Spillways

- Spillway evacuates extra flood volume to downstream

- Guidelines in the selection of spillway capacity:

To function properly, spillway must have sufficient capacity to discharge floods which are likely to occur during the lifetime of the dam

a) Consider PMF for dams
- likely to be subject to excessive damage & loss of lives
- having large reservoir capacity
- in close vicinity of settlements

b) Take a reasonable risk for dams:
- with no serious damage possibility at downstream

Spillway Design Flood (SDF)

Risk ⇔ return period

$$Risk = 1 - q^n = 1 - \left(1 - \frac{1}{T_r}\right)^n$$

n = lifetime of the structure

q = prob. of nonoccurrence

T_r = return period

- Risk α 10^{-4} - 10^{-8}

- To select an appropriate risk, consider:
  - flood attenuation in the reservoir
  - capacity of outlet works
  - available storage

- Selection of SDF
  - Prescriptive standards (e.g. USACE guidelines)
  - Risk analysis (Minimization of total cost)

commonly used return periods

Spillway of	T_r (yr)
Earth-fill dam	≈ 15000
Rock-fill dam	≈ 10000
Gravity dam	≈ 1000
Diversion weir	≈ 100

to decrease risk of overtopping

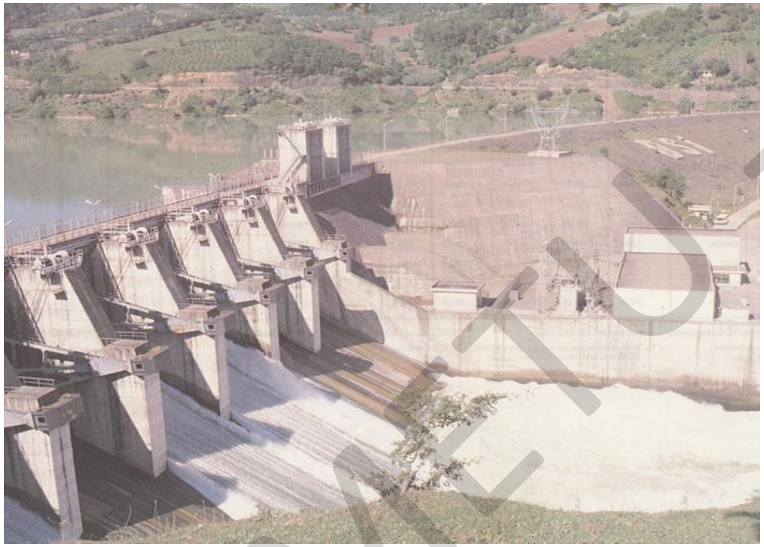
if a gravity dam overtops
it is not that big of a problem

Overflow spillways:

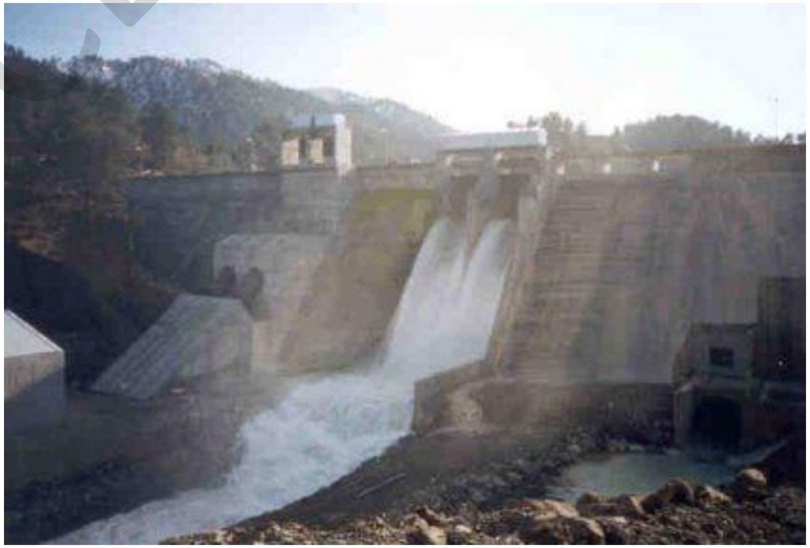
- (Ogee-crested, S-shaped)
- flood wave passes over the crest
- used on gravity, arch & buttress dams
- used as a separate structure at one side of fill dams

Types of overflow spillway

- uncontrolled (ungated, free)
- controlled (gated, guided) ← allows storage of more water



(a)

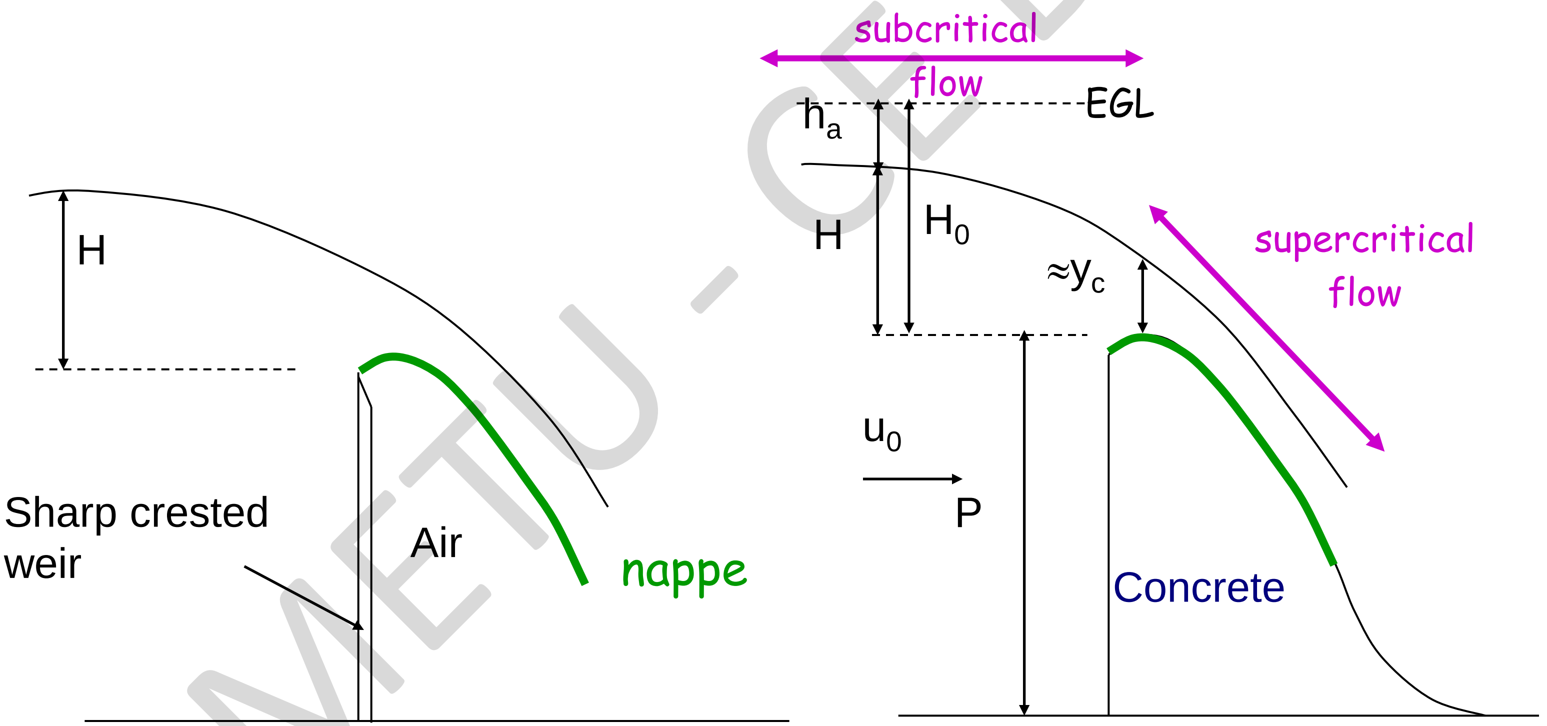


(b)

(a) Spillway of Suat Uğurlu Dam (b) Spillway of Suçatı Dam

The shape of an ideal spillway is the lower nappe of a sharp crested weir for Q = Q_{des}

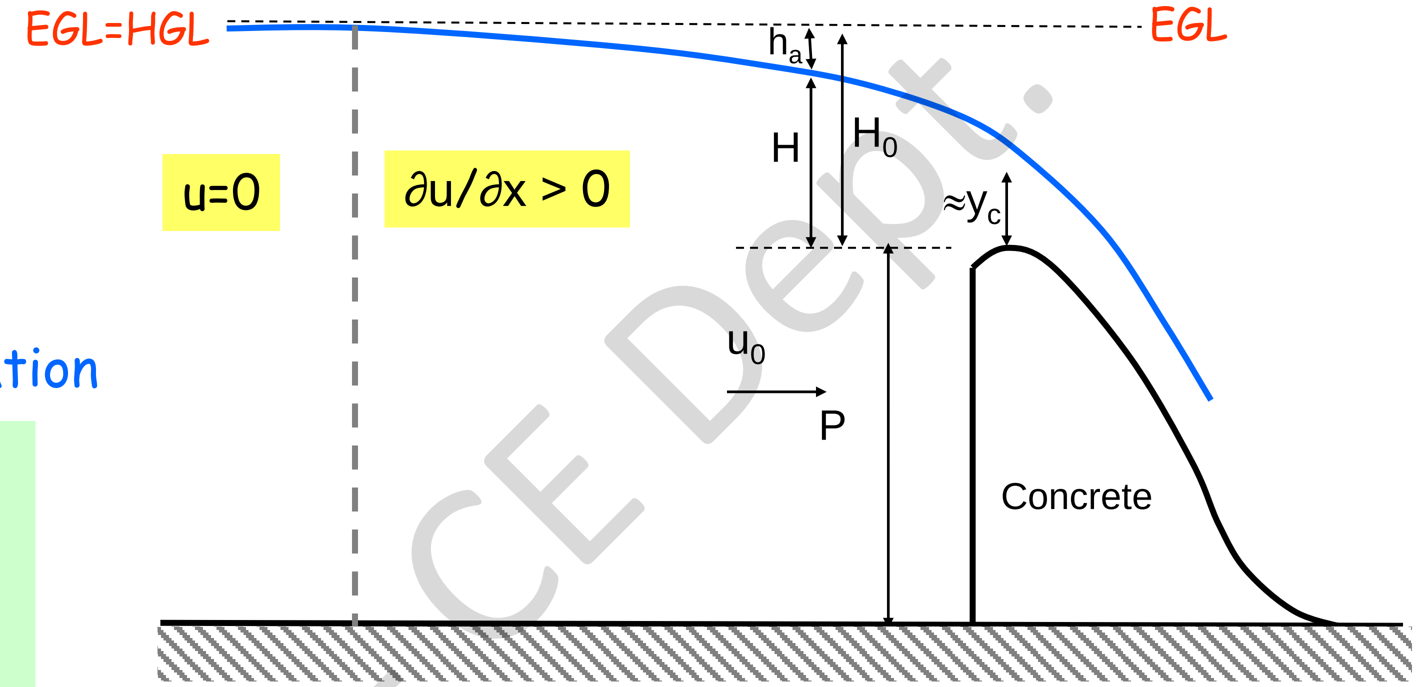
- natural shape
- involves atm. pressure both along the lower & upper boundaries



Sharp crested weir
(used to measure flow)

Overflow spillway
y_c = critical depth

Construction of an Overflow Spillway



Discharge Equation

$$Q_0 = \int_{\frac{u_0^2}{2g}}^{H_0} u\,dA$$

$$Q_0 = C_0 L H_0^{3/2}$$

Q_0 = design discharge

C_0 = design discharge coefficient

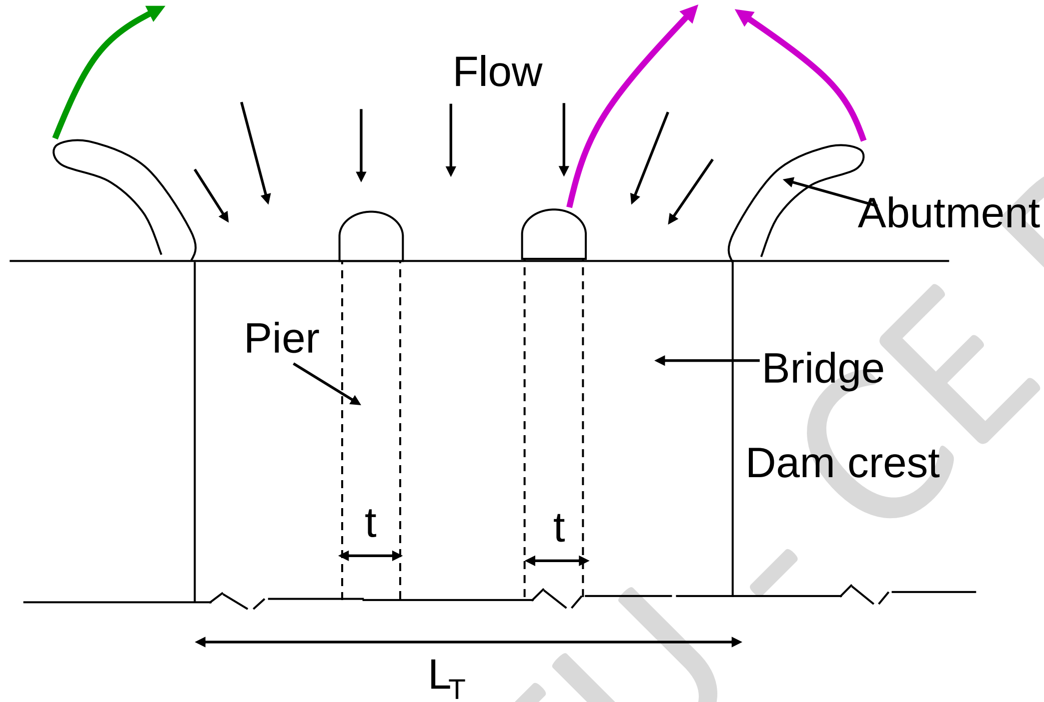
L = effective crest length

H_0 = total head over the spillway crest

dimensional analysis of C_0?

