. Upstream profile of the crest

- 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:

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

- Where the upstream profile extends up to $x = -0.27H_d$
- The corresponding y value is equal to **0.126 H_d**.

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Discharge of Overflow Spillway

The discharge over an overflow spillway is given by

$$Q = CL_e H_D^{3/2}$$

where Q = discharge, m^3/s

C = coefficient of discharge

L_e = effective length of crest of spillway, m

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

$$H_D = H_d + H_a$$

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

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A. Coefficient of discharge, C , of Overflow spillway

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:

- ✓ Depth of approach, p
- ✓ Heads differing from design head
- ✓ Upstream face slope
- ✓ Downstream apron interference and downstream submergence

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Effect of Depth of Approach

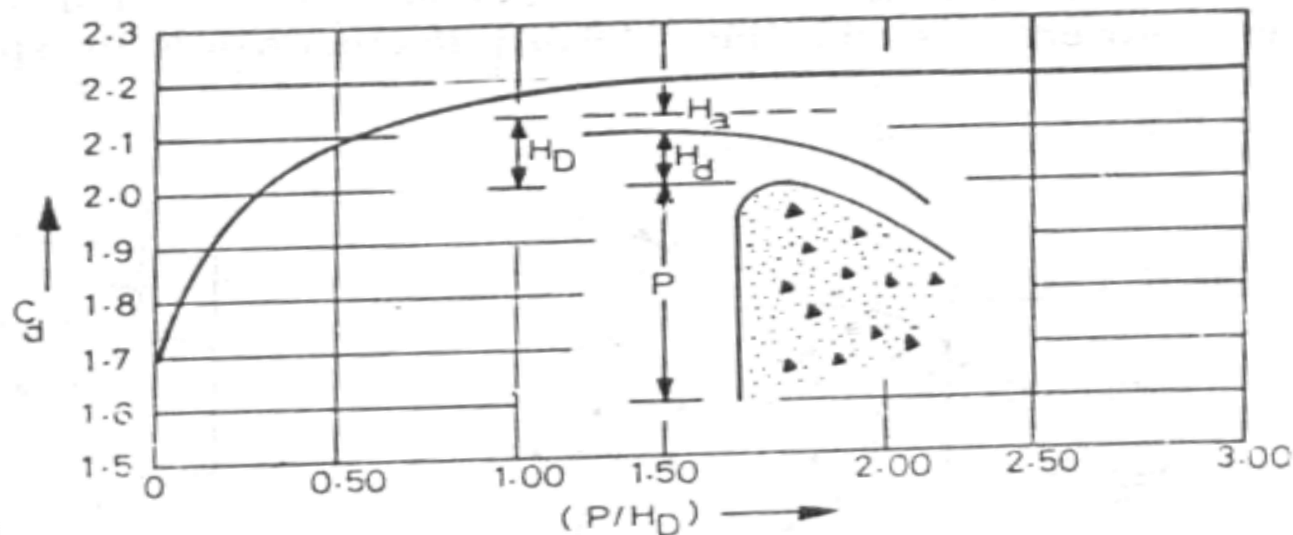
- Model tests have shown that the effect of approach velocity is negligible when the height of the spillway above the streambed is equal to or greater than $1.33 H_d$ ($P \geq 1.33 H_d$)
- where H_d is the design head excluding the head due to velocity of approach
- Such spillways are known as high overflow spillways.

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A plot of C versus P/H_D is shown below

It may be observed from this plot that there is a marked increase in the value of C till the height of the spillway (P) becomes equal to twice the design head H_D .



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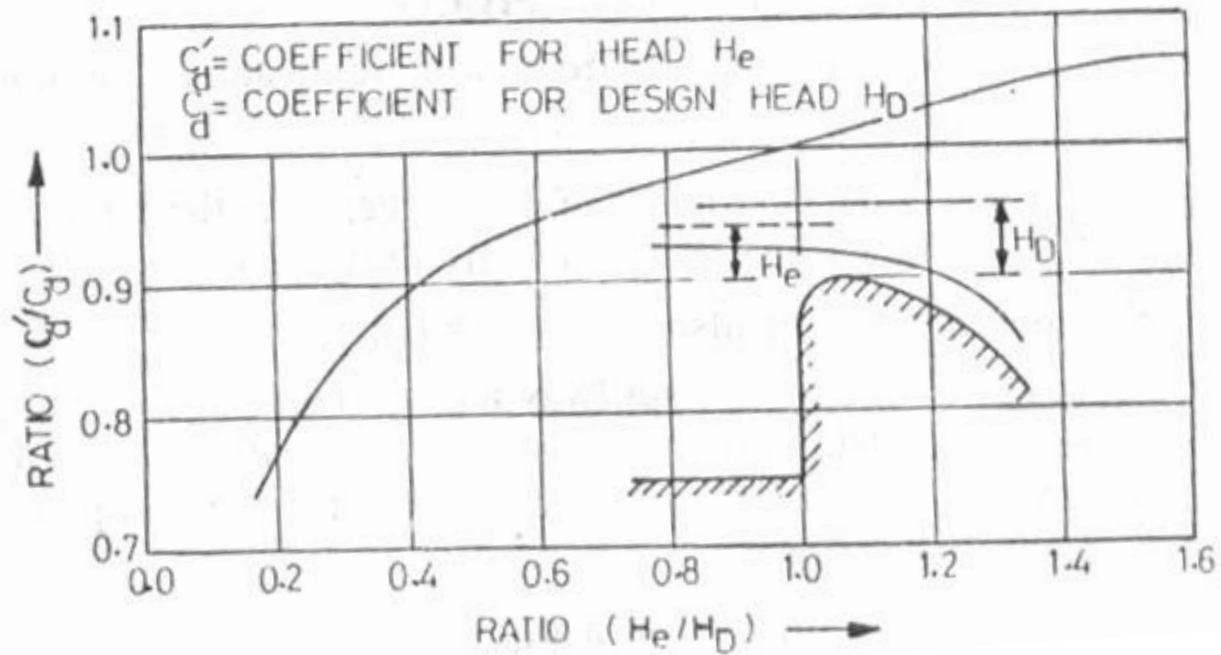
Effect of heads differing from the design head:

The plot of (C/C') versus (H_e/H_D) for a spillway of height P above stream bed greater than $1.33 H_d$, where C is coefficient of discharge corresponding to the actual head of flow H_e and C' is the coefficient of discharge corresponding to the design head H_D .

- It may be observed from this plot that with increase in the value of (H_e/H_D) the value of (C/C') increases.
- However, the design head should not be less than about 80% of the maximum head in order to avoid the possibility of cavitation

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

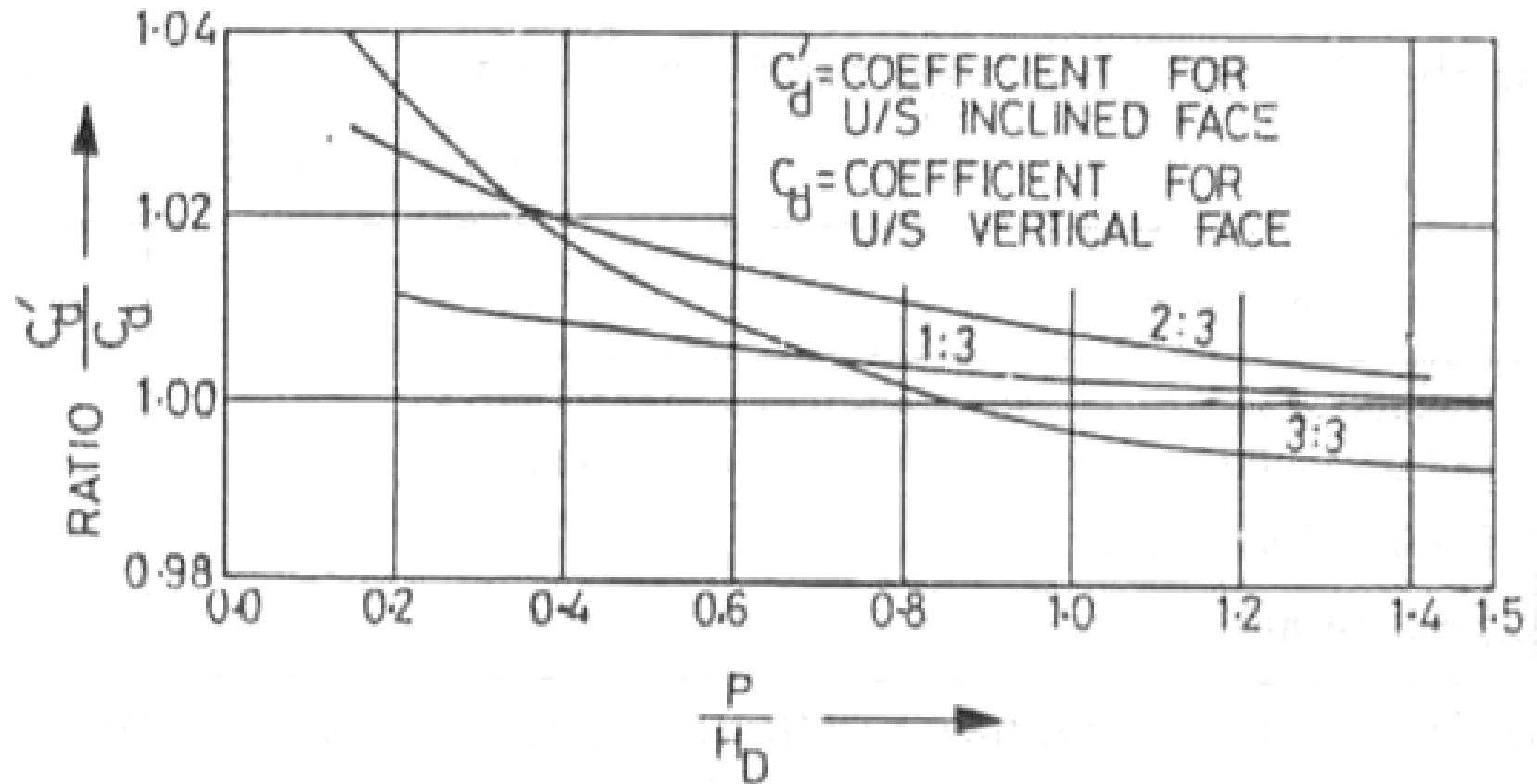
$$C = C' \left(\frac{H_e}{H_D} \right)^{0.12}$$

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

Effect of upstream face slope: For small values of the ratio (P/H_D) a spillway with sloping upstream face has a higher coefficient of discharge than a spillway with vertical upstream face.

