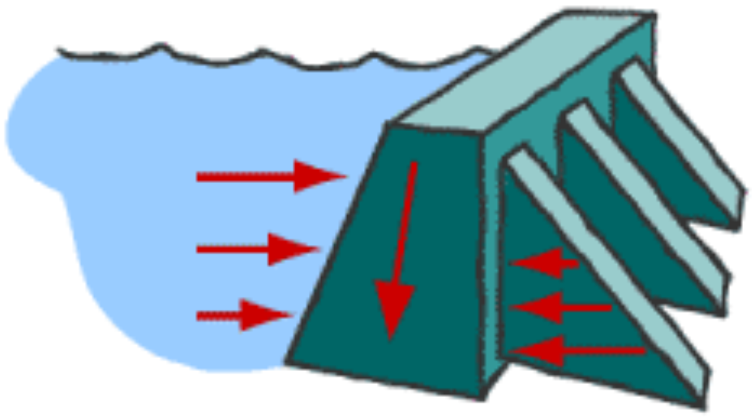
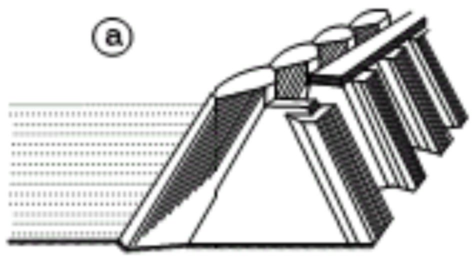
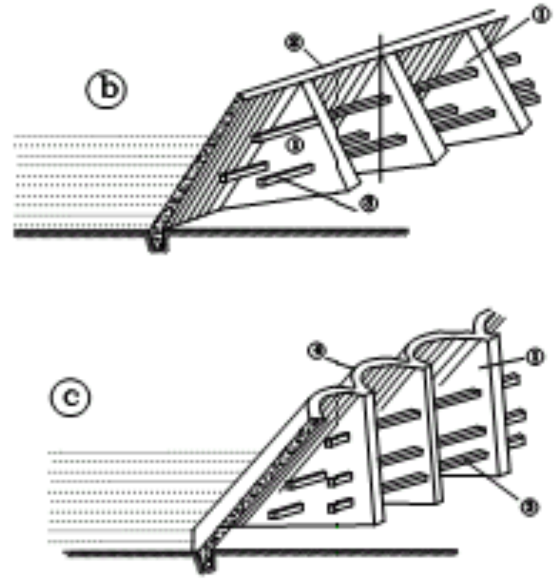
3.7 Buttress Dams



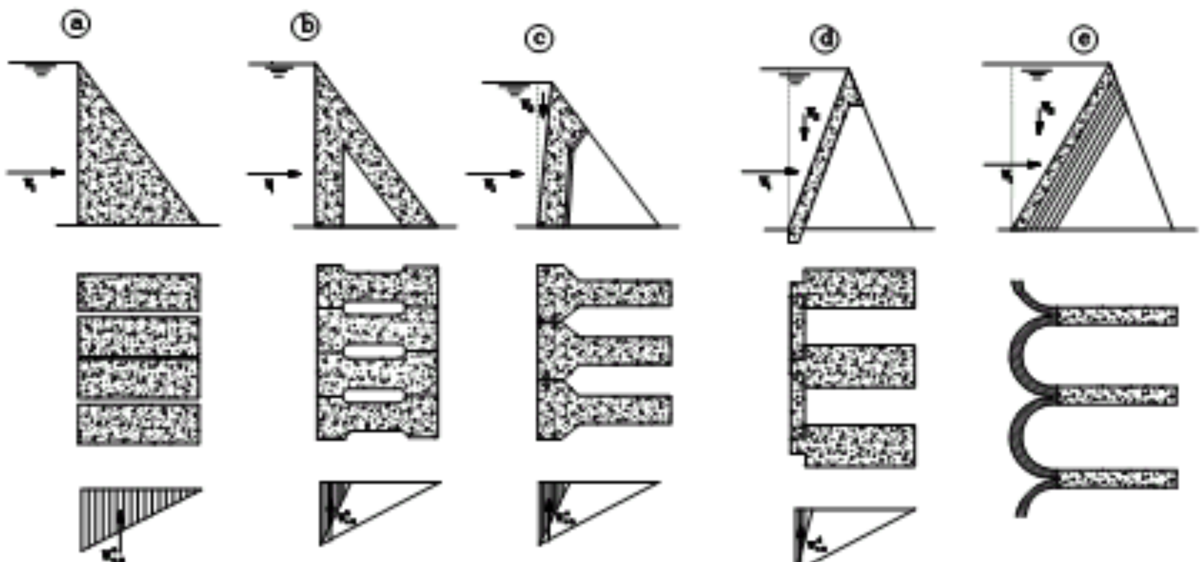
Sloping slab which transmits the water trust to a series of buttress at right angles to the axis of the slab

This type of structure can be considered even if the foundation rocks are little weaker.





Typical sections of Buttress dams



Buttress Dams

- ✓ made of concrete reinforced with steel.
- ✓ typically spaced across the dam site every 6 to 30 metre
- ✓ sometimes called hollow dams
- ✓ require less concrete than gravity dams
- ✓ but not necessarily less expensive to build.

- ❑ Costs associated with the complex work of forming the buttresses or multiple arches may offset the savings in construction materials.

- ❑ Buttress dams may be desirable, however, in locations with foundations that would not easily support the massive size and weight of gravity dams.

3.8 Spillways

- Structural component of the dam that evacuates flood wave
- Safety valve of the dam
- *DESIGN RETURN PERIOD*:

From 100-year for a diversion weir to 15,000-year or more (Probable Maximum Flood-PMF) for earth-fill dams

3.8.1 Types of Spillways

- More common types are:

  (1) Overflow (Ogee crested)
  (2) Chute
  (3) Side Channel
  (4) Shaft
  (5) Siphon

- Most spillways are of overflow types due to its large capacity and high adaptability.

3.8.1.1 Overflow Spillways

→ Allows the passage of flood wave over its crest

→ Used on often concrete gravity, arch & buttress dams

→ Constructed as a separate reinforced concrete structure at one side of the fill-type dams

→ Classified as uncontrolled (ungated) & controlled (gated).

Hinze dam (Gold Coast Qld, Australia)

Ogee Spillways

Ideal Spillway Shape:

The underside of the nappe of a sharp-crested weir when

$$Q = Q_{max}$$

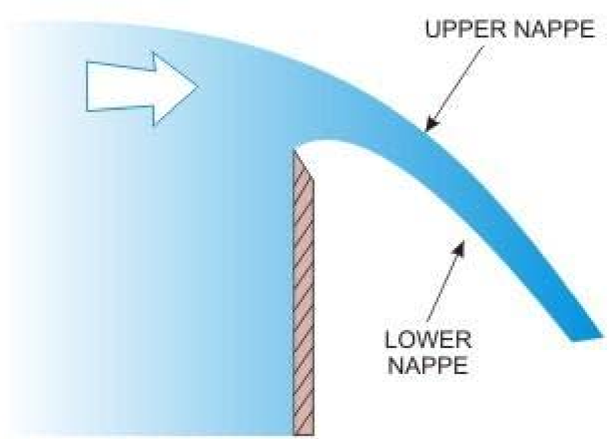
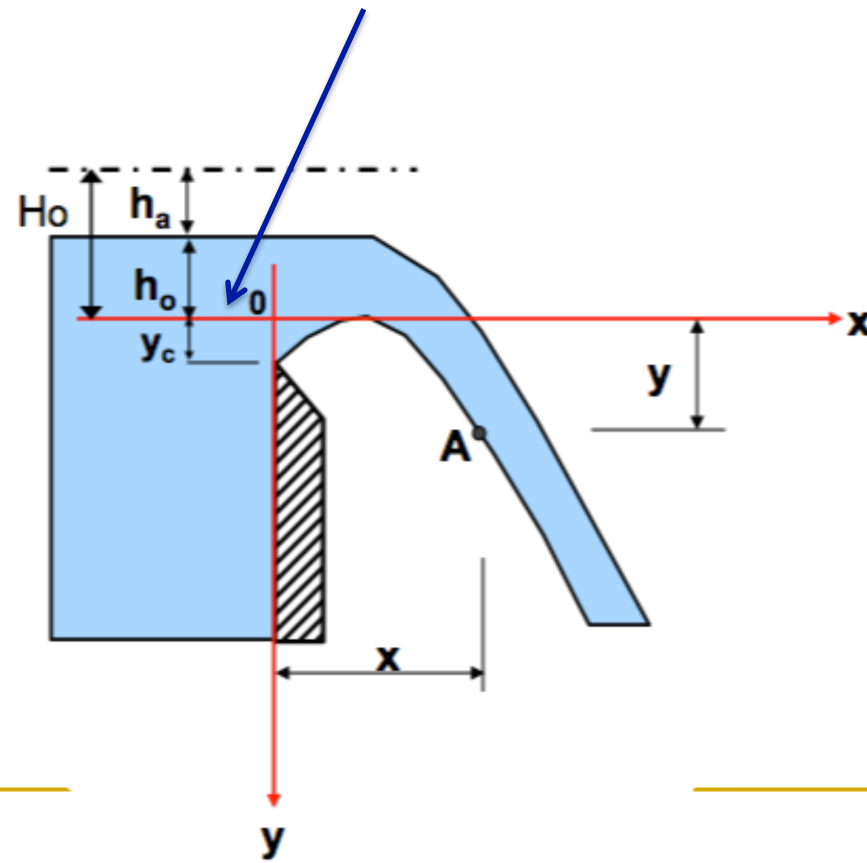


FIGURE 6. Outflow from a free-falling weir , properly ventilated from below



Ogee Spillways

A- Design Discharge of Spillway:

- If crest is uncontrolled or gates are fully opened (integrating velocity distribution):

$$Q_o = C_o \, L \, H_o^{3/2}$$

C_o: Discharge Coefficient

L: Effective Crest Length

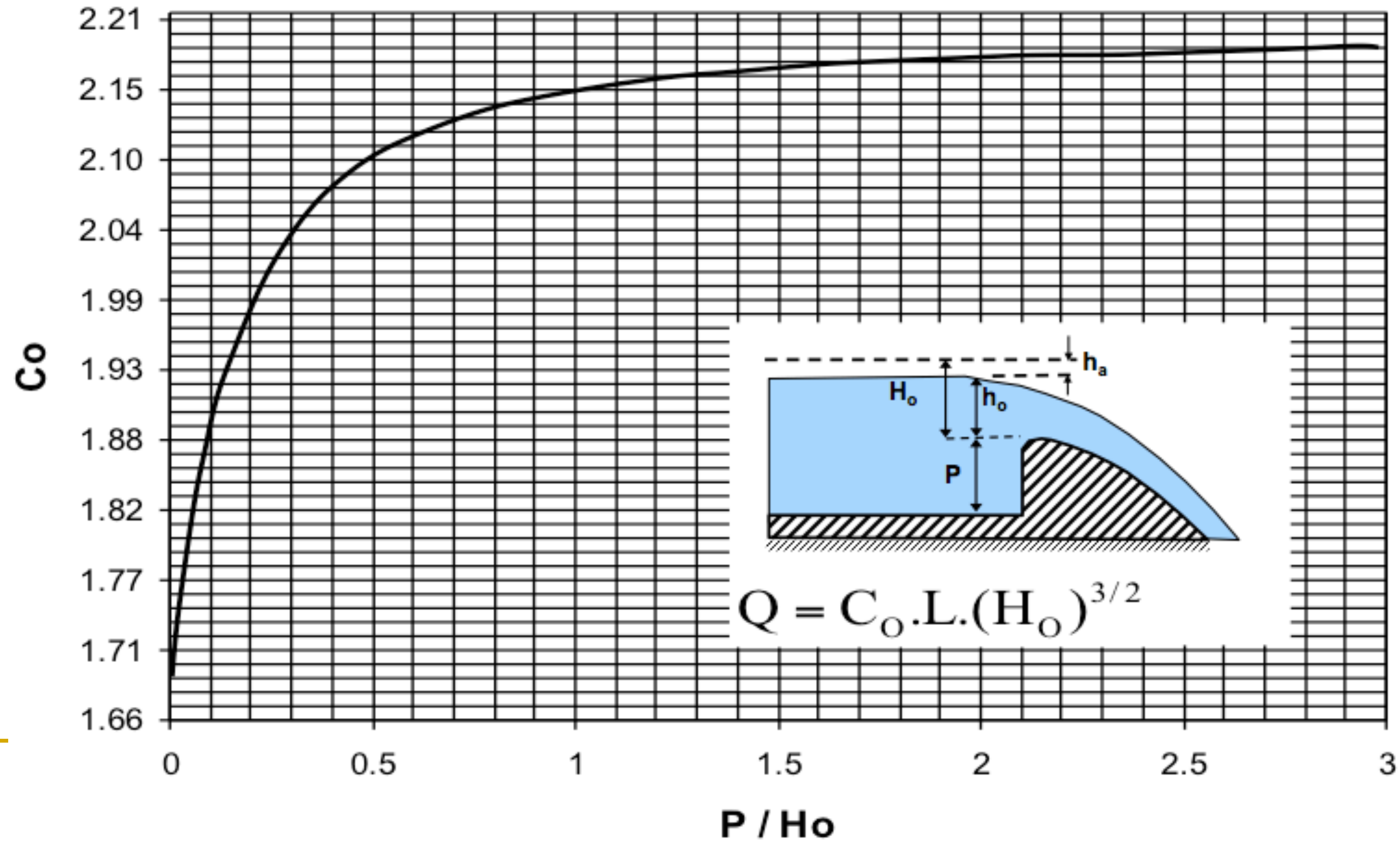
H_o: Total Head $H_0 = h_o + h_a$ over spillway crest