- to avoid development of vortexes
- to facilitate gentle flow cond.s

rounded to minimize hydraulic disturbances



Piers are for:

1. Piers divide the spillway in various chutes s.t. gentle flow conditions prevails in narrower chutes
2. If one gate is located over L_T, it will be very hard to operate it. So couple of smaller gates are installed.

effective crest length, $L = L' - 2(NK_p + K_a)H_0$

L' = net length = $L_T - Nt$

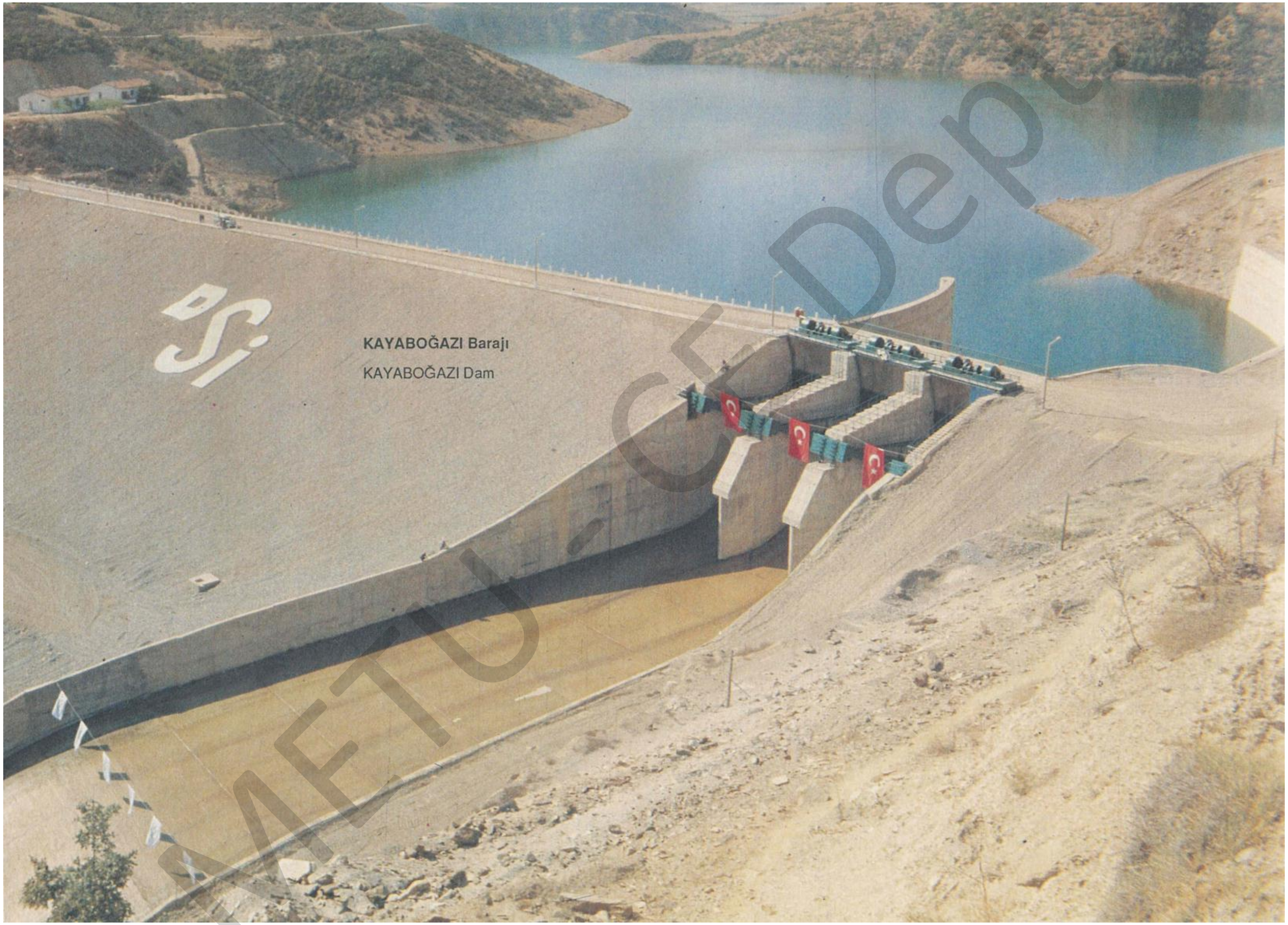
L_T = total crest length

t = thickness of each pier

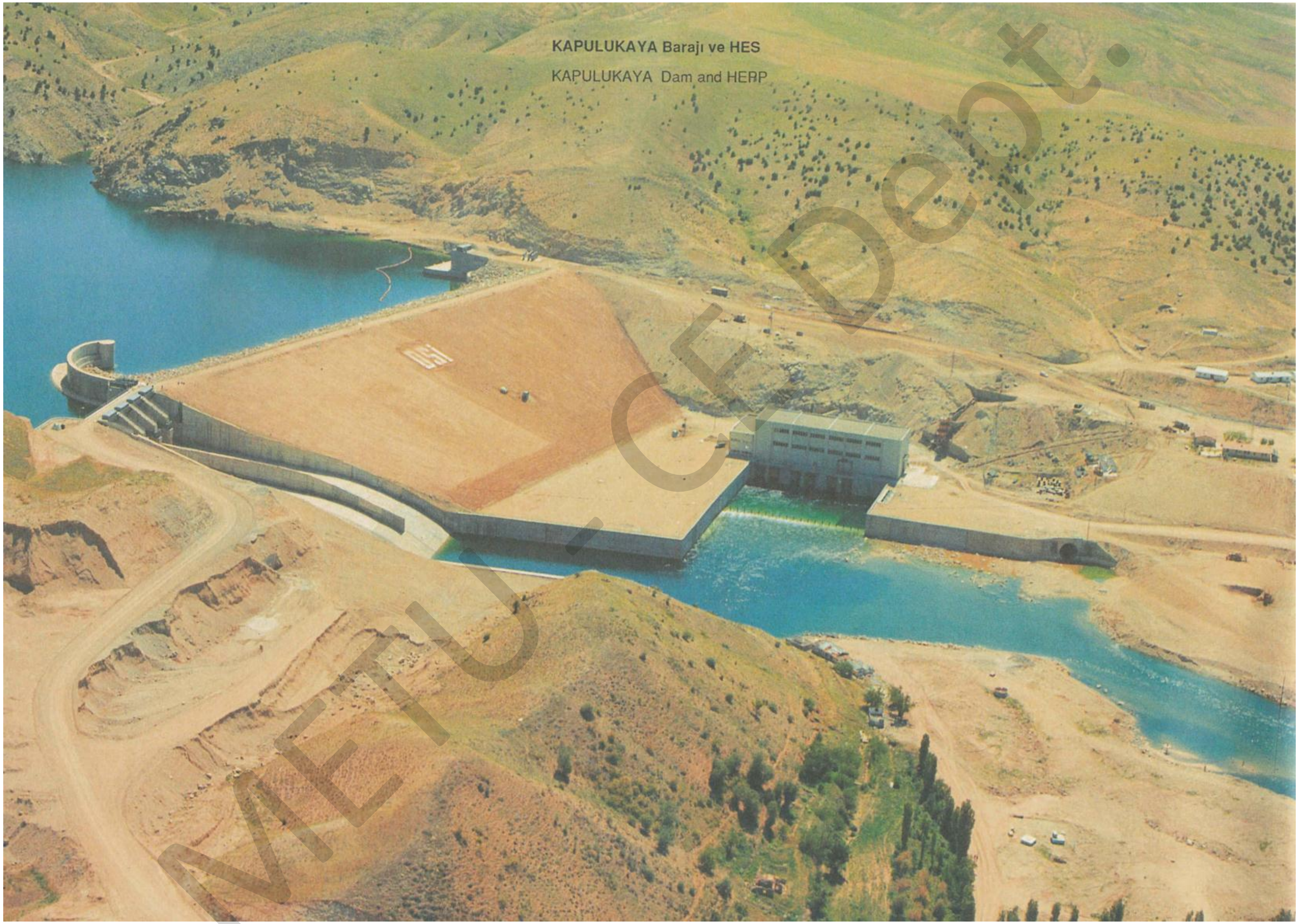
N = number of bridge piers

K_p = pier contraction coefficient (0-0.02)

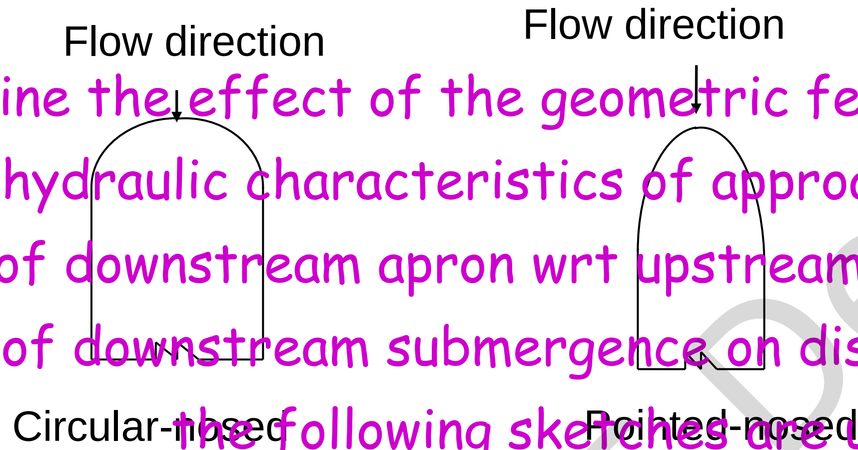
K_a = abutment contraction coefficient (0-0.2)



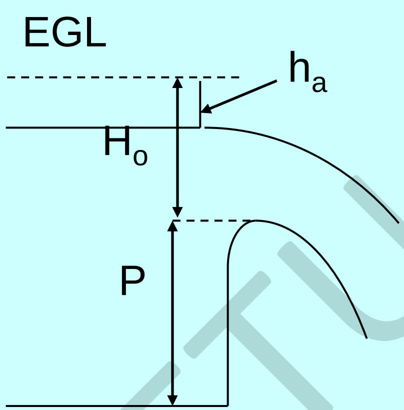
Kayaboğazı Dam on Kocaçay (Susurluk) River



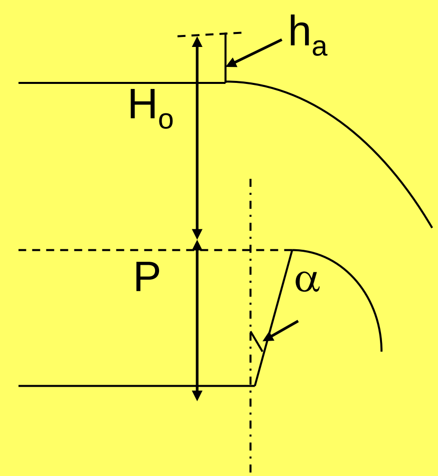
Kapulukaya Dam on Kızılırmak



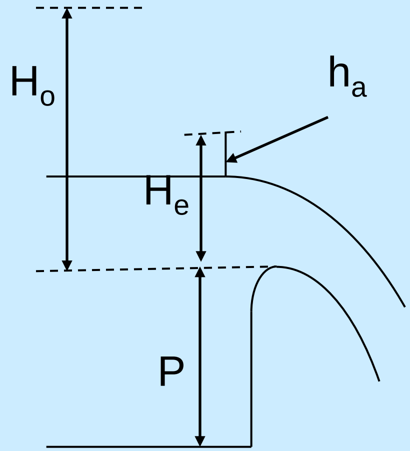
To examine the effect of the geometric features of spillways,
hydraulic characteristics of approaching flow,
level of downstream apron wrt upstream energy level and
degree of downstream submergence on discharge coefficient
the following sketches are used:



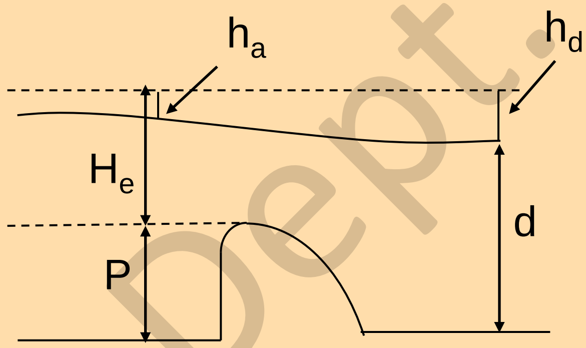
1. Vertical upstream face under design case



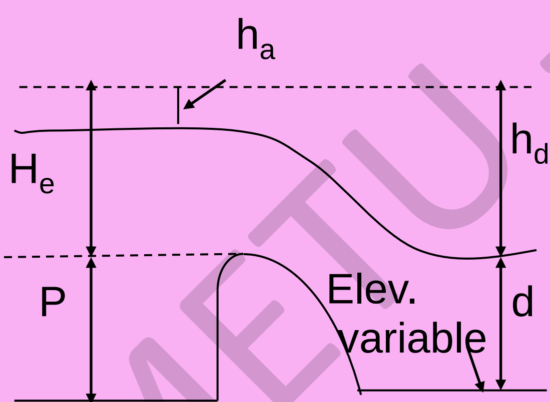
2. Sloping upstream face under design case



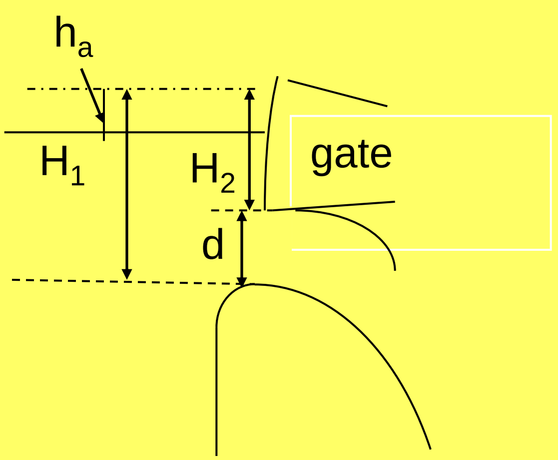
3. Existing heads other than design head



