

**WOLLO UNIVERSITY**  
**KOMBOLCHA INSTITUTE OF TECHNOLOGY**  
**Hydraulic and Water Resources Engineering Department**

*Course outline*

**Course Name: Dam Engineering II**

**Course Code: WRIE3153**

**Credits: 2**

**Program: B.Sc. in Water Resource and Irrigation Engineering**

**Module name: Hydraulic Structures & Hydropower Engineering**

**Prerequisites: Dam Engineering I**

**Instructor: Yassin Y.**

**Course Objectives:**

- *The objective of this course is to enable students to be familiar with the ancillary components of dams or appurtenant structures which are provided for dam safety and environmental considerations.*
- *To understand the need of energy dissipation and outlet works in dam design and construction; explain the functions of intakes, gates, dam instrumentation and surveillance after completing this course.*

**Course Description:**

- *This course is designed to aware students about the need of dam outlets, and factors to be considered in outlet works are also component parts of the course.*
- *As a safety measure of dam, spillways, types of spillways, hydraulic design, construction, aeration and cavitations of spillways, energy dissipaters, intakes structures ,gates and valves are also included.*

**Course Contents**

**1. Dam outlet works**

- 1.1. Introduction
- 1.2. The design flood
- 1.3. Sedimentation in reservoirs
- 1.4. Cavitations
- 1.5. Spillways
- 1.6. Bottom outlets

**2. Energy dissipation**

- 2.1. Energy dissipation on spillways
- 2.2. Stilling basins
- 2.3. Plunge pools
- 2.4. Energy dissipation at bottom outlets

### **3. Intake structures**

- 3.1.** Introduction
- 3.2.** Bottom intake
- 3.3.** Surface intake

### **4. Gates and valves**

- 4.1.** General
- 4.2.** Crest gates
- 4.3.** High-head gates and valves
- 4.4.** Tidal barrage and surge protection gates
- 4.5.** Hydrodynamic forces acting on gates

### **Assessment**

50% Assessment and 50% Final examination.

10% Assignment, 20% Test, 10% Project, 10% Term paper

**Attendance:** Students should attend at least 85% of the lecture.

### **References**

- Arora, K., (2002). Irrigation, Water power and Water Resources Engineering, 4th Edition, A.K. Jain, New Delhi.
- Novak, P., et al. (2007). Hydraulic structures, 4th Edition, Taylor and Francis, London.
- USBR, (1973). Design of Small Dams, 2nd Edition, US gov't Printing Office, Washington D.C.
- Creager, W.P., J.D., Justin, and J. Hinds, (1945). Engineering for Dams (VOL I,II and III).
- Bhart Singh, R.S. Varshney (1995). Engineering for embankment dams, A.A.Balkema publishers, USA.
- Vischer, D.L & W.H. Hager, (1997). Dam Hydraulics, John Wiley & Sons, New York.

## 1. Dam outlet works

### 1.1 Introduction

Outlet works are hydraulic structures used to convey water from a reservoir to a point downstream of a dam. It serves to regulate or release water impounded by a dam. It may release incoming flows at a retarded rate, as in the case of detention dam; divert incoming flows into canals or pipelines, as in the case of diversion dam; or release stored waters at such rates as may be dictated by downstream needs, evacuation considerations, or a combination of multi-purpose requirements.

Dam outlet works consist generally of spillways and bottom (high-head) outlets. Spillways are basically dam appurtenances ensuring a safe passage of floods from the reservoir into the downstream river reach.

Most of the water, which is stored in a reservoir for irrigation, water supply or power generation purposes, is stored below the spillway crest level. The spillway is provided at normal pool level, such that the floods are discharged safely above the spillway. But, in order to draw water from the reservoir as and when needed, for irrigation, water supply, power generation etc it is absolutely necessary that outlet works are provided either through the body of the dam or adjacent to it through some hillside at one end of the dam, this water may be discharged to the downstream channel below the dam or may be transported at distances where required (to some power house, etc) through pipes or canals. The opening a pipe or tunnel provided for this withdrawal of water is known as a dam out let.

In certain instances the out let works of a dam maybe used as a service spillway in conjunction with an auxiliary or secondary spillway. In this event the usual outlet works installation might be modified to include a bypass overflow, so that the structure can serve both as an outlet work and spillway.

An outlet works may also act as a flood control regulator, to release waters temporarily stored in flood control storage space or to evacuate storage in anticipation of flood inflows. Further, the outlets may serve to empty the reservoir to permit inspection, to make needed repairs, or to maintain the upstream face of the dam or other structures normally inundated.

#### Classification of Outlet Works

Outlet works can be classified according to their purpose, their physical and structural arrangement, or their hydraulic operation.