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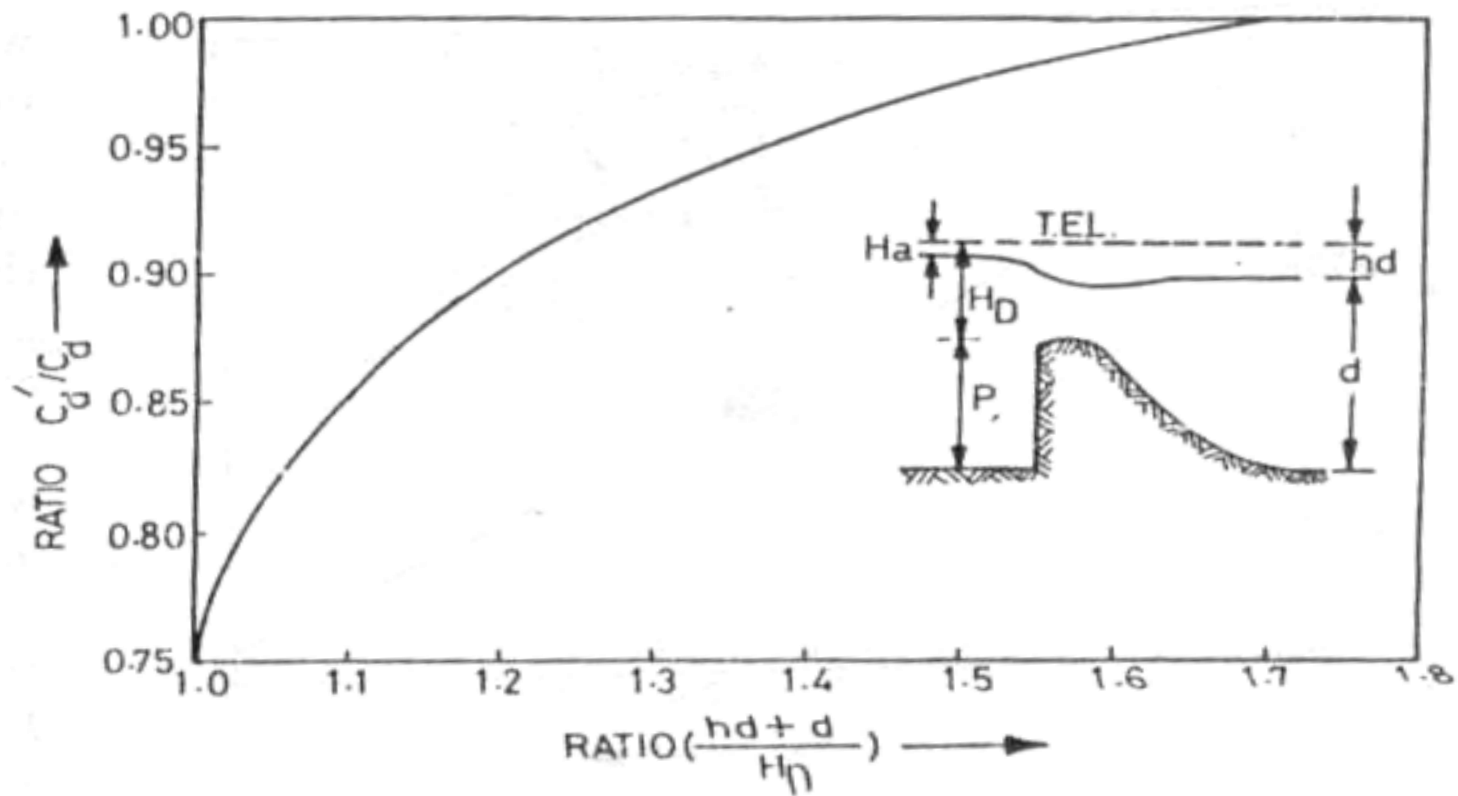
Downstream apron interface and submergence

effects: The coefficient of discharge is reduced due to submergence.

- 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 $(h_d + d)/H_D$ exceeds 1.7, the downstream apron is found to have negligible effect on the coefficient of discharge. But there may be a decrease in C due to tail water submergence.

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

$$L_e = L - 2 (NK_p + K_a) H_D$$

Where L_e = effective length of crest

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

H_D = total head on crest including velocity head

N = number of Piers

K_p = Pier contraction coefficient

K_a = abutment contraction coefficient

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The pier contraction coefficient, K_p depends on

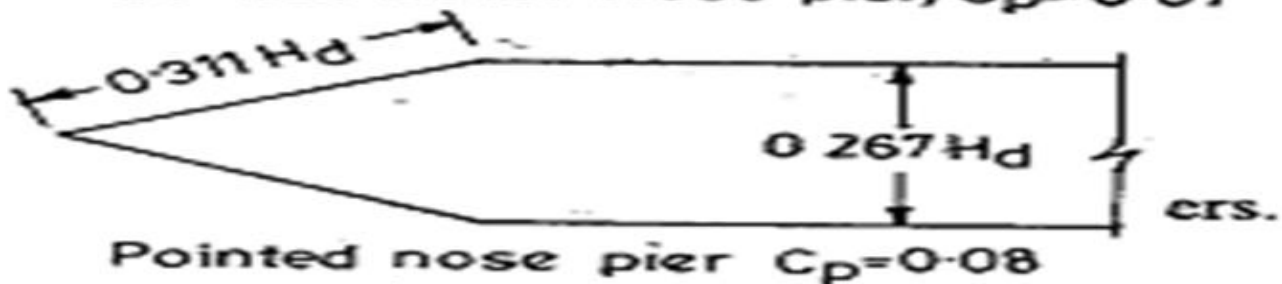
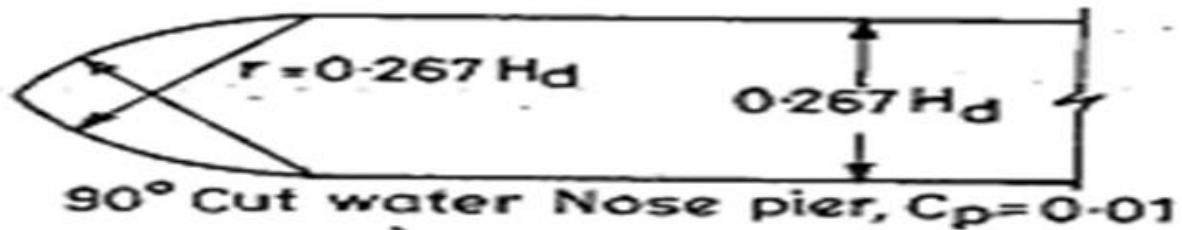
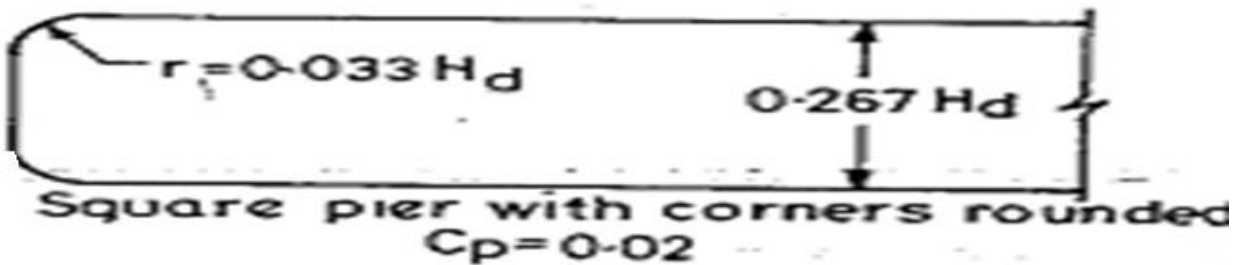
- i) Shape and location of pier nose;
- ii) Thickness of pier;
- iii) Velocity of approach; and
- iv) Ratio of actual head to design head.

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

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

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Various shapes of piers.

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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:

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

3.2 Energy Dissipaters

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- The water flowing over the spillway acquires a lot of kinetic energy by the time it reaches near the toe of the spillway due to the conversion of potential energy into kinetic energy.
- If the velocity of the water is not reduced, large-scale scour can take place on the downstream side near the toe of the dam and away from it.
- In order to protect the channel bed against scour, the kinetic energy of the water should be dissipated before it is discharged in to the down stream.

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- For the dissipation of the excessive kinetic energy possessed by the water the two common methods adopted are:
 - ✓ By converting the supercritical flow into subcritical flow using hydraulic jump.
 - ✓ By directing the flow of water into air and then making it falls away from the toe of the structure using different types of buckets.
- The choice of the energy dissipating method is governed by the tail water depth & the characteristics of the hydraulic jump, if formed ,at the toe.

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- If the tail water depth at the site is not approximately equal to that required for a perfect hydraulic jump, a bucket type energy dissipating device is usually provided.

Characteristics of a hydraulic jump

- Hydraulic jump is a sudden and turbulent rise of water which occurs in an open channel when the flow changes from the supercritical flow state to the subcritical state.
- It is accompanied by the formation of extremely turbulent rollers and considerable dissipation of energy.

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Types of jumps

- The type of jump and its characteristics depends mainly upon the froude number of the incoming flow or initial froude number(F_1), given by

$$F_1 = V_1 / \sqrt{gy_1}$$

Where V_1 = the mean velocity flow before the hydraulic jump

Y_1 = the pre-jump depth

- For the formation of a hydraulic jump the initial froud number F_1 should be greater than one.
- Different types of hydraulic jump are as follows

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1. undular jump : is formed when $F_1 = 1.0$ to 1.70 . in this case the water surface shows some undulation and the energy dissipation is about 5%.
2. Weak jump : when $F_1 = 1.70$ to 2.50 , a weak hydraulic jump occurs. The energy dissipation is about 20 %.
3. Oscillating jump : occur when $F_1 = 2.50$ to 4.5 . and the energy dissipation is between 20 to 40%.
4. Steady jump: occur when $F_1 = 4.5$ to 9.0 . it has very good performance, and most of the hydraulic structures utilize this type of jump for the dissipation of energy. The energy dissipation is between 45 to 70%.
5. Strong jump : occur when $F_1 > 9.0$. it causes a rough water surface with strong surface waves downstream. The energy dissipation is between 70 to 85%. Because of rough action, a strong jump is avoided in spillways, as far as possible.