$h_a = u_0^2/2g$ (Approaching velocity head)

Ogee Spillways

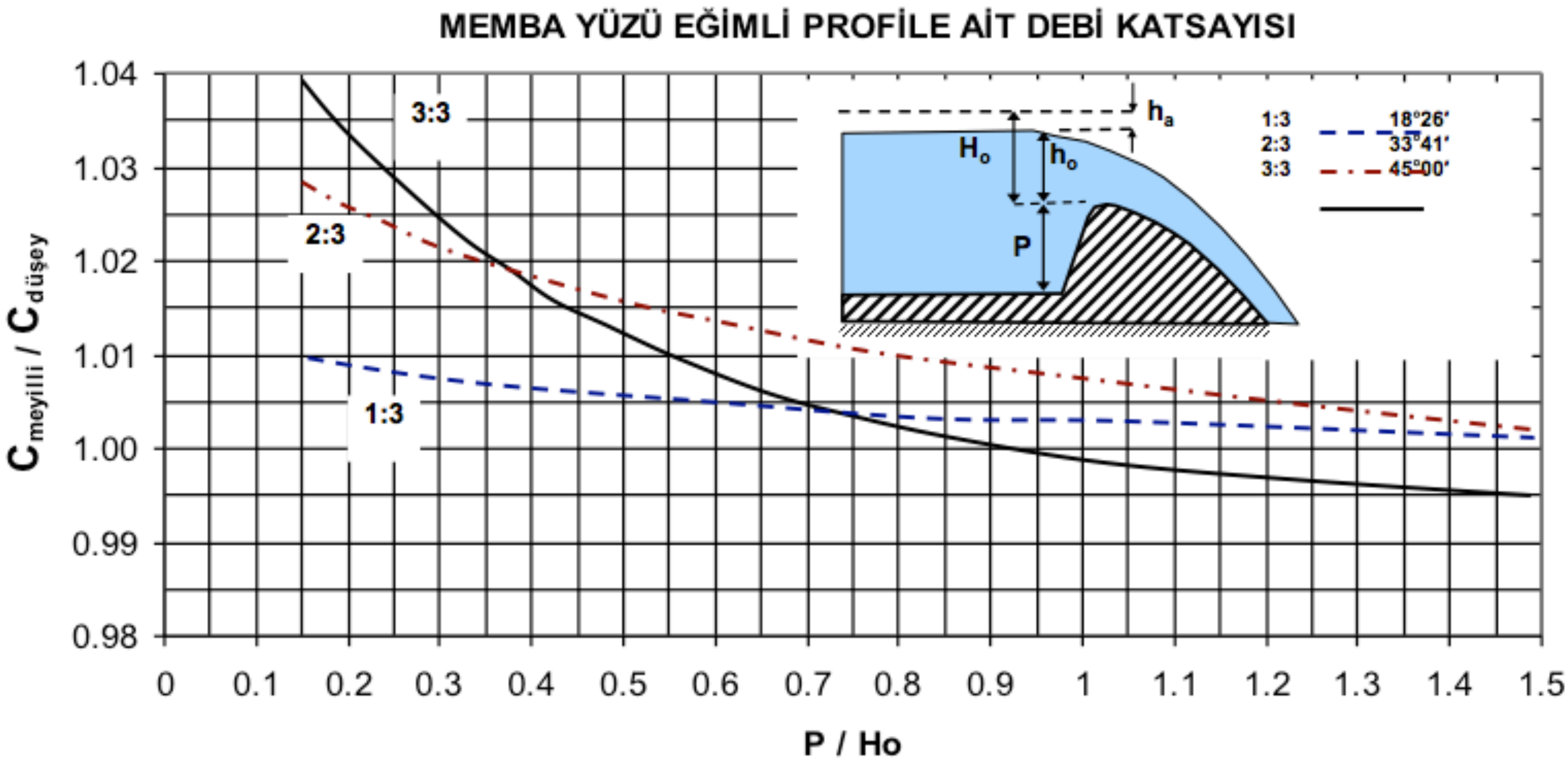
C_o (*Discharge Coefficient*)**:** Determined from Fig. 2.15 for the vertical overflow spillways as a function of P (spillway height) / H_o (total head)

► The overall C_o → multiplying each effect of each case below



Ogee Spillways

Coefficient of discharge for sloping upstream face



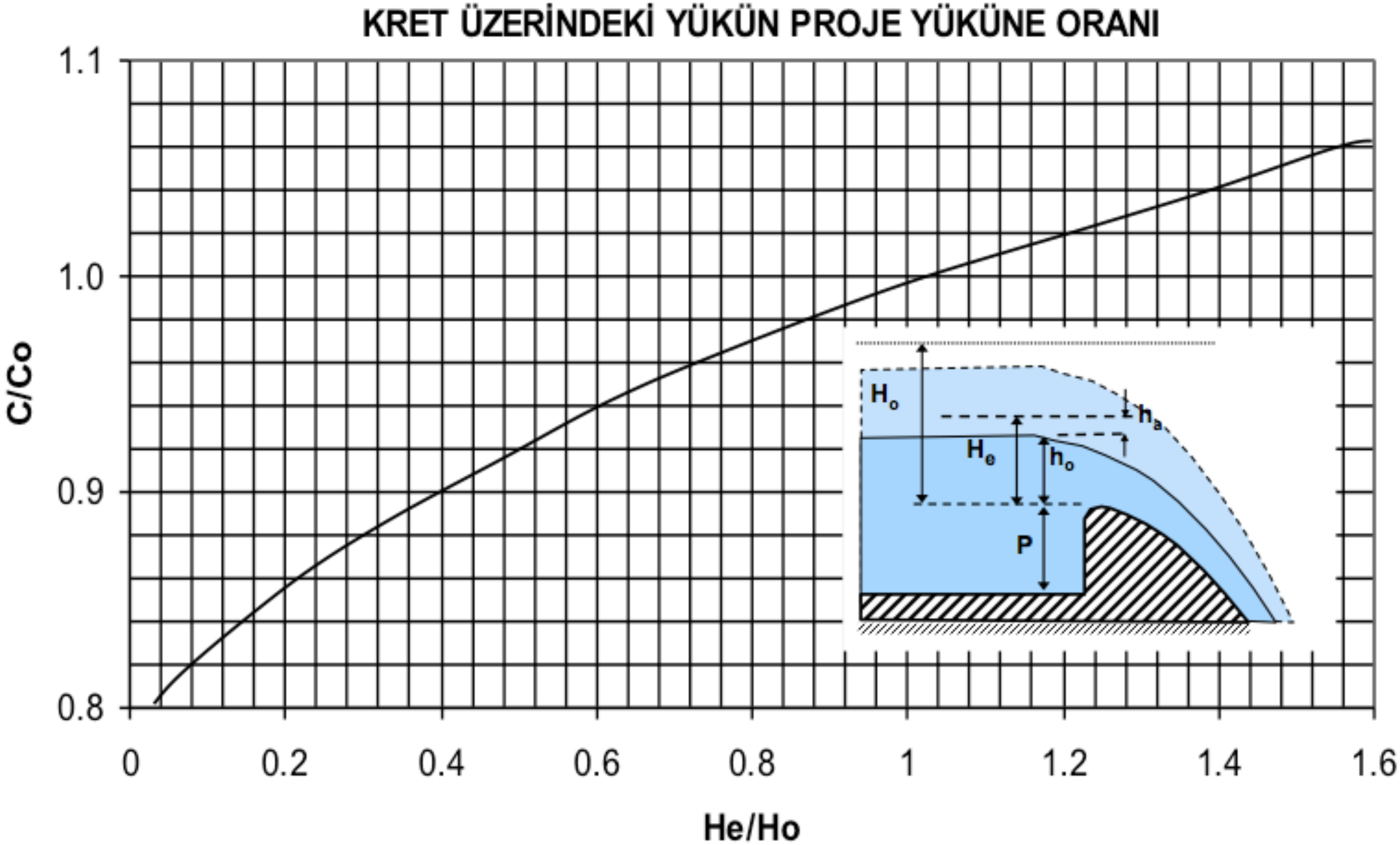
Ogee Spillways

Spillways are seldom operated with their design heads since the design head corresponds to high return periods

→ discharge coefficient for an existing total operating head (He) needs to be determined

Ogee Spillways

Coefficient of discharge for heads other than design head

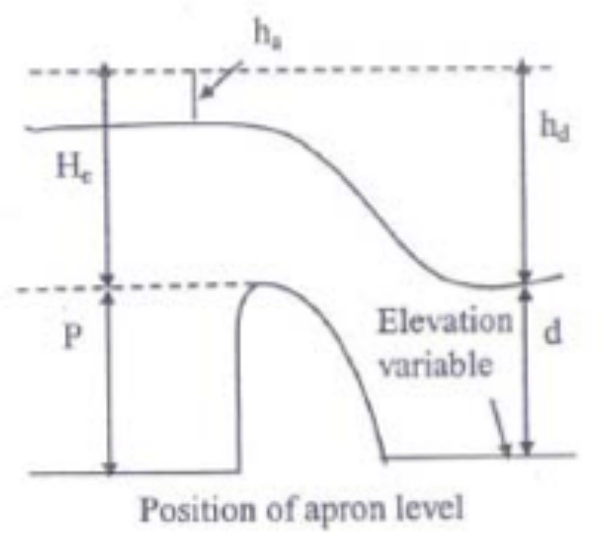


Ogee Spillways

⦿ For low spillways, (spillways of diversion weirs) the level of apron and submergence would also affect the flow conditions.

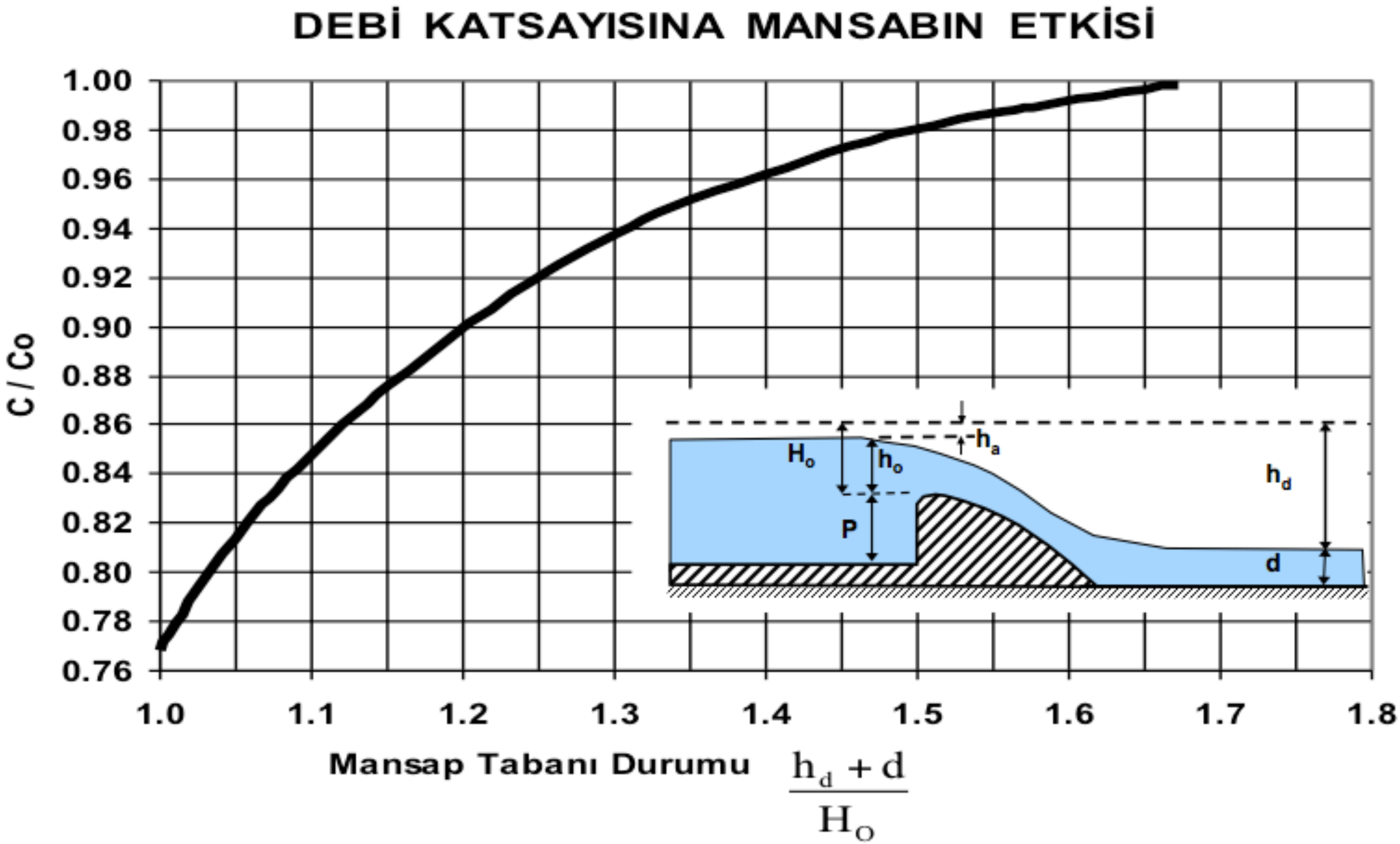
⦿ For a given fixed upstream energy level, the elevation of the apron has a direct influence on the total head available at the downstream.

