4. DESIGN OF SPILLWAYS

A reservoir will overflow if its capacity is less than the difference between the volumes of inflow and outflow.

A spillway is designed to prevent overtopping of a dam at a place that is not designed for overflow.

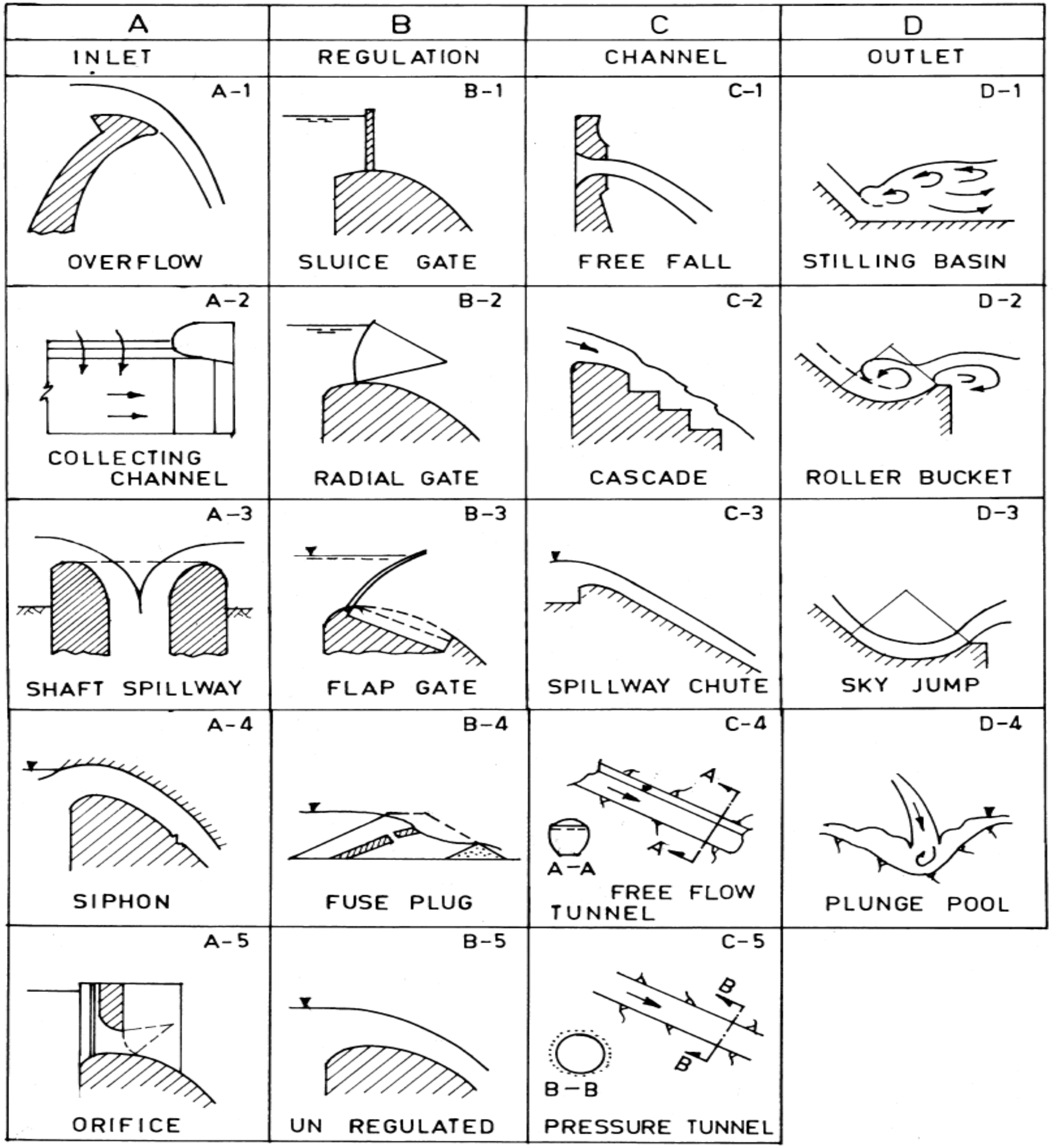
Principal function of a spillway is to pass down the surplus water from the reservoir into the downstream river.

Spillway is a safety structure against dam overtopping.

A spillway is used to maintain optimum reservoir levels before and during flood-control operations by releasing excess flood water.

40 % of the dam failure hazards is due to inadequate spillway capacity.

Classification of Spillway (shown in Vischer et al, San Francisco,1988).



Classification of spillways according to the most prominent feature

- Ogee spillway
- Chute spillway
- Side channel spillway
- Shaft spillway
- Siphon spillway
- Straight drop or overfall spillway
- Tunnel/Culvert spillway
- Labyrinth spillway
- Stepped spillway

Aspects involved in spillway design

- Hydrology
  - ➢ Estimation of inflow discharge
  - ➢ Selection of spillway design flood
  - ➢ Determination of frequency of spillway use

- Topography and geology
  - ➢ Type and location of spillway

- Utility and operational aspects
  - ➢ Serviceability

- Constructional and structural aspects
  - ➢ Cost-effectiveness

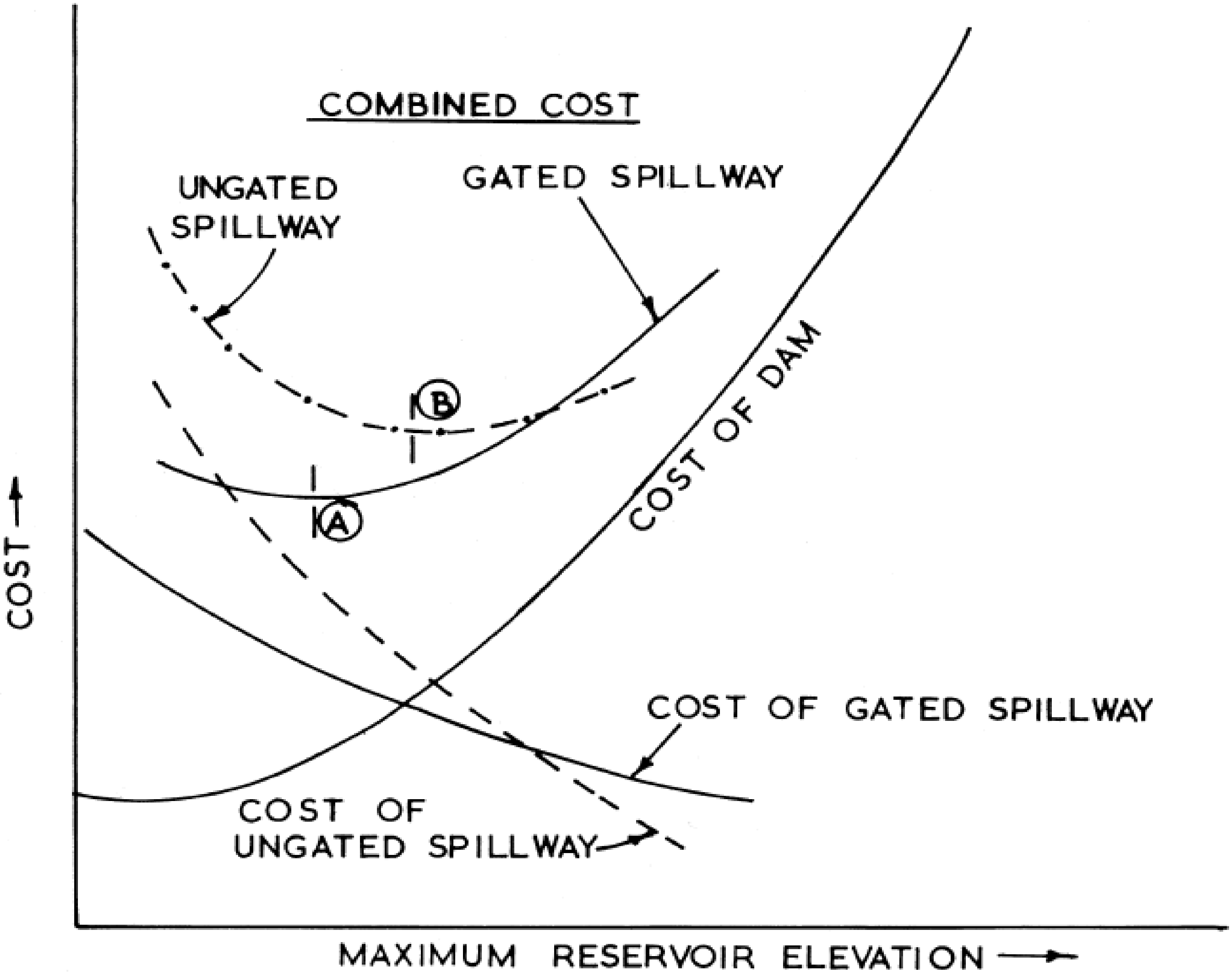
- After a spillway control device and its dimensions have been selected, the maximum spillway discharge and the maximum reservoir water level should be determined by flood routing.

- Other components of the spillway can then be proportioned to conform to the required capacity and to the specific site conditions and a complete layout of the spillway can be established.

- Cost estimates of the spillway and the dam should be made.

- Comparisons of various combinations of spillway capacity and dam height for an assumed spillway type, and of alternative types of spillways allow selection of an economical spillway type.

- Ungated-gated spillways


COMBINED COST
UNGATED SPILLWAY
GATED SPILLWAY
B
A
COST OF DAM
COST
COST OF GATED SPILLWAY
COST OF UNGATED SPILLWAY
MAXIMUM RESERVOIR ELEVATION

4.1 Design Discharge of Spillway

- The lifespan of a dam is of the order of 100 years.
- The design discharge may be related to the maximum flood discharge that may occur within this period.
- Probability of occurrence of a discharge that can seriously damage the system should be minimum. As an example (depending on project size and country regulations) :
- For $\mathbf{Q_{100}}$ optimum flow conditions observed.
- For $\mathbf{Q_{1000}}$ some adverse flow conditions may be tolerated, but there should be no damage.
- For $\mathbf{Q_{10000}}$ minor damage may be tolerated but system should not fail.

- Another approach is based on the concept of probable maximum flood (PMF). The most extreme combination of basic parameters is chosen, and no return period is specified.

- This design flood has to be diverted without a dam breaching.

- Often, a full reservoir level is assumed and all intakes for power plants etc. are blocked, and (N-1) spillway outlets are in operation.

- Whether the bottom outlet can be accounted for diversion is a question, but there is a tendency to include it in the approach.

4.2 Overflow Structures

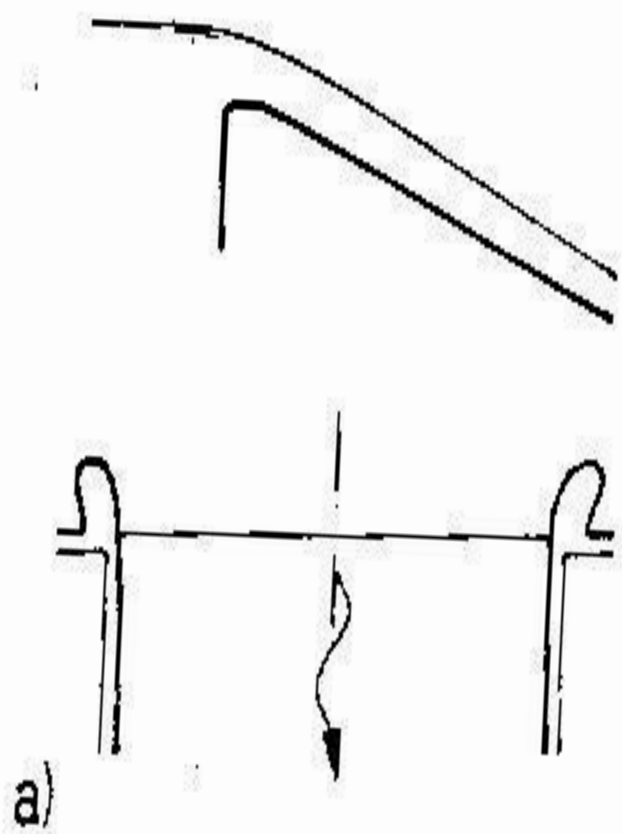
- Depending on the site conditions and hydraulic particularities an overflow structure can be of various designs:

    - Frontal overflow,
    - Side-channel overflow, and
    - Shaft overflow.