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Jump Height and Tail water Rating Curves

- Hydraulic jump can form in a horizontal rectangular channel when the following relation is satisfied between the pre-jump depth (y_1) and post – jump depth (y_2).

$$y_2 = \frac{y_1}{2} \left[-1 + \sqrt{1 + 8F_r^2} \right]$$

Where y_1 = pre-jump (initial) depth

y_2 = post- jump (sequent) depth

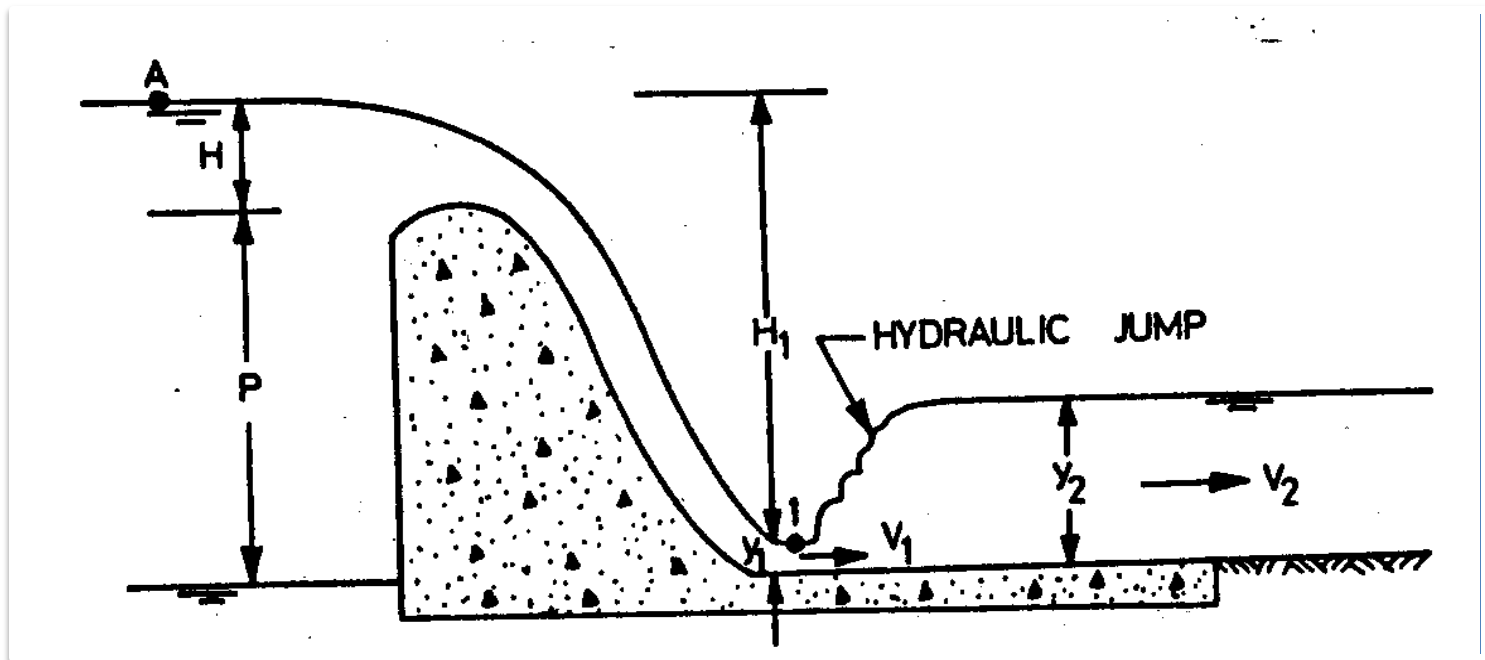
F_{r1} = Froude number of the incoming flow

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- For a given discharge intensity q over a spillway, y_1 , will be equal to q/v_1 ; and v_1 (mean velocity of incoming flow) is determined by the drop H_1 , ($v_1 = \sqrt{2gH_1}$) if head loss is neglected



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- The corresponding value of the sequent depth Y_2 can be determined from eqn . Like wise for different values of the discharge intensity, the value of the sequent depth y_2 can be computed.
- A plot is then made between discharge intensity q and sequent depth y_2 is known as the Jump height curve(JHC).
- The value of tail water depth y_2' corresponding to different values of q may be obtained by actual gauge discharge observations and plots of y_2' versus q prepared, known as tail water rating curve (TWRC)

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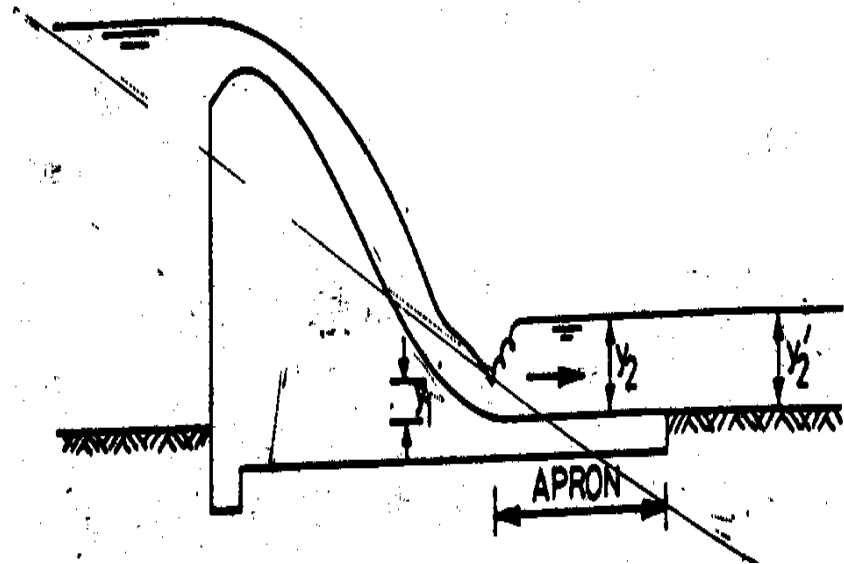
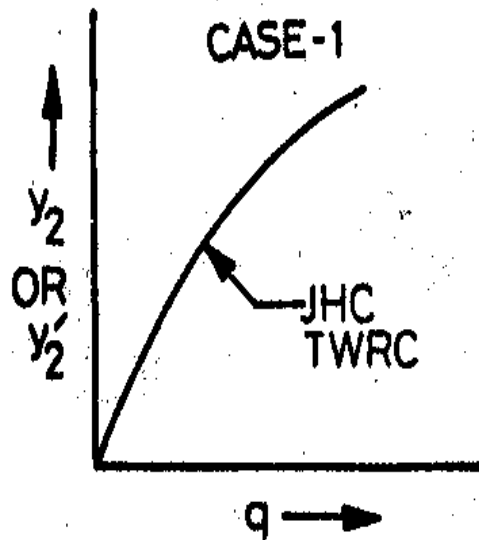
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- If J.H.C. and T.W.R.C. are plotted on the same graph, five possibilities exist regarding the relative positions of these curves
 - I. T.W.R.C. (y_2) coinciding with JHC curve for all discharges
 - II. T.W.R.C. (y_2) lying above the JHC curve for all discharges
 - III. T.W.R.C. (y_2) lying below the JHC curve for all discharges
 - IV. T.W.R.C. (y_2) lying below the JHC curve for smaller discharges and lying above JHC curve for larger discharges
 - V. T.W.R.C. (y_2) lying above the JHC curve for smaller discharges and lying below the JHC curve for larger discharges
- The energy dissipation arrangement that can be provided is dependent upon the relative positions of T.W.R.C. and y_2 curve.

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Condition 1 (T.W.R.C. (y_2) coinciding with JHC for all discharges)



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In this case there is no need special measure for the formation of hydraulic jump, as a perfect jump will always form at the toe.

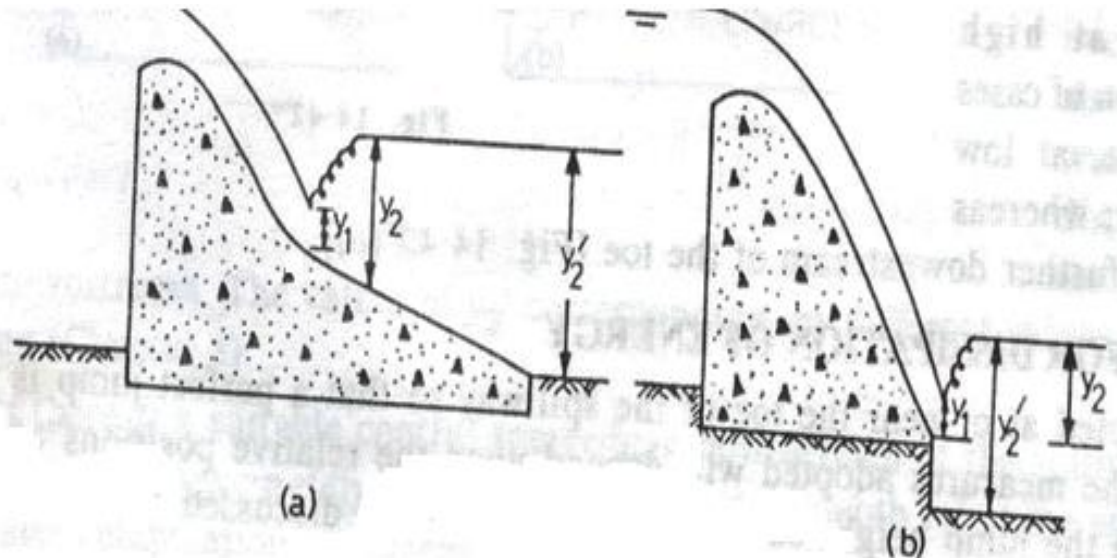
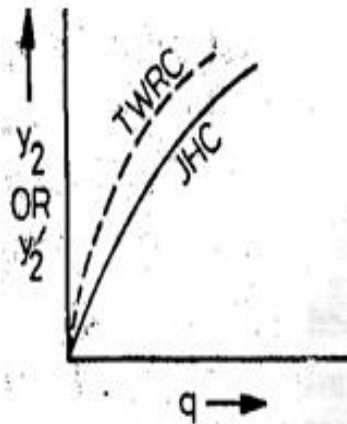
- simple horizontal concrete apron may be provided on the down stream of the toe for the protection of the river bed.
- The length of a horizontal apron is equal to the length of the jump corresponding to the maximum discharge over the spillway

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Condition 2: (T.W.R.C. (y_2)) lying above the JHC for all discharges)

- In this case the jump forming at toe will be drowned out by tail water, and little energy will be dissipated.