⦿ The lower the apron elevation, the greater the total available head at the downstream and hence greater discharge coef.



Ogee Spillways

Ratio of discharge coefficient due to apron effect



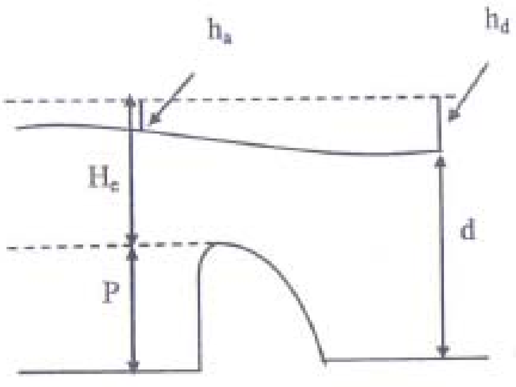
Ogee Spillways

⊙ Submergence imposes a retarding effect to the approaching flow because of lowered available head between the upstream and downstream.

⊙ Therefore, the spillway discharge coefficient for a submerge case decreases as the submergence is pronounced.

⊙ However, submergence is only critical for low spillways.

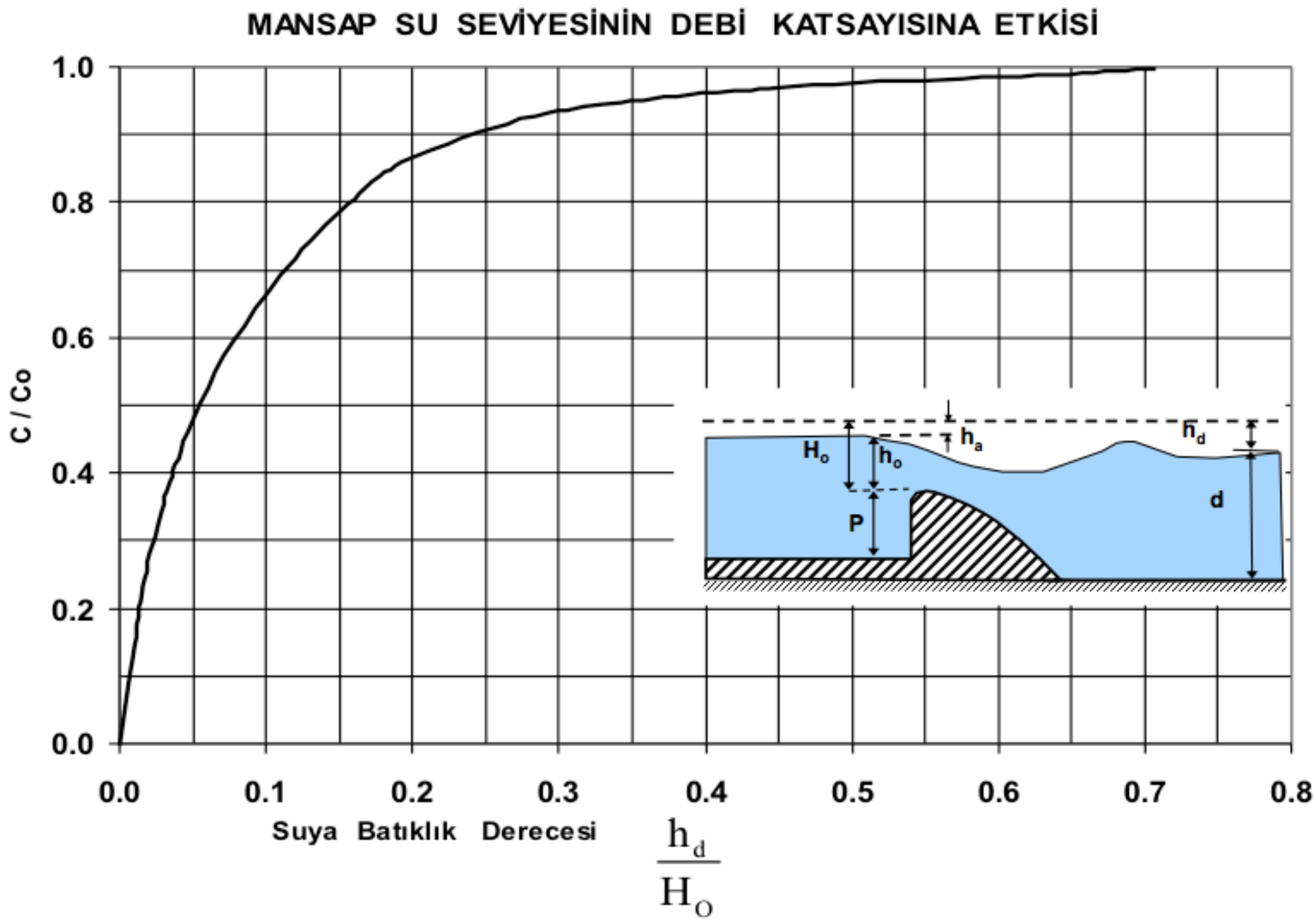
❖ **Overall spillway discharge coef is obtained by multiplying the effects of each aforementioned case.**



Submergence effect

Ogee Spillways

Ratio of discharge coefficient due to tailwater effect



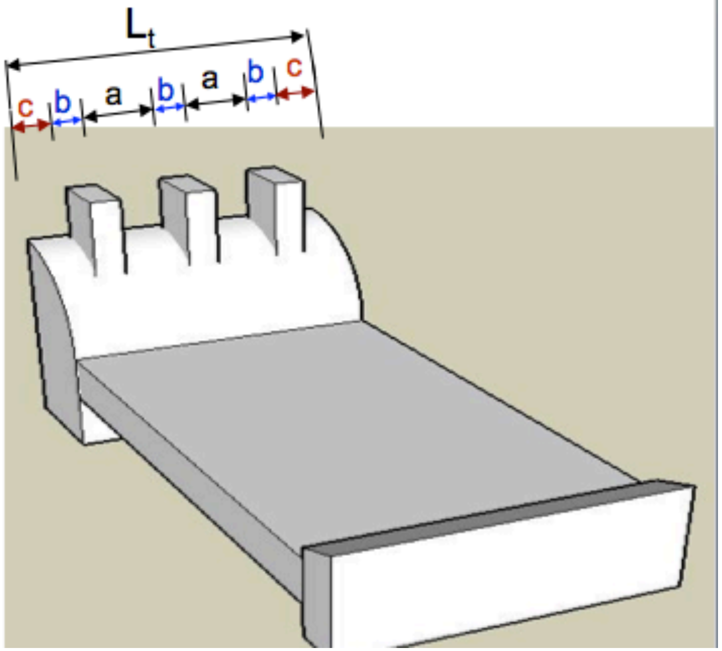
Ogee Spillways

L: Effective Crest Length

$$L_1 = 2 \times c + (N-1) \times a$$

$$L_1 = L_t - N \times b$$

$$L = L_1 - 2 \times (N \times K_p + K_a) \times H_o$$

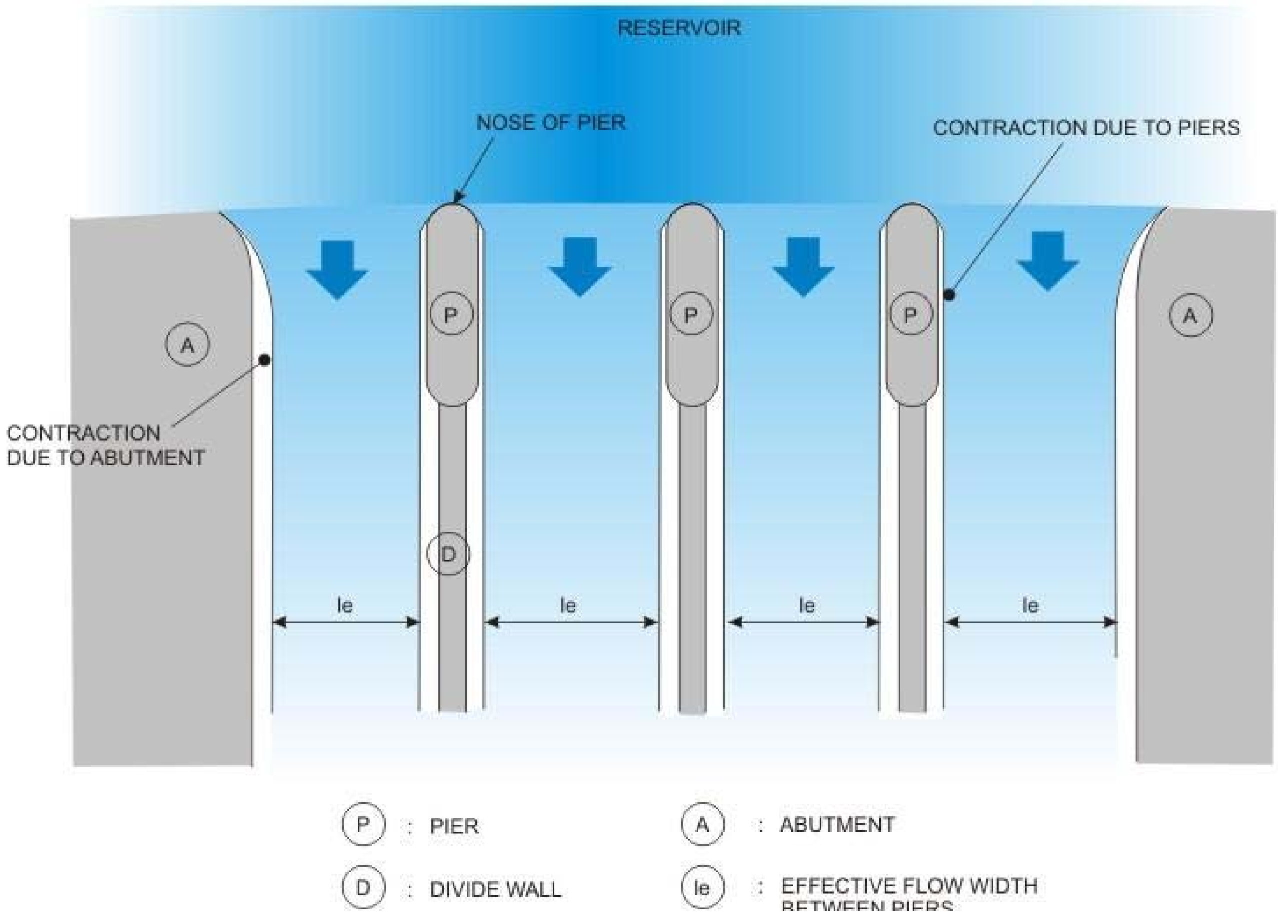


L = Etkili kret uzunluğu (m)
L_t = Toplam kret uzunluğu (m)
L_1 = Net kret uzunluğu (m)
N = orta ayak adedi
Kp = Orta ayaklara ait büzülme katsayısı
Ka = Kenar ayaklara ait büzülme katsayısı
Ho = Toplam proje yükü (m)

Ogee Spillways

L: Effective Crest Length

Reason for the reduction of the net length may be appreciated from:



Ogee Spillways

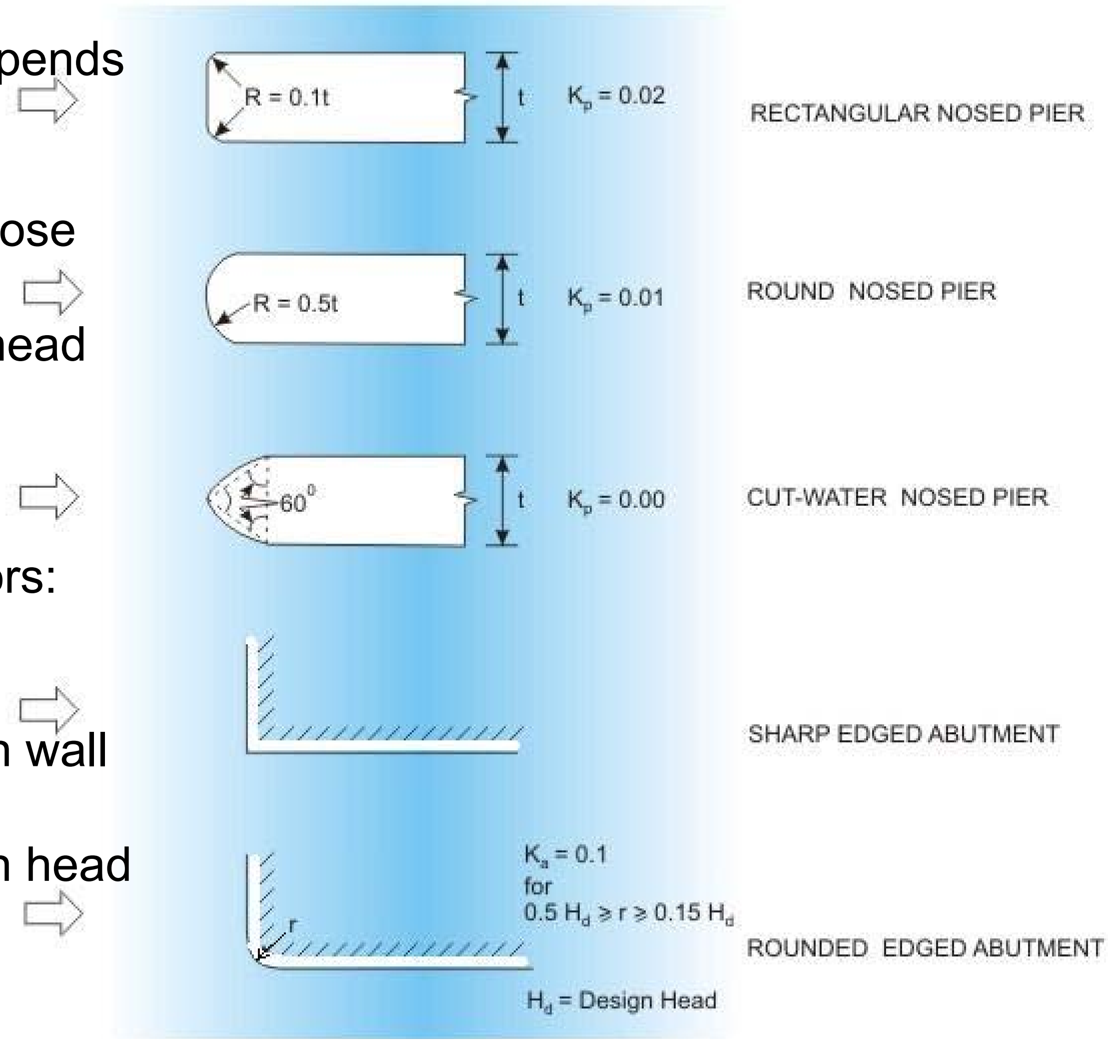
L: Effective Crest Length

Pier contraction coefficient K_p depends upon following factors:

1.Shape & location of the pier nose
2. Thickness of the pier
3.Head in relation to the design head
4. Approach velocity

Abutment contraction coefficient depends upon the following factors:

1. Shape of abutment
2. Angle btw upstream approach wall & the axis of flow
3. Head, in relation to the design head
4. Approach velocity



Ogee Spillways

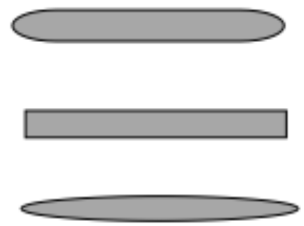
L: Effective Crest Length

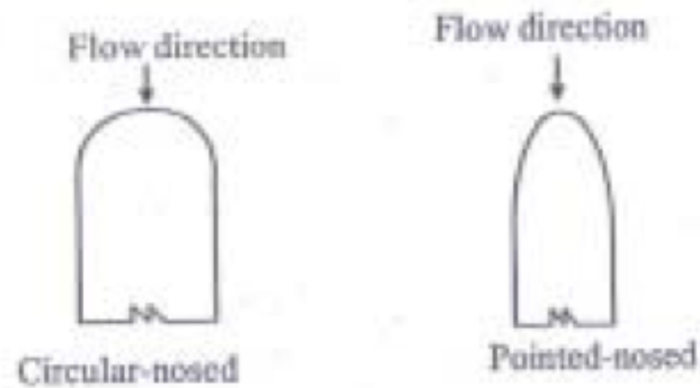
Kp

0.035~0.01

0.1

0.00





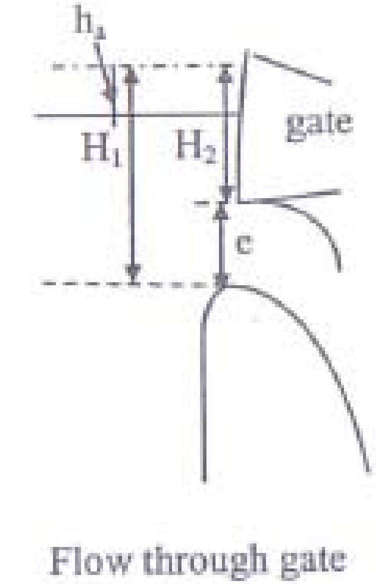
Coefficient	Value	Description
K_p	0.02	Square nosed piers with corners rounded by r=0.1H
	0.01	Rounded nosed piers
	0	Pointed nosed piers
K_a	0.20	Square abutments with head wall 90° to the direction of flow
	0.10	Rounded abutments with head wall 90° to the direction of flow when $0.1H_d<r<0.15H_d$
	0	Rounded abutments where $r > 0.5H_d$ and head wall is placed not more than 45° to the direction of flow

Ogee Spillways

B- Design Discharge of Spillway:

♦ If the gates are partially opened:

$$Q = 2/3\ (2g)^{0.5}\ C\ L\ (H_1^{3/2} - H_2^{3/2})$$



C: Discharge Coefficient (determined from Fig. 2.20)

L: Effective Crest Length

H_1 & H_2: Heads (see the Fig. 2.20 for definition)

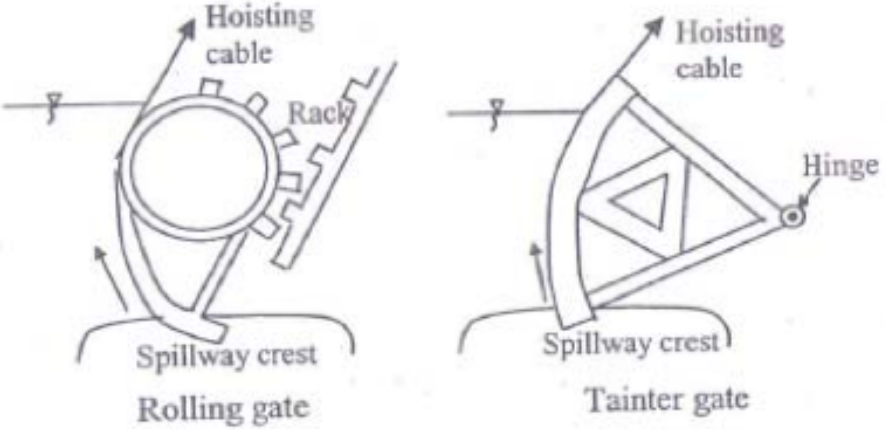
3.8 Spillway Crest Gates

Provide additional storage above the crest.

→ See Fig. 2.21 for Primitive types of gates.
→ See Fig. 2.22 for Underflow gates.

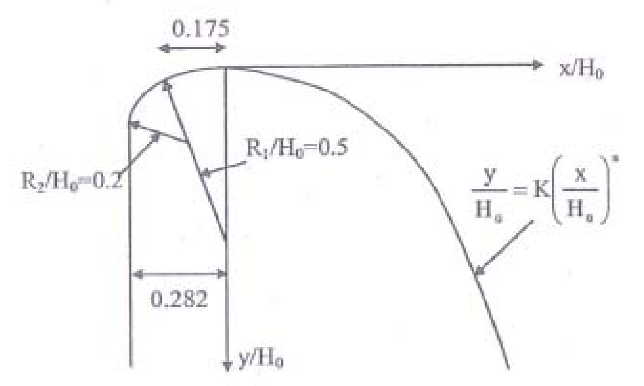
Common types:

- Radial gates (easy operation & small friction)
- Rolling drum gates
- Vertical lift gates



3.8 Spillway Crest Profiles

- The standard overflow spillway crest profile for a vertical upstream face is recommended by USBR (1987).
- $K \approx 0.5$ and $n \approx 1.85$
- If the head on the spillway is greater than H_0, the pressure over the spillway face may drop below the atmospheric pressure and separation and cavitation may occur.
- The upstream face of the crest is formed by smooth curves in order to minimize the separation and inhabit the cavitation.



Standard crest profile of an overflow spillway (USBR,1987)