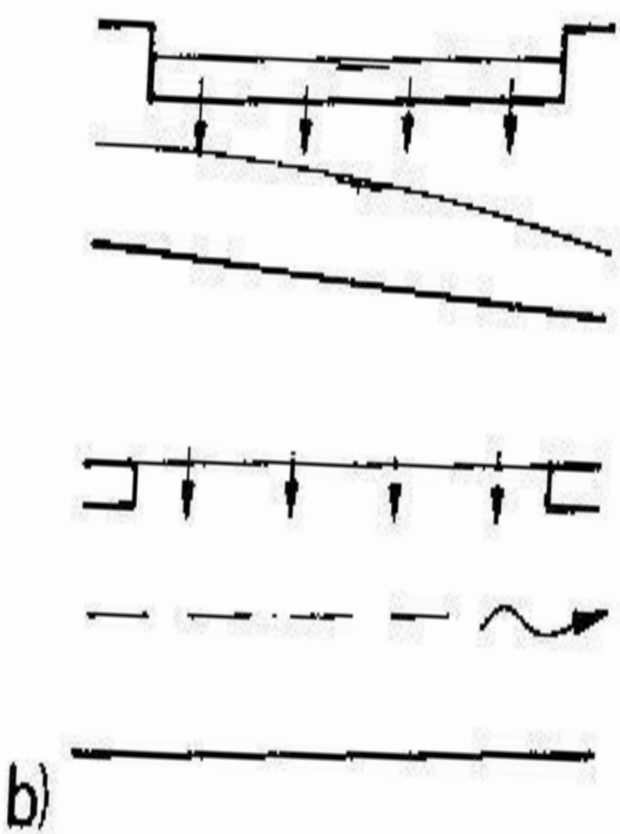
- Other types of structures such as labyrinth spillway use a frontal overflow but with a crest consisting of successive triangles or trapezoids in plan view.

- Still another type is the orifice spillway in the arch dam.

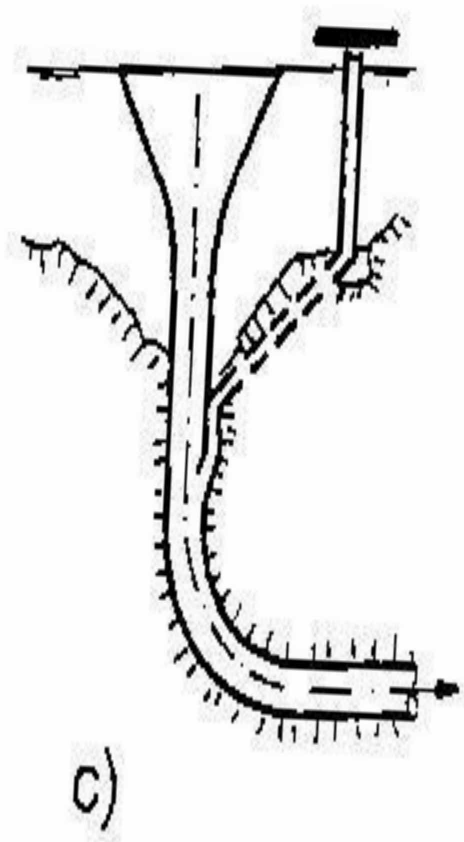
- The non-frontal overflow type of spillways are used for small and intermediate discharges, typically up to design floods of 1000 m^3/s.



Frontal Overflow



Side Overflow



Shaft Overflow

4.2.1 Frontal Overflow

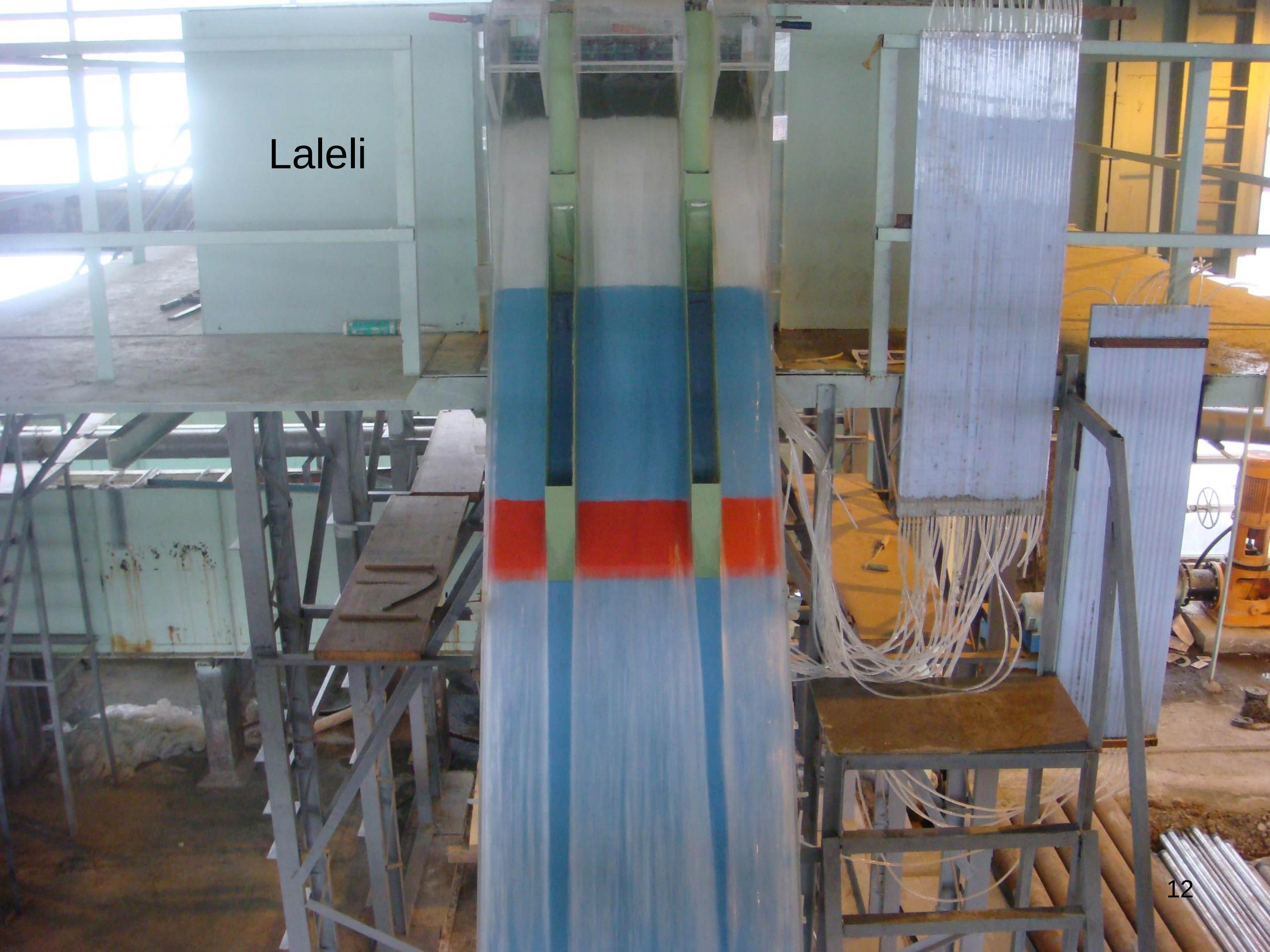
- The frontal type of overflow is a standard overflow structure, both due to simplicity and direct connection of reservoir to tailwater. It can normally be used in both arch and gravity dams.

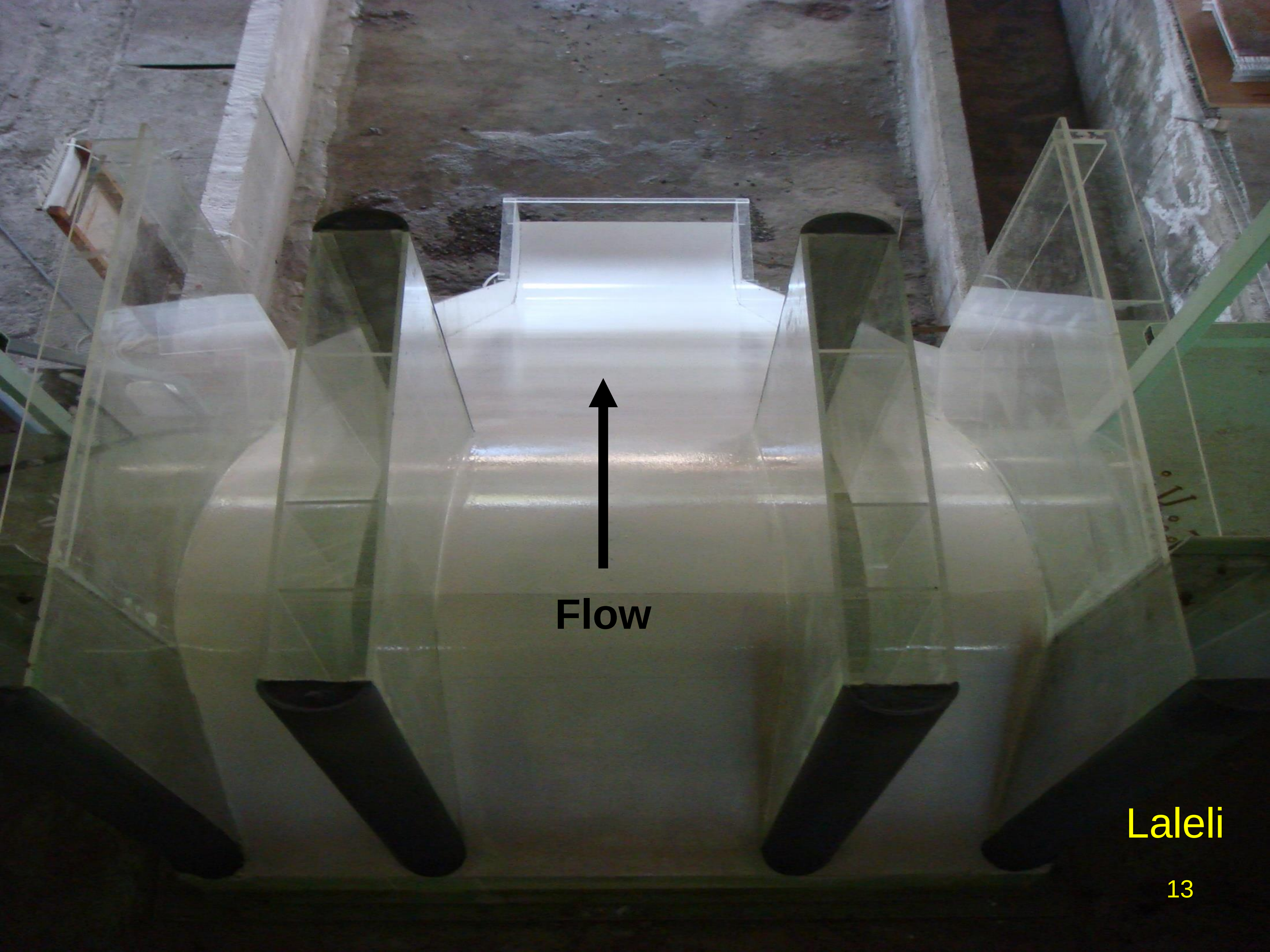
- The frontal overflow can easily be extended with gates and piers to regulate the reservoir level, and to improve the approach flow to spillway.

- Gated overflows of 20 m gate height and more have been constructed, with a capacity of 200 m^3/s per unit width. Such overflows are thus suited for medium and large dams, with large floods to be conveyed to the tailwater.

- Particular attention has to be paid to cavitation due to immense heads that may generate pressure below the vapor pressure in the crest domain.