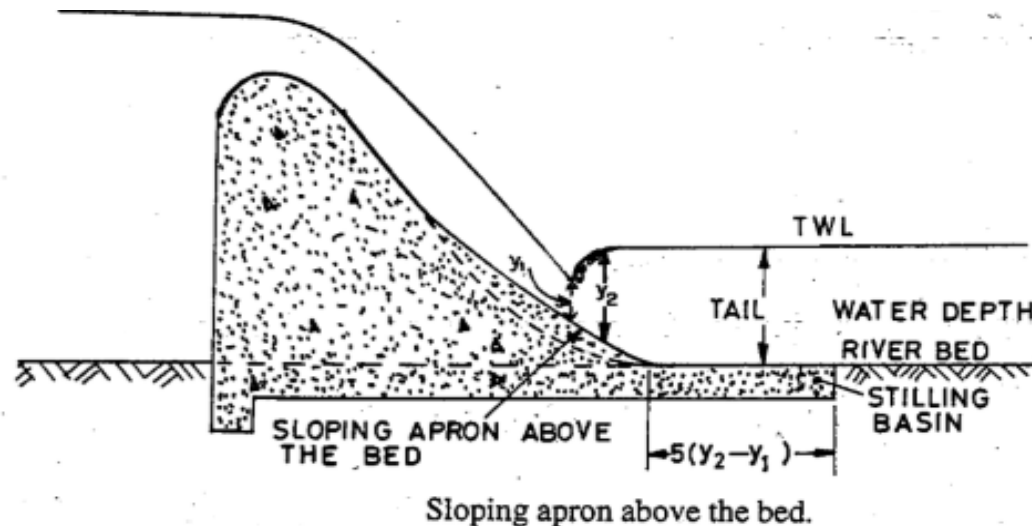


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- The problem can be solved
 - i. By constructing a sloping apron above the river bed level extending from the spillway surface to the toe as shown in the fig.

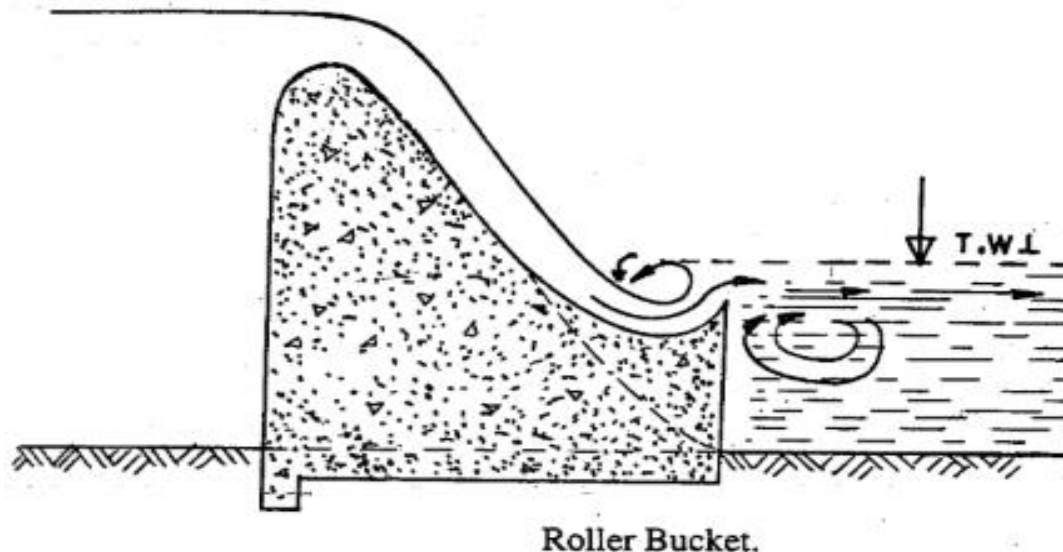
The sloping apron raises the level of the point where the hydraulic jump is formed.



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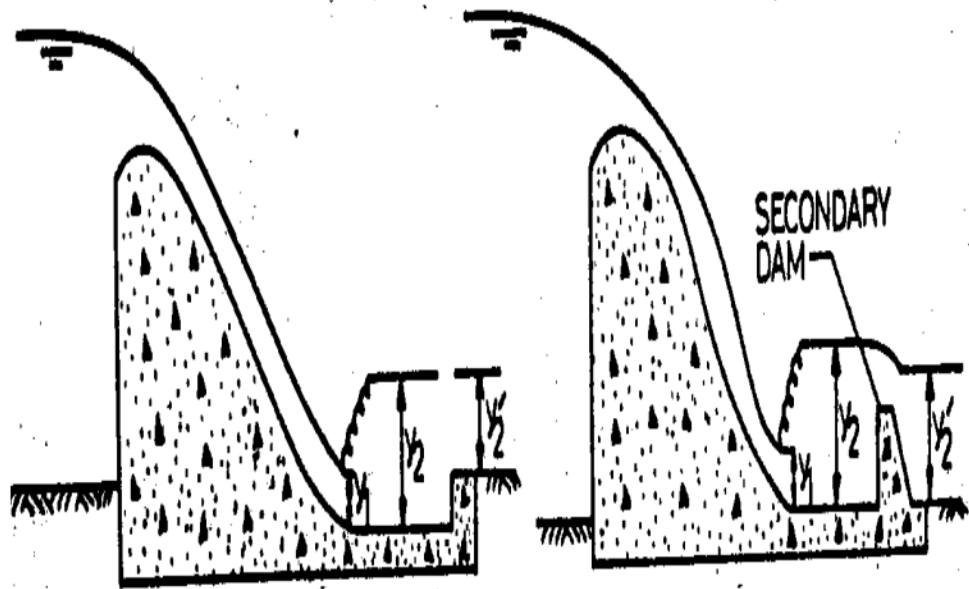
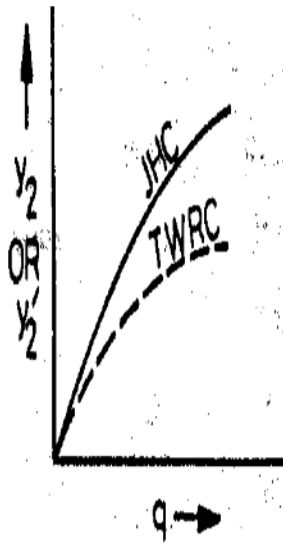
ii. The second solution of this problem can be in the form of providing a roller bucket type of energy dissipater. It consists of an apron which is upturned sharply at end as shown as in fig.



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Condition 3: (T.W.R.C. (y_2)) lying below the JHC for all discharges)



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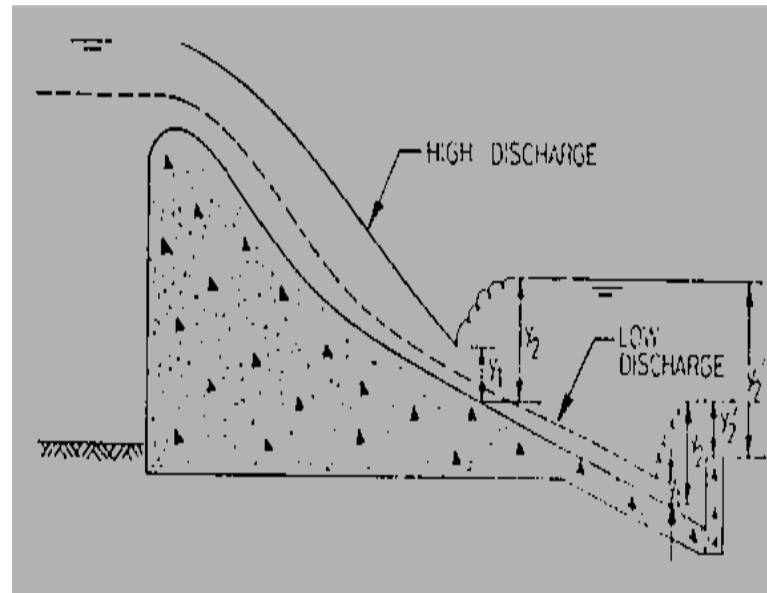
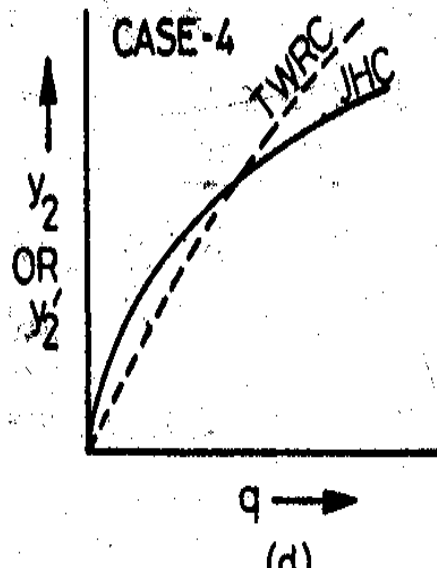
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- In this case the jump will develop at a certain section far downstream of the toe of the spillway.
- This is the **most frequent one**, and shows that a stilling basin (with a depressed horizontal apron) is required for all discharges in order to produce a jump close to the toe of the spillway.

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Condition 4: T.W.R.C (y_2) lying below the JHC curve for smaller discharges and lying above JHC for larger discharges



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In this case the following measures may be taken to develop jump close to the spillway.

- Provide a stilling basin with an end sill for developing a jump at low discharges and combine the basin with a sloping apron for developing a jump at high discharges.
- Provide a sloping apron which lies partly above and partly below the riverbed so that jump will develop at lower portion of the apron at low discharges and at higher portion of the apron at high discharges.

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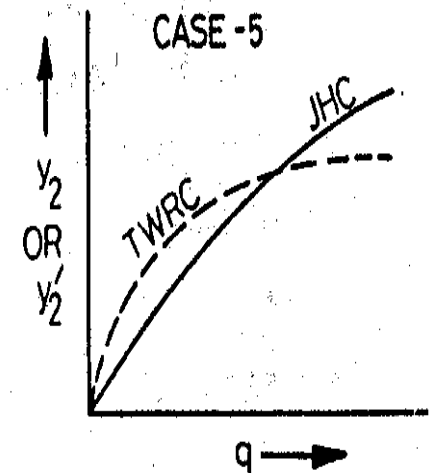
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Condition 5 T.W.R.C. (y_2) lying above the JHC curve for smaller discharges and lying below the JHC curve for larger discharges

This case is similar to case four but the range of discharge is different.

A sloping apron is provided which is partly Below the river bed and partly above the River bed.

In this case the jump will form in the upper Portion of the apron at low discharges, and In the lower portion at high discharge.



Cont....

60

Stilling Basin

- A stilling basin consists of a short, level apron at the foot of the spillway
- The function of the basin is to decelerate the flow sufficiently to ensure the formation of a hydraulic jump within the basin.
- A hydraulic jump can be stabilized in the stilling basin by using appurtenances(or accessories) such as chute blocks, basin blocks and end sill.

Cont....

61

Types of stilling basins

- There are various types of stilling basins.
- The type of stilling basin most suitable at a particular location mainly depends upon the initial froude number (F_1) and the velocity V_1 of the incoming flow.
- The stilling basins are usually rectangular in plan and made of concrete.
- The length of the basin, measured in the direction of flow, depends upon the sequent depth y_2 and initial froude number F_1 . and it is different for different type of basin.

Cont....

62

- The following type of basins are commonly used in practice.
- U.S.B.R basins has standardized stilling basins for different range of froude number.