3.8 Spillway Crest Profiles

- Hidrolik Profil

•Creager Profili
•Ogee Profili

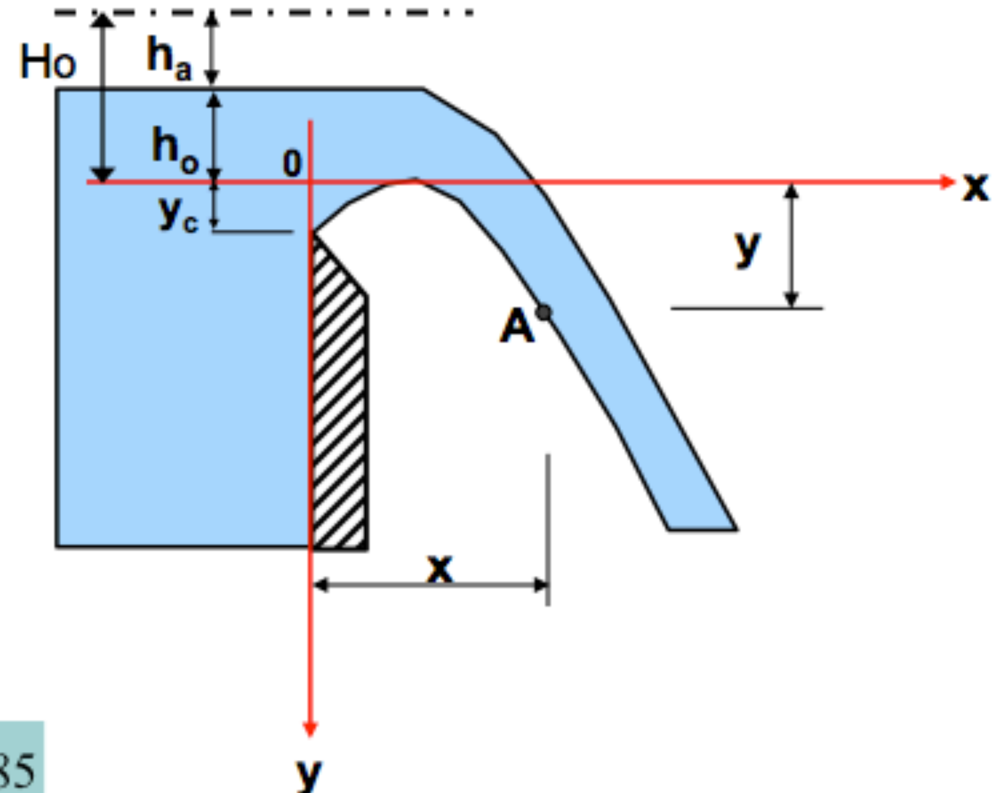
$$y = k.H_o^{1-n}.x^n$$

k = 0.461 ~ 0.556
n = 1.80 ~ 2.00

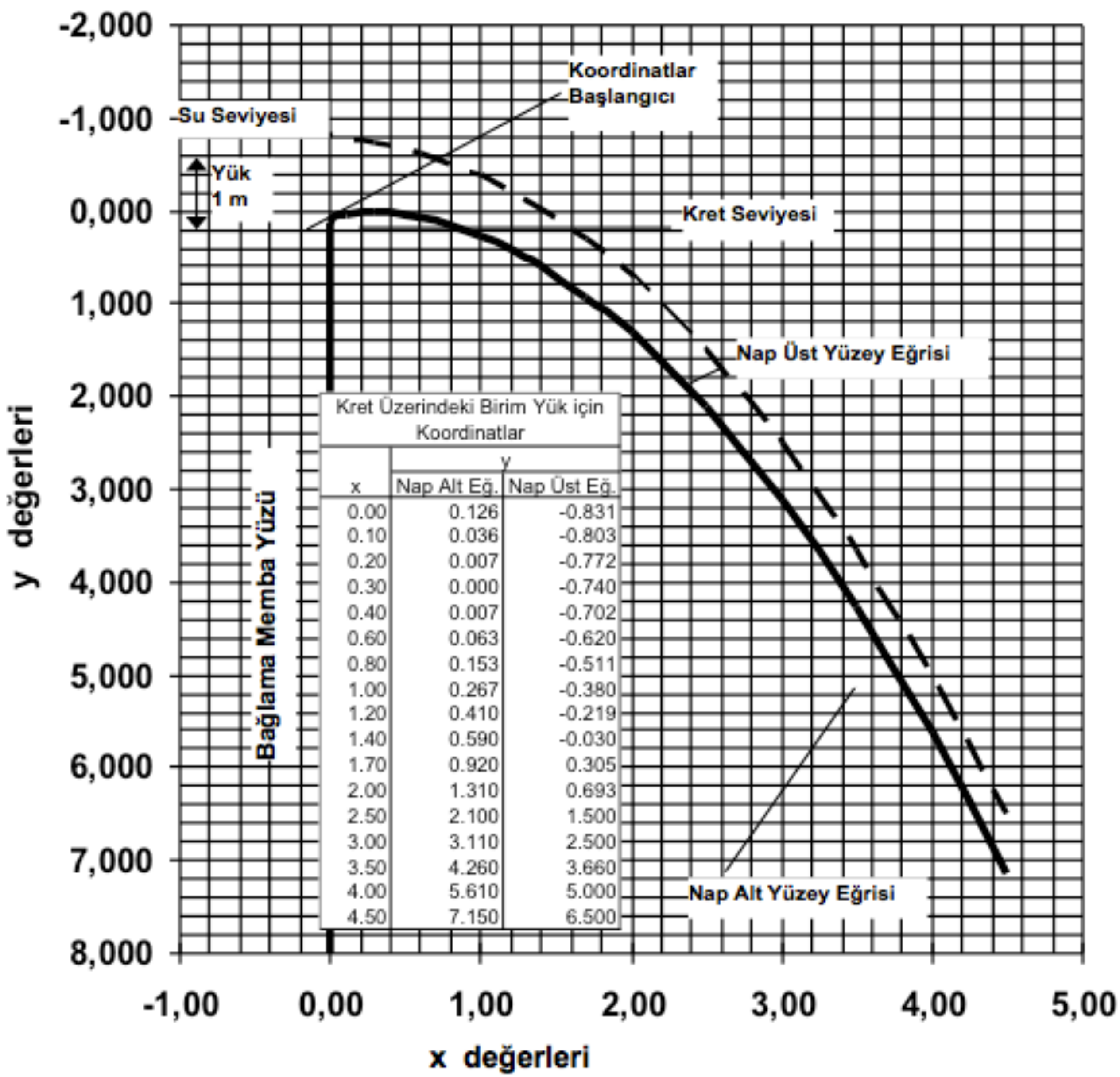
Creager' de
k = 0.47
n = 1.80

Ogee profili

$$y = \frac{0.50}{H_o^{0.85}}.x^{1.85}$$



3.8 Spillway Crest Profiles

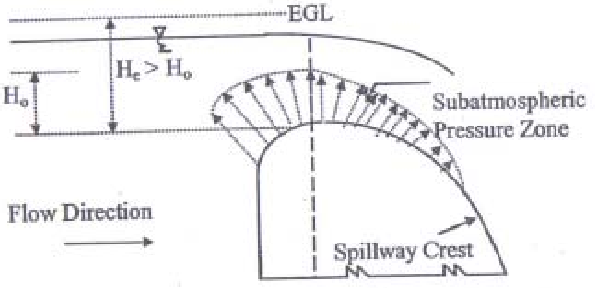


Kret Üzerindeki Birim Yük için Koordinatlar

x	y	
	Nap Alt Eğ.	Nap Üst Eğ.
0.00	0.126	-0.831
0.10	0.036	-0.803
0.20	0.007	-0.772
0.30	0.000	-0.740
0.40	0.007	-0.702
0.60	0.063	-0.620
0.80	0.153	-0.511
1.00	0.267	-0.380
1.20	0.410	-0.219
1.40	0.590	-0.030
1.70	0.920	0.305
2.00	1.310	0.693
2.50	2.100	1.500
3.00	3.110	2.500
3.50	4.260	3.660
4.00	5.610	5.000
4.50	7.150	6.500

3.8 Spillway Crest Profiles

- The shape of the crest as well as the approach flow characteristics are important for the bottom pressure distribution of the spillway face.
- At the crest of the spillway, the streamlines have a curvature.
- For heads less than the design head, $H_e<H_0$,
  - the curvature of streamlines is small and
  - the pressure over the spillway crest is greater than atmospheric pressure but still less than hydrostatic pressure.
- When the curvature is large enough under a high head $H_e>H_0$ over the crest, internal pressure may drop below the atmospheric pressure.
- With the reduced pressure over the spillway crest for $H_e>H_0$, overflowing water may break the contact with the spillway face, which results in the formation of vacuum at the point of separation and cavitation may occur.

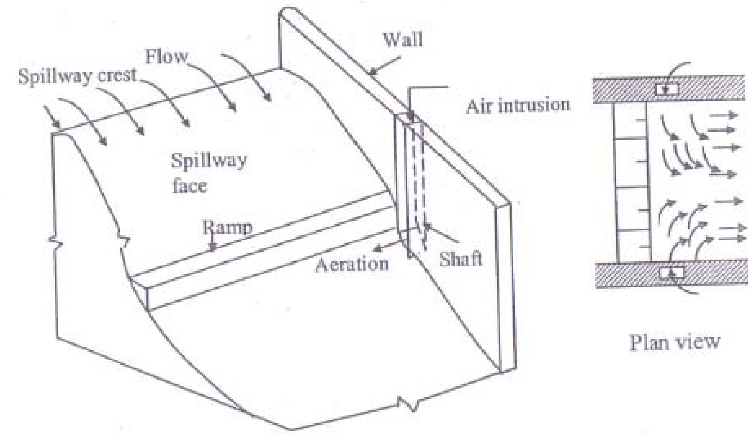


Development of negative pressure at the spillway crest for $H_e>H_0$

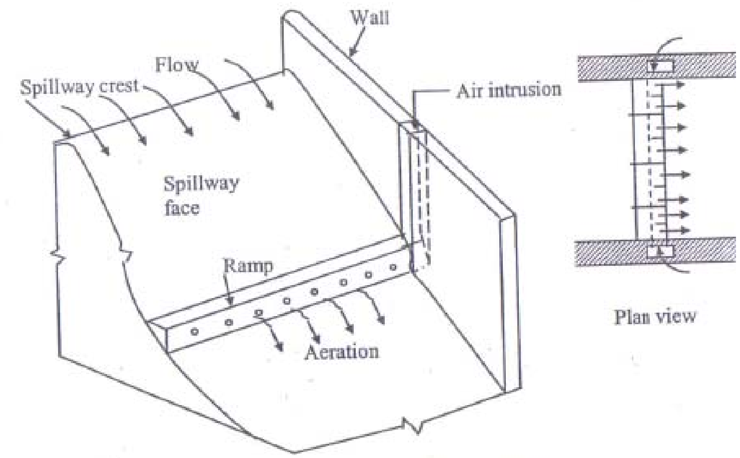
3.8 Spillway Crest Profiles

Spillway Crest Profile

- To prevent cavitation, sets of ramps are placed on the face of overflow spillways such that the jet leaves the contact with the surface.
- Ramps are provided at locations where the natural surface air entrainment does not suffice for the concrete protection against cavitation.
- Air is then introduced by suction into the nappe created by the ramp through vertical shafts to increase the negative pressure to atmospheric pressure.



a) Chute aeration without distribution duct



b) Chute aeration with distribution duct

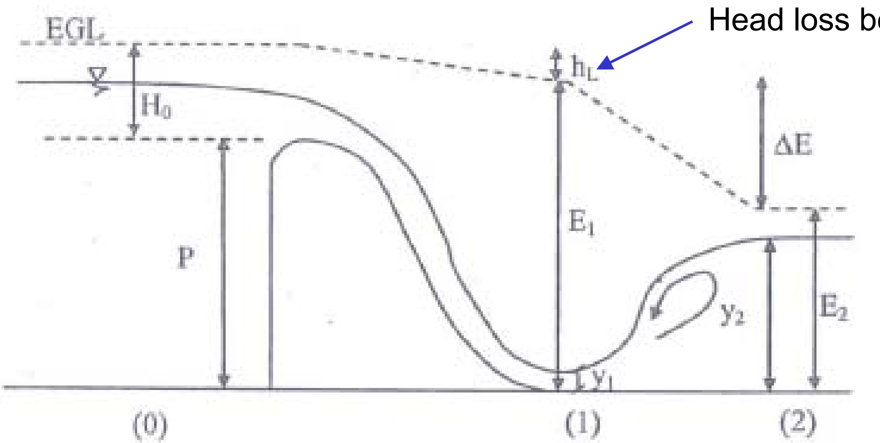
3.8 Spillway Crest Profiles

ATATURK DAM

Overflow Spillway

Energy Dissipation at the Toe of Overflow Spillway

- Excessive turbulent energy at the toe of an overflow spillway can be dissipated by the hydraulic jump.
- To protect the streambed, a **stilling basin (energy dissipation basin)** having a thick mat foundation (**apron**) may be formed.
- Energy equation between section (0) and (1)



Flow condition at the toe of an overflow spillway

$$P + H_0 = y_1 + \frac{u_1^2}{2g} + h_L$$

$h_L \approx 0.1 u_1^2/(2g)$

y_1=?

$$P + H_0 = y_1 + 1.1\frac{u_1^2}{2g} = y_1 + 1.1\frac{q^2}{2gy_1^2}$$

The sequent depth:

y_2=?

$$\frac{y_2}{y_1} = \frac{1}{2}\left(\sqrt{1+8F_{r1}^2} - 1\right) = \frac{1}{2}\left(\sqrt{1+\frac{8q^2}{gy_1^3}} - 1\right)$$

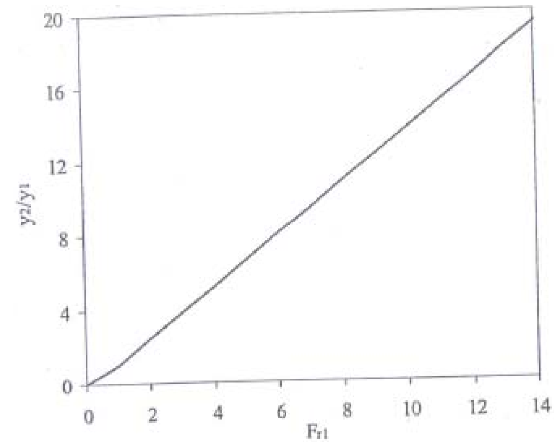
Energy Dissipation at the Toe of Overflow Spillway

- The strength of the hydraulic jump is measured by the depth ratio, y_2/y_1.
- As the depth ratio increases, the hydraulic jump becomes stronger.
- For $F_{r1}>2$,

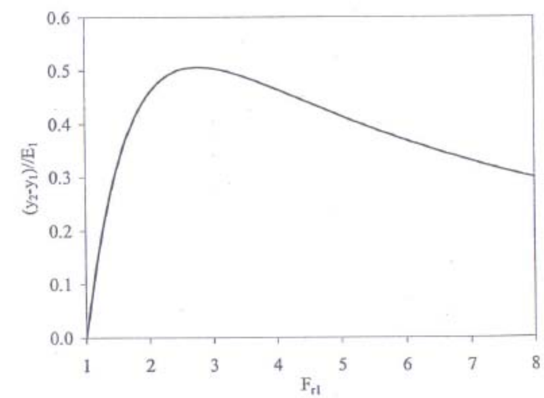
$$\frac{y_2}{y_1}=\frac{1}{2}\left(\sqrt{1+8F_{r1}^2}-1\right)=\frac{1}{2}\left(\sqrt{1+\frac{8q^2}{gy_1^3}}-1\right) \Rightarrow \frac{y_2}{y_1}=\sqrt{2}F_{r1}-\frac{1}{2}$$

- Dimensionless height of the jump $\Delta y=y_2-y_1$

$$\frac{\Delta y}{E_1}=\frac{\sqrt{1+8F_{r1}^2}-3}{F_{r1}^2+2}$$



Variation of depth ratio of the hydraulic jump against Froude number.



Variation of dimensionless height of the jump against Froude number.

Adapted from Lecture Notes of Dr. Bertuğ Akıntuğ
Middle East Technical University Northern Cyprus Campus

Energy Dissipation at the Toe of Overflow Spillway

- The energy loss through the hydraulic jump in a rectangular basin is given by

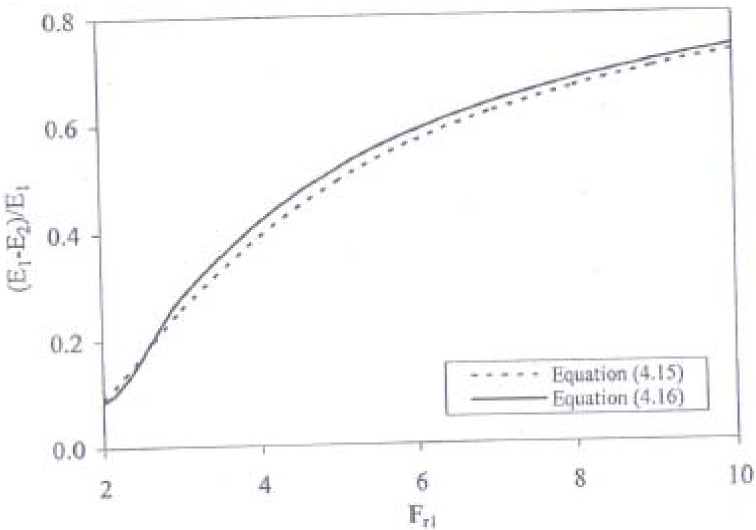
$$\Delta E = E_1 - E_2 = \frac{(y_2 - y_1)^3}{4 y_1 y_2} \qquad (4.14)$$

- Percent energy loss through the hydraulic jump in a rectangular stilling basin is

$$\frac{E_1 - E_2}{E_1} = \frac{\Delta E}{E_1} = 1 - \frac{(8F_{r1}^2 + 1)^{3/2} - 4F_{r1}^2 + 1}{8F_{r1}^2 (2 + F_{r1}^2)} \qquad (4.15)$$