4. Submergence effect

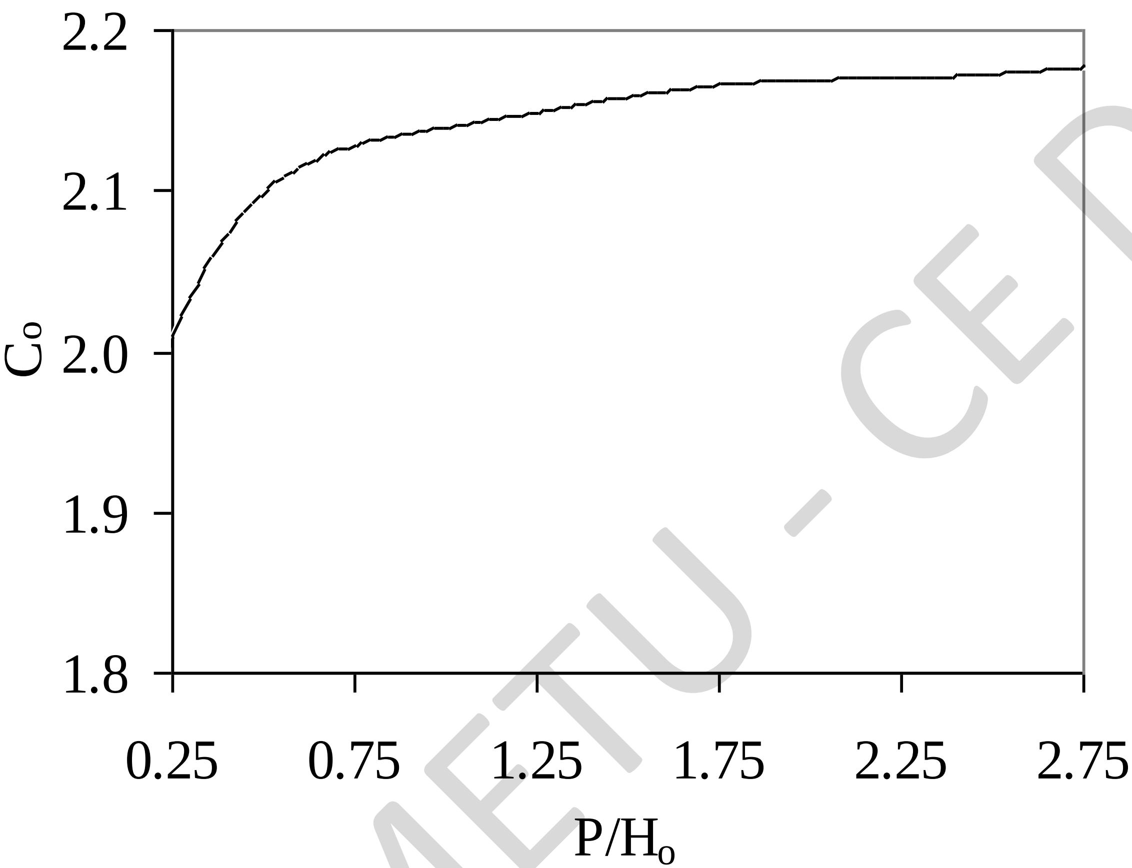


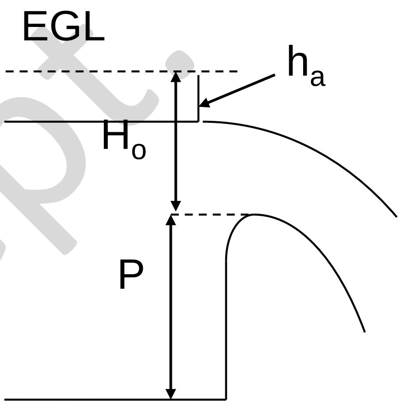
5. Position of apron level



Flow through gate

1. For vertical upstream face





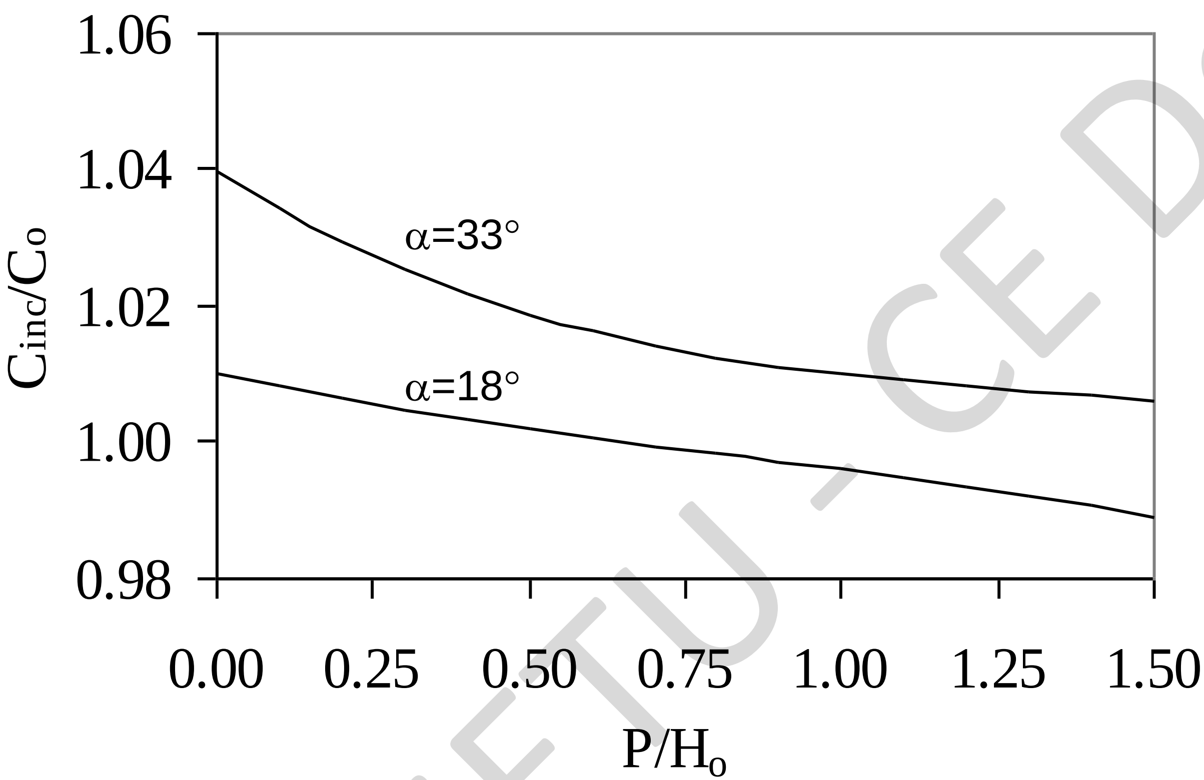
$$Q_0 = C_0 L H_0^{3/2}$$

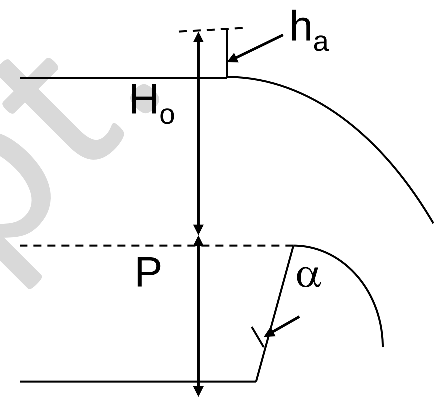
Q_0 and P given. Find H_0

Both H_0 and C_0 unknown!

1. assume H_0
2. calculate P/H_0
3. read C_0
4. calculate Q_0
5. check w/ design discharge

2. For inclined/sloping upstream face



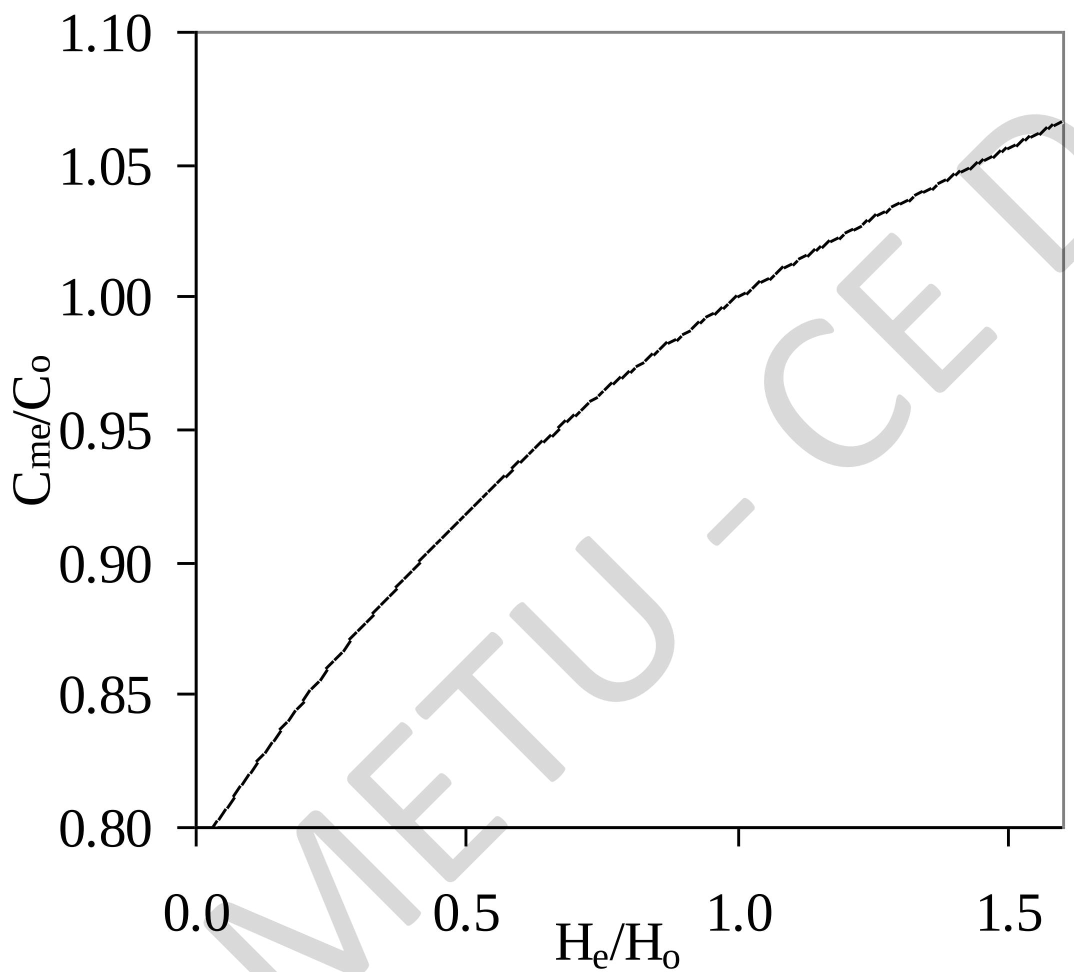


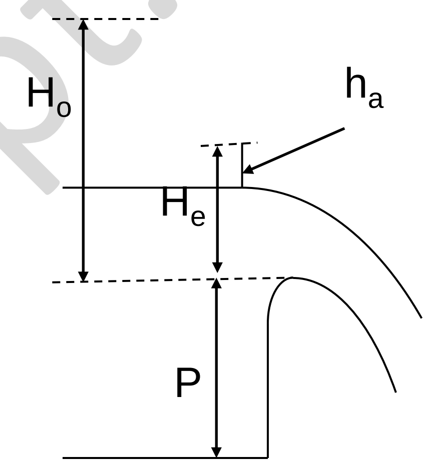
why sloped upstream required?

to increase stability, its weight should be big enough to provide the necessary resisting forces

First irrespective of shape of upstream face determine C_0 then use this figure to determine C_{inc}/C_0 and calculate C_{inc} from this ratio

3. For existing head other than design head



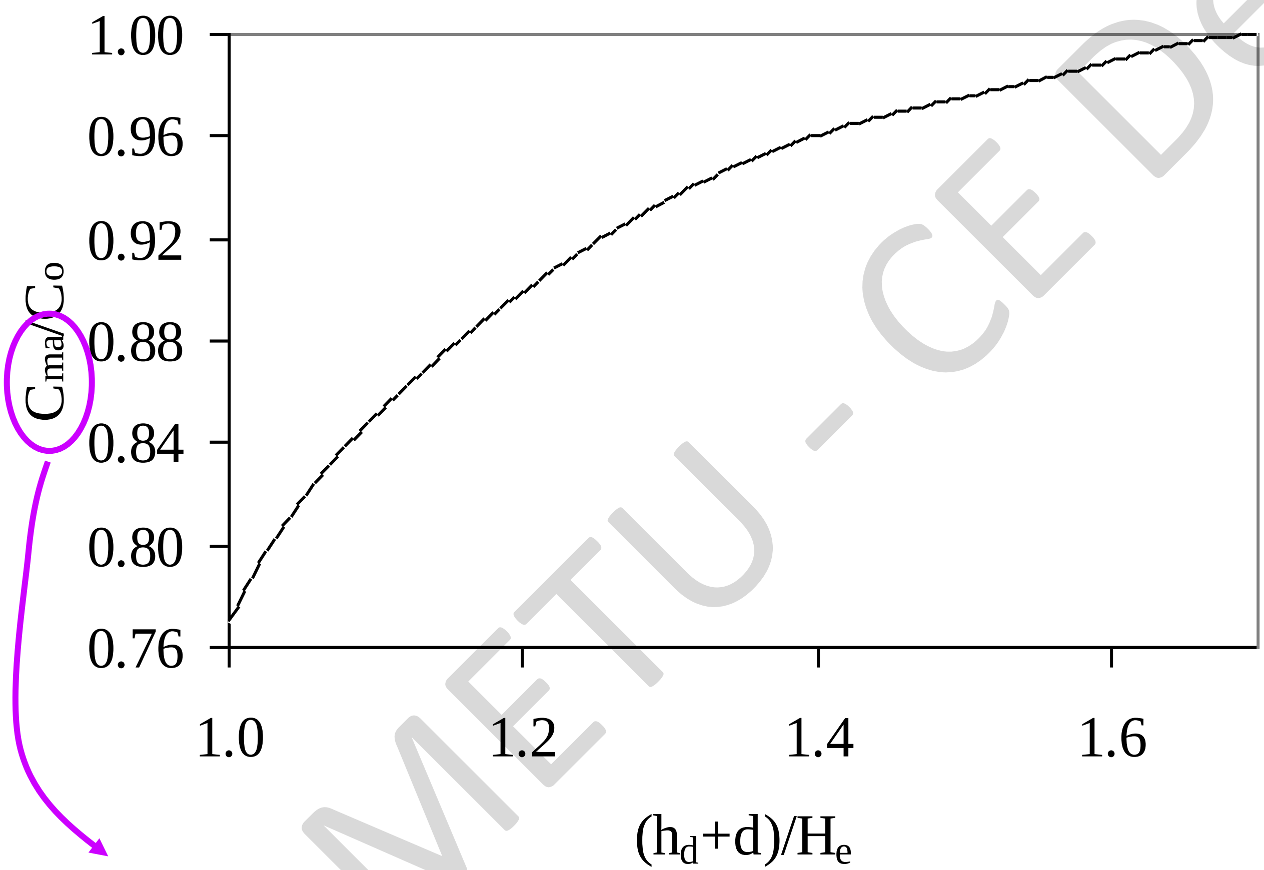


H_e is he total head

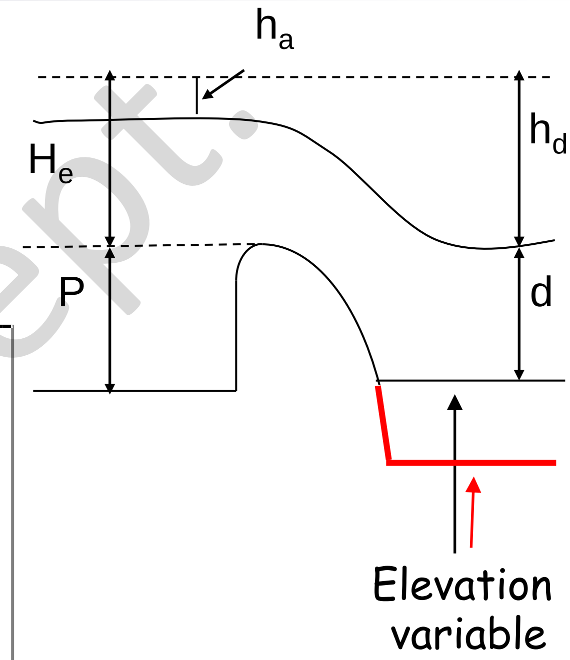
If Q_e is given, use trial and error to determine H_e

$$Q_e = C_e L H_e^{3/2}$$

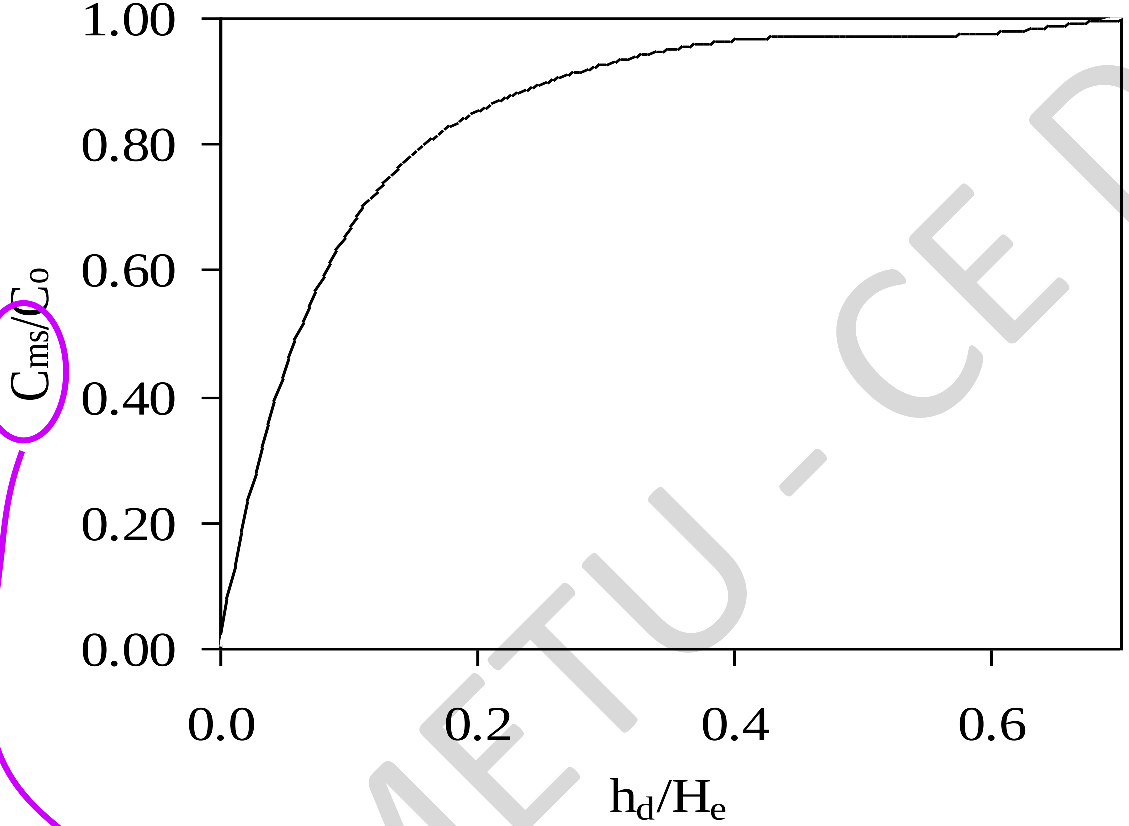
4. Position of apron level



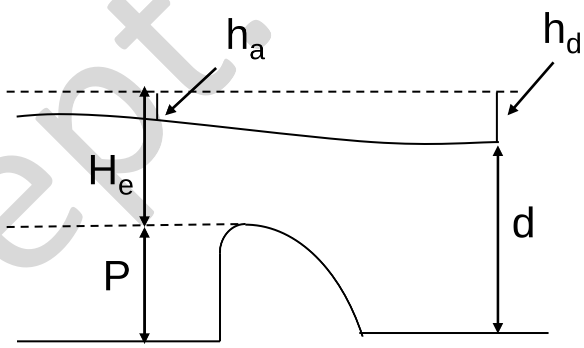
discharge coeff. that accounts
for the apron effect