1. U.S.B.R. basins for Froude's number between 1.7 and 2.5.

- For this case only a horizontal apron needs to be provided.
- As the flow in this case does not have much turbulence usually no accessories are required to be provided.
- However, the apron should be sufficiently long to contain the entire jump over it.
- Length of apron = $5 Y_2$, and where Y_2 is the sequent depth.

Cont....

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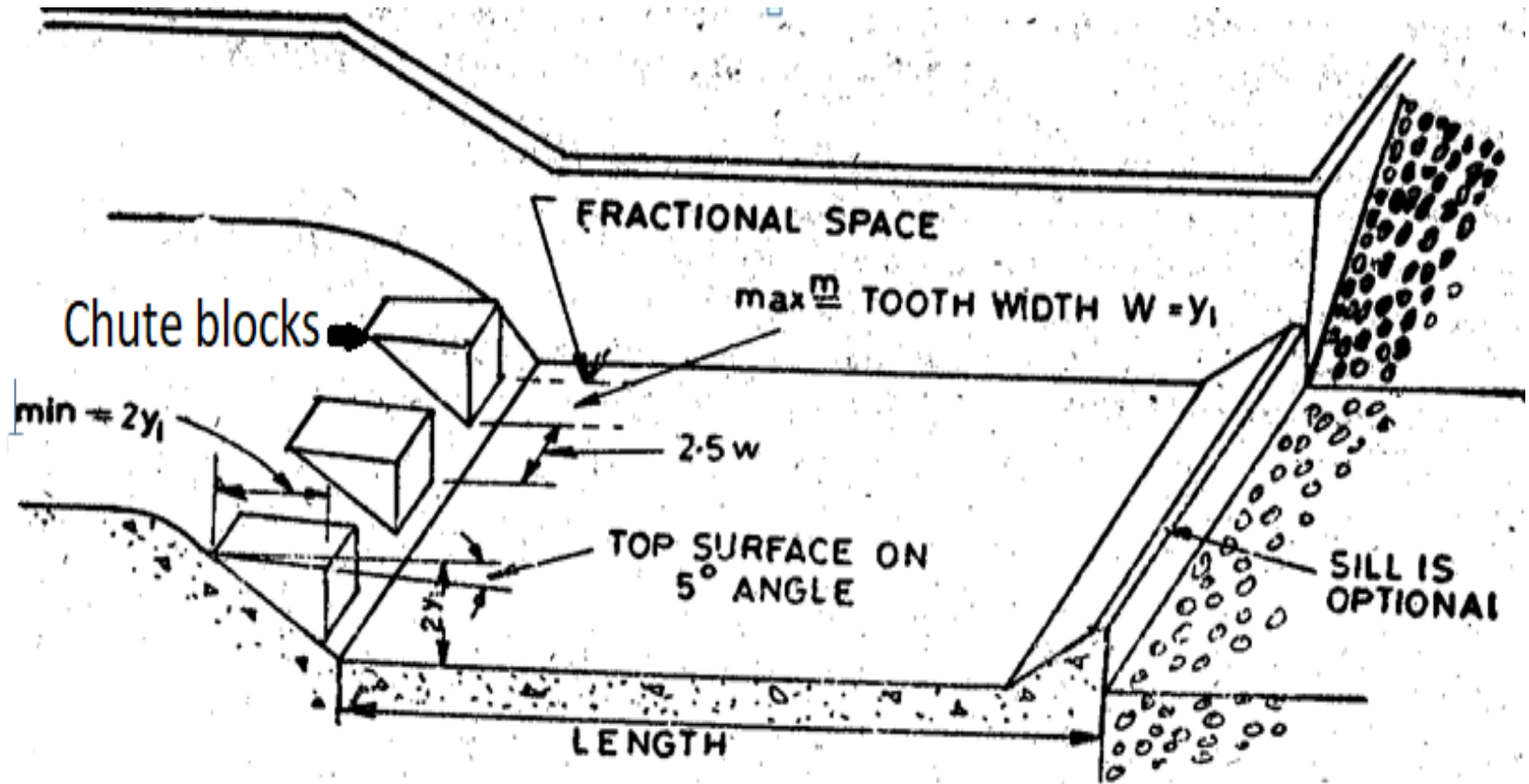
2. U.S.B.R. basins for Froude's number between 2.5 and 4.5

- For this range of Froude number Type IV stilling basin has been found to be effective for dissipating the energy of flow.
- The basin is provided with chute blocks.
- The length L of the stilling basin may be obtained for different values of F_1 from the following table:

F_1	2	3	4	5
L/Y_2	4.3	5.3	5.8	6

Cont....

64



U.S.S.R. Stilling Basin IV (F_1 lies between 2.5 and 4.5).

Cont....

65

3. U.S.B.R. basins for Froude number higher than 4.5

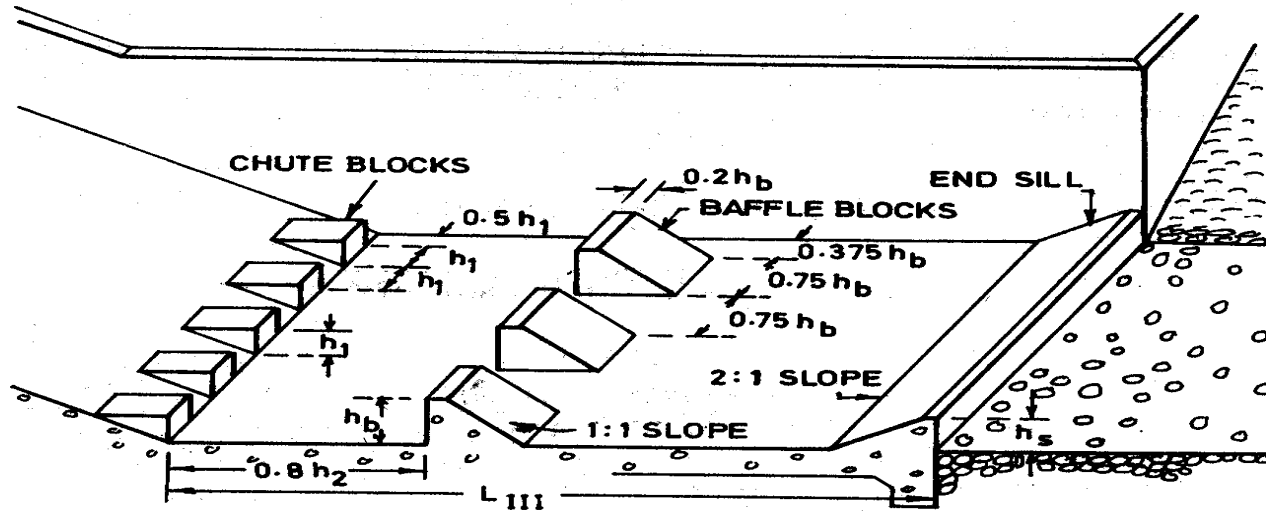
- For this case depending upon the velocity of incoming flow, two types of stilling basin have been developed as indicated below.

**(a) When the velocity of incoming flow is less than 15m/s,)
Type III stilling basin** have been adopted.

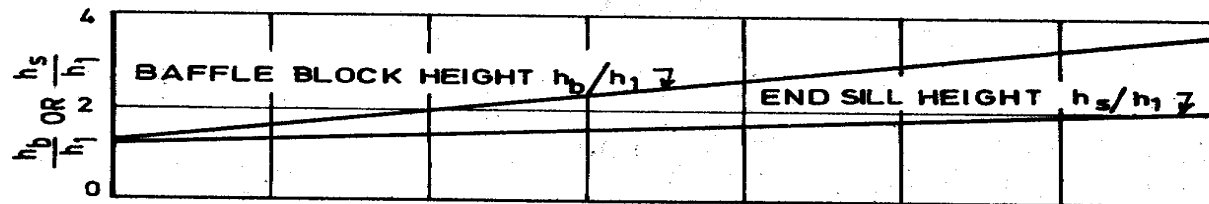
- This basin is provided with CHUTE BLOCKS, BAFFLE BLOCK and an ENDSILL.

Cont....

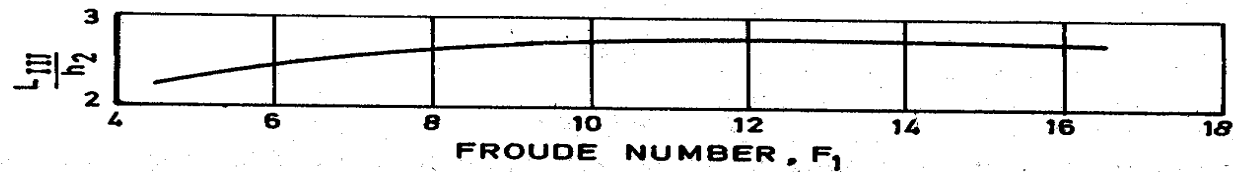
66



(a) TYPE III BASIN DIMENSIONS



(b) HEIGHT OF BAFFLE BLOCKS AND END SILL



(c) LENGTH OF STILLING BASIN

Cont....

67

F1	5	6	8	10	12	14
L/y2	2.3	2.5	2.6	2.7	2.8	2.8
hb/y1	1.5	1.7	2.0	2.3	2.7	3.0
Hs/y1	1.2	1.3	1.5	1.6	1.7	1.8

Cont....

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(b) When the velocity of the incoming flow is exceeds 15 m/s,

- **TYPE II STILLING BASIN** may be adopted.
- In this basin only chute blocks are provided and instead of a solid end sill a dentate sill is provide.
- In this basin baffle blocks are not provided because
 - Due to high velocity of incoming flows these blocks will be subjected to excessive large impact forces and
 - There is a possibility of cavitation along the downstream face of theses blocks and adjacent floor of the basin due to large negative pressure developed in the region.