Laleli



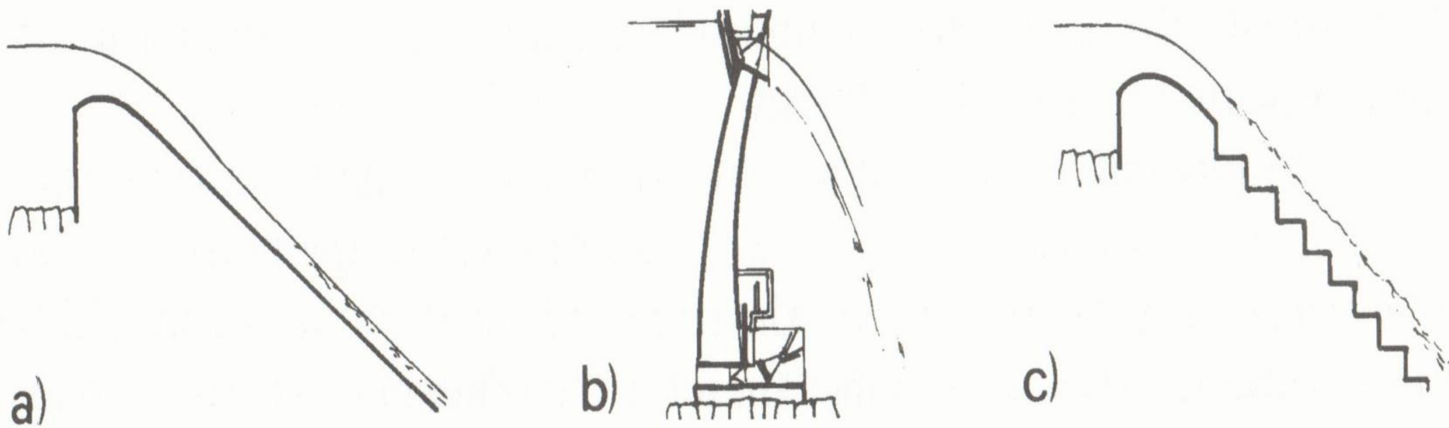


Laleli

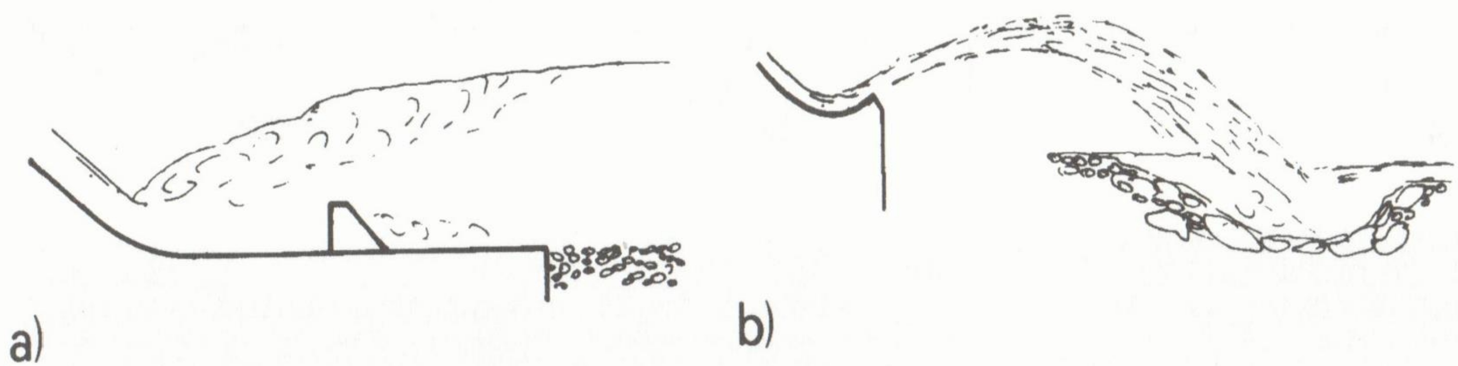

Laleli

- The gate piers have to be carefully shaped in order to obtain a symmetric approach flow.
- The downstream of frontal overflow may have various shapes. Usually, a spillway is connected to the overfall crest as a transition between overflow and energy dissipator.
- The crest may abruptly end in arch dams to include a falling nappe that impinges on the tailwater.
- Another design uses a cascade spillway to dissipate energy right away from the crest end to the tailwater, such that a reduced stilling basin is needed.
- The standard design involves a smooth spillway that convey flow with a high velocity either directly to the stilling basin, or to a trajectory bucket.

Connection between frontal overflow and dissipator (a) chute spillway, (b) free fall, (c) cascade



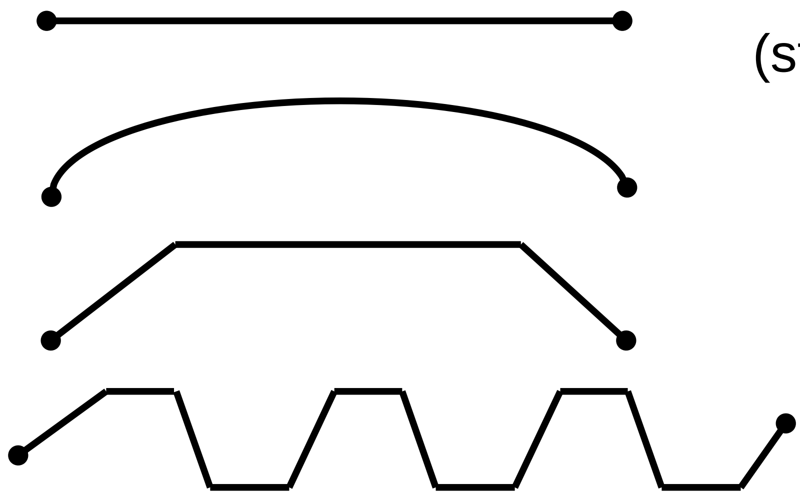
Connections between spillway and tailwater (a) stilling basin, (b) trajectory basin



Crest Shapes

Overflow structures of different shapes are:

1. Straight (standard)
2. Curved
3. Polygonal
4. Labyrinth



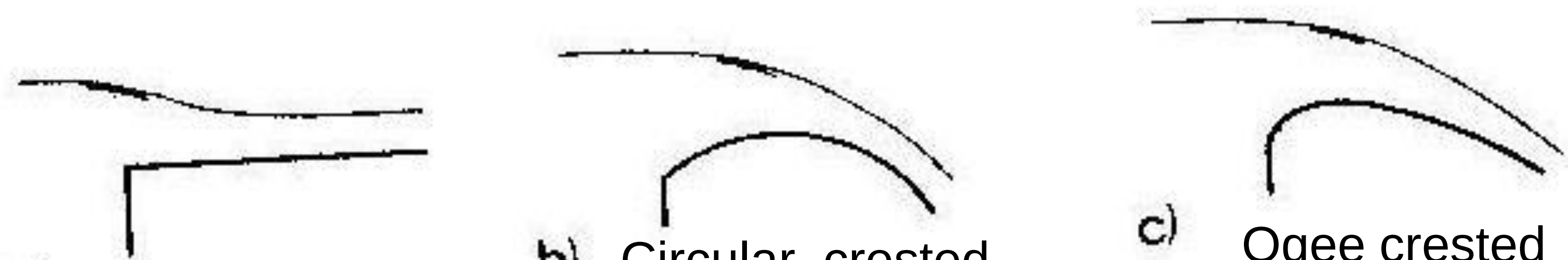
Plan view

The labyrinth structure has an increased overflow capacity with respect to the width of the structure.

Labyrinth spillway

Standard Crest Shape

- In order to have a symmetric downstream flow, and to accommodate gates, the rectangular cross section is used almost throughout.



a) Broad crested

b) Circular crested

c) Ogee crested (Standard)

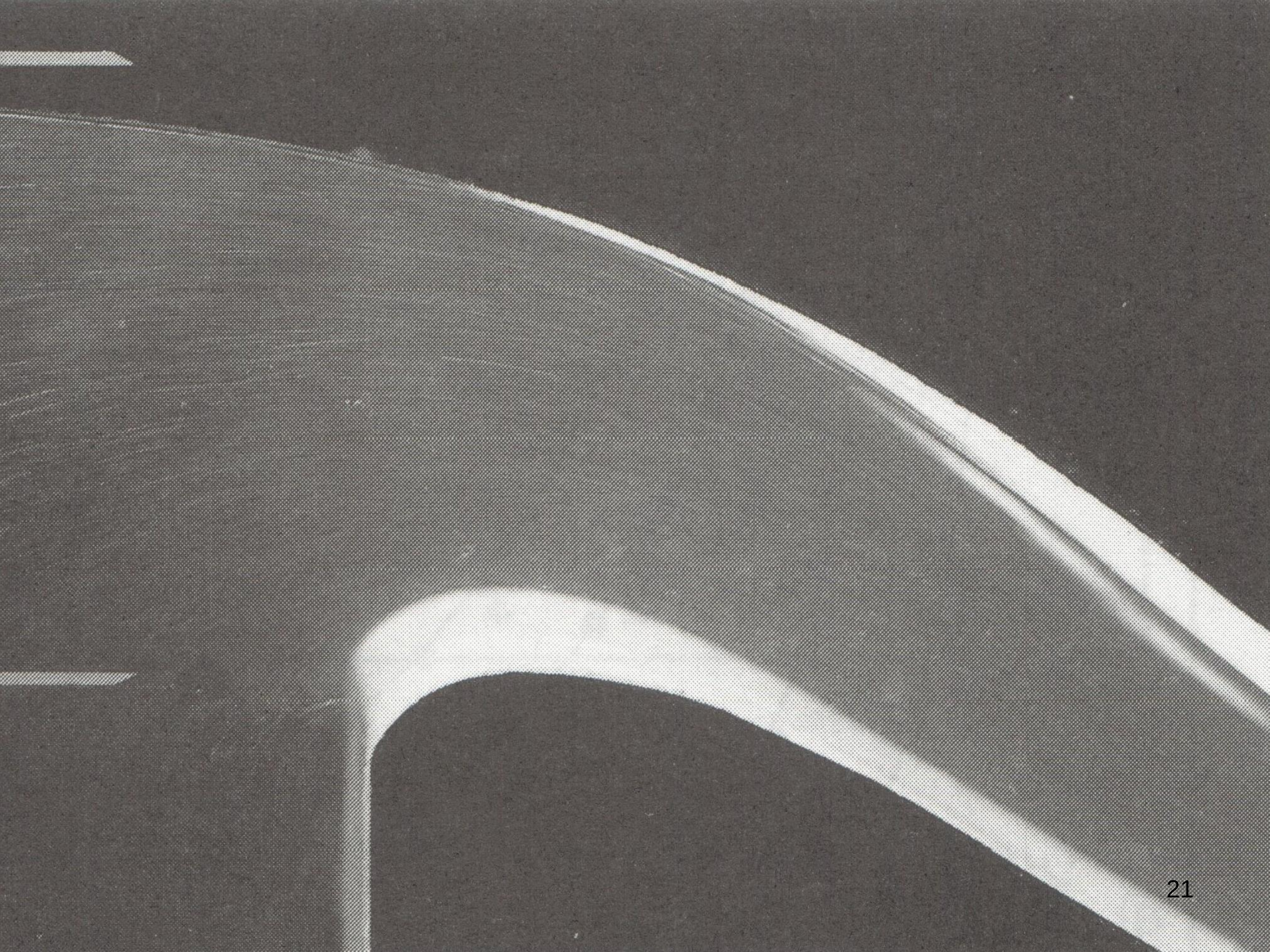
- For heads larger than 3 m, the standard overflow shape should be used.
- Although its cost is higher than the other crest shapes, advantages result both in capacity and safety against cavitation damage.

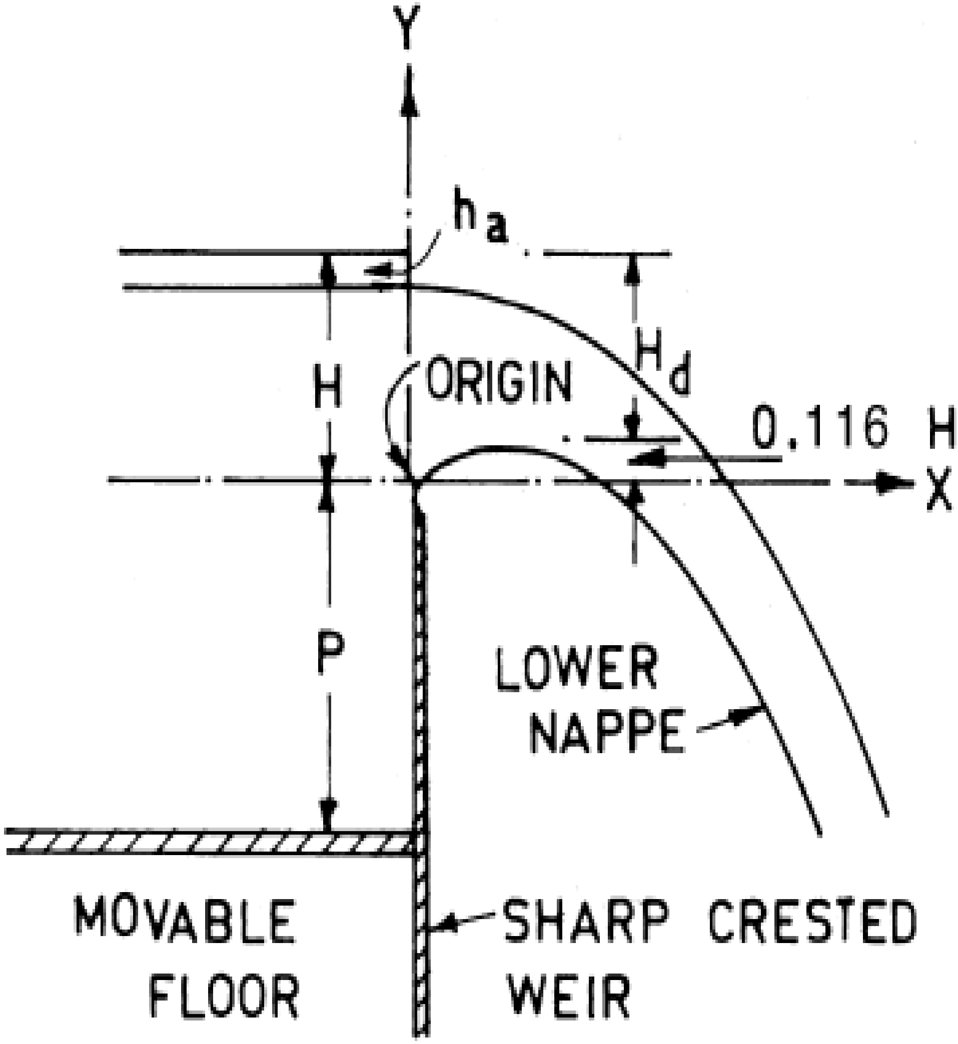
- When the flow over a structure involves curved streamlines with the origin of curvature below the flow, the gravity component of a fluid element is reduced by the centrifugal force.