5. Submergence effect



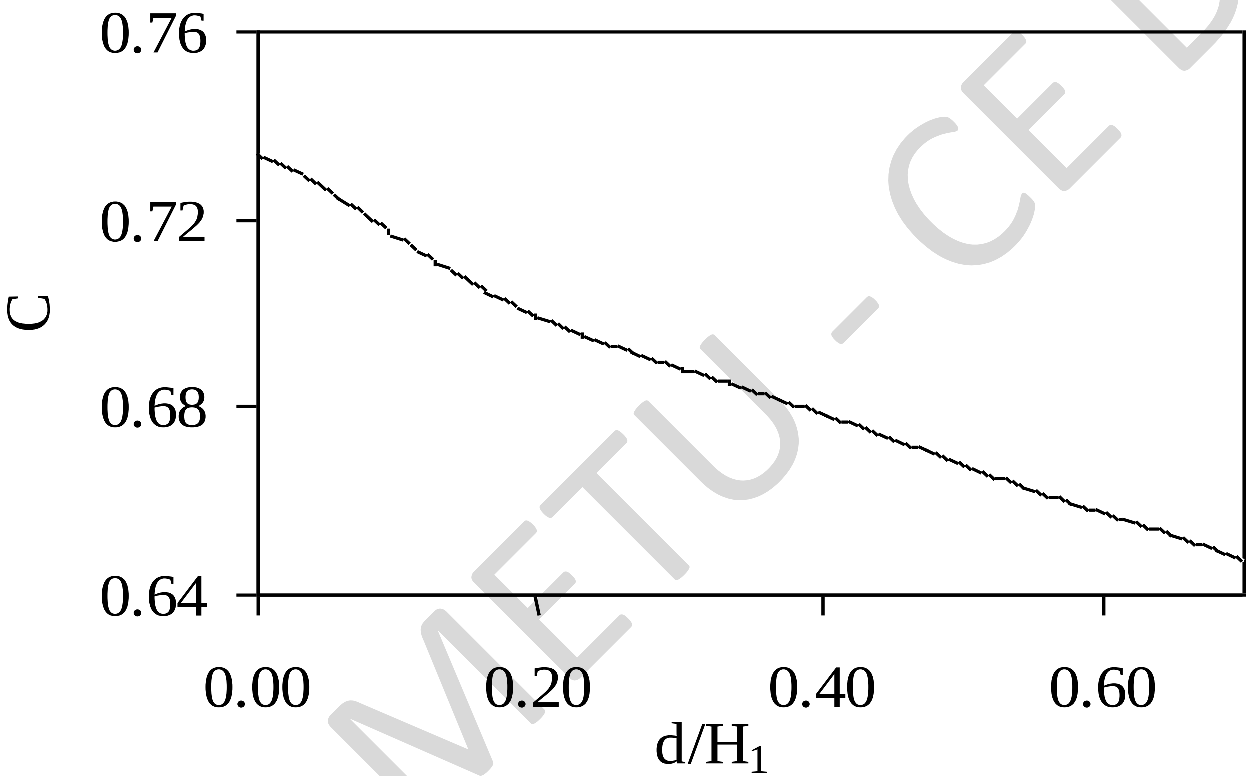
discharge coeff.
submerged case

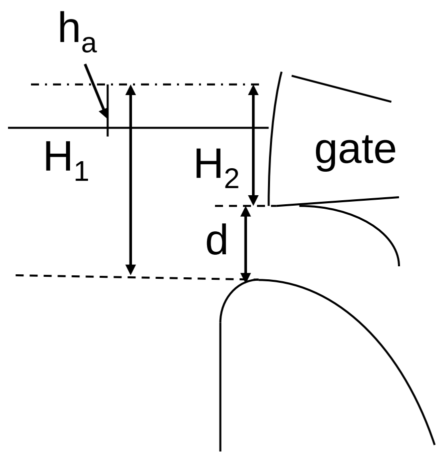


For partially open gates

$$Q = \frac{2}{3}\sqrt{2g}\,CL(H_1^{3/2} - H_2^{3/2})$$

g = gravitational acceleration
L = effective crest length
C = discharge coeff



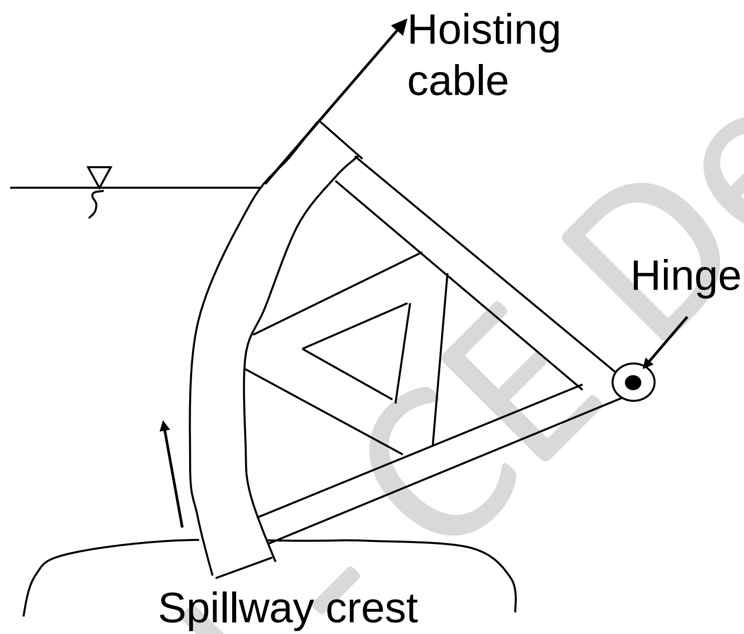


Crest Gates

- Gates attain additional storage !
- Small increase in h → large increase in storage

- Common types of gates
  - Vertical lift gate
  - Tainter (radial) gate
  - Rolling (drum) gate

Reservoir surface area may reach very big values at the spillway crest elevation. Therefore even a few meters of water storage above the spillway crest may correspond to a huge volume of additional water



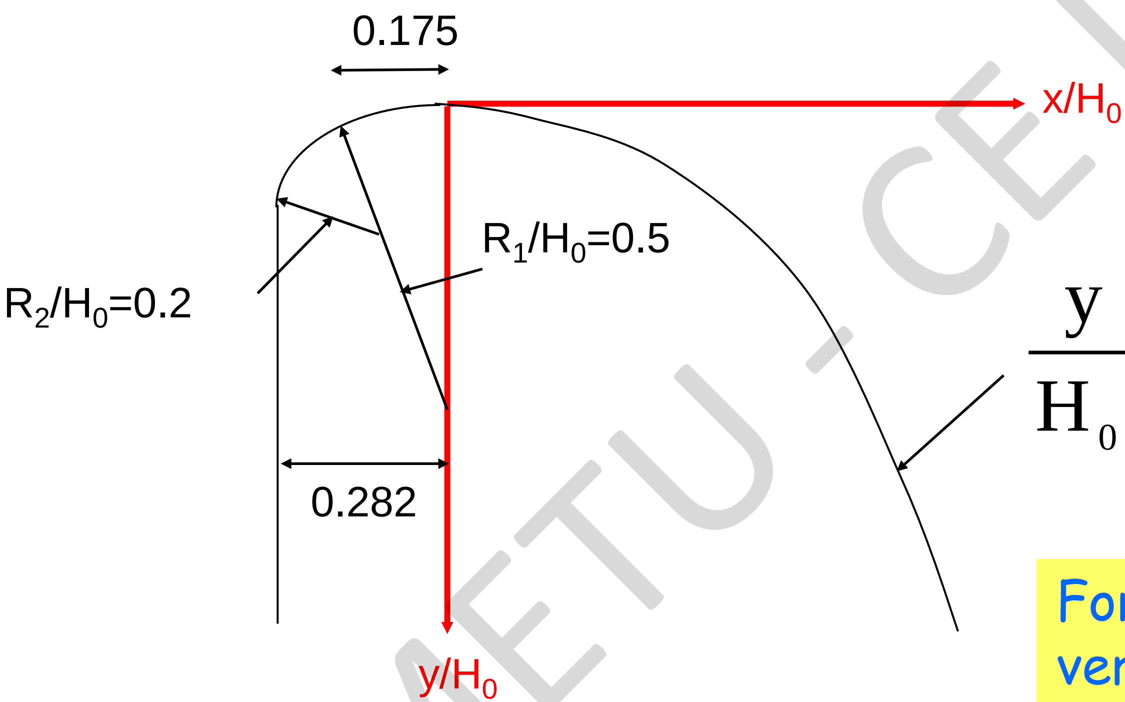
Tainter gate (radial gate)

- have wider application
- easy to operate

Tainter (radial) gates at spillway crest

Spillway Crest Profile

USBR Standard crest profile (1987)



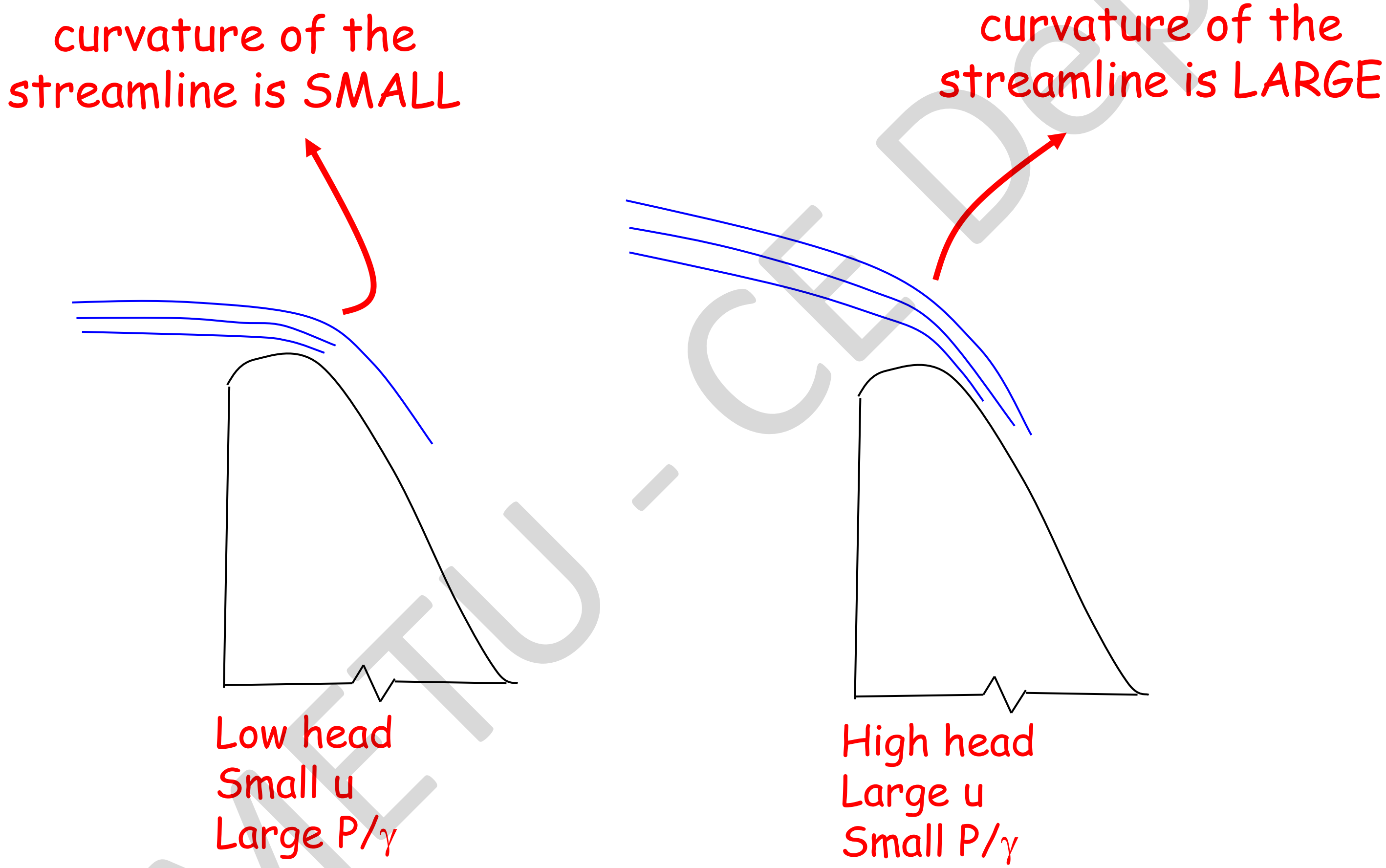
$$\frac{y}{H_0} = K\left(\frac{x}{H_0}\right)^n$$

For an overflow spillway with vertical upstream face

$K \approx 0.5$

$n \approx 1.85$

Pressure distribution at spillway face



u = velocity
P = pressure
γ = specific wt of water

1) For $H_e < H_0$ → curvature of streamlines is SMALL:

$(P/\gamma) > (P/\gamma)_{atm}$ (but < hydrostatic)

2) For $H_e > H_0$ → curvature of streamlines is LARGE:

$(P/\gamma) < (P/\gamma)_{atm}$

H_0 = design head
H_e = present head

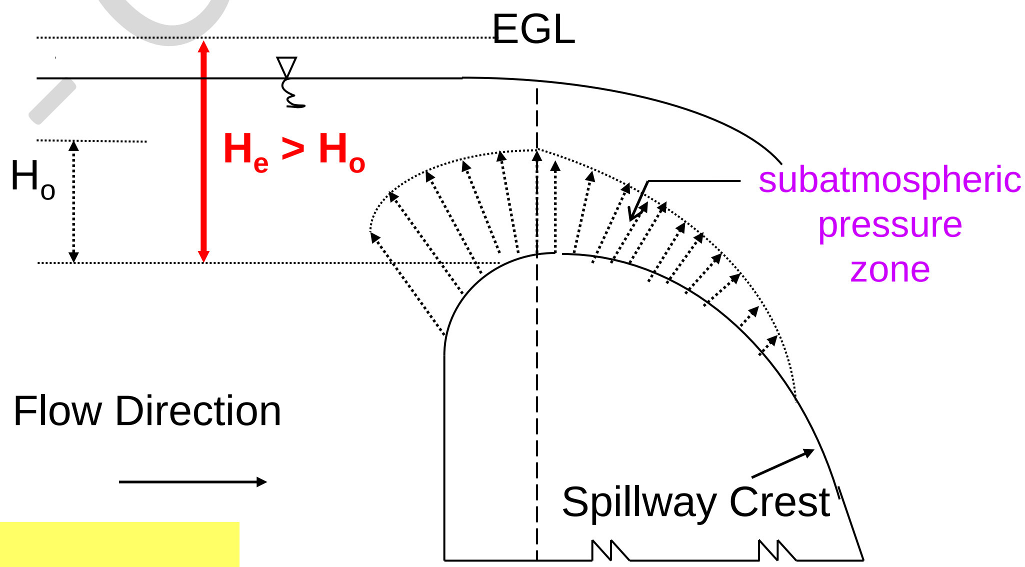
reduced pressure at the spillway crest

may ↓ cause

overflowing water to break the contact with the spillway face

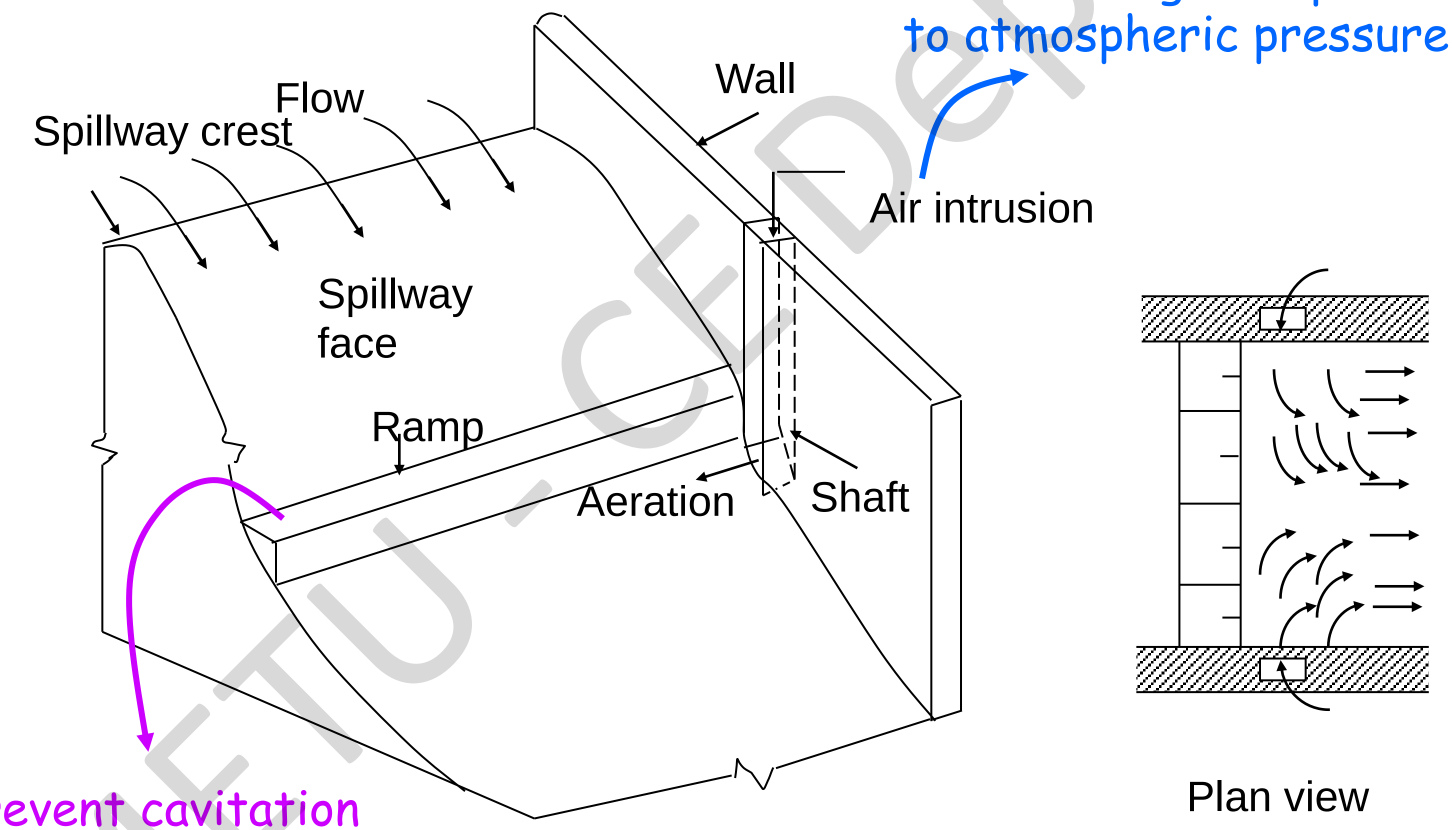
↓

formation of vacuum at the point of seperation → cavitation



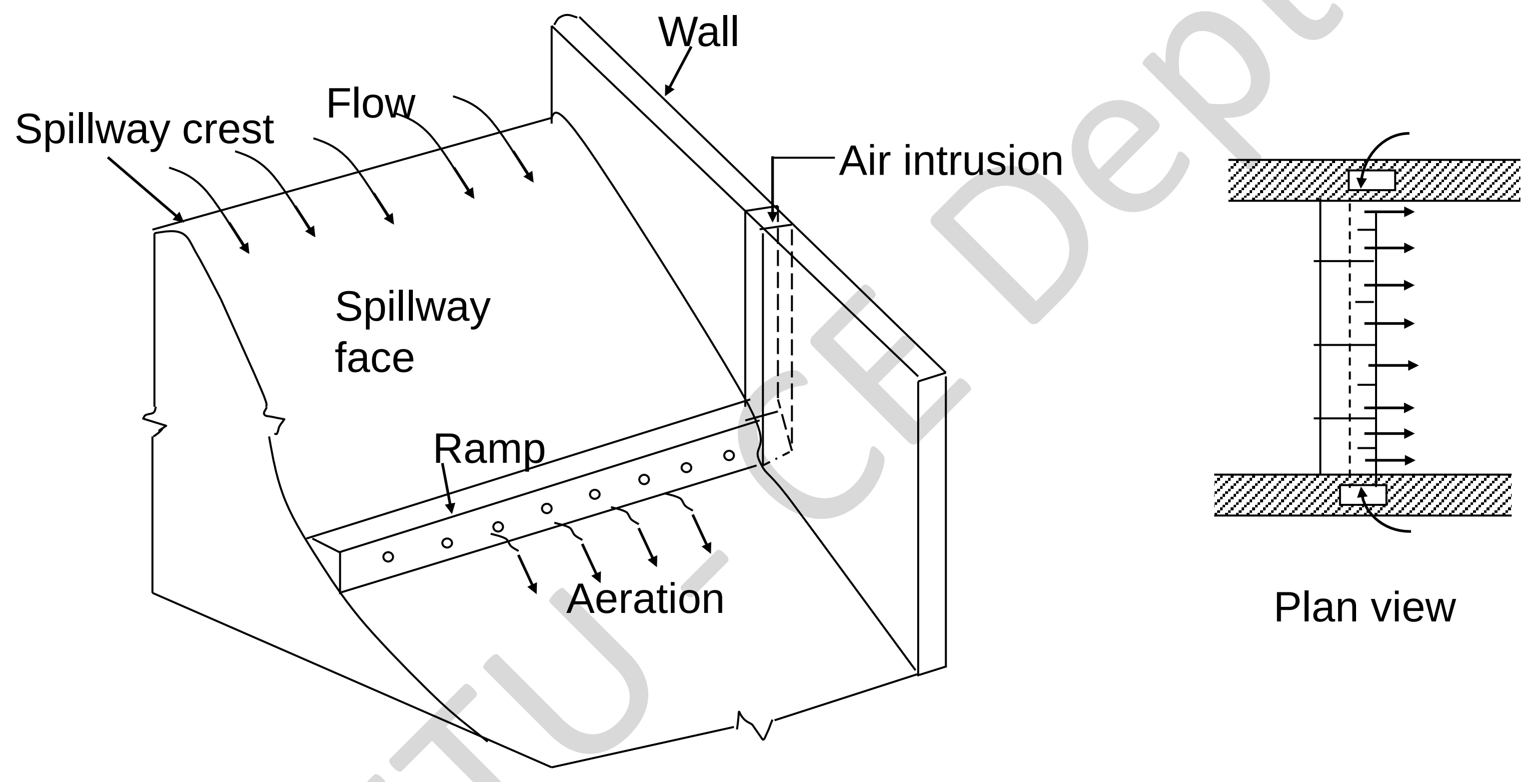
$H_e > H_0$ → Formation of vacuum → Possibility of cavitation

to increase negative pressure to atmospheric pressure

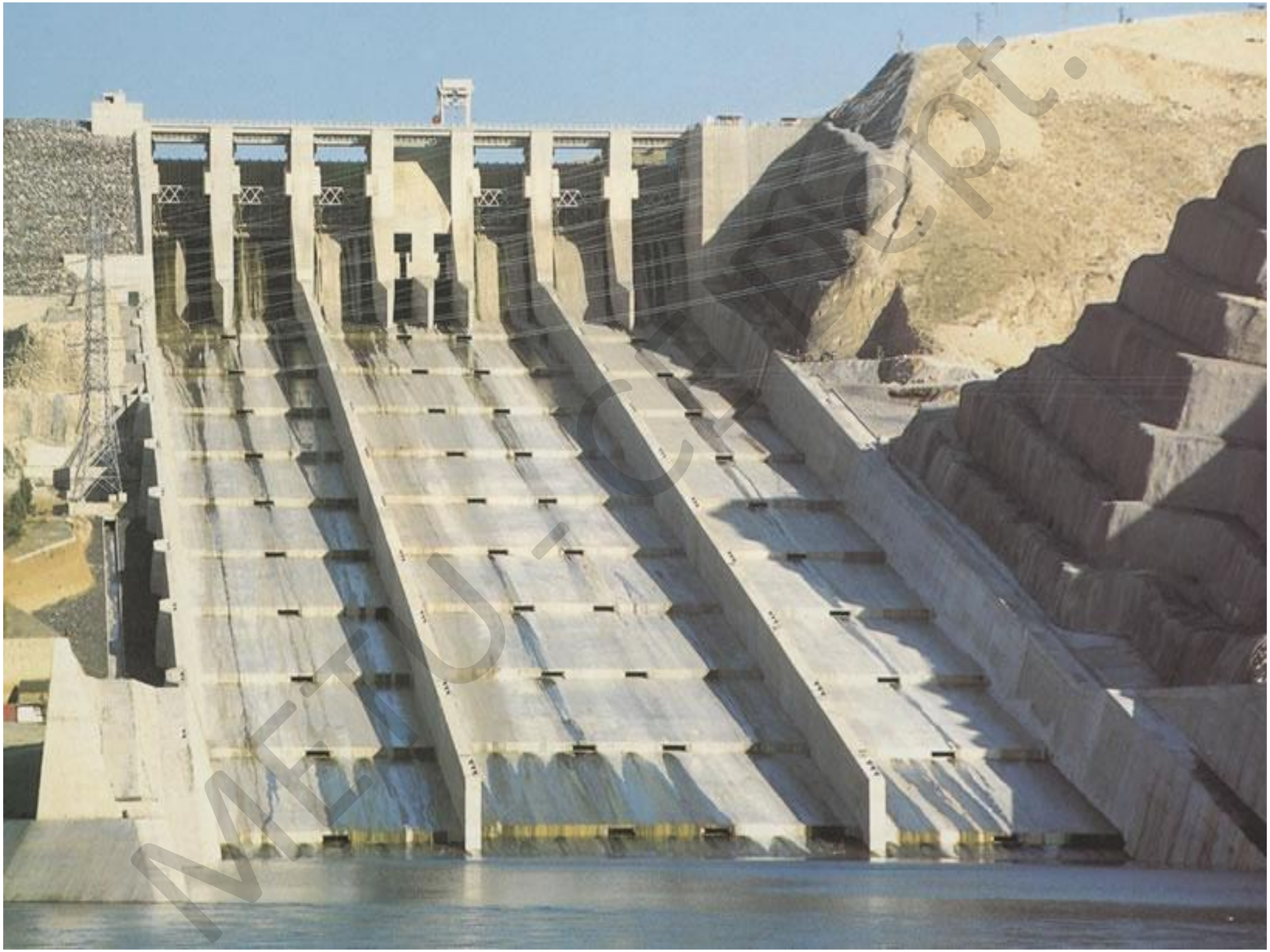


to prevent cavitation sets of ramps are placed on the face of the overflow spillway

a) Chute aeration without distribution duct @ locations where natural air entrainment does not suffice for concrete protection



b) Chute aeration with distribution duct



Ramp application at the face of Atatürk Dam's spillway

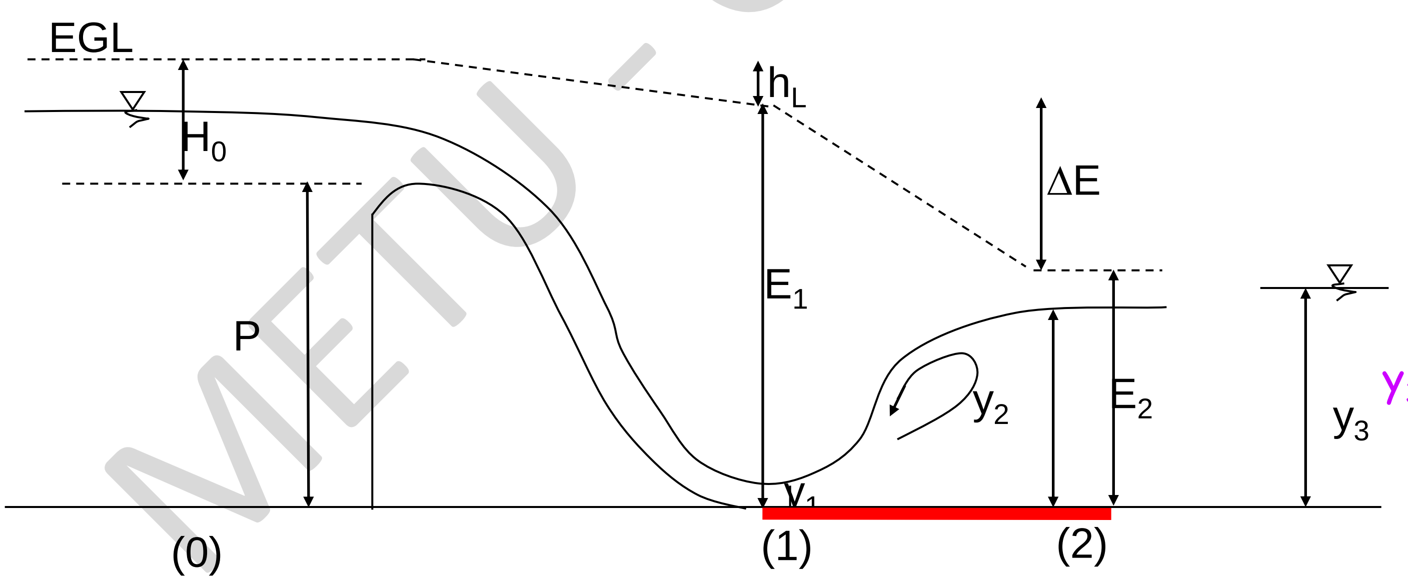
Energy Dissipation at Spillway Toe

Dissipation of energy at spillway toe:

- deflecting jet to the air
- forming hydraulic jump

  (regime from supercritical to subcritical)

energy dissipation basin → a stilling basin having a thick mat foundation called the apron & walls of sufficient height is formed to confine hydraulic jump at the downstream of the spillway



y_3 = tailwater depth

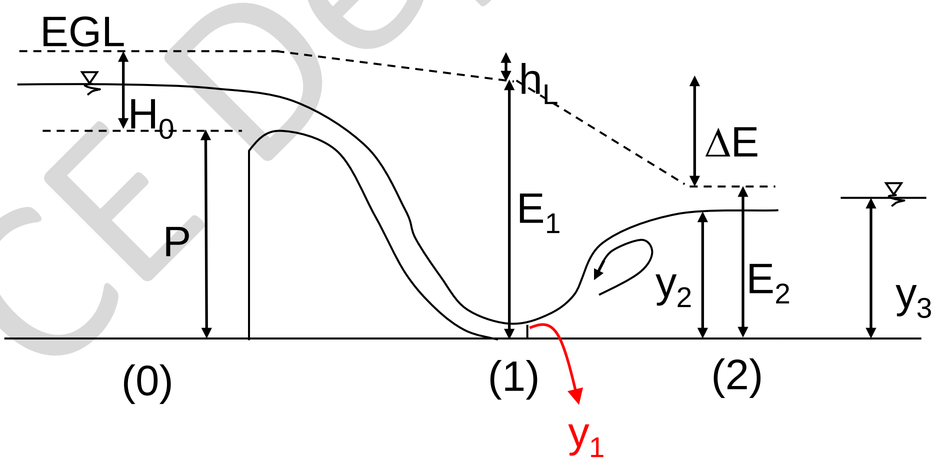
y_1 = initial depth of hyd. jump

y_2 = sequent depth

To find y_1 when Q_0 and L are known

Energy equation between sections (0) and (1):

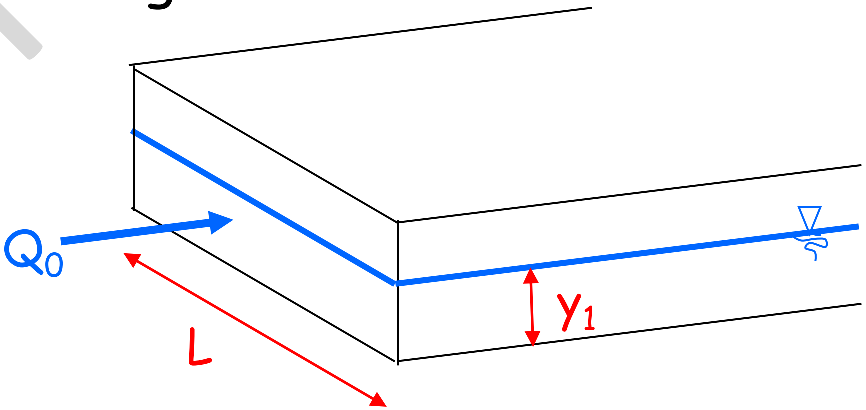
$$P + H_0 = y_1 + \frac{u_1^2}{2g} + h_L$$



- $h_L \approx 0.1 u_1^2/(2g)$
- Stilling basins are rectangular in cross-section

$q = Q_0/L$

$u_1 = q/y_1$



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

Solve for the initial depth of hydraulic jump, y_1

$y_1 \Rightarrow$ supercritical root (positive smaller root)

- To find y2 → energy eqn. b/w 1 and 2 can not be used since at 2 both y2 and ΔE are unknown

  write momentum equation b/w 1 and 2

$$\frac{y_2}{y_1} = \frac{1}{2}\left(\sqrt{1+8F_{r1}^2} - 1\right) \qquad F_{r1} = \frac{u_1}{\sqrt{gy_1}}$$

$$\Delta E = E_1 - E_2 = \frac{(y_2 - y_1)^3}{4y_1y_2}$$

Relative magnitudes of y_2 and y_3 dictate location of hydraulic jump

Hydraulic jump at the apron $\Rightarrow$ best solution !

$\therefore$ Avoid formation of hydraulic jump away from toe !

- jump at spillway face $\Rightarrow$ operational difficulty !
- jump at further downstream $\Rightarrow$ erosion problem !

Case 1: $y_3 = y_2$ ← ideal condition

- hydraulic jump forms just at the toe of the spillway

- USBR Type 1 basin: A horizontal apron

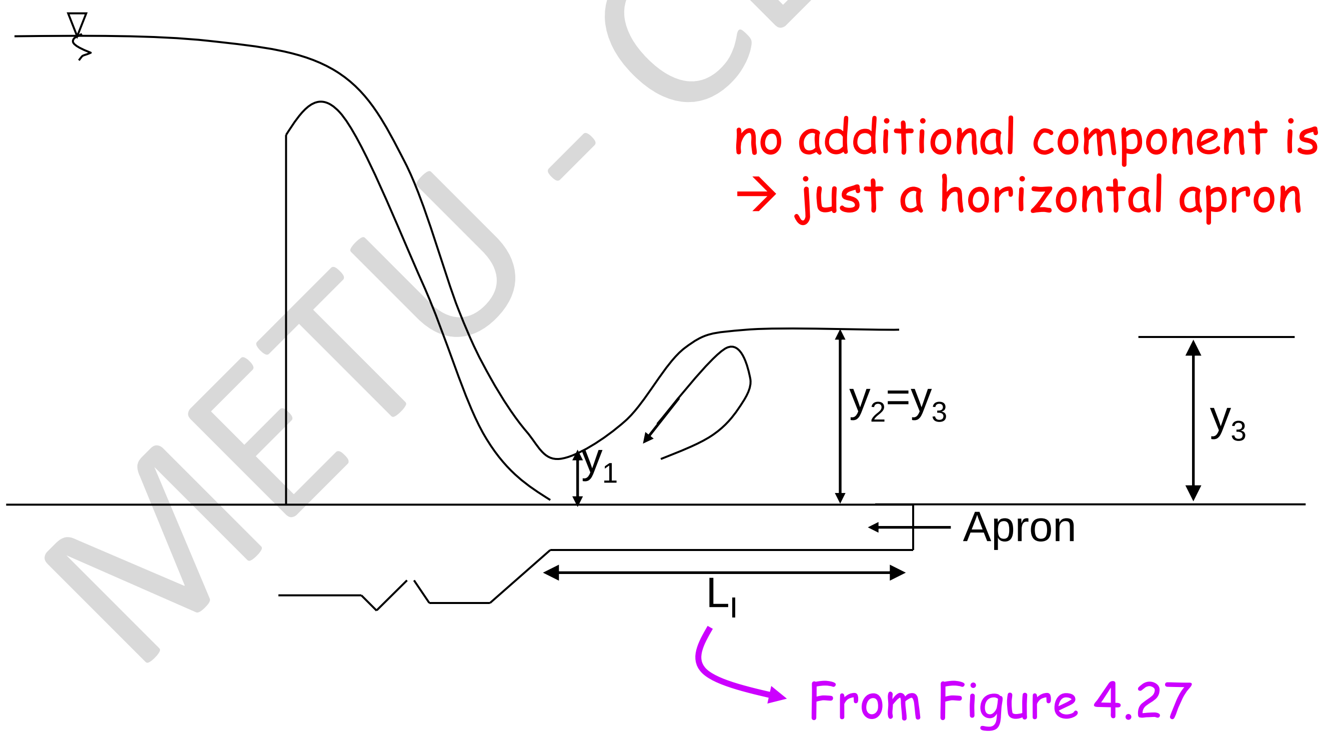
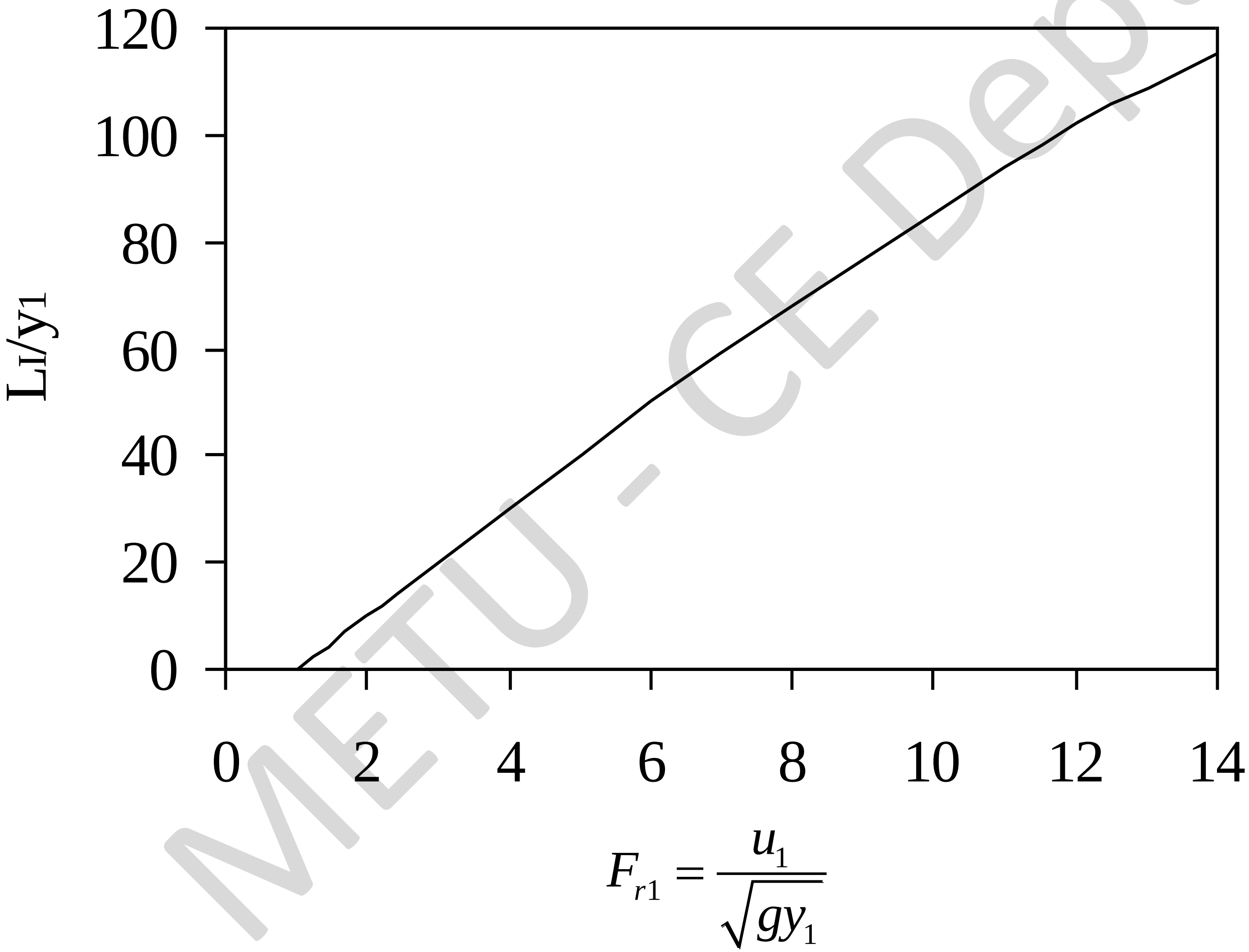


Figure 4.27

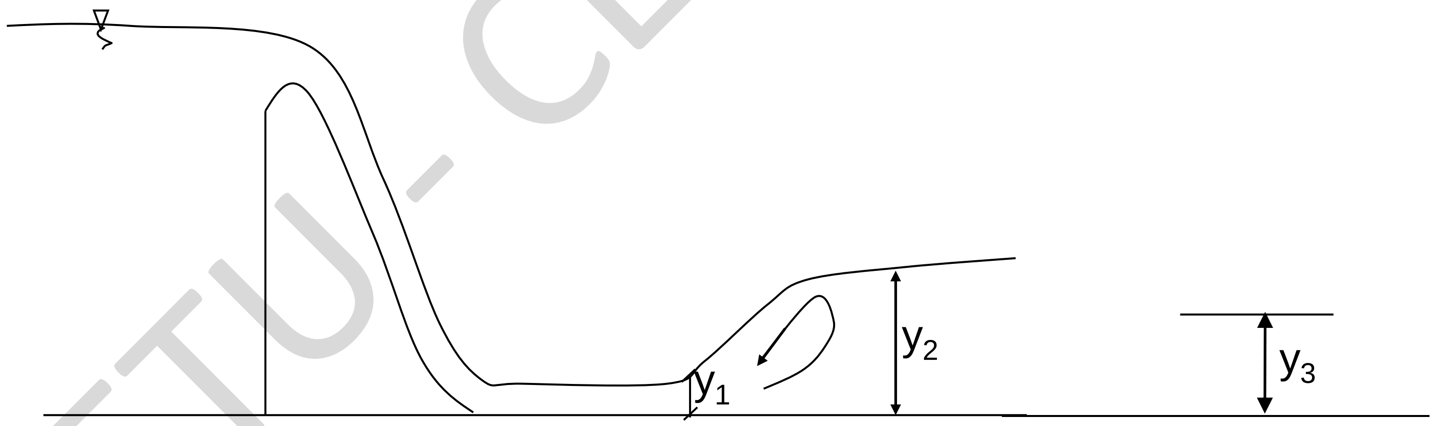


L_I = length of USBR Type I basin

Case 2: $y_3 < y_2$

$\therefore$ the jump moves toward the downstream !