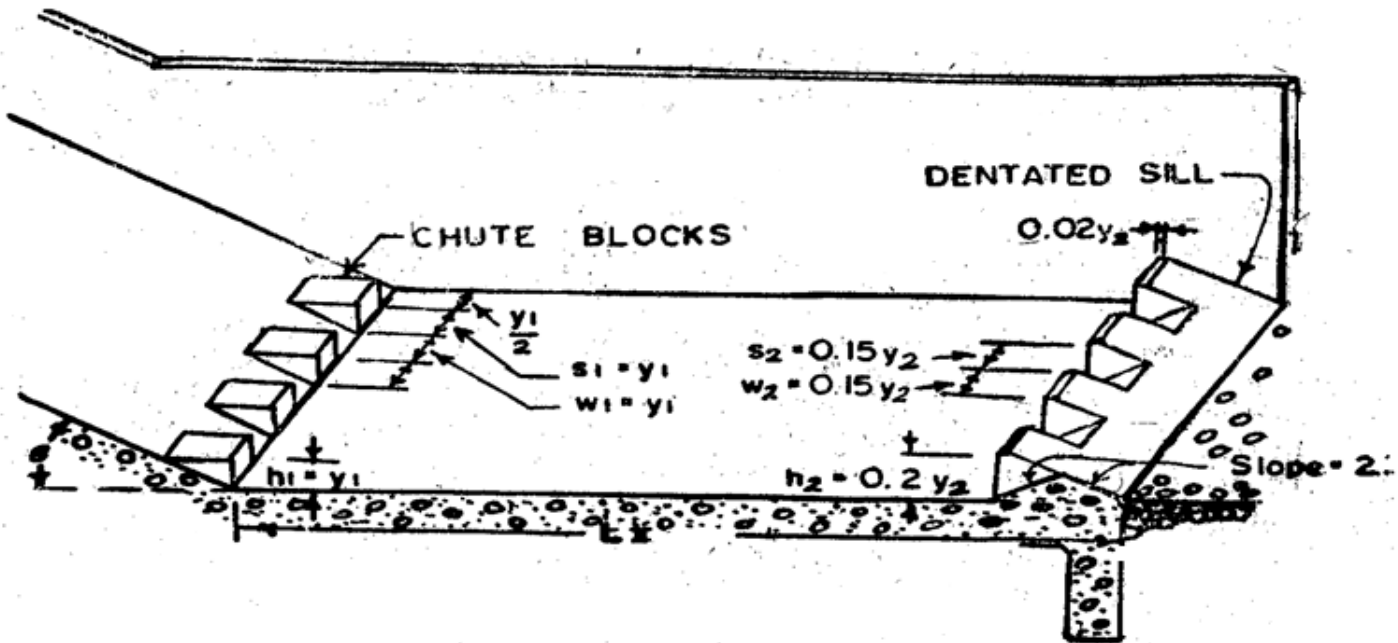
Cont....

69

- Due to baffle blocks being eliminated in this case the dissipation of the energy is primarily accomplished by hydraulic jump ,
- Hence, the length of the basin will be greater than that indicated for the **TYPE III BASIN**.

Cont....

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

F_1	5	6	8	10	12	14
L/Y_2	3.85	4.0	4.2	4.3	4.3	4.3

Cont....

71

Bucket type energy dissipaters

- Bucket type energy dissipaters consist of an upturned bucket provided at the toe of the spillway.
- The bucket type energy dissipaters may be used only for overflow type spillways.
- This type of energy dissipation becomes more economical than the method of stilling basins when the Froude number F_1 of the incoming flow exceeds 10,
- because in such cases the difference between the initial depth and sequent depth is quite large and a very long and deep stilling basin is required.

Cont....

72

- Moreover the bucket type energy dissipaters may be used with any tail water condition.
- However, this type of energy dissipater may be used only when the river bed is composed of stiff rock.
- The bucket type energy dissipaters are of the following three types:
 - a) Solid roller Bucket
 - b) Slotted roller Bucket
 - c) Ski jump (or flip or trajectory bucket)

Cont....

73

- The solid or slotted roller bucket may be used where the tail water depths are too large as compared to the sequent depths required for the formation of the hydraulic jump.
- Both these buckets remain submerged in tail water and hence these are also termed as submerged bucket type energy dissipaters.

Cont....

74

A. Solid roller Bucket

- A solid roller bucket consists of a bucket like apron with a concave circular profile of large radius and a deflector lip as shown in figure.
 - When the water flows over the bucket the entire sheet of water leaving the bucket is deflected upward by the bucket lip and **two elliptical rollers** are developed as shown in the figure.
- i. surface rollers:-** these are formed on the surface of the bucket. for the flow conditions the rollers move counter clockwise direction with in the region above the bucket.

Cont....

75

ii. **Ground rollers** :- these rollers are developed at the river bed immediately downstream of the bucket. These move in the clockwise direction for the flow conditions shown in the figure.

➤ the movement of the two types of rollers and consequent balance of the incoming flows causes the dissipation of the energy.

Cont....

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- This type of bucket is not effective if the river bed is higher on its down stream. The radius of bucket is usually taken as
- Radius of the Bucket: **$R = 0.6\sqrt{H' * H_d}$**
 - Where H' = fall from crest of spillway to bucket invert in meter.
 - And H_d = Head over crest in meters
- Vente Chow's Formula **$R = 0.306 * 10^k$**
 - Where $k = (v_1 + 6.4 H_d + 4.88) / (3.6 H_d + 19.5)$
 - V_1 = velocity of flow at the toe of spillway in m/s

Cont....

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B. Slotted Roller Bucket

- A slotted roller bucket is a modified form of a solid roller bucket. It is also used when the tail water depth is greater than the sequent depth.
- A slotted roller bucket, like a solid roller bucket, consists of a bucket like apron with a concave circular profile of large radius but it has a slotted (or dentated) deflector lip at the downstream. Thus a slotted bucket has a number of teeth.
- In general the hydraulic action of the slotted bucket has the same characteristics as that of a solid bucket.
- Thus in the case of the slotted bucket also the same two rollers are developed.

Cont....

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- However in this case the water leaves the lip of the bucket at a flatter angle and only a part of it is deflected upwards.
- Thus surface boil is considerably reduced, and less violent ground rollers occurs which results in a smoother flow on the downstream side.
- Moreover in this case the bed material is neither deposited nor carried away from the bucket lip, also any debris which might get into the bucket is immediately washed out through the slots.

79

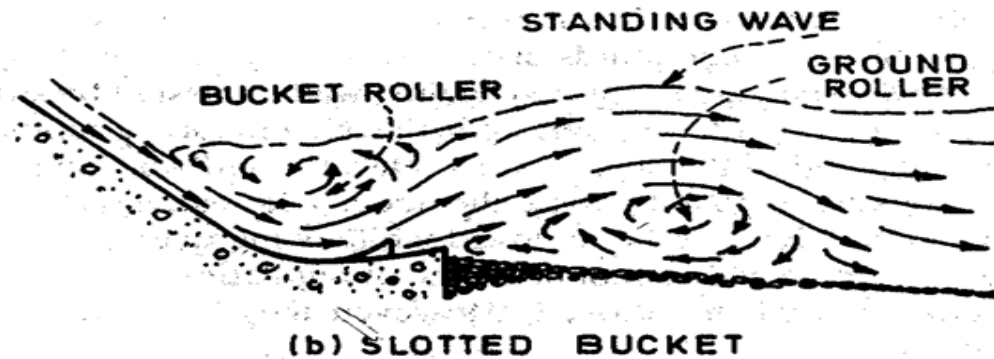
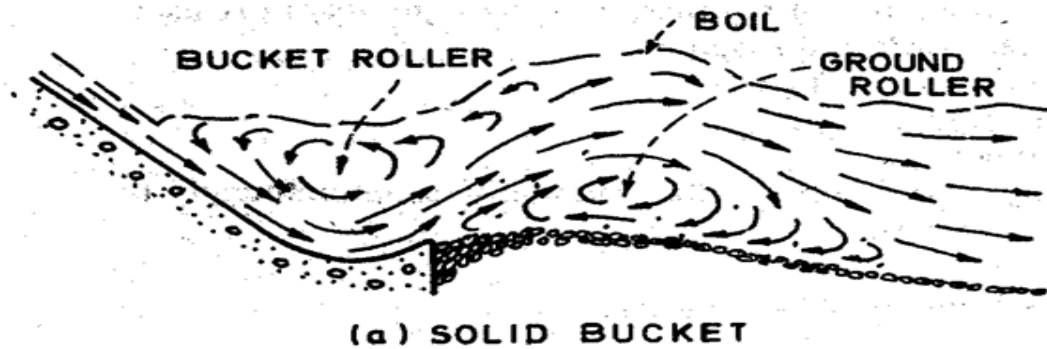


Roller Buckets

Cont....

80

Energy Dissipation as Roller



Roller Formation in Roller Buckets

Cont....

81

C. SKI JUMP BUCKET

- A ski jump bucket may be used where the tail water depth, is less than sequent depth required for the formation of hydraulic jump and the river bed is composed of stiff rock.
- The lip of the bucket is so shaped that the entire sheet of the water flowing over the bucket is deflected as a free jet which falls back into the river channel at a safe distance away from the spillway.
- Thus in this case energy is dissipated by air resistance, breaking of the jet into bubbles and the impact of the falling jet against the river bed and tail water.

3.3 Dam outlets or sluices ways

82

- Dam outlets are provided in the body of the dam or its abutments below the crest level of the spillways so that water can be released for the various purposes for which the dam was constructed.
- i.e. dam outlets are opening for withdrawal of water from the dam.
- The outlet of most of the dams consists of one or more sluice ways with their inlets at about minimum pool level.
- In most of the cases, a number of outlets are generally provided at different levels: as a single large capacity outlet may be structurally unsatisfactory or difficult to construct.

Cont....

83

- More over by having more number of smaller capacity outlets, greater control on discharge can be obtained, which can be varied as & when desired.
- Hence, when wider fluctuations in the demand are anticipated, it is always, advisable to go in for more number of small capacity sluiceways, although it may prove to be some what costlier as compared to a few large capacity sluiceways.

Cont....

84

Location of dam outlets

- An outlet is a closed conduit formed in the body of the dam. It may also be in the form of a pipe or tunnel that passes through the hill side at one end of the dam.
- The function of an outlet is to discharge the stored water in to the channel d/s. For a concrete (or masonry) dam, the outlets pass through the body of the dam and are called sluice ways.