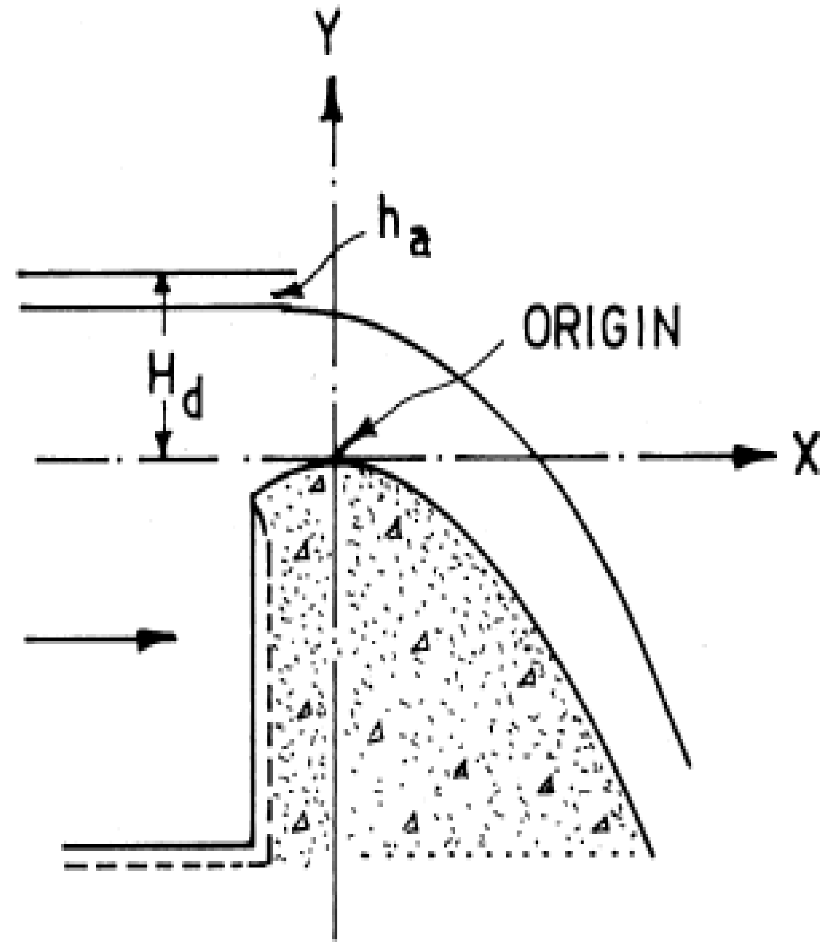
- If the curvature is sufficiently large, the internal pressure may drop below the atmospheric pressure and even attain values below the vapor pressure for large structures.

- Then cavitation may occur with a potential cavitation damage. As discussed, the overflow structure is very important for the dam safety. Therefore, such conditions are unacceptable.

- For medium and large overflow structures, the crest is shaped so as to conform the lower surface of the nappe from a sharp-crested weir.




Y
h_a
H_d
H
ORIGIN
0.116 H
X
P
LOWER
NAPPE
MOVABLE
FLOOR
SHARP CRESTED
WEIR


Y
h_a
ORIGIN
H_d
X
OGEE PROFILE

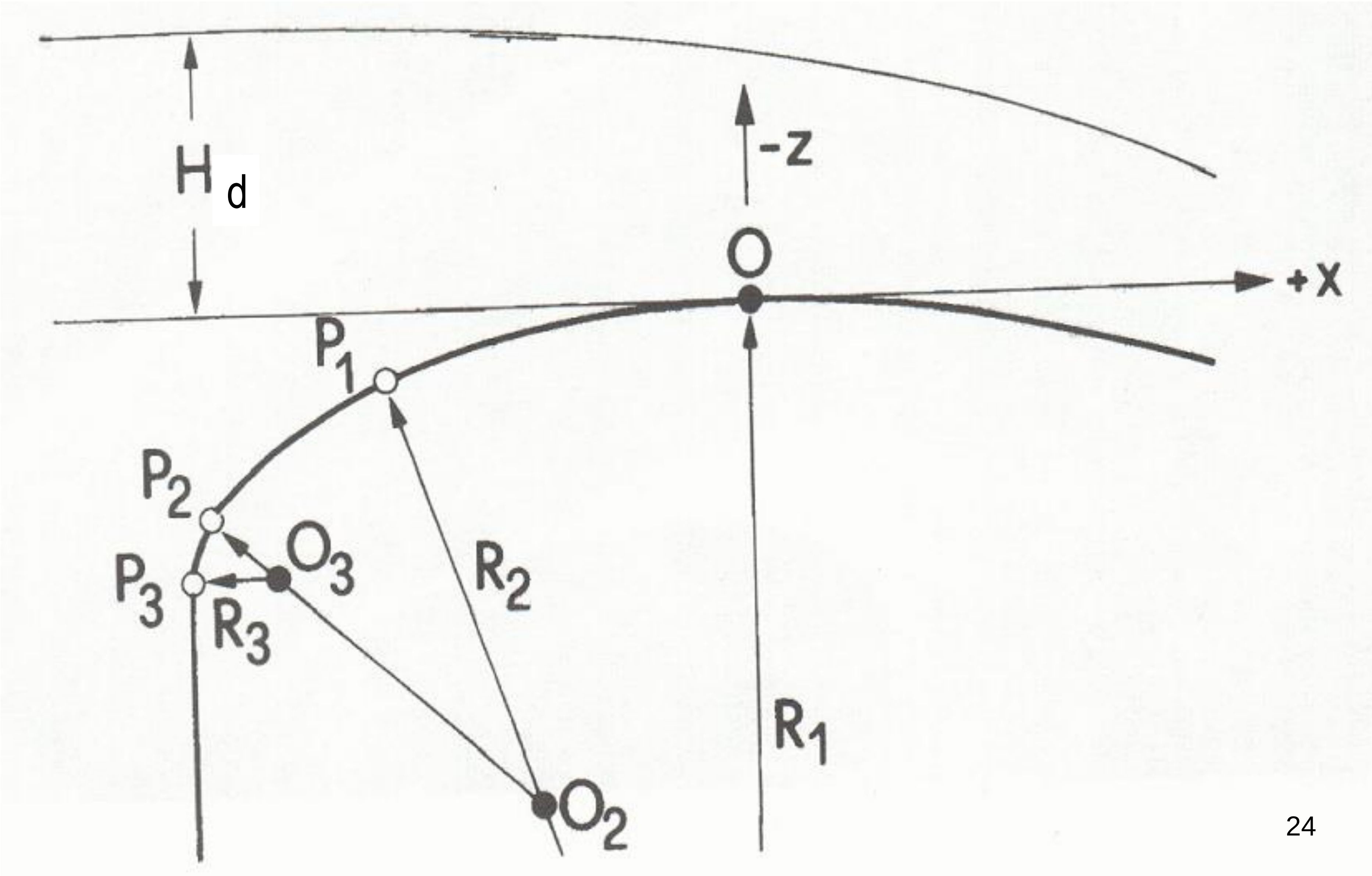
- The shape of the crest is important regarding the bottom pressure distribution. Slight modifications have a significant effect on the bottom pressure, while the discharge characteristics remain practically the same.

- The geometry of the lower nappe cannot simply be expressed analytically. The best known approximation is due to US Corps of Engineers (USCE1970).

- They proposed a three arc profile for the upstream quadrant and a power function for the downstream quadrant, with the crest as origin of Cartesian coordinates (x, z).

- The significant scaling length for the standard overflow structure is the so-called design head, H_d.

USCE Crest Shape



The radii of the upstream crest profile are:

$$\frac{R_1}{H_d} = 0.50, \qquad \frac{R_2}{H_d} = 0.20, \qquad \frac{R_3}{H_d} = 0.04$$

The origins of curvature O_1, O_2, and O_3, as well as the transition points P_1, P_2, and P_3, for the upstream quadrant are;

Point	**O_1**	**O_2**	**O_3**	**P_1**	**P_2**	**P_3**
x/H_d	0.00	-0.105	-0.242	-0.175	-0.276	-0.2818
z/H_d	0.500	0.219	0.136	0.032	0.115	0.136

- The downstream quadrant crest shape was originally proposed by Craeger as:

$$\frac{z}{H_d} = 0.50\left(\frac{x}{H_d}\right)^{1.85}, \quad \text{for x} > 0$$

- This shape is used up to so-called tangency point with a transition to the straight-crested spillway.

- The disadvantage of USCE crest shape is the abrupt change of curvature at locations P_1 to P_3 and at the origin. Such a crest geometry can not be used for computational approaches due to the curvature discontinuities.

- The crest shape given above for vertical spillways for which the velocity of approach is zero, i.e.; for $H_d/P \rightarrow 0$, where P is the height of the spillway. In general, the shape of the crest depends on:

  - The design head H_d,
  - The inclination of the upstream face,
  - The height of the overflow section above the floor of the entrance channel (which influences the velocity of approach to the crest).

Discharge Characteristics

The discharge over an ogee crest is given by the formula:

$$Q = C_d LH(2gH)^{1/2}$$

where*:*

Q = discharge

C_d = discharge coefficient

L = overflow crest length

H = total head on the crest

The discharge coefficient is influenced by a number of factors:

- The depth of approach
- Relation of actual crest shape to the ideal nappe shape
- Upstream face slope
- Downstream apron interface
- Downstream submergence

The discharge coefficient may be written as function of relative head up to $H^{+} = H/H_{d} = 3$

$$C_d = \frac{2}{3\sqrt{3}}\left[1 + \frac{4H^{+}}{9 + 5H^{+}}\right]$$

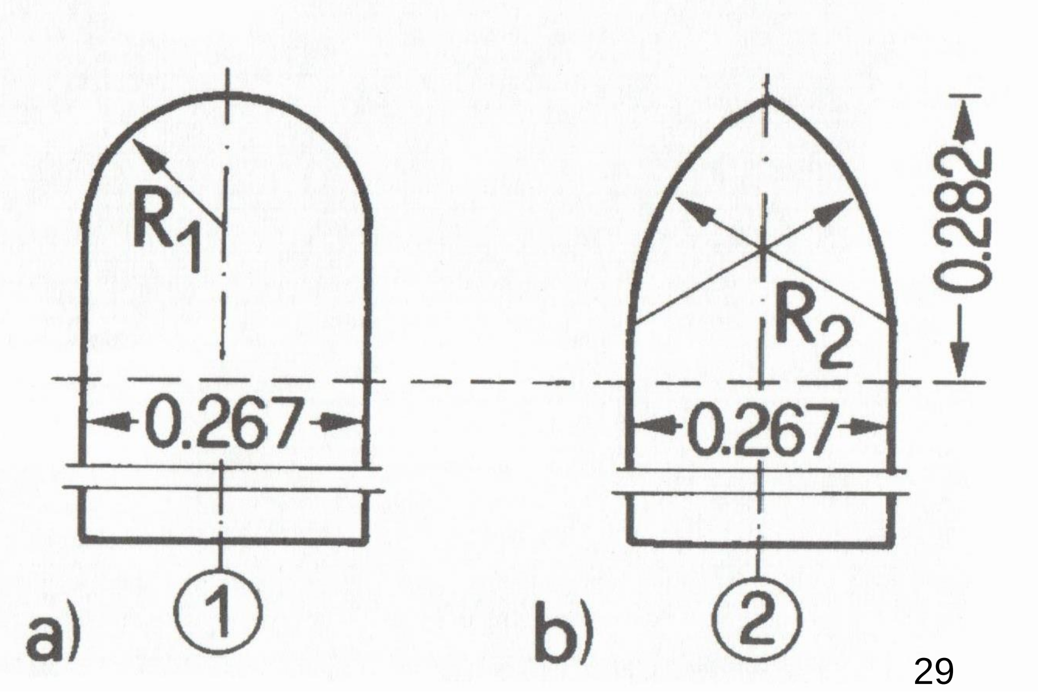
For $H^{+} \rightarrow 0$, the overflow is shallow and almost hydrostatic pressure occurs. Then overflow depth is equal to the critical depth and the discharge coefficient is $C_{d} = 0.385$. For the design flow $H^{+} = 1$ and $C_{d} = 0.495$.

Pier Effects

Piers on overflow structures are provided:

- to improve approach flow conditions
- to mount overflow gates
- to divide the spillway into sub-channels
- to aerate the chute flow at the pier ends

Two typical front pier shape designs are standard. Numbers to be multiplied by the design head, H_d.