- Classification according to purpose
  - ✓ River outlets - empties directly into the river for river flow requirements

- ✓ Canal outlets - discharges into a canal
- ✓ Flood control - to release water temporarily stored in flood control storage space or to evacuate storage in anticipation of flood inflows
- ✓ Emergency drawdown - to empty the reservoir to permit inspection, to make needed repairs, or to maintain the upstream face of the dam or other structures normally inundated.
- ✓ Additional spillway capacity - the structure can serve as both an outlet works and a spillway.
- ✓ Diversion during construction
- ✓ Control of sedimentation of reservoirs - by draining out sediment-laden water.
- ✓ Power production, Irrigation, public water supply, etc
- According to the type of waterway
  - ✓ Open channel waterway
  - ✓ Closed conduit waterway, or
  - ✓ Closed waterway in a conduit or in tunnel
- According to its hydraulic operation
  - ✓ Gated or ungated,
  - ✓ Pressure flow (for a closed conduit) - for part or all of its length or
  - ✓ Free flow waterway (for closed conduit)

### Capacity of Outlets

Outlet work controls are designed to release water at specific rates, as dictated by downstream needs, flood control regulation, storage consideration, or legal requirements.

- Determination of the proper outlet capacity where flood control is a major function is dependent on the following factors
  - (i) Magnitude of design flood for the project
  - (ii) Reservoir storage capacity required for control purposes & spillway capacity
  - (iii) Downstream channel capacity.

Note: For efficient and adequate flood control, channel improvement should be made to increase the capacity.

- Irrigation outlet capacities are determined from reservoir operation studies and must be based on a consideration of a critical period of low runoff when reservoir storages are low and daily irrigation demands are at their peak. The most critical draft from the reservoir, considering such demands (commensurate with remaining reservoir storage) together with prior rights or other needed releases, generally determines the minimum irrigation outlet capacity.
- If an outlet is to serve as a service spillway, the required discharge for this purpose may fix the outlet capacity.

- For emptying the reservoir for inspection or repair, the volume of water to be evacuated and the allotted emptying period may be the determining conditions for establishing the minimum outlet capacity.
- Here again, the inflow into the reservoir during the emptying period must be considered. The capacity at low reservoir level should be at least equal to the average inflow expected during the maintenance or repair period. The size of an outlet conduit for a required discharge varies inversely with the available head for producing the discharge.

### **Positions, Alignment and Arrangement of Outlet Works**

In order to attain the required discharge capacity, the outlet must be placed sufficiently below minimum reservoir operating level to provide head for effecting outlet works flows. Outlet works for small detention dams are generally constructed near riverbed level since permanent storage space, except for silt retention, is ordinarily not provided. For dams impounding water for irrigation, domestic use, or other conservation purposes, the outlet works must be placed low enough to draw the reservoir down to the bottom of the allocated storage space; however, it might be placed at some level above the riverbed, depending on the elevation of the established minimum reservoir storage level.

### **Layout of Outlet Works**

The layout of a particular outlet works will be influenced by many conditions relating to the hydraulic requirements, to the site adaptability and the interrelation of the outlet works to the construction procedures, and to other appurtenance of the development. Thus, an outlet works leading to a high-level canal or into a closed pipeline might differ from one emptying into the river. In certain instances, the proximity of the spillway may permit combining some of the outlet works & spillway components into a single structure. As an example, the spillway and outlet works layout might be arranged so that discharge from both structures will empty into a common stilling basin.

The topography and geology of the site may have a great influence on the layout selection. Some sites may be suited only for a cut-and-cover conduit type of outlet works, while at other sites either a cut-and-cover conduit or a tunnel can be selected. Unfavorable foundation geology, such as deep overburdens or inferior foundation rock, will obviate the selection of a tunnel scheme. On the other hand, sites in narrow canyons with steep abutments may make a tunnel outlet the only choice.

### **Arrangement of Outlet Works**

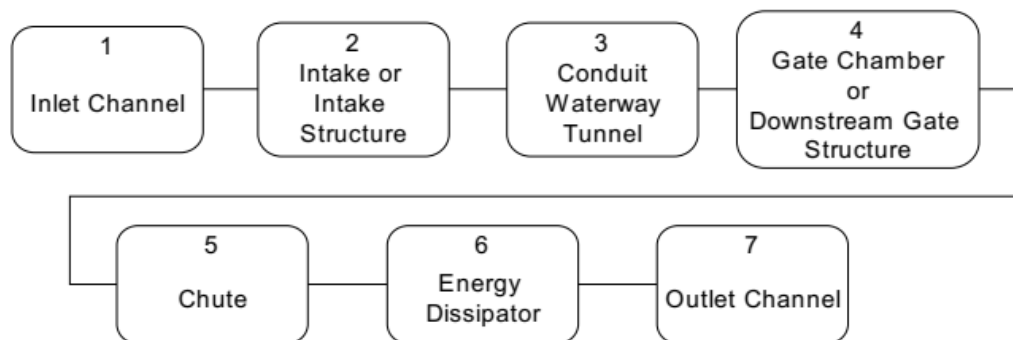
An outlet work for a low dam, whether it is to divert water into a canal or release it to the river, often may consist of an open channel or a cut-and-cover structure placed at the dam abutment. Where the outlet is to be placed through a low earth fill embankment, a closed-type structure

might be used which may consist of single or multiple units of buried pipe or box culverts placed through or under the embankment. Flow for such an installation could be controlled by gates placed at the inlet or at the intermediated point along the conduit, such as the crest of the embankment, where a shaft would be provided for gate operation.

For higher earth fill dams where an open channel outlet structure would not prove feasible, the outlet might be carried through, under, or around the dam as a cut-and-cover conduit or through the abutment as a tunnel. Depending on the position of the control device, the conduit or tunnel could be free flowing, flowing under pressure for a portion of its length, or flowing under pressure for its entire length. Intakes might be arranged to draw water from the bottom of the reservoir, or the inlet sills might be placed at some higher reservoir level. For a concrete dam the outlet works installation is usually carried through the dam as a formed conduit or a sluice, or as a pipe embedded in the concrete mass. Intakes and terminal devices can be attached to the upstream and downstream faces of the dam. Often the outlet is formed through the spillway overflow section, using a common stilling basin to dissipate both spillway and outlet flows.

#### Components of Outlet Works

An outlet work is consists of all or some of the following components.



- Conveyance - (1), (3), (5), (7)
- Control - (2), (4)
- Energy dissipation - (6)

#### a. The design flood

The required maximum flow rate which passes through the spillway, depends on the design flood, discharge capacity of the other outlet works and the existing storage capacity. The selection of the design flood (reservoir inflow) hydrograph is one of the most important tasks in dam design; it depends on the dam location and the type of dam and the procedure for its determination. The methods used for the calculation of floods could be developed from historical

records of maximum observed floods, empirical and regional formulae, flood envelope curves and flood frequency analysis to modern methods based on rainfall analysis and conversion to runoff.

The maximum out flow rate and the maximum rise in water surface over the spillway is determined by the step-by-step method of flood routing. Please review your Hydrology course for flood routing.

To carry out the flood routing the following items are required

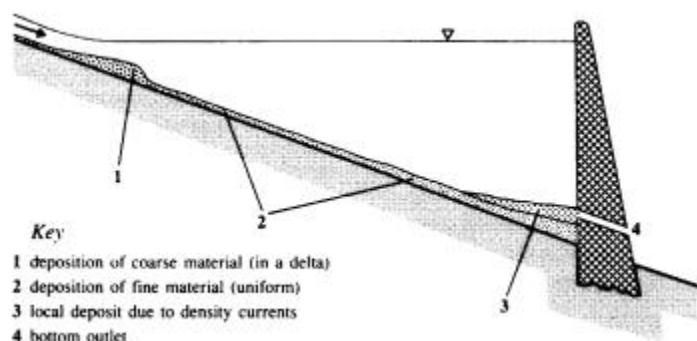
- i) the inflow hydrograph
- ii) the storage elevation curve for the reservoir
- iii) The outflow elevation curve.

b. Sedimentation in reservoirs

The assessment of economic viability, safety and cultural considerations as well as the environmental and social impact assessment should form an integral part of any large dam project; this assessment has many facets, one of the most important being the estimation of the sediment deposition in a reservoir and its 'life'. Sediment run-off in many rivers is continuously increasing – mainly as a result of human influence. Sediment concentration in rivers fluctuates greatly and is a function of sediment supply and discharge.

The loss of storage is only one deleterious effect of sedimentation in reservoirs; others are increased flood levels upstream of the reservoir, retrogression of the river bed and water levels downstream of the dam, the elimination of nutrients carried by the fine sediments, the effect of sedimentation on the reservoir water quality, etc. At present, many reservoirs have a life expectancy of only 100 years. A useful life of a reservoir less than, say, 200 years should certainly be a matter of concern, and one has to consider whether the drastic environmental effects are outweighed by the economic advantages during a relatively short effective life.

The detailed computation of the amount of sediment deposited in a reservoir requires not only knowledge of the quantity and composition of the incoming sediment but also of the reservoir operation and cross-sections along the reservoir.



## Deposition stages in a reservoir

Reservoir capacity can be preserved by

- (a) Minimizing the sediment input into the reservoir,
- (b) Maximizing the sediment throughflow, or
- (c) The recovery of storage

*Minimizing sediment input* is by far the most effective measure, and can be achieved by optimal choice of the location of the reservoir, the prevention of erosion in the catchment by soil conservation methods (afforestation, terracing, vegetation cover, etc.), the trapping of sediment in traps or by vegetative screens on the tributaries upstream of the reservoir, or by bypassing heavily sediment-laden flows during floods from an upstream diversion structure to downstream of the dam; (in a diversion tunnel a compromise has to be