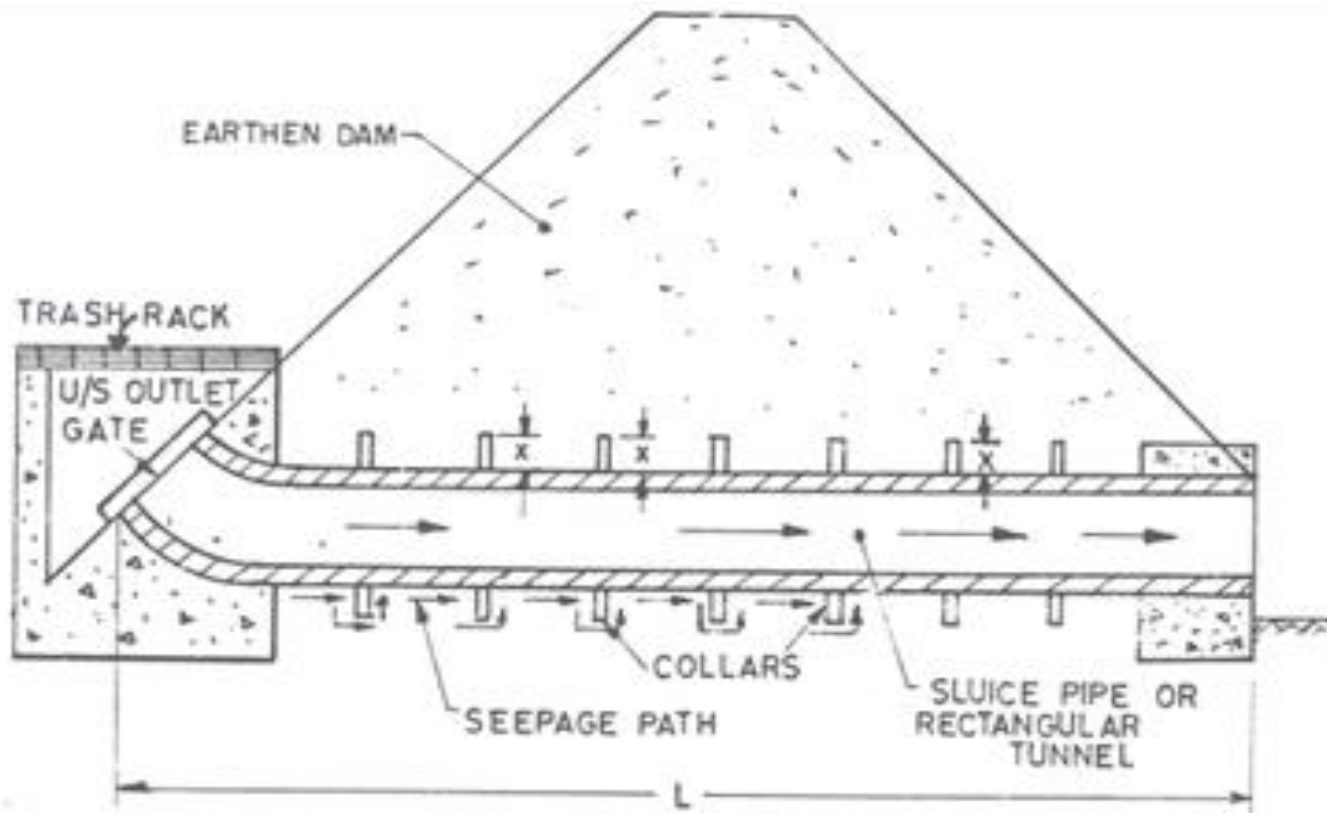
Cont....

85

- For earthen dams, it is preferred to place the outlets outside the limits of the embankments.
- However , for small earth dams, sometimes the outlet conduits are permitted to pass through the body of the dam.
- In this case projecting collars, must be provided so as to reduce seepage along the outside of the conduit. The seepage is thus, reduced with the collar by increasing the length of the seepage path at least 25 percent.

Cont....

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Cont....

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Discharge through an Outlet

- The outlet should be designed to have the required discharge capacity . The discharge through the outlet can be determined by the equation:

$$Q = C_d A \sqrt{2gH}$$

Where C_d = coefficient of discharge whose value depends upon various factors such as type of gates & trash rack provided, conduit friction, transition etc...

H = differential head causing flow i.e. the difference of u/s & d/s water level

A = area of outlet sluice

Cont....

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In an outlet work the following hydraulic losses should be considered:-

- i) **Iterance loss**:- depends up on the shape & size of the entrance

It may be taken as $0.5 \frac{V^2}{2g}$ for square edged entrance and equal to

$0.04 \frac{V^2}{2g}$ for bell mouthed iterance

Where:- V is the flow velocity through the conduit.

- ii) **Friction Loss**: - depends on the surface of the conduit, its size & length and velocity

$$hl = f \frac{L}{D} \frac{V^2}{2g}$$

Where: - f= friction factor

L = length

D= diameter

Cont....

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- iii) **The head loss through the gate** : depends up on the type of gate & valve provided
- A loss of about $0.2 \frac{V^2}{2g}$ may be taken for fully open gate & butterfly valve
 - Head loss is taken as nil for ring follower gates
- iv) **The head loss through the trash rack** depends up on the design of the trash rack and the velocity through it

Velocity in trash rack(m/s)	Head loss in meters
0.15	0.006
0.3	0.03
0.45	0.09
0.62	0.15

The net effective head which is responsible for flow, should be taken as $H_{\text{eff}} = \text{differential head (H)} - \text{head loss}$

Then the discharge is calculated as

$$Q = A \sqrt{2 g H_{\text{eff}}}$$

Cont....

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Intake structure

- An intake structure is required at the entrance of an outlet conduit when the outlet is not an integral part of the dam.
- An independent intake structure is constructed through which the water is withdrawn from the river (or the reservoir).
- The primary function of the intake structure is to permit withdrawal of water from the reservoir (or river) over a predetermined range of reservoir levels and thus to protect the conduit from being damaged or clogged by ice, trash, debris, waves etc.

Cont....

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- An intake structure may vary from a simple concrete blocks supporting the end of the conduit pipe to huge concrete towers, depending up on the various factors such as reservoir characteristics, capacity and discharge requirements, climatic conditions etc.

Types of intake structures

i) Simple submerged intakes it consists of a simple concrete blocks or a rock filled timber crib supporting the starting end of the withdrawal pipes as shown below

Cont....

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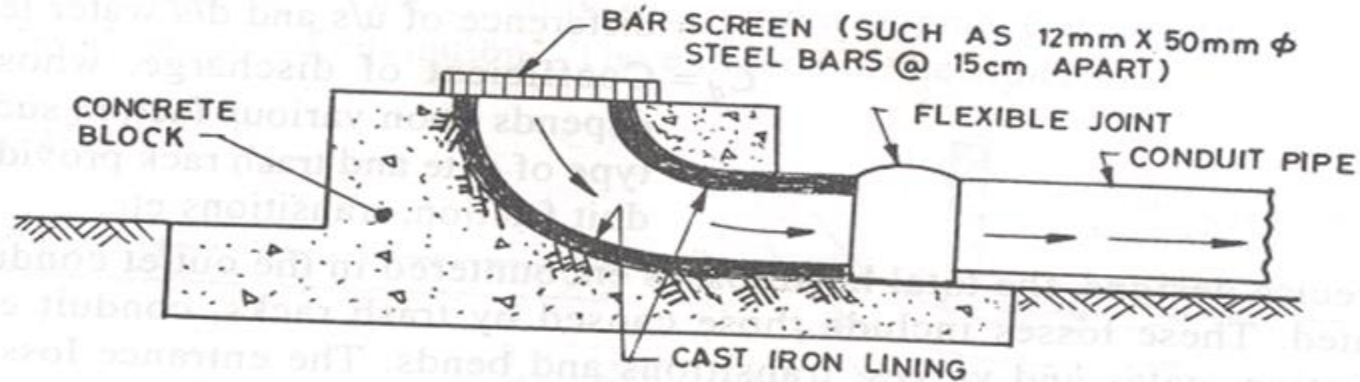


Fig Simple concrete block submerged intake

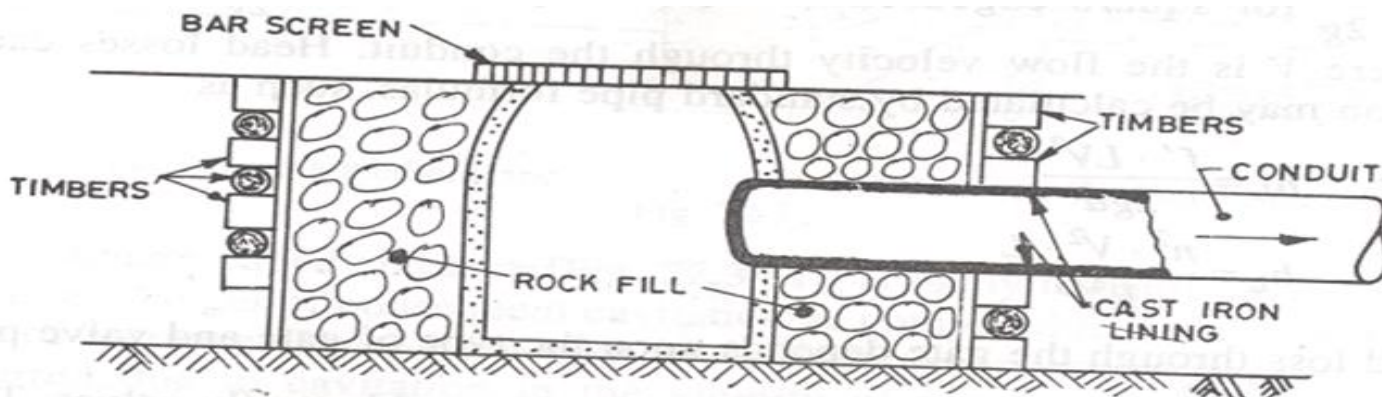


Fig Rock filled timber crib submerged intake

Cont....

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ii) Intake Towers

- Intake towers are generally used on large projects and where there are large fluctuations of water level.
- Openings at various levels called ports are generally provided in these concrete towers, which may help in regulating the flow through the towers and permits some selection of the quantity of water to be withdrawn.
- There are two major types of intake towers, viz.
 - i) Wet intake towers
 - ii) Dry intake towers

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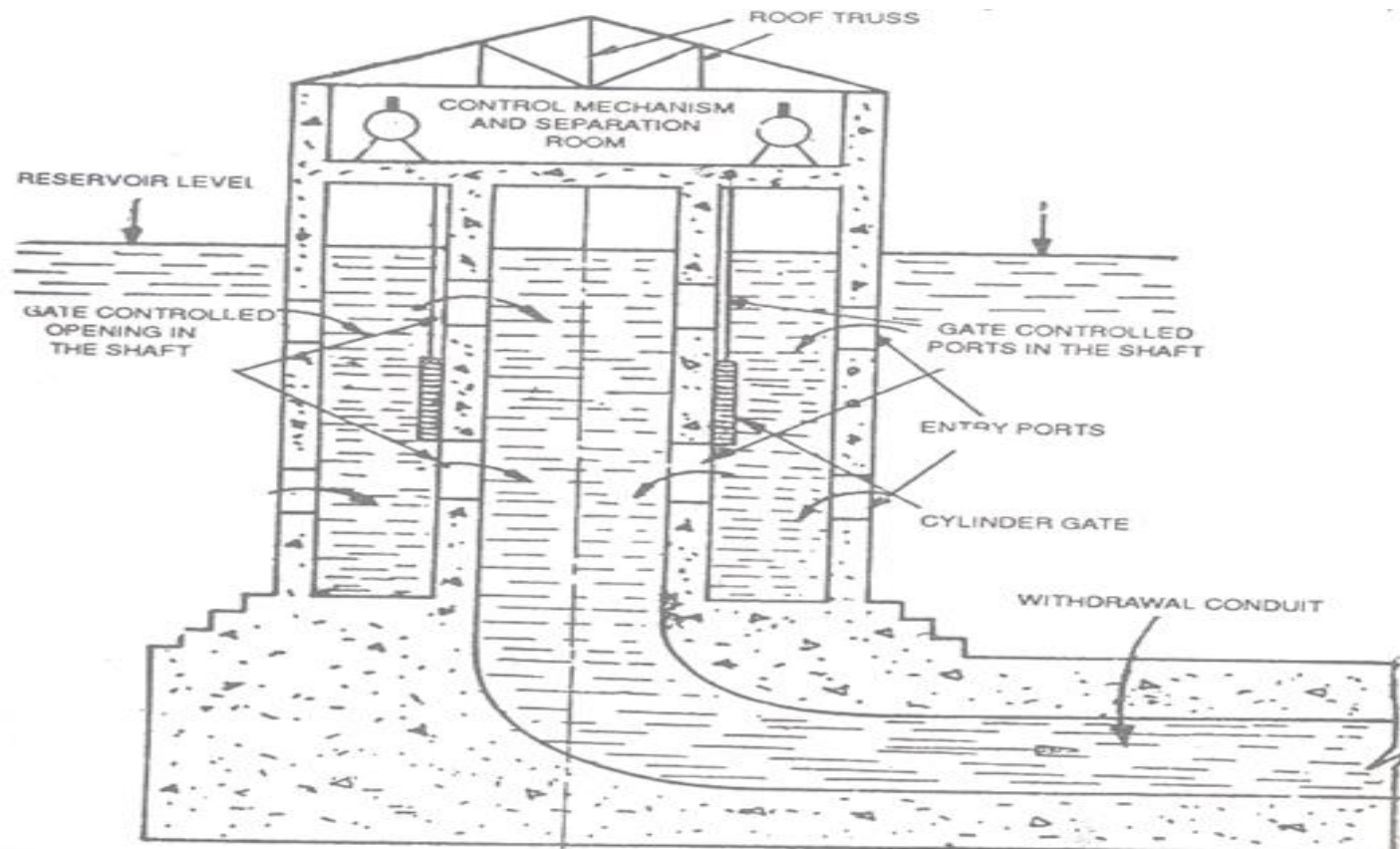
94

i) Wet intake tower

- It consists of a concrete cylindrical well filled with water to the level of the reservoir.
- There is a vertical shaft inside the well which is connected to the withdrawal conduit.
- The water enters the well through entry ports. It then enters the vertical shaft through gate-controlled ports(openings).
- A control room is usually constructed over the wet intake tower.

Cont....

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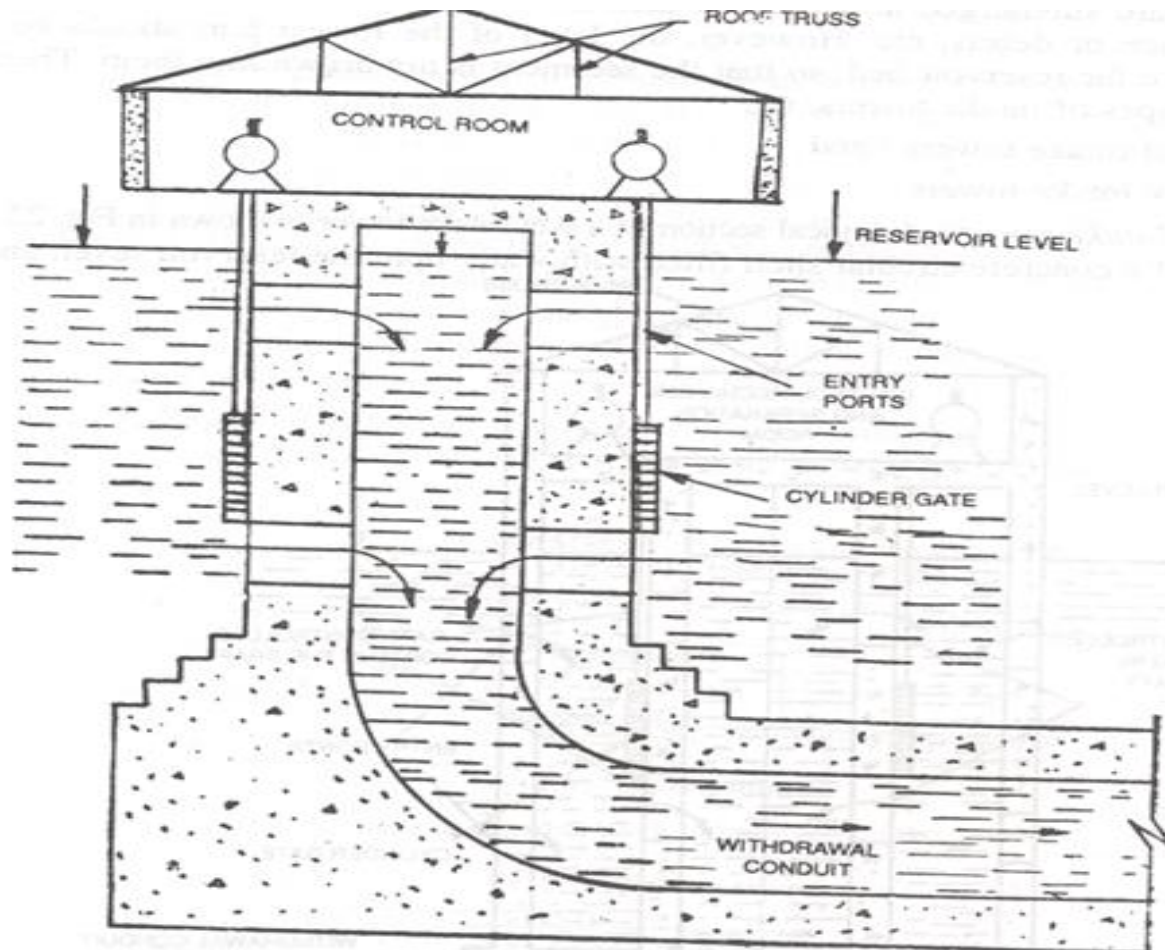
96

Dry intake tower

- This also consist of a concrete cylindrical well. However, in this case, the water is not filled in the well, but it is directly drawn into the conduits through the gated entry ports.
- Thus in a dry –intake tower, there is no water inside the tower, whereas in a wet-intake tower there will be water even when the ports gates are closed.

Cont....

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Cont....

98

- Intake towers are huge structures standing in the river (reservoir), and hence should be located so as not to interfere with navigation,
- and must be properly designed so as to withstand the worst possible combination of various forces, such as hydrostatic pressure, wind and earth quake forces, and forces caused by waves, ice and debris etc..

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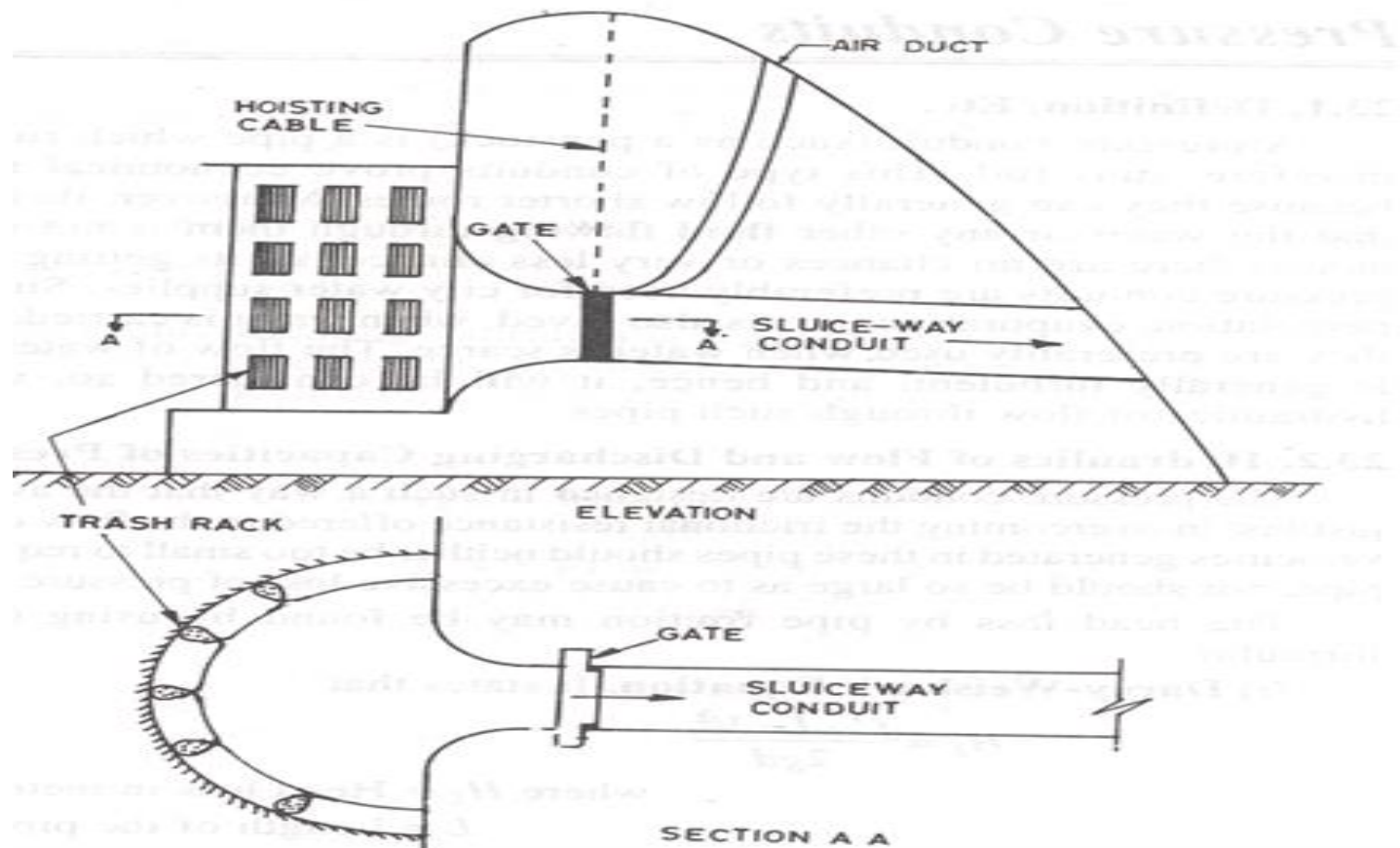
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Trash racks

- The entrance to intakes and dam outlets are generally covered with trash racks so as to prevent the entry of debris, ice etc in to the conduit.
- These racks are generally bar screens, made from steel bars spaced at 5 to 15 cm center to center in both directions depending upon the maximum size of the debris required to be excluded from entering the conduit.
- The velocity of flow through the trash rack is kept low (generally less than 0.62m/s) so as to minimize losses.

Cont....

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Half Cylinder-trash Rack and Tractor Gate installation

THANK YOU