- Crest piers and abutments cause contraction of the flow, reduction in the effective length of the crest, and cause reduction in discharge.

$$L' = L - 2(NK_p + K_a)H_e$$

where L = Effective length of the crest for calculating discharge

L' = Net length of the crest

N = number of piers

K_p = Pier contraction coefficient

K_a = Abutment contraction coefficient

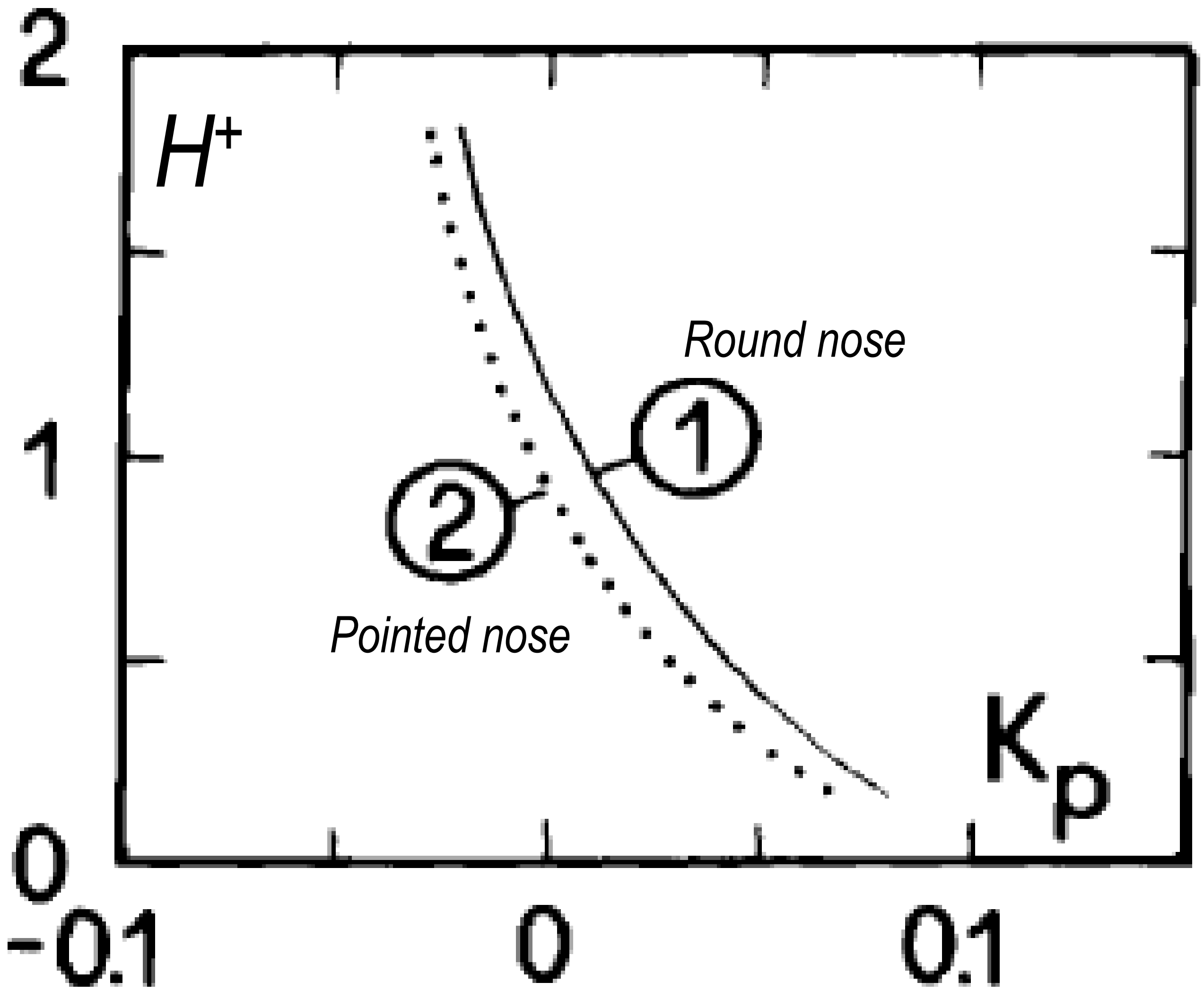
H_e = Total head on the crest

K_p = 0.2 for square abutments

K_p = 0.1 for abutments rounded by radius between $(0.15\sim0.5)H_d$

K_p = 0. for abutments rounded by radius $>0.5H_d$

Pier contraction coefficients



- Tailwater end of an overflow pier corresponds to an abrupt expansion of flow.
- Because the spillway flow is supercritical, standing shock waves have their origins at the pier ends, which will propagate all along the chute.
- In order to suppress pier waves two designs are available:
    - Either sharpening the pier end both in width and height, or
    - Continue with the pier as a dividing wall along the chute
- Both designs are not ideal, because even a slim pier end perturbs the flow and dividing walls may be costly especially for long spillways.

Free Surface Profile

- The free surface over an overflow structure is important in relation to freeboard design and for gated flow

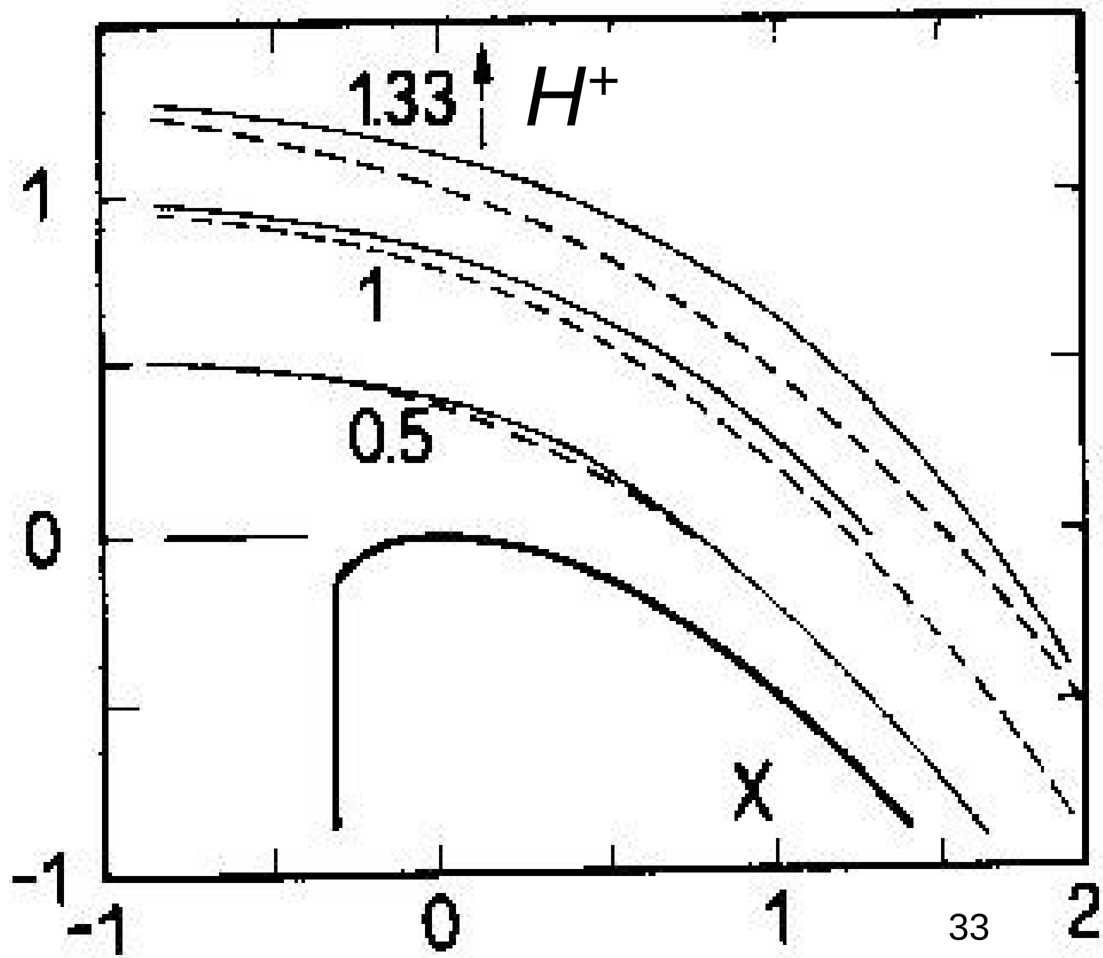
- A generalized approach for plane flow over standard-shaped overflow crest can be written as:

$$S = 0.75\left[(H^+)^{1.1} - X/6\right]$$

- for $-2 < X/(H^+)^{1.1} < +2$

- where $S = s/H_d$, $X = x/H_d$

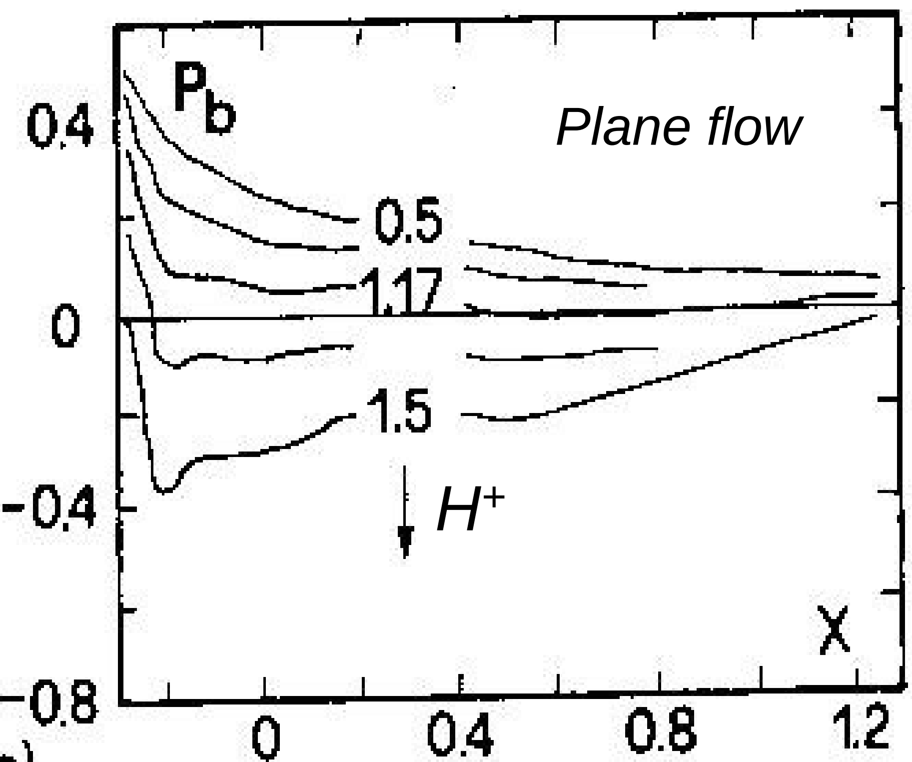
- The surface elevation s is referred to the crest level upstream from the crest origin, and to the bottom elevation downstream from the crest.