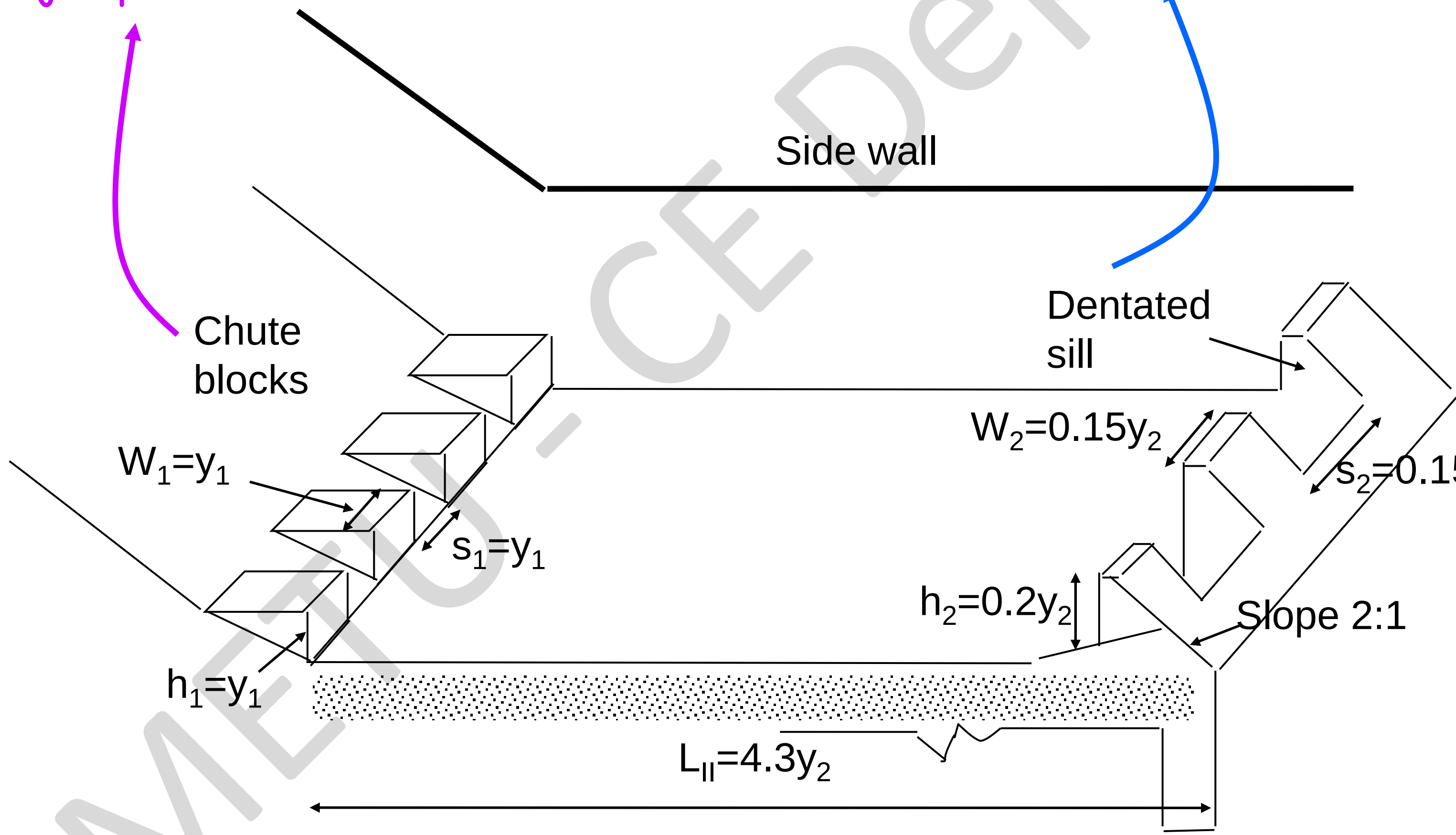
Very speedy flow ⇨ destructive effect on apron



- Excavate foundation by Δ so that jump is forced to occur in the basin
- USBR Basins Types 2, 3, 4

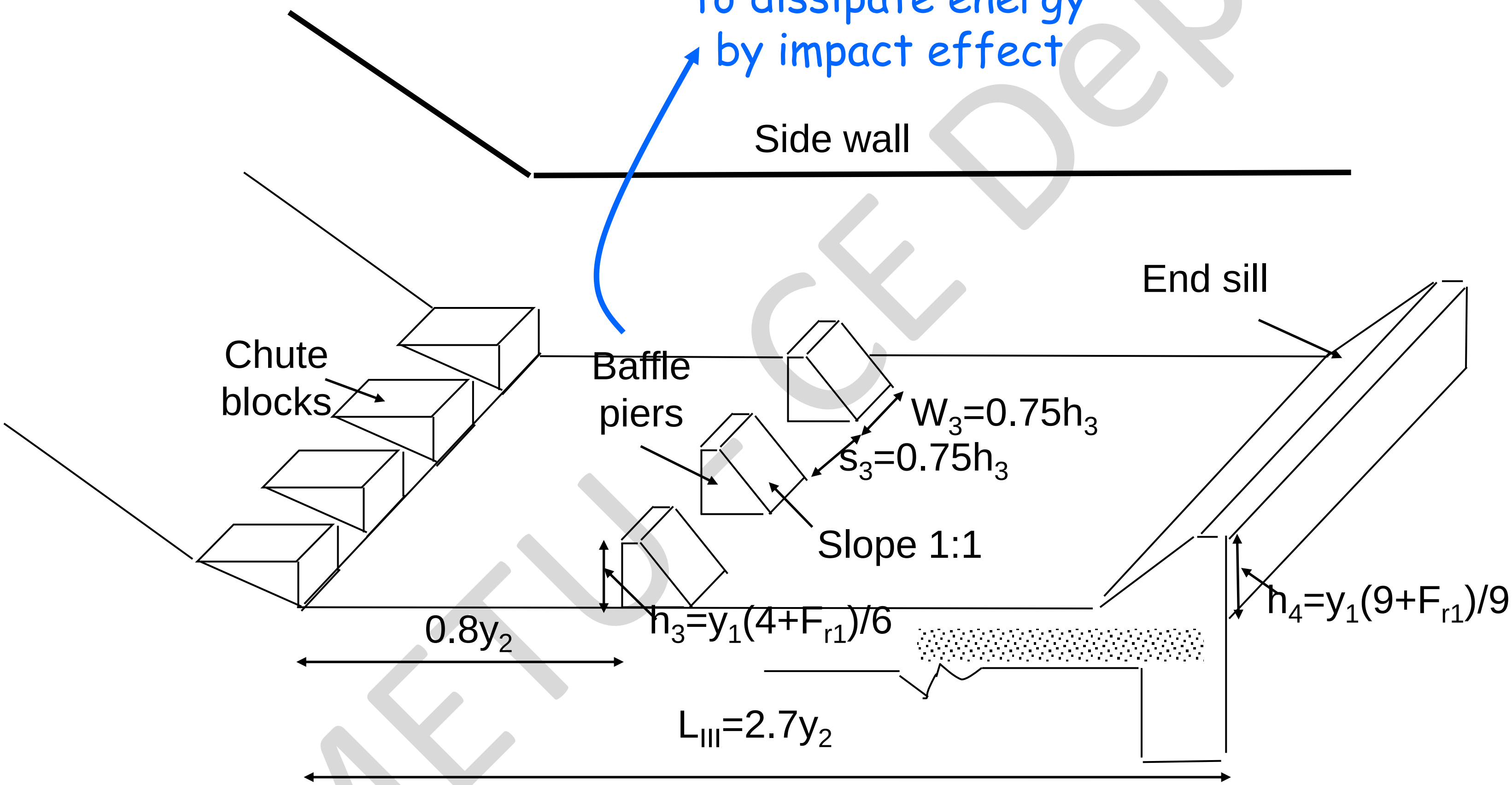
- to channelize the flow
- they shorten the length of the jump & stabilize it

- to further decrease the length of the jump &
- to control scour at the downstream of the basin

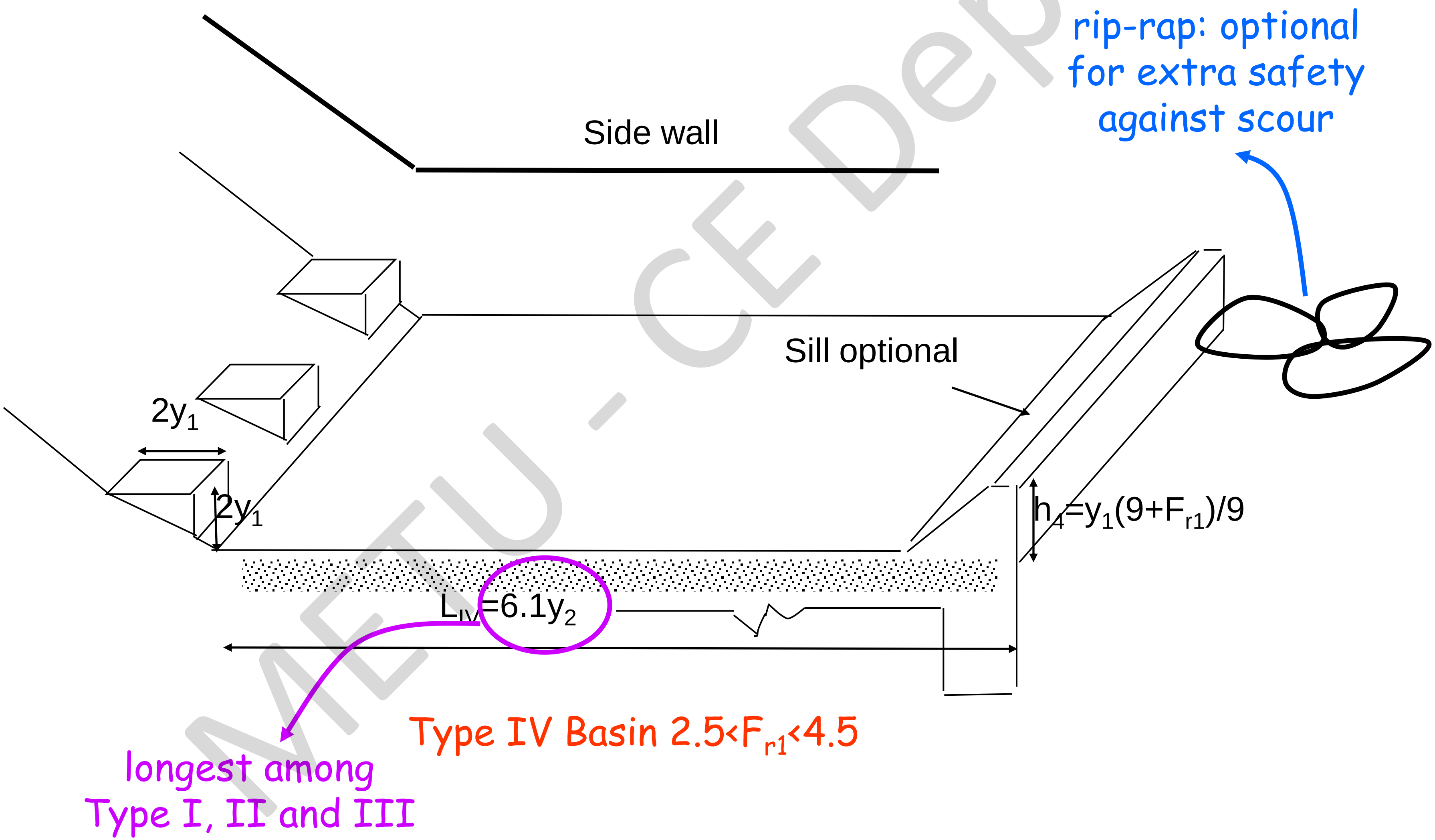


Type II Basin, $F_{r1}\geq 4.5$, $u_1\geq 15$ m/s if velocities are too high baffles are not good ← cavitation may occur

to dissipate energy by impact effect



Type III Basin, $F_{r1} \geq 4.5$, $u_1 < 15$ m/s

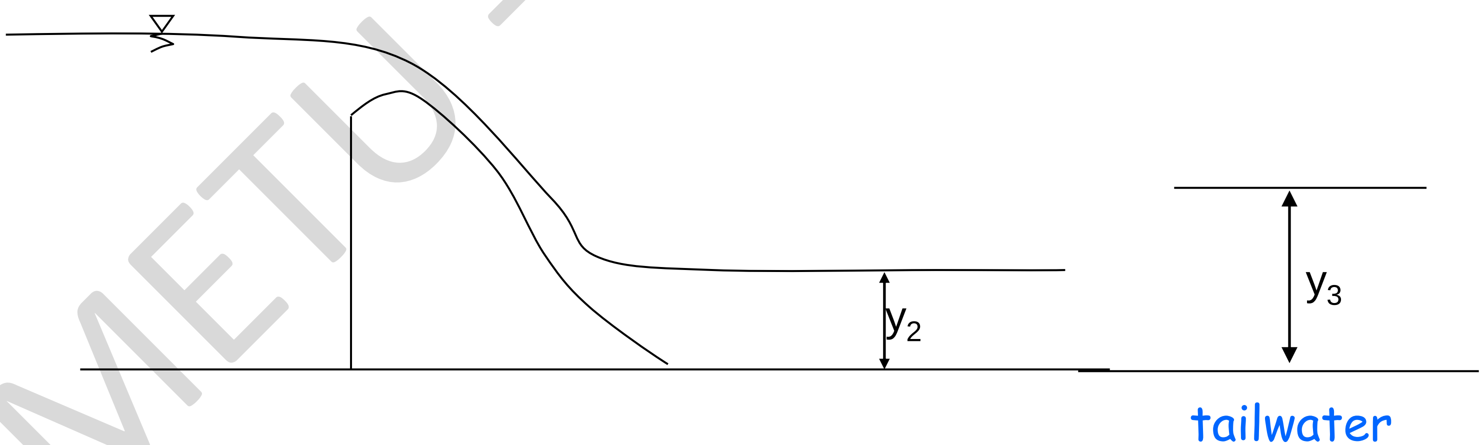


rip-rap: optional for extra safety against scour
Side wall
Sill optional
2y1
2y1
h4=y1(9+Fr1)/9
LIV=6.1y2
Type IV Basin 2.5<Fr1<4.5
longest among Type I, II and III

Case 3: $y_3 > y_2$

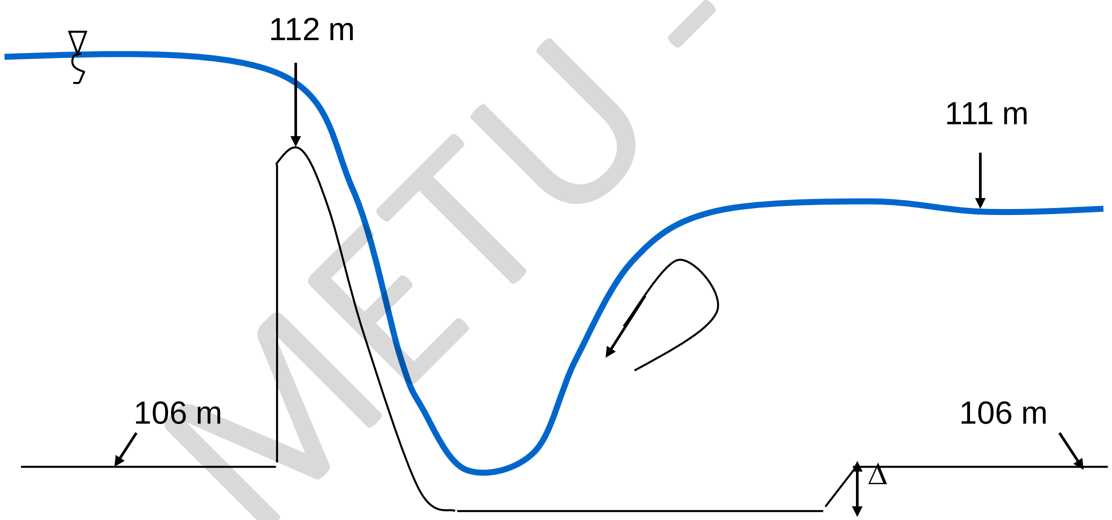
$\therefore$ jump moves towards the spillway face !

- A long sloping apron (USBR type 5 basin)
- culvert outlet (USBR type 6 basin)
- deflector bucket (USBR type 7 basin)



Example 15 Consider the overflow spillway shown in the figure below. The design spillway discharge Q_0 is 400 m^3/s. The total crest length of the spillway is 25 m. There are two rounded nosed piers, each one-meter thick. Headwalls of rounded abutments make angle less than 45° with the flow direction. Ignoring the headlosses over the spillway and over a possible downstream end sill, determine:

a) the design spillway head;
b) the required lowering of the river bed for the stilling basin;
c) the type and dimensions of the stilling basin.