- For $F_{r1}>2$, above equation can be simplifed to

$$\frac{\Delta E}{E_1} = \left(1 - \frac{\sqrt{2}}{F_{r1}}\right)^2 \qquad (4.16)$$



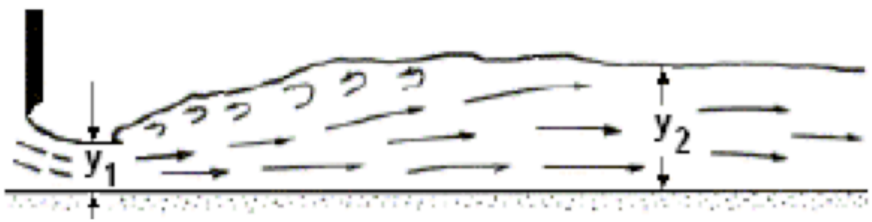
Variation of % energy loss against Froude number

Adapted from Lecture Notes of Dr. Bertuğ Akıntuğ
METU Northern Cyprus Campus

Energy Dissipation at the Toe of Overflow Spillway

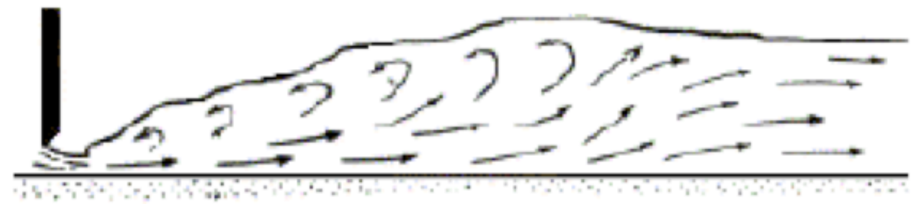
- Hydraulic jumps can be classified according to the value of F_{r1}.
  - For ($F_{r1} \leq 1.7$) → Undular jump
  - For ($1.7 < F_{r1} < 2.5$) → Prejump stage
  - For ($2.5 \leq F_{r1} < 4.5$) → Transition stage
  - For ($4.5 \leq F_{r1} < 9.0$) → Well-balanced jump
  - For ($F_{r1} > 9.0$) → Effective jump (highly rough downstream)

Energy Dissipation at the Toe of Overflow Spillway



$1.7 < Fr < 2.5$

$2.5 < Fr < 4.5$



$4.57 < Fr < 9.0$

$Fr > 9.0$

Energy Dissipation at the Toe of Overflow Spillway

- The location of the hydraulic jump is governed by the depth of tailwater.

Table 4.4 Selection criteria for the stilling basin.

Type of basin	F_{r1}	Limitations and characteristics
I	All ranges	▪ Not economic ▪ the jump entirely depends on the tailwater and it may sweep away from the basin if $y_2>y_3$
II	≥4.5	▪ The basin length is smaller than basin I by 33% and disperses the energy within the basin ▪ Suitable for high dams ▪ Its construction is a little complicated because of the formwork of the dentated sill and chute blocks.
III	≥ 4.5	▪ Suitable for small dams and diversion weirs where $u_1 < 15$ m/s ▪ The basin length is smaller than basin I by 60%, but it is more difficult to construct because of the form works of the chute blocks, baffle piers, and end sill.
IV	$2.5 < F_{r1} < 4.5$	▪ Suitable for small dams and diversion weirs ▪ The basin length is the same as the length of basin I, but it guarantees the occurrence of the jump within the basin and reduces waves resulting from imperfect jumps

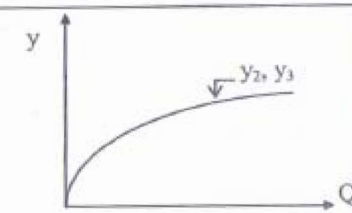
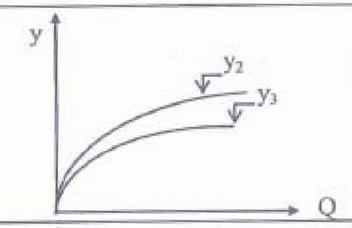
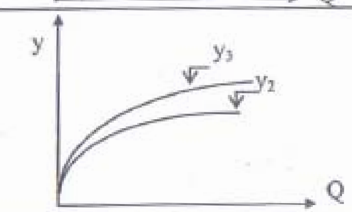
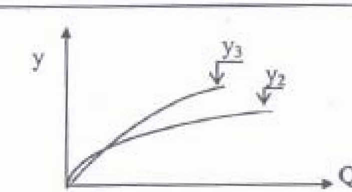
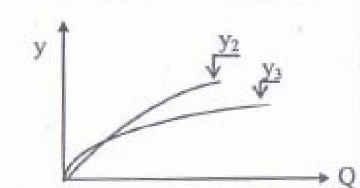
Energy Dissipation at the Toe of Overflow Spillway

$Fr < 1.7$	Düşü havuzuna ve enerji kırıcı bloklara gerek yoktur.
$1.7 < Fr < 2.5$	Havuz yapılır, eşik ve enerji kırıcı bloklara gerek yoktur.
$2.5 < Fr < 4.5$	Havuz, eşik ve şut yapılır. Tip I havuzu seçilir.
$Fr > 4.5$ ve $V < 15$ m/s	Havuz, şut, eşik ve enerji kırıcı bloklar yapılır. Tip II Havuzu seçilir.
$Fr > 4.5$ ve $V > 15$ m/s	Havuz, şut, eşik ve enerji kırıcı bloklar yapılır. Tip III Havuzu seçilir.

Energy Dissipation at the Toe of Overflow Spillway

- The location of the hydraulic jump is governed by the depth of tailwater.
- y_2: Sequent depth
- y_3: Tailwater depth at spillway toe.

Table 4.5 Summary of sequent depth and tailwater interference at spillway toe.

Case		Designation	Remedial measure
1		y_2 and y_3 coincide at all flows	USBR Type 1 basin
2		y_2 is always greater than y_3	USBR Types 2, 3, 4 basins
3		y_3 is always greater than y_2	USBR Type 5 or Type 7 basins
4		y_2 is greater than y_3 at low flows and smaller at high flows	USBR Type 5 basin with an end sill
5		y_3 is greater than y_2 at low flows and smaller at high flows	USBR Types 2,3,4 basins

Energy Dissipation at the Toe of Overflow Spillway

- The location of the hydraulic jump is governed by the depth of tailwater, y_3.

Case 1: (Sequent depth,y_2) = (Tailwater depth,y_3)

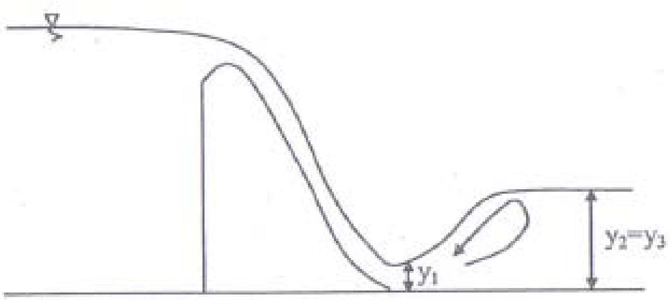


Figure 4.26 Flow conditions for $y_2 = y_3$.

A horizontal apron with a certain thickness may be constructed for this case.

Length of the apron, L_I, is determined from Fig.4.27.

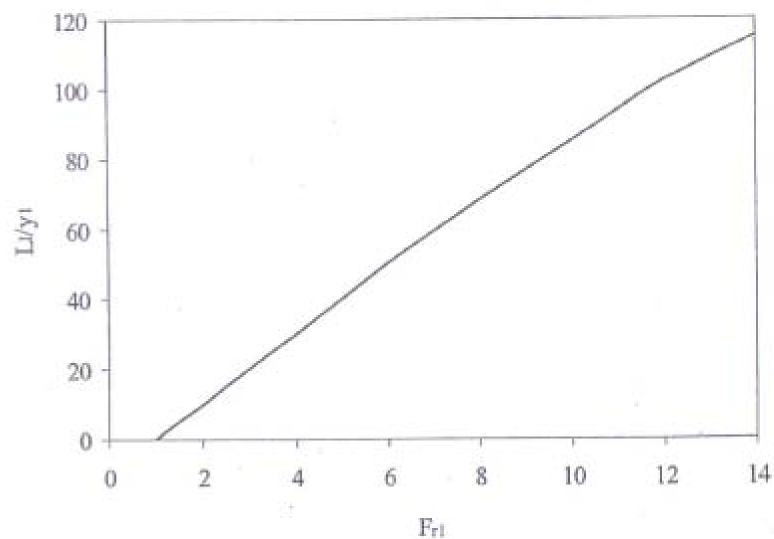


Figure 4.27 Determination of length of the USBR type 1 basin (Peterka, 1964).

Energy Dissipation at the Toe of Overflow Spillway

- The location of the hydraulic jump is governed by the depth of tailwater.

Case 2: $y_3 < y_2$

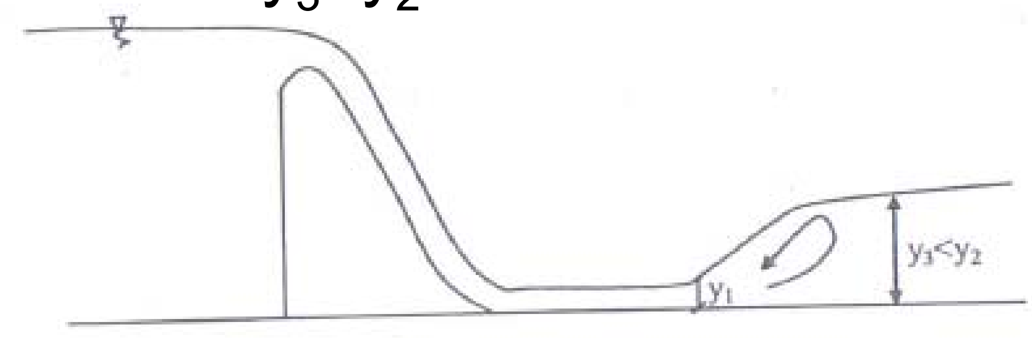


Figure 4.28 Flow conditions for $y_3 < y_2$.

This case should be eliminated since water flows at a very high velocity having a destructive effect on the apron.

Energy Dissipation at the Toe of Overflow Spillway

- **Case 2:**
  - **Chute blocks** channelize the flow and shorten the length of jump and stabilize it.
  - **Baffle piers** dissipate energy by impact effect.
  - Baffle piers are not suitable for very high velocities because of the possibility of cavitation.

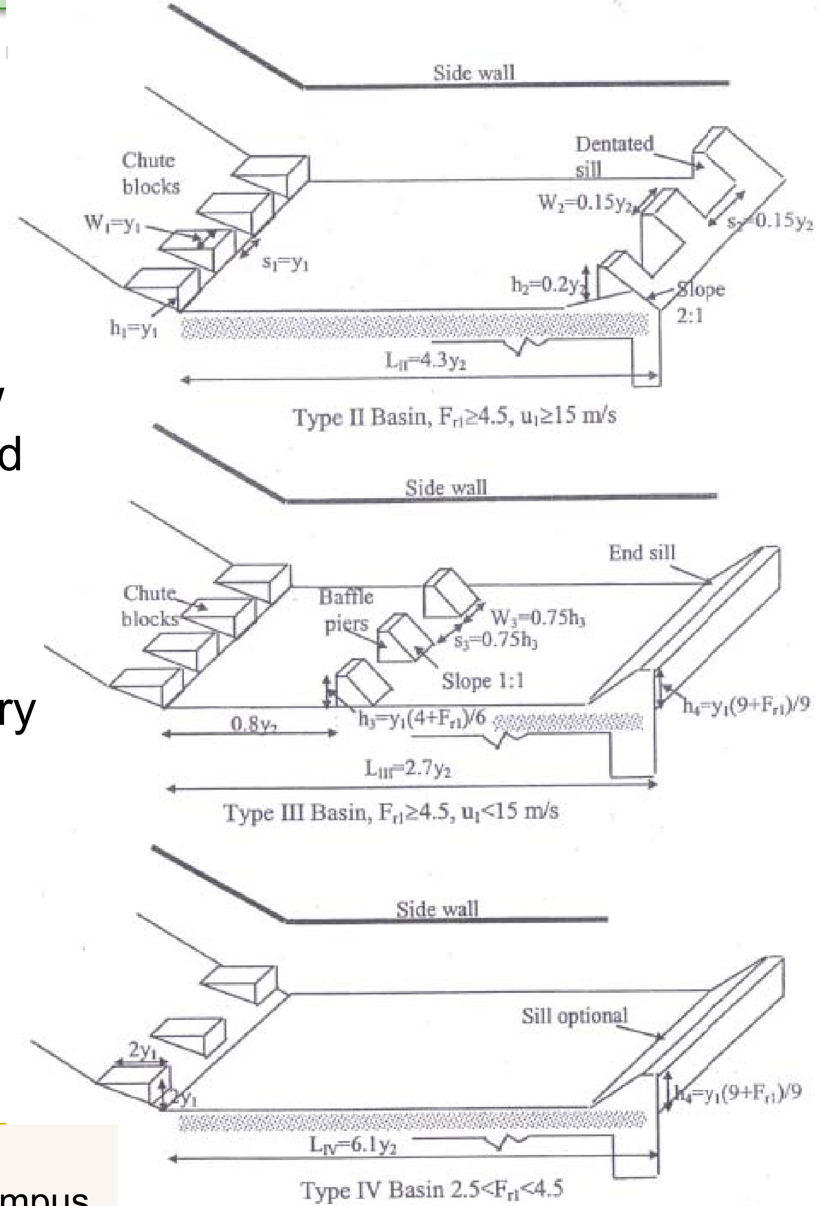


Figure 4.29 Types of the USBR stilling basins (Peterka, 1964; Henderson, 1966).