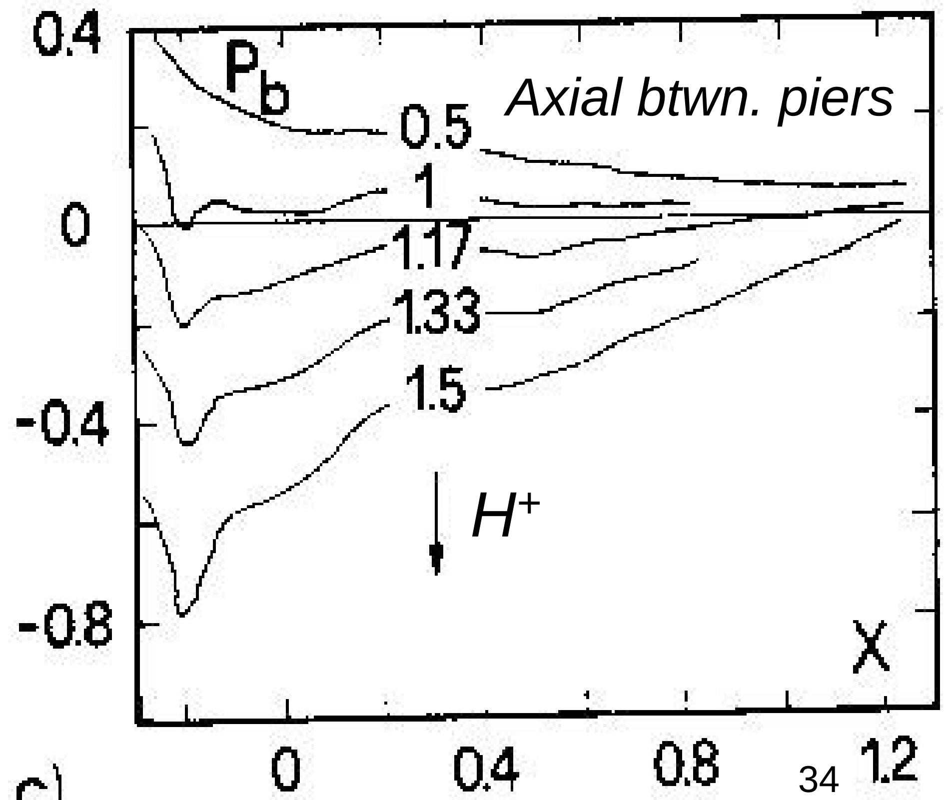


Bottom pressure characteristics

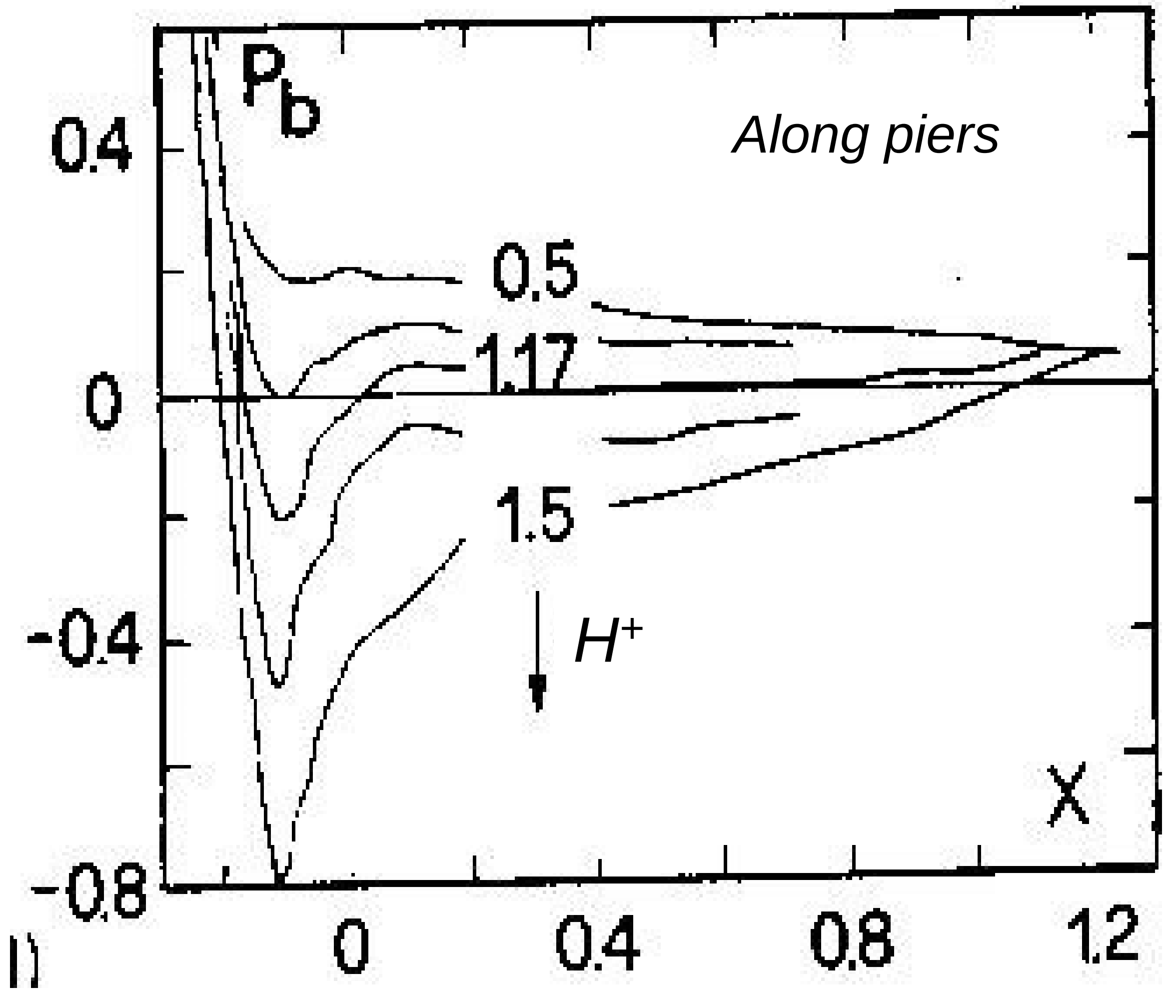
- The bottom pressure distribution $p_b(x)$ is important, because it yields:
  - ➢ an index for the potential danger of cavitation damage, and
  - ➢ the location where piers can end without inducing separation of flow.

The nondimensionalized bottom pressure heads ($P_b = p_b/\gamma H_d$) for various H^+ *values are* shown in figures below.





- The most severe pressure minima along the piers due to significant streamline curvature effects.



- A generalized formula for the minimum pressure in plane flow is given as

$$\overline{P}_m = (1 - H / H_d) \qquad \text{where} \qquad \overline{P}_m = p_m / \gamma H$$

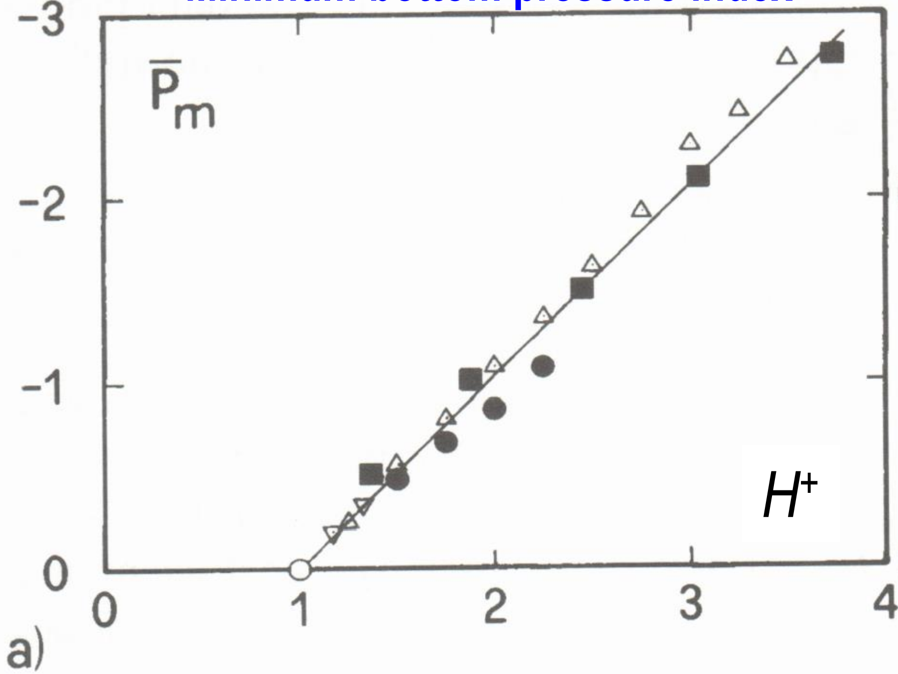
- Variation of the discharge coefficient with H^+ *and* α *is shown.*

- The crest bottom pressure index ($\overline{P}_c = p_c / \gamma H$) *is shown as function of* H^+

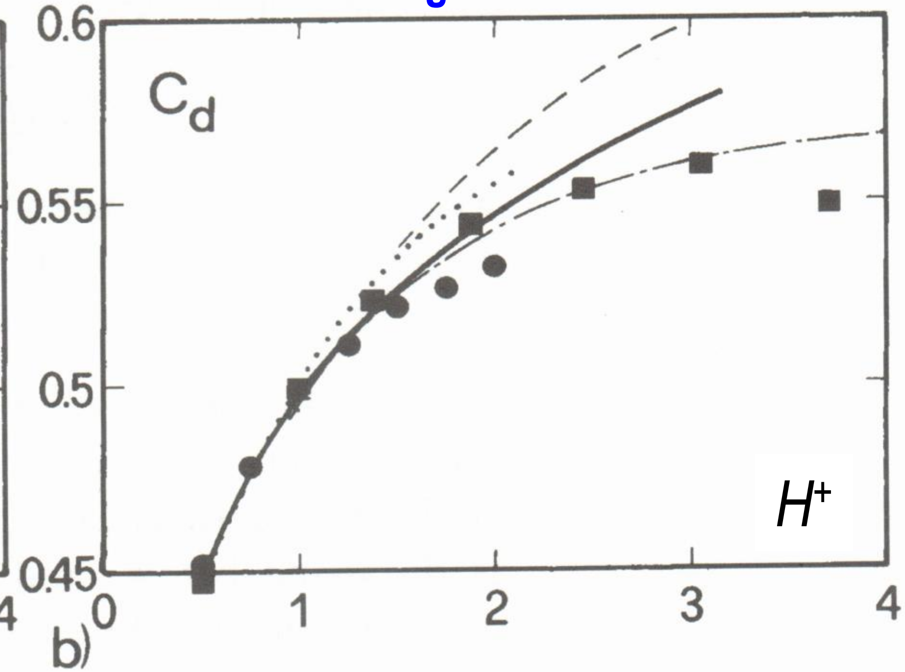
- The location of zero bottom pressure is also shown. The data vary with H^+ and chute angle α.

$$X_0 = x_0 / H_d = 0.9 \tan\alpha \, (H / H_d - 1)^{0.43}$$

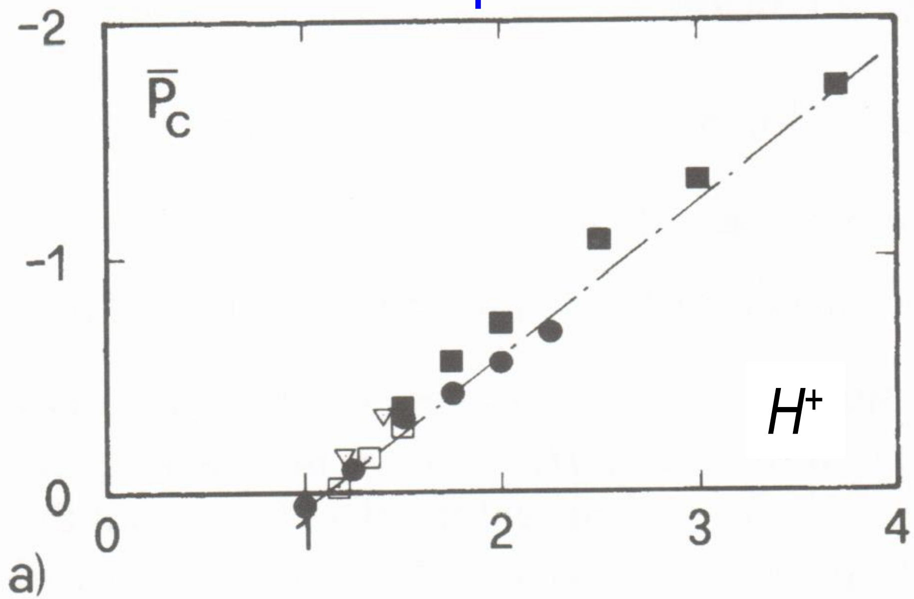
Minimum bottom pressure index



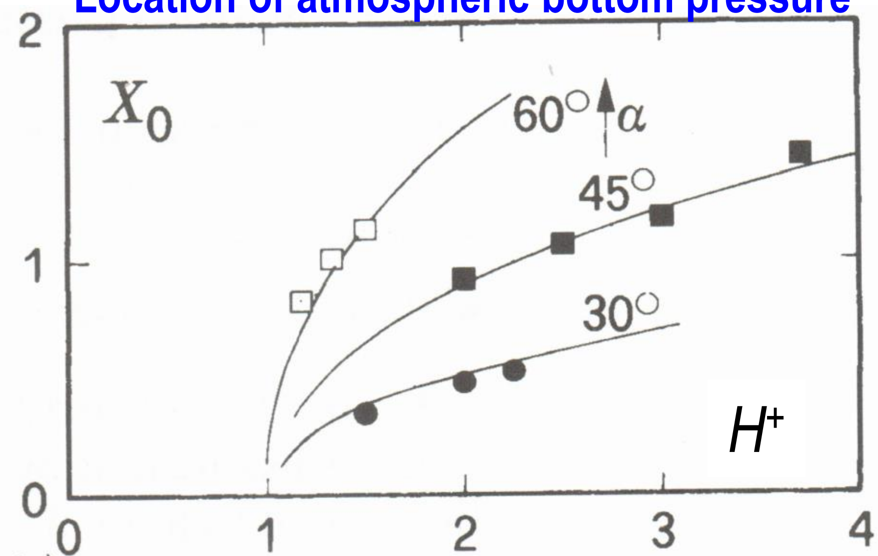
Discharge coefficient



Crest bottom pressure index



Location of atmospheric bottom pressure



Cavitation Design

Standard overflow with

$H^{+} < 1$ under designed

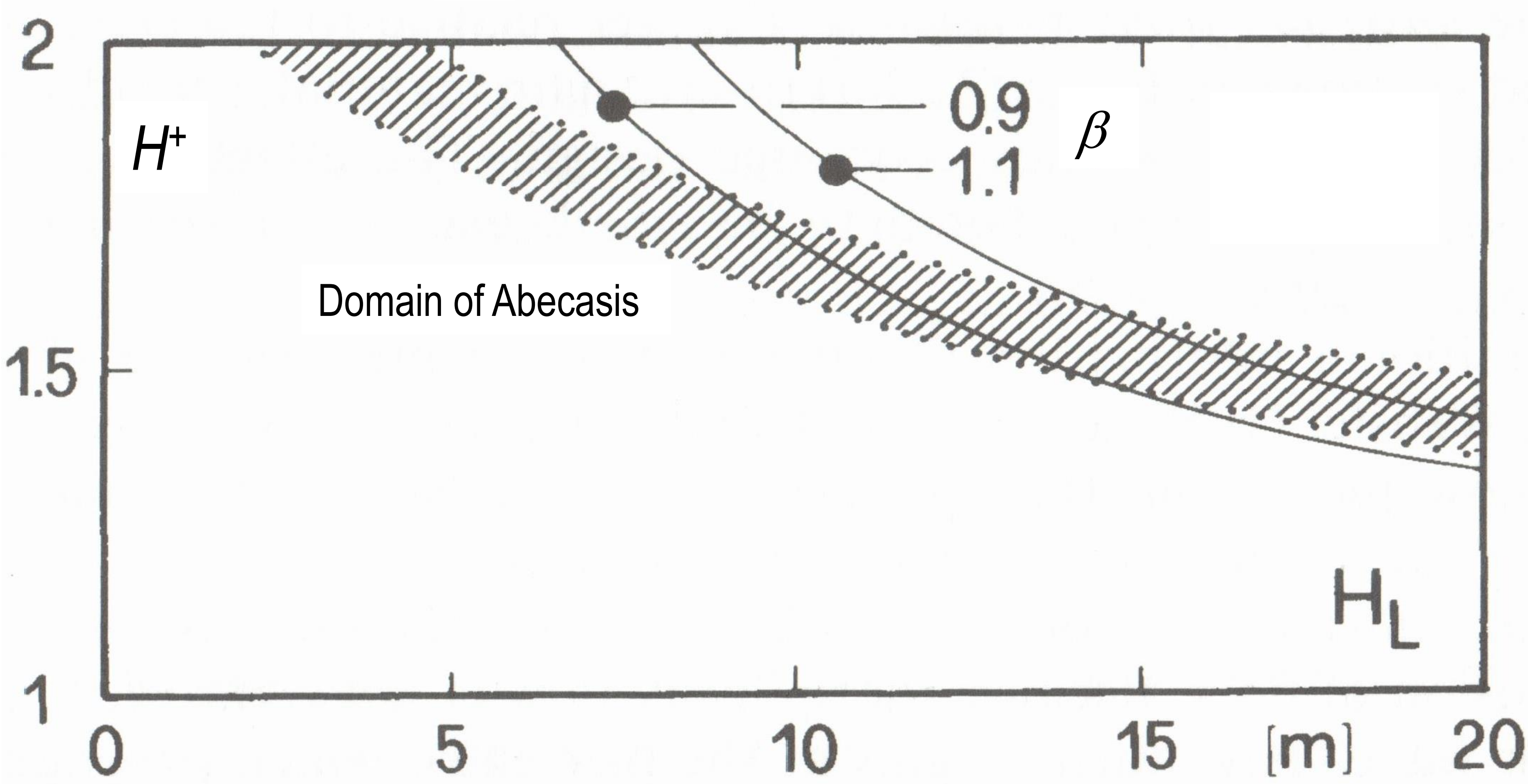
$H^{+} > 1$ over designed and thus sub atmospheric bottom pressures.