Q_0 = 400 m^3/s
L = 25 m
t = 1 m
2 rounded piers

Solution

a) the design spillway head

$$L = L' - 2\,(N\,K_p + K_a)\,H_0 \qquad L' = 25 - 2*1 = 23\,\text{m}$$

K_p= 0.01, K_a= 0 (from Table 4.3, for rounded nose and angle < 45°)

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

$\neq Q_0 \rightarrow$ continue

$\neq Q_0 \rightarrow$ continue

$= Q_0 \rightarrow$ stop $\rightarrow H_0 = 4.04$ m

Therefore, the total head is H_0 = 4.04 m.

Elevation of total energy line at the upstream:

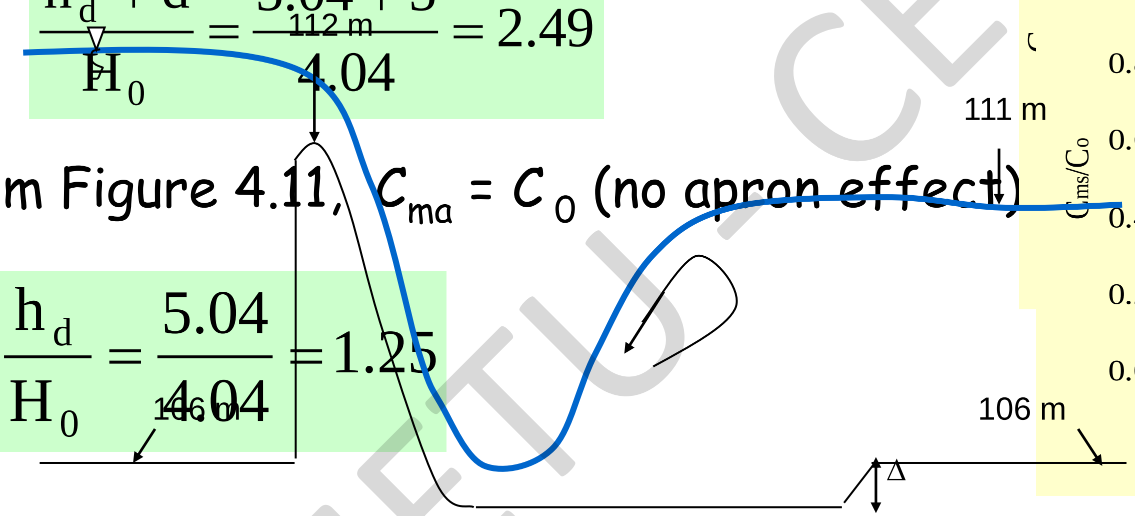
EGL_1 = 112 + 4.04 = 116.04 m.

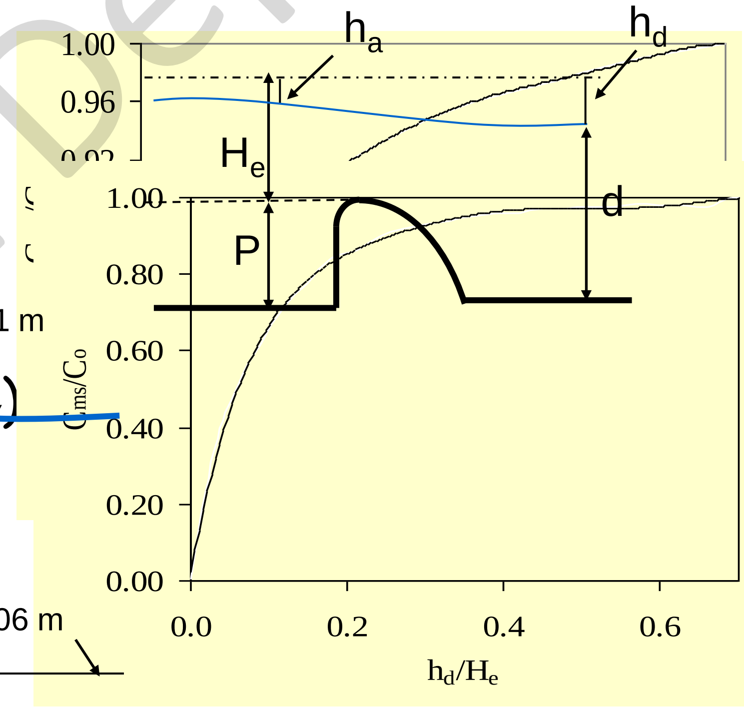
Let us check the effects of apron level and submergence on the spillway discharge.

The value of h_d, as required in Figures 4.11 and 4.12, is obtained as

h_d = 116.04 - 111 = 5.04 m.

$$\frac{h_d + d}{H_0} = \frac{5.04 + 5}{4.04} = 2.49$$





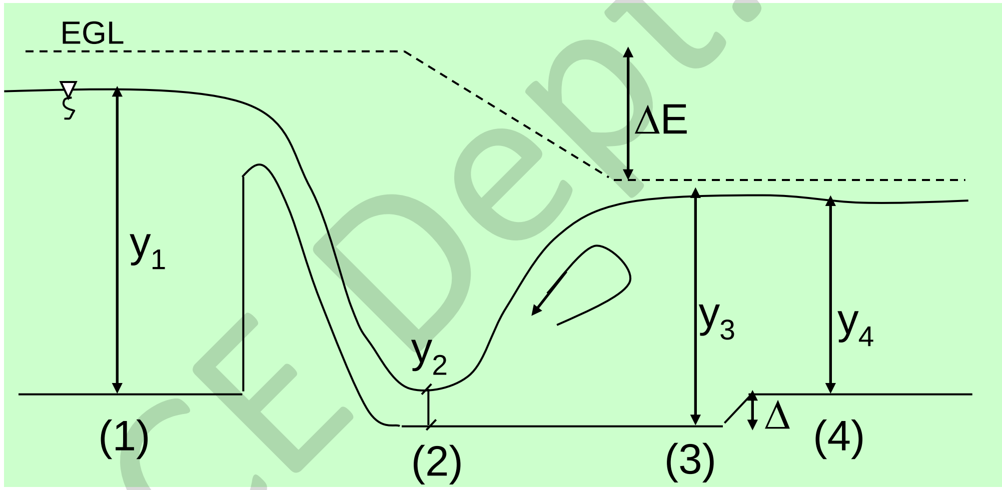
from Figure 4.11, $C_{ma} = C_0$ (no apron effect)

$$\frac{h_d}{H_0} = \frac{5.04}{4.04} = 1.25$$

from Figure 4.12, $C_{ms} = C_0$ (no submergence effect)

Downstream conditions would not retard the flow.

b) If the headloss over the spillway face and the end sill are ignored, the total energy loss between sections (1) and (4) will be only due to hydraulic jump.



In the tailwater:

q= 400/25 = 16 $m^3/s/m$,

y_4= 5 m,

u_4= 16/5 = 3.2 m/s

$$F_{r4} = \frac{u_4}{\sqrt{gy_4}} = \frac{3.2}{\sqrt{9.81*5}} = 0.46$$

F_{r4}= 0.46 < 1.0. Since subcritical flow conditions exist in the tailwater, a hydraulic jump occurs.

Vel. head at (4) $$h_{a4} = \frac{u_4^2}{2g} = \frac{(3.2)^2}{2*9.81} = 0.52\,\mathrm{m}$$

Elevation of the total energy line at the tailwater:

EGL_4 = 111 + 0.52 = 111.52 m

$\Delta E = EGL_1 - EGL_4$ = 116.04 -111.52 = 4.52 m.

$$\Delta E = \frac{(y_3 - y_2)^3}{4 y_3 y_2}$$ and $$\frac{y_3}{y_2} = \frac{1}{2}\left(\sqrt{1 + 8F_{r2}^2} - 1\right)$$

Value of $8F_{r2}{}^2$ is $$8F_{r2}{}^2 = \frac{8\,u_2^2}{g\,y_2} = \frac{8\,q^2}{g\,y_2^3} = \frac{8*16^2}{9.81\,y_2^3} = \frac{208.77}{y_2^3}$$

Value of y_3 is entered into the ΔE eqn. as: $$y_3 = \frac{y_2}{2}\left(\sqrt{1 + \frac{208.77}{y_2^3}} - 1\right)$$

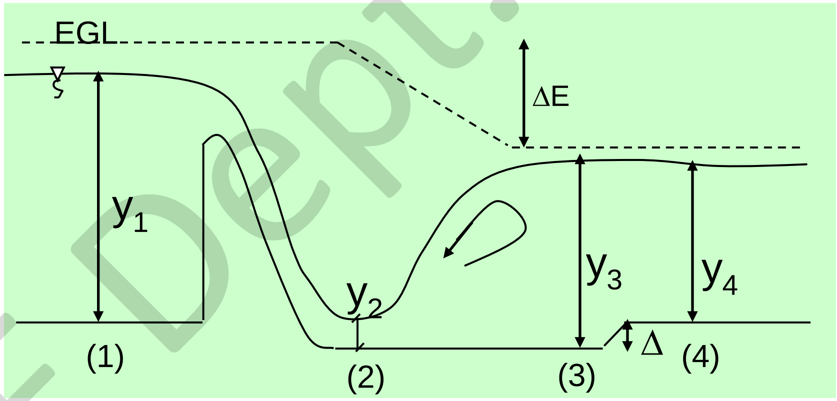
$$\frac{\left[\frac{y_2}{2}\left(\sqrt{1+\frac{208.77}{y_2^3}} - 1\right) - y_2\right]^3}{2y_2^2\left(\sqrt{1+\frac{208.77}{y_2^3}} - 1\right)} = 4.52m$$

By trial and error, the value of y_2 is determined from this equation as 1.15 m.

Then $$y_3 = \frac{y_2}{2}\left(\sqrt{1+\frac{208.77}{y_2^3}} - 1\right) = \frac{1.15}{2}\left(\sqrt{1+\frac{208.77}{1.15^3}} - 1\right) = 6.18\,m$$

The same value can also be obtained from Table 4.6

To use Table 4.6, find y_c then $\Delta E/y_c$



$$y_c = \sqrt[3]{\frac{q^2}{g}} = \sqrt[3]{\frac{16^2}{9.81}} = 2.966$$

$$\frac{\Delta E}{y_c} = \frac{4.52}{2.966} = 1.51$$ → $y_3/y_2 = 5.372$

$y_3 = 5.372 * 1.15 = 6.18$ m

Since $y_3 > y_4$, river bed should be excavated by a certain depth of Δ, which is determined from energy equation between sections (3) and (4)

$$y_3 + \frac{q^2}{2gy_3^2} = \Delta + y_4 + \frac{q^2}{2gy_4^2}$$ → $\Delta = 1.0$ m

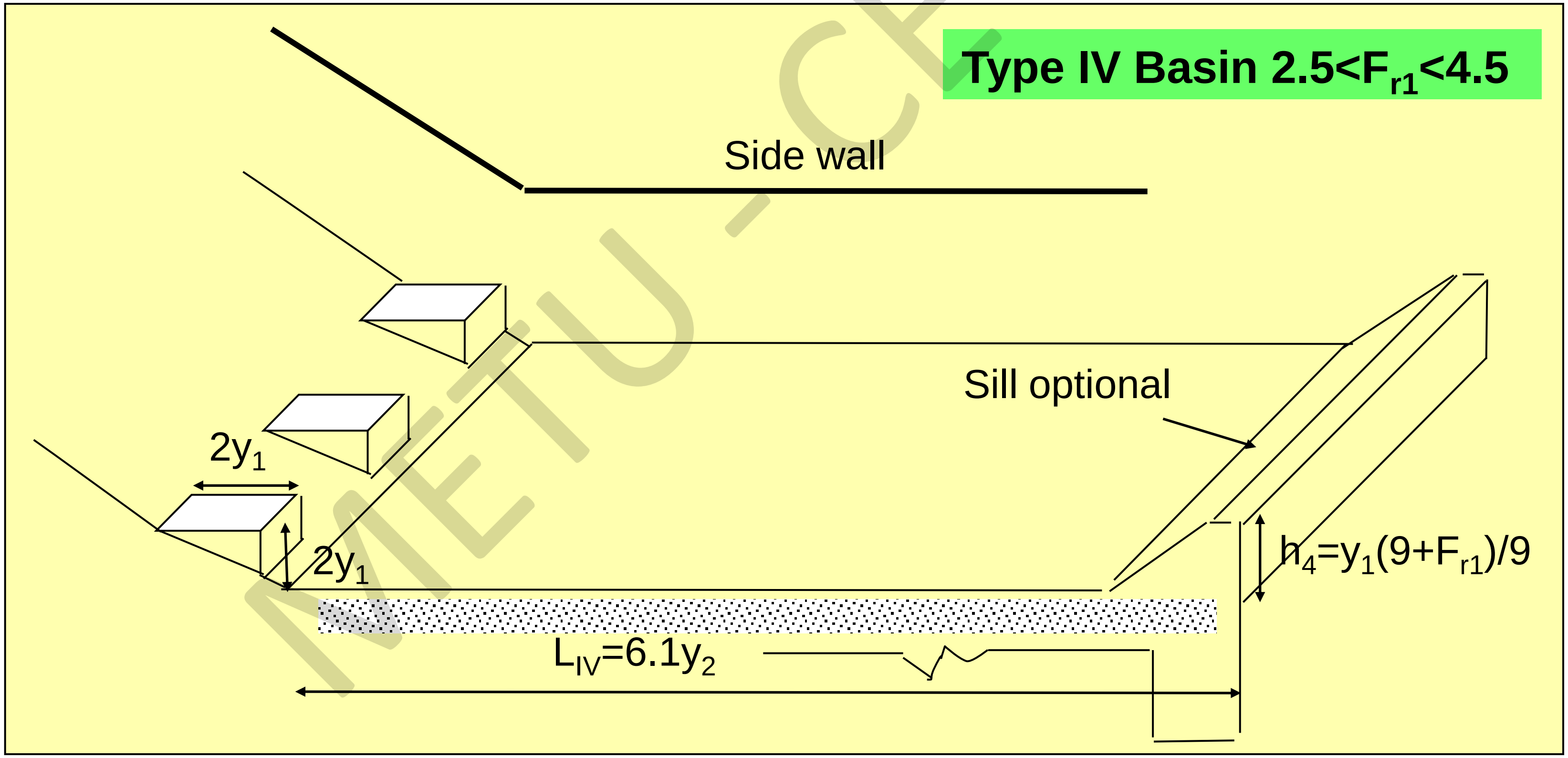
Then, bottom elevation of the stilling basin is 106 - 1 = 105 m.

c) Type and dimensions of the stilling basin:

$$u_2 = \frac{16}{1.15} = 13.91\,\text{m/s}$$

$$F_{r2} = \frac{13.91}{\sqrt{9.81*1.15}} = 4.14$$

Since $2.5<F_{r2}<4.5$ and $u_2<15$ m/s, USBR Type IV basin will be designed, using characteristic dimensions given in Figure 4.30.



Length of basin → $L_4 = 6.1\, y_3 = 6.1 * 6.18 = 37.7$ m.

Height of chute blocks above the stilling basin → $h_1 = 2\, y_2 = 2.30$ m.

Height of the end sill can also be computed according to the relation given in Figure 4.30 for USBR type 4 basin as:

$$h_4 = \frac{y_2\left(9 + F_{r2}\right)}{9} = \frac{1.15\left(9 + 4.14\right)}{9} = 1.68\,\mathrm{m}$$

However, the minimum value of Δ required to confine the jump at the spillway toe is 1.0 m as determined before.

So, Δ = 1.0 m is considered to be satisfactory for economic reasons.