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Energy Dissipation at the Toe of Overflow Spillway

- **Case 2:**
  - The force acting on a baffle pier is

$$F_p = 2\gamma A E_1$$

where γ: Specific weight of water (kN/m^3),

A: area of the upstream face of the pier in m^2,

E_1: The specific energy at section 1 in m.

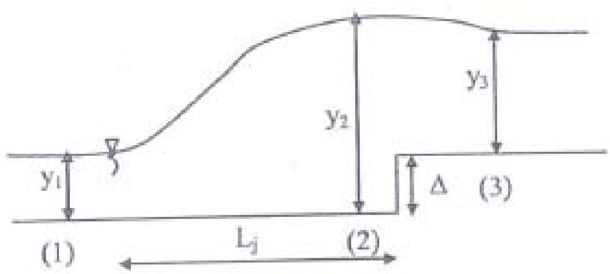
Solid of dentated sills are placed to reduce the length of the jump and control scour downstream of the basin.

Energy Dissipation at the Toe of Overflow Spillway

To find the stilling basin depth, Δ (h_4):

$$\left(\frac{y_3}{y_1}\right)^2 = 1 + 2F_{r1}^2\left(1 - \frac{y_1}{y_3}\right) + \alpha\left(\alpha - \sqrt{1 + 8F_{r1}^2} + 1\right)$$

where $\alpha = \Delta / y_1$



- The line of minimum F_{r1} $\qquad \frac{y_3}{y_1} = F_{r1}^{2/3}$
- The length of jump, L_j: $\qquad L_j = 5(y_3 + \Delta)$

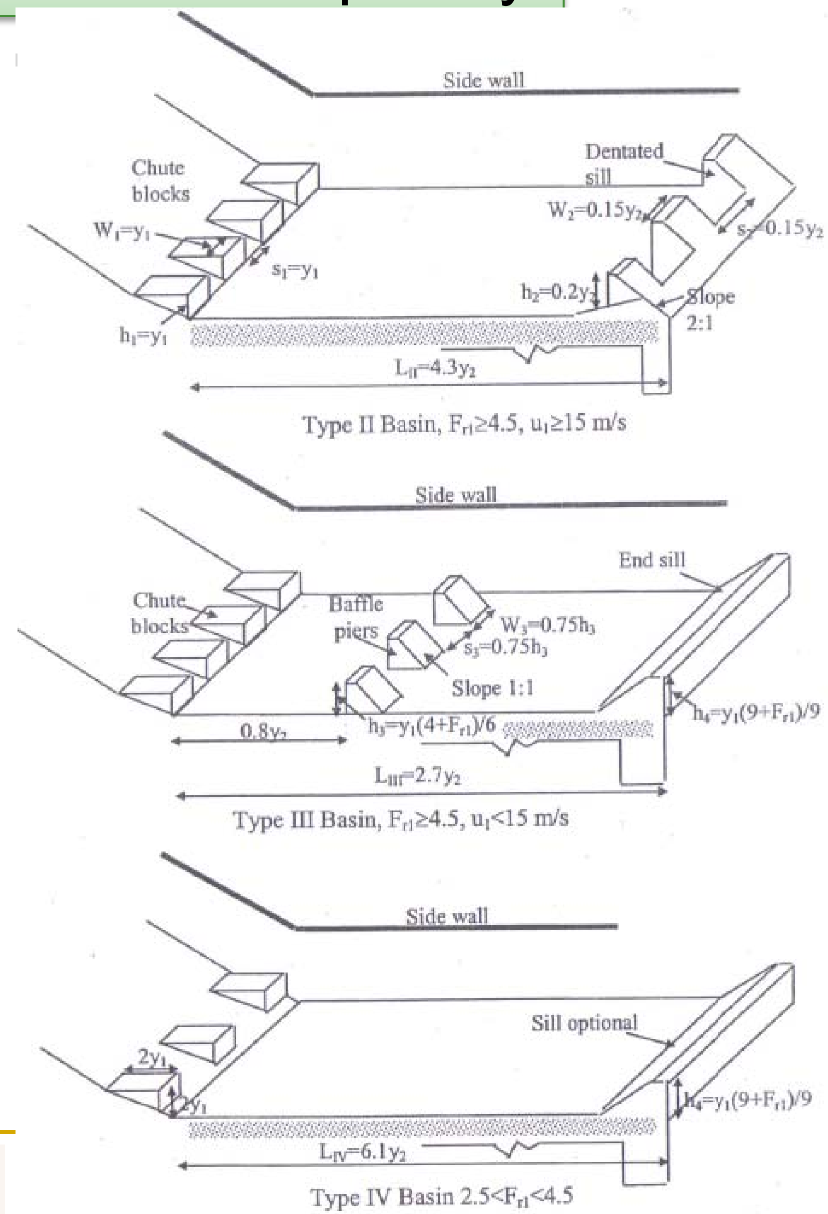


Figure 4.29 Types of the USBR stilling basins (Peterka, 1964; Henderson, 1966).

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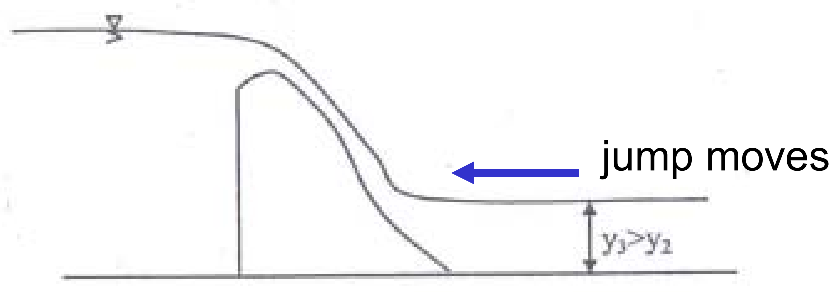
Energy Dissipation at the Toe of Overflow Spillway

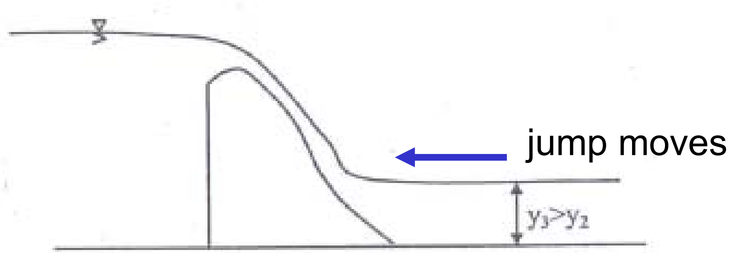
Case 3: $y_3>y_2$



- Different modes of energy dissipation may be considered:
  - A long sloping apron (USBR type 5 basin)
  - A culvert outlet (USBR type 6 basin)
  - A deflector bucket (USBR type 7 basin)
- Selection of the best type is normally dictated by
  - The required hydraulic conformity,
  - Foundation conditions, and
  - Economic considerations

Energy Dissipation at the Toe of Overflow Spillway

Case 3: $y_3 > y_2$



- A **deflector bucket** may be used.

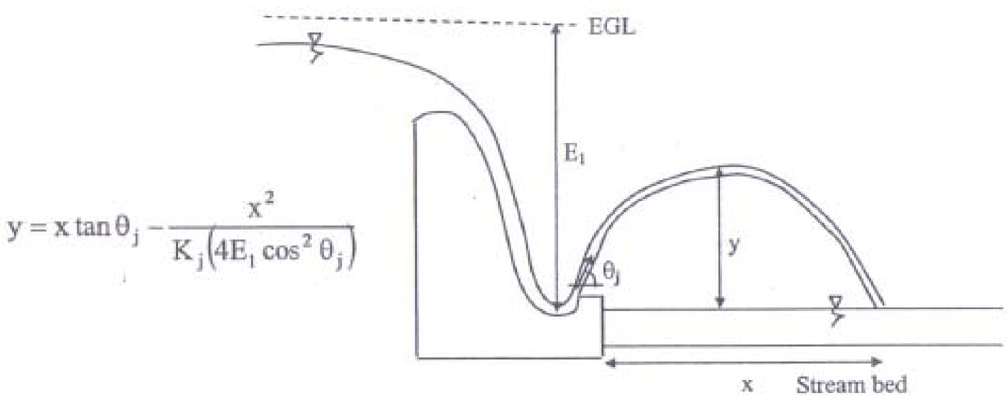


Figure 4.34 Flow conditions for deflector buckets.

K_j: factor (unity for theoretical jet).
E_l: total head at the bucket.

The max. value of x will be $2K_jE_l$ when leaving angle is 45º.

Special care must be taken in case of loose bed material.

Extra measure may be taken to prevent the stream bed erosion induced by the action of inclined jet.

Energy Dissipation at the Toe of Overflow Spillway

Case 4: $y_2 > y_3$

- Sequent depth of the hydraulic jump y_2 is greater than the tailwater depth y_3 at low flows and smaller at the high flows.
- USBR Type 5 basin with an end sill can be used for this case.

Case 5: $y_3 > y_2$

- Sequent depth of the hydraulic jump y_3 is greater than the tailwater depth y_2 at low flows and smaller at the high flows.
- USBR Type 2,3, and 4 basin can be selected for this case.

Chute Spillways

- variously called as open channel or trough spillway
- discharge is conveyed from the reservoir to the downstream river level through an open channel
- placed either along a dam abutment or through a saddle
- mostly used in conjunction with embankment dams
- simple to design and construct
- constructed successfully on all types of foundation materials, ranging from solid rock to soft clay.

Chute Spillways

Ordinarily consist of

an entrance channel,
a control structure,
a discharge channel,
a terminal structure,
& an outlet channel.

Often, the axis of the entrance channel or that of the discharge channel must be curved to fit the topography.

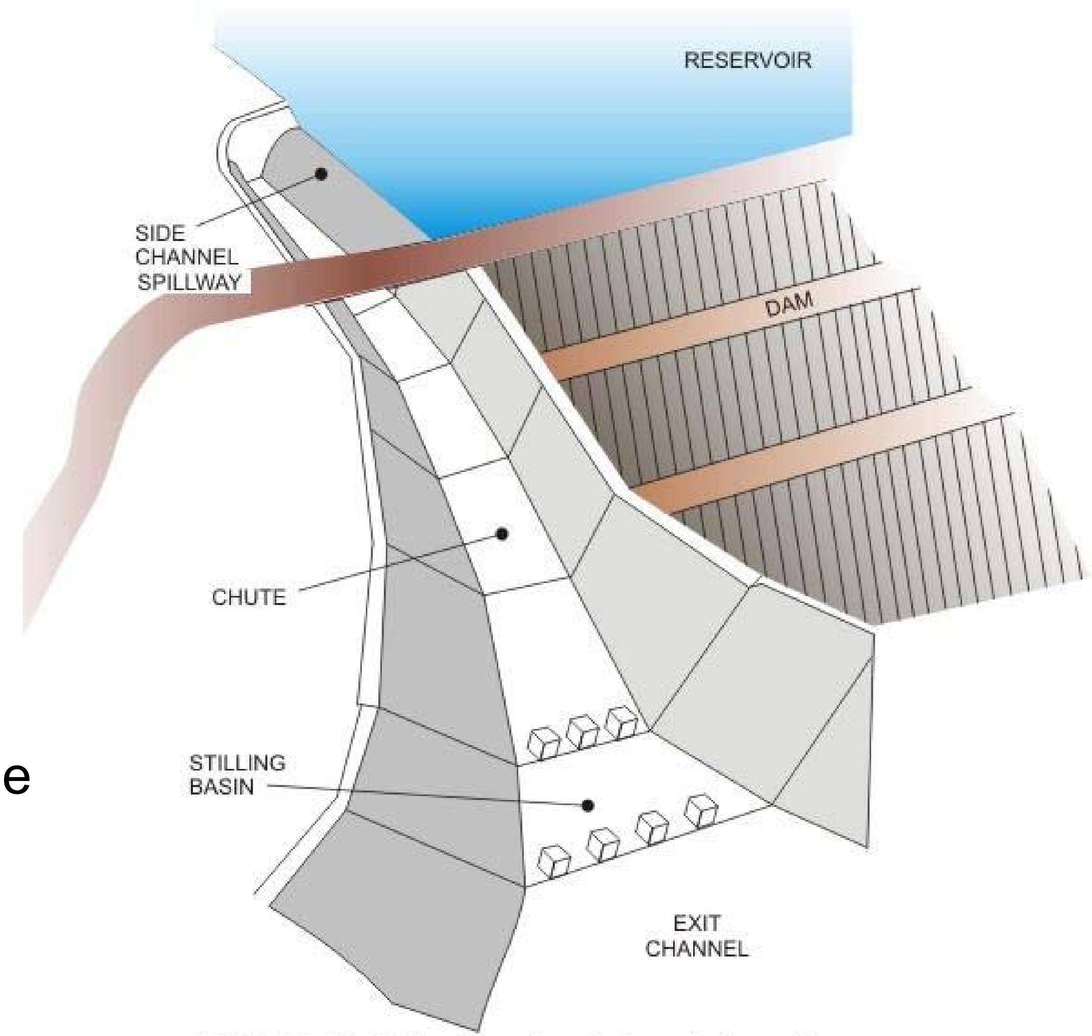
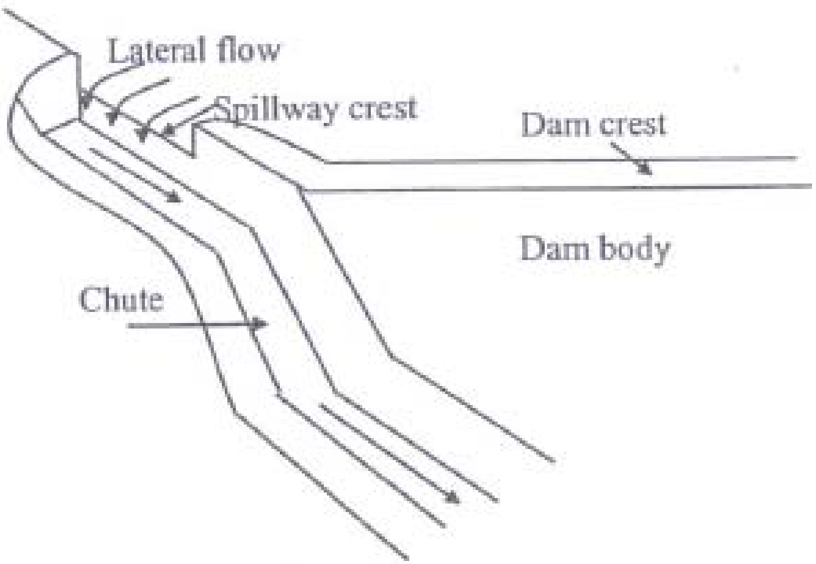


FIGURE 10. Side channel entry to a chute spillway

Side Channel Spillways

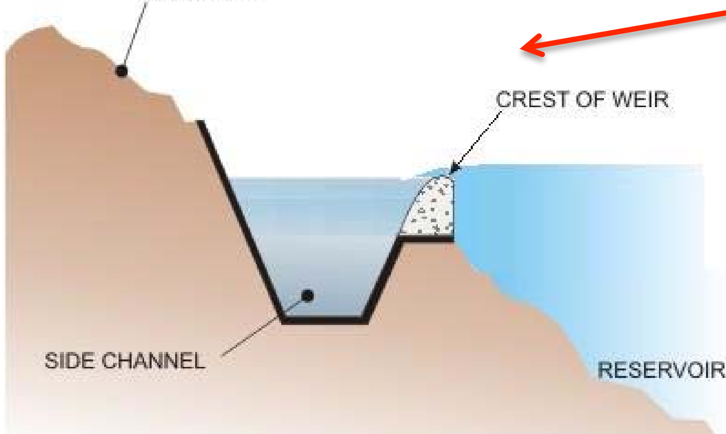
- A side channel spillway is one in which the control weir is placed approximately parallel to the upper portion of the discharge channel
- Suitable in narrow valleys where sufficient crest length is not available



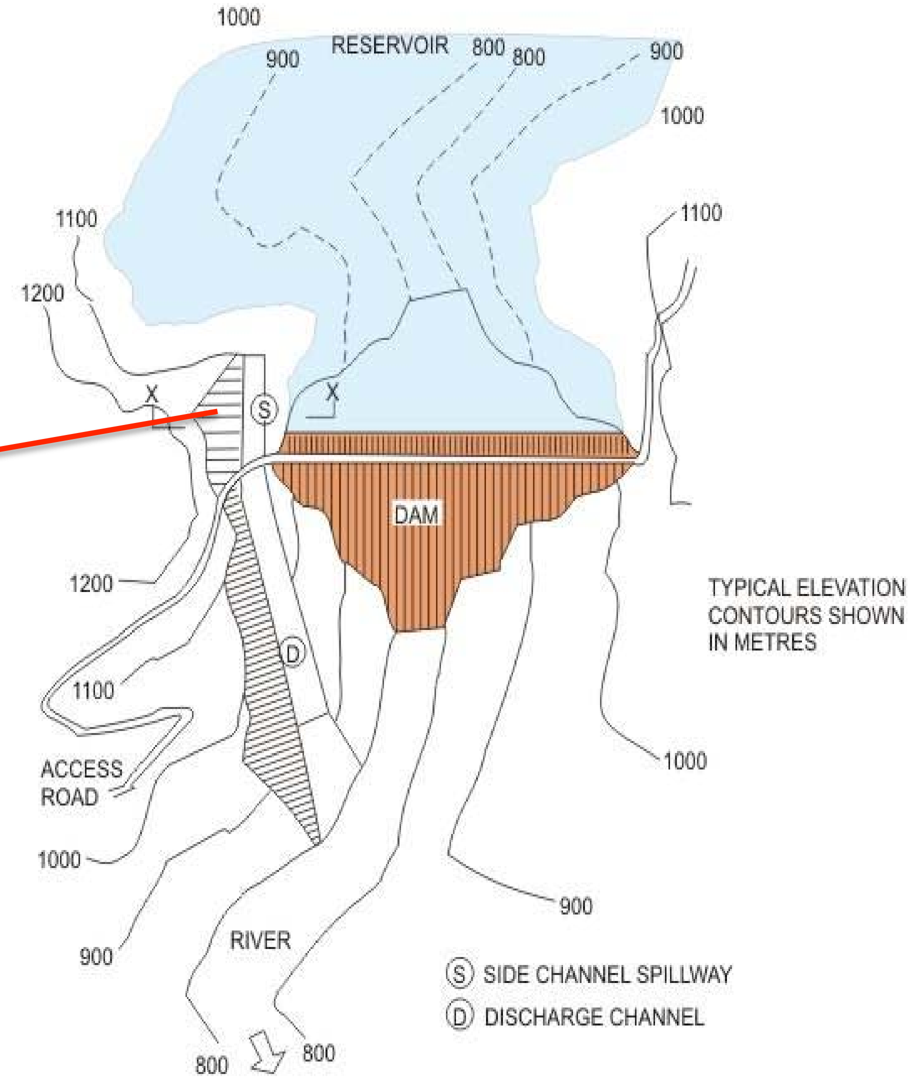
Side channel spillway

Hoover Dam side channel spillway

Side Channel Spillways



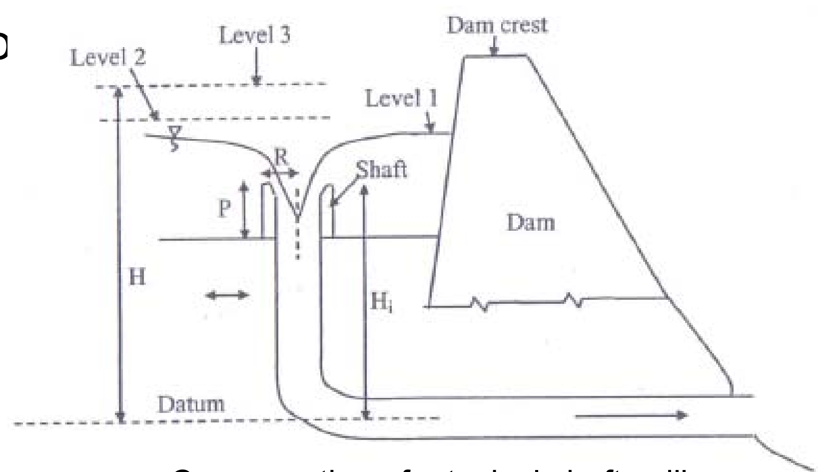
Magnified sectional view X-X through the side channel spillway



. Plan of an embankment dam showing side channel spillway and chute channel

Shaft Spillways

- If a sufficient space is not available fo an overflow spillway, a shaft spillway may be considered.
- In the site of shaft spillway
  - Seismic action should be small,
  - Stiff geologic formation should be available, and
  - Possibility of floating debris is relativeıy small.
- Flow conditions in the spillway:
- Level 1 → a weir flow $Q = C_s(2\pi R)H_0^{3/2}$
- Level 2 → midway between weir flow and pipe flow
- Level 3 → pressurized pipe flow.



Cross-section of a typical shaft spillway

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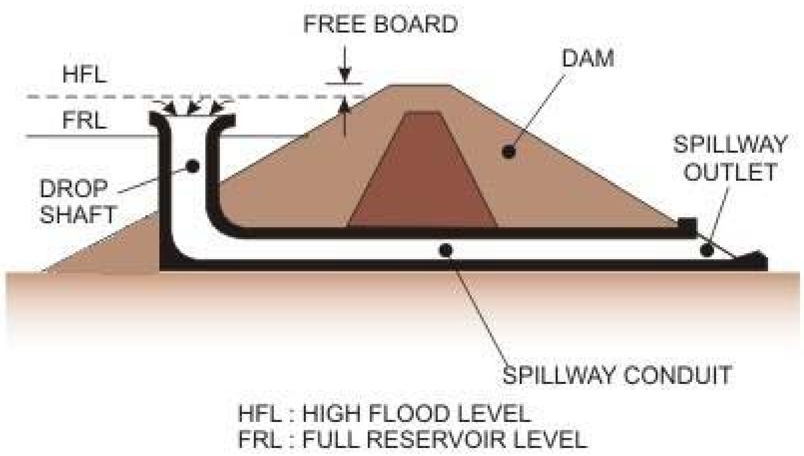
Shaft Spillways

- When the shaft is completely submerged, further increase in head will not result in appreciable increase in discharge.
- Not suitable for large capacity and deep reservoirs because of stability problems.
- Special designs required to handle cavitation damage at the transition between shaft and tunnel.
- Repair and maintenance difficult.
- Rare application in Türkiye (Alakır dam).



Siphon Spillways

- A siphon spillway may be constructed in the body of a concrete dam when space is not available for an overflow spillway.
- It has a limited capacity.
- Discharge $Q = C_d A (2gh)^{1/2}$

  where

  C_d: discharge coefficient (≈0.9)

  A: flow area of siphon

  h : the elevation difference between the upstream water level and end of the barrel. When the downstream end is submerged, h is elevation difference between the upstream and downstream water levels.

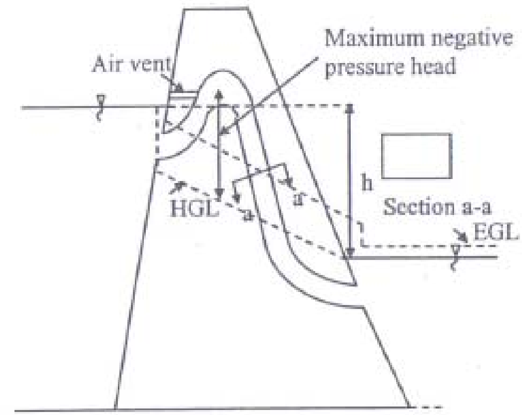
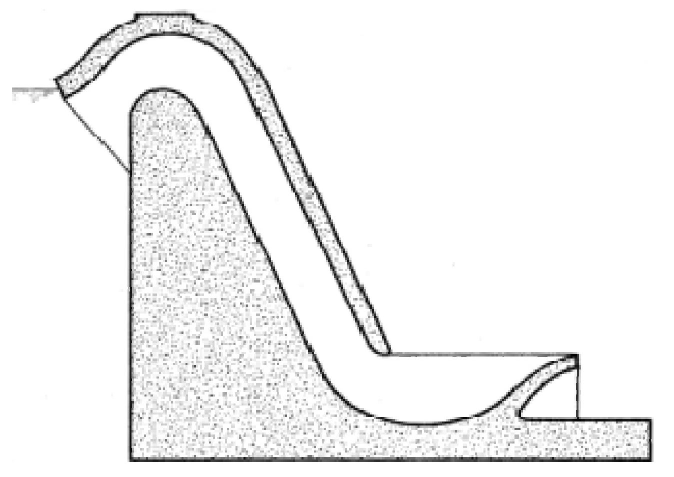


Figure 4.39 Cross-section of a typical siphon spillway.



Cross-section of a typical siphon spillway

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Overflow Spillway

In the selection of a spillway, the following steps are to be considered:

- A spillway with certain dimensions is selected.
- The maximum spillway discharge and maximum lake elevation are determined through reservoir flood routing performed for design conditions.
- Other dimensions are determined.
- Cost of dam and spillway are determined.
- The above steps are repeated for:
  - various combinations of dam height and reservoir capacities using elevation storage relationship of reservoir, and
  - various types of spillways.
- The most economical spillway type and optimum relation of spillway capacity to the height of dam are chosen.

Overflow Spillway

- In the economic analysis, following should be considered:
  - repair and maintenance costs,
  - the hydraulic efficiency of each type of spillway.
- Most of the spillways in Turkey are of the controlled overflow type.
- The relation between the length of overflow spillway and the total cost of the dam must be analyzed to achieve an optimum solution.

- Spillway length ⇗ the cost of the spillway ⇗
- Spillway length ⇗ the water level ⇘ the cost of the dam ⇘
- There is an optimum spillway length, which minimizes the total cost of construction.

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