- Initially overdesign of dam overflows was associated with advantages in capacity.
- However the increase in discharge coefficients C_d for $H^{+} > 1$ is relatively small, but the decrease of minimum pressure, P_m, is significant.
- Overdesigning, thus adds to the cavitation potential.
- Generally, one assumes an incipient pressure head: $\frac{p_{vi}}{\gamma} = -7.6\,\mathrm{m}.$

- The limit head, H_L , for incipient cavitation to occur is

$$H_L = \left[\beta\left(1 - H^+\right)\right]^{-1} \left[p_{vi} / \gamma\right]$$

- The constant β was introduced to account for additional effects, such as the variability of p_{vi} with H^+



Overflow Gates

- The overflow structure has a hydraulic behavior that the discharge increases significantly with the head on the overflow crest.

- Overflow may be regulated to a desired or prescribed reservoir level using gates.

- The head on the turbines may be increased compared to ungated overflow.

- During the floods, if the reservoir is full, the gates are completely open to promote the overflow.

- The hydraulics of gates on overflow structures involves three major problems to be considered:
    - Discharge characteristics
    - Crest pressure distribution
    - Gate vibration

Currently most large dams are equipped with gates for a flexible operation.

The advantages of gates at overflow structure are:

- Variation of reservoir level,
- Flood control,
- Benefit from higher storage level.

The disadvantages are:

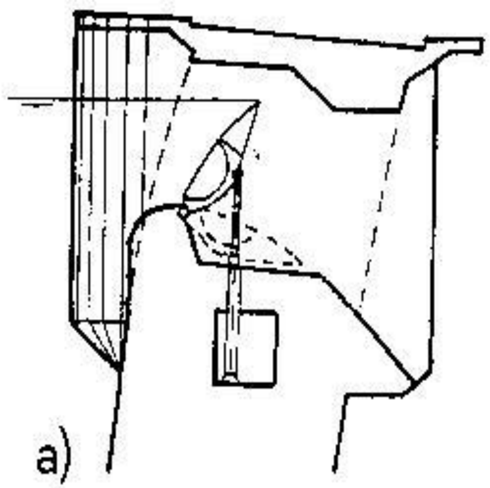
- Potential danger of malfunction,
- Additional cost, and maintenance.

Depending on the size of the dam and its location, one would prefer the gates for:

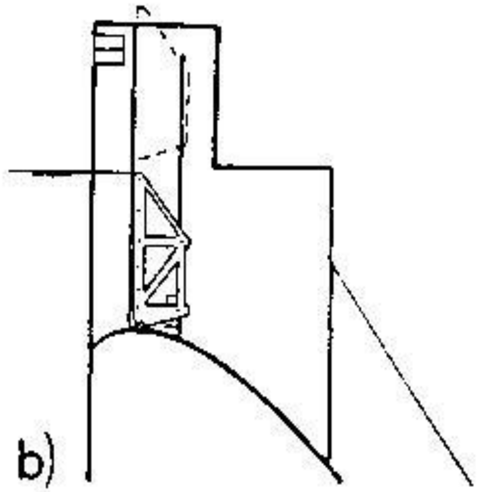
- Large dams,
- Large floods, and
- Easy access for gate operation.

Three types of gates are currently favored:

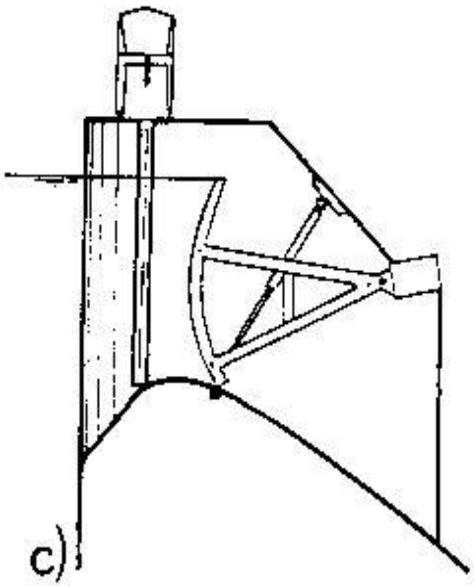
- Hinged flap gates,
- Vertical lift gates,
- Radial gates.



Flap Gate



Vertical Gate



Radial Gate

- The flaps are used for a small head of some meters, and may span over a considerable length.

- The vertical gate can be very high but requires substantial slots, a heavy lifting device, and unappealing superstructure.

- The radial gates are most frequently used for medium or large overflow structures because of

  - their simple construction,
  - the modest force required for operation and
  - absence of gate slots.

- They may be up to 20m X 20m, or also 12 m high and 40 m wide. The radial gate is limited by the strength of the trunnion bearings.

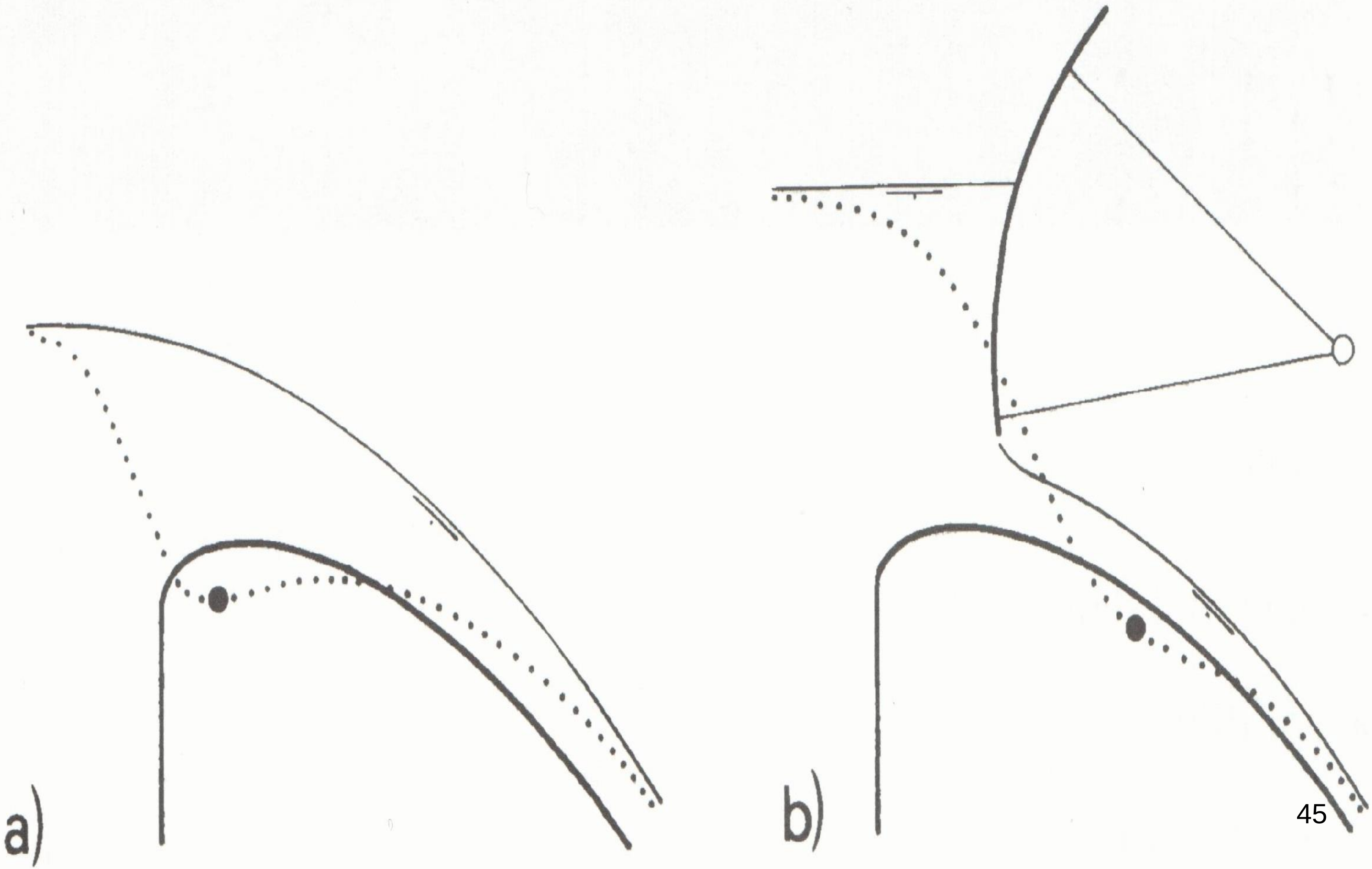
- For safety reasons, there should be a number of moderately sized gates rather than a few large gates.

- For the overflow design, it is customary to assume that the largest gate is out of operation.

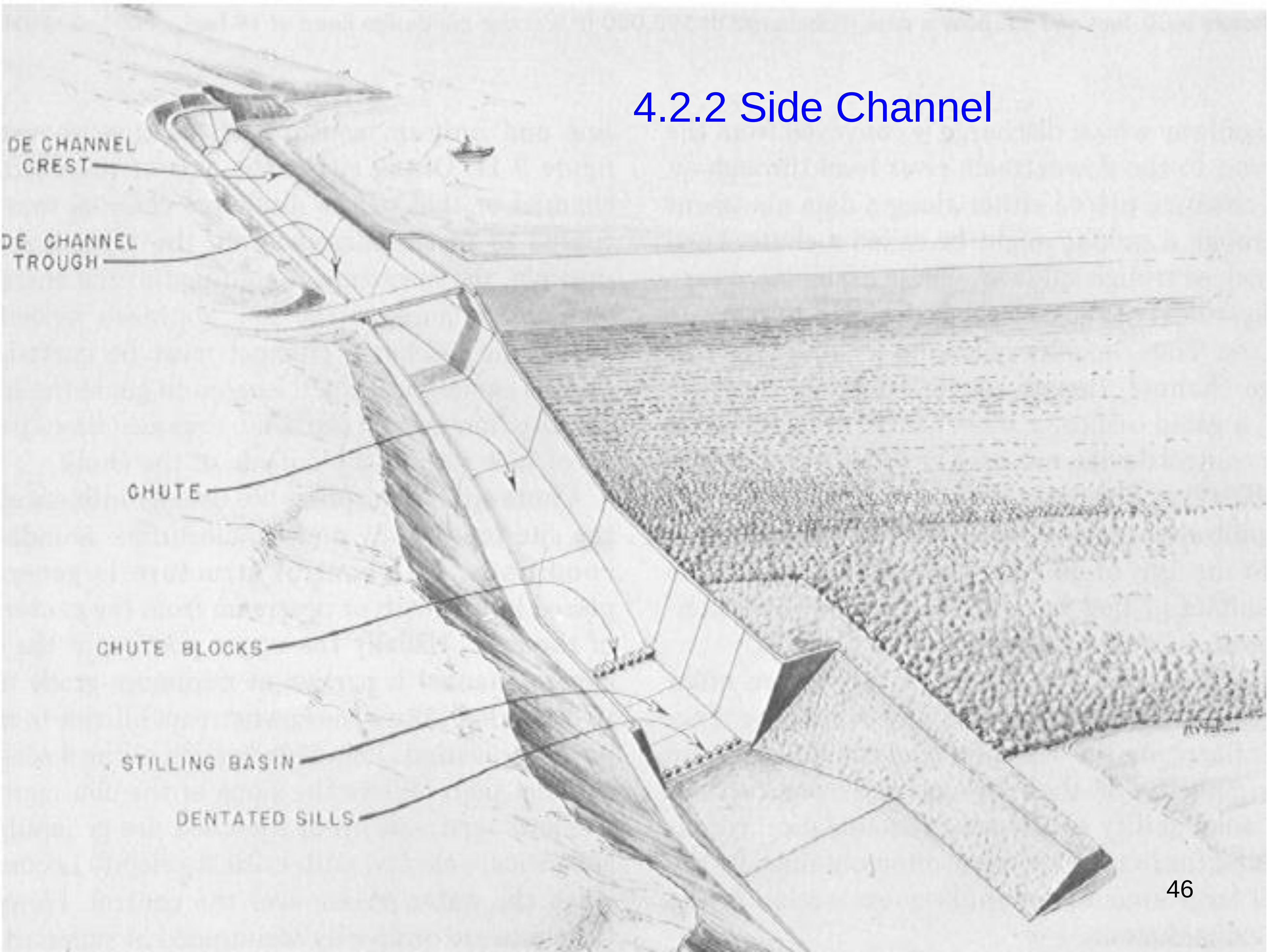
- The regulation is ensured by hoist or by hydraulic jacks driven by electric motors.

- Stand-by diesel-electric generators should be provided if power failures are likely.

Bottom pressure profile



4.2.2 Side Channel



- Side channels are often considered at sites where:

  - a narrow gorge does not allow sufficient width for the frontal overflow,
  - impact forces and scour are a problem in case of arch dams,
  - a dam spillway is not feasible, such as in the case of an earth dam,
  - when a different location at the dam site yields a simpler connection to the stilling basin.

- Side channels consist of a frontal type of overflow structure and a spillway with axis parallel to the overflow crest.

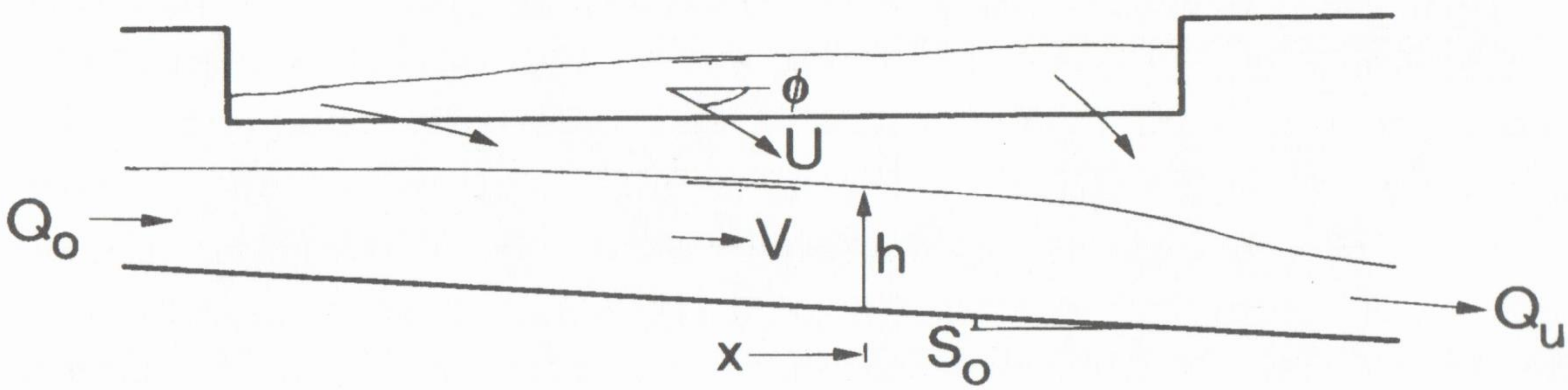
  - The specific discharge of overflow structure is normally limited to 10 $m^3/s/m$, but for lengths of over 100 m.
  - The overflow head is limited to say 3 m.
  - Not equipped with gates.

Hydraulic design

The 1D equation for the free surface profile can be derived from momentum considerations (Chow 1959)

$$\frac{\mathrm{d}h}{\mathrm{d}x} = \frac{S_o - S_f - \left[2 - \frac{U\cos\phi}{V}\right]\frac{Q(\mathrm{d}Q/\mathrm{d}x)}{gA^2} + \frac{Q^2(\partial A/\partial x)}{gA^3}}{1 - \mathbf{F}^2}$$

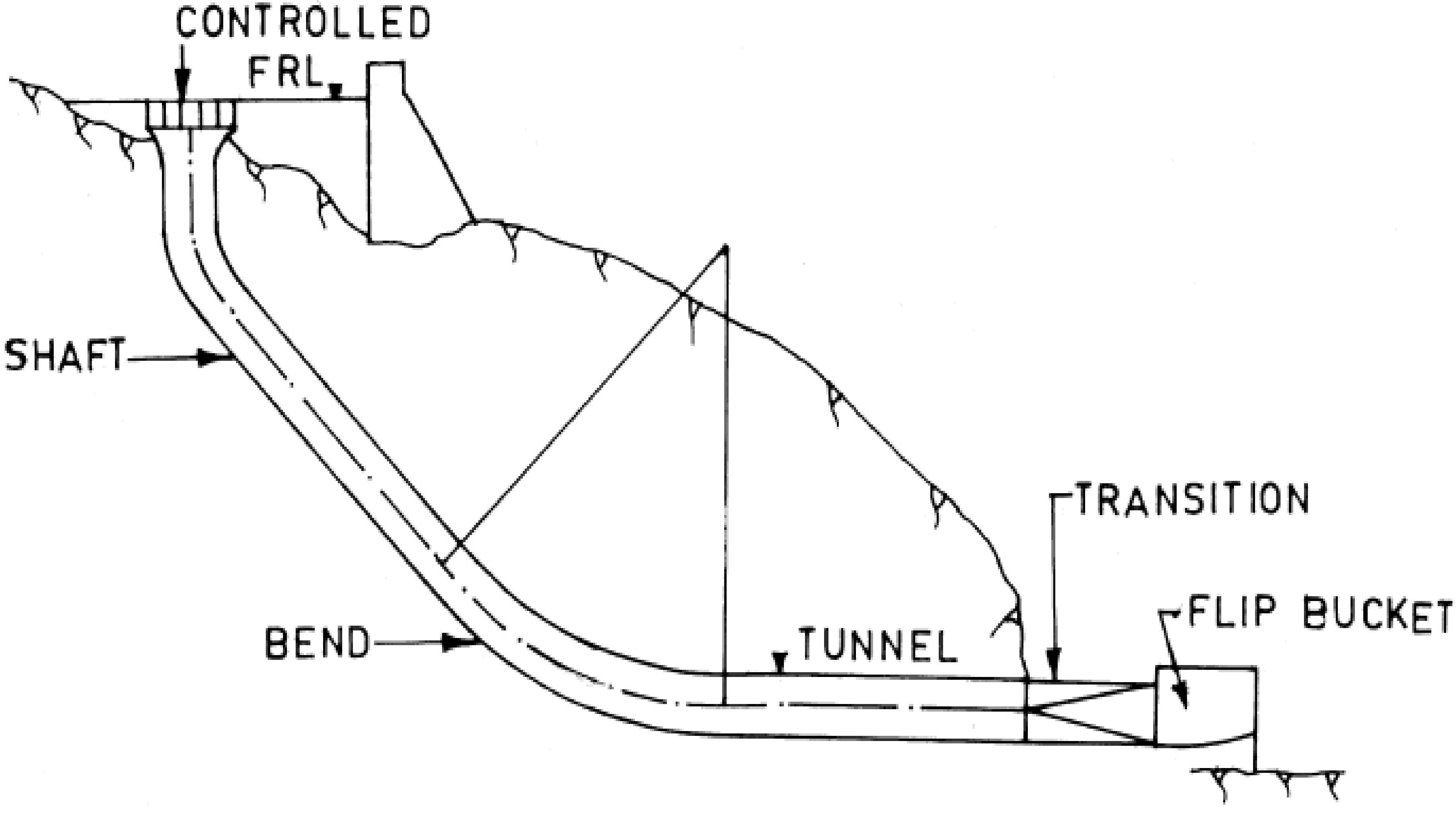
$$\mathbf{F}^2 = Q^2(\partial A/\partial h)/(gA^3)$$



4.2.3 Morning Glory Overfall

- The shaft type spillway has proved to be economical, provided the diversion tunnel can be used as a tailrace. The main elements are:
  - The intake,
  - The vertical shaft with a bend,
  - The almost horizontal spillway tunnel, and,
  - Energy dissipator.
- Air by aeration conduits is provided in order to prevent cavitation.
- Also, to account for flood safety, only non-submerged flow is allowed such that free surface flow occurs along the entire structure, from the intake to the dissipator.
- Used for dams with small to medium design discharges ($<1000\ m^3/s$).
- The structure has a circular standard-crested overfall

Elements of a shaft spillway




Morning Glory Overfall

- Morning Glory Overfall is advantageous when:

  - seismic action is small,
  - the horizontal spillway may be connected to the existing diversion channel,
  - floating debris is insignificant,
  - space for the overflow structure is limited,
  - geologic conditions are excellent against settlement, and

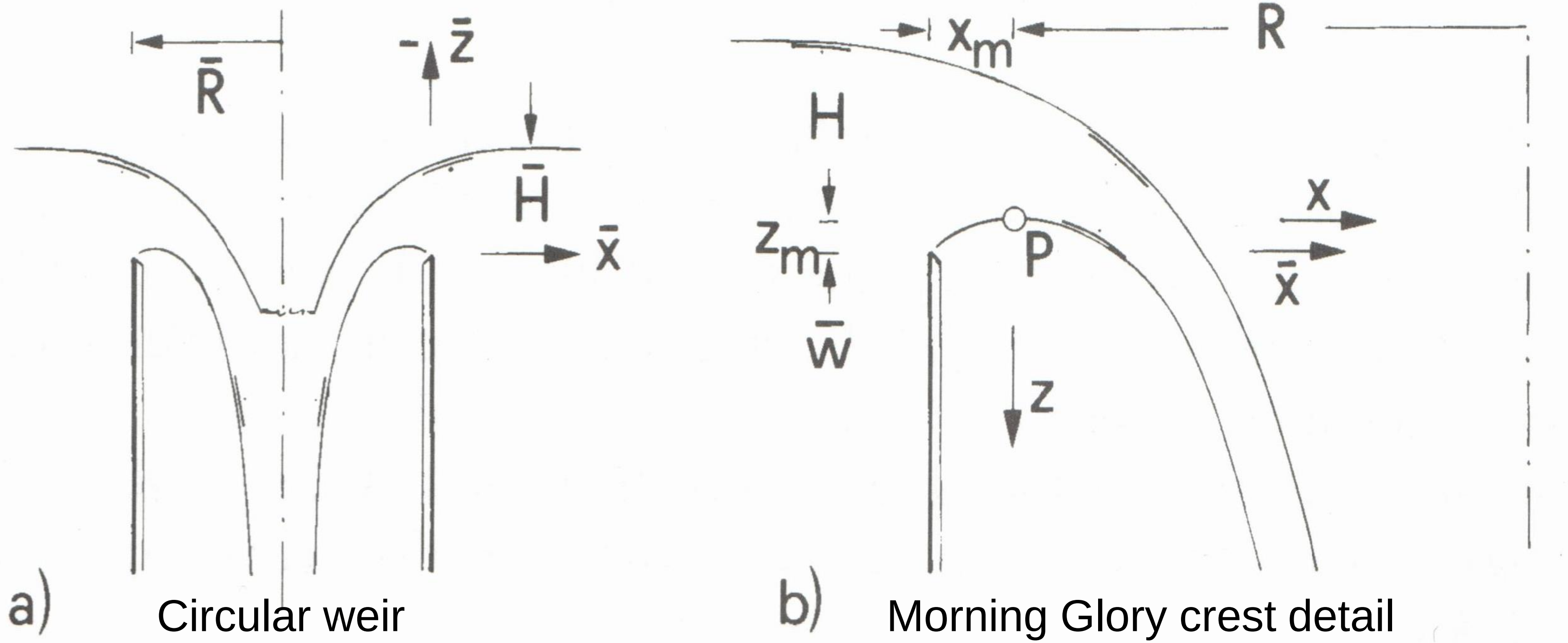
- Location of the Morning Glory

  - The intake is prone to rotational approach flow, which should be inhibited with a selected location of the shaft relative to the reservoir topography and the dam axis.

  - The radial flow may be improved with piers positioned on overfall crest.

Crest shape

- The shape of the Morning Glory overfall is a logical extension of the standard overfall crest. Experiments were performed on circular sharp crested weir.



Circular weir

Morning Glory crest detail

- All quantities referring to the weir are over barred.
- The overflow head relative to the sharp crest is $\overline{H}$ and the coordinate system $(\overline{x}, \overline{z})$ is located at the weir crest.

Discharge

- The discharge over a Morning Glory overfall structure is in analogy with the straight-crested overfall

$$Q = C_d\, 2\pi R\, (2gH^3)^{1/2}$$

$$C_d = 0.515\, [1 - 0.20(H/R)]$$

for the range of $\quad 0.2 \leq H/R \leq 0.5$

- An initial value of *H* or *R* may be assumed for a fixed *H/R* ratio to start the computations.

- Shaft radius R_s can be determined from

$$R_s = 1 + 0.1R \qquad \text{(in meters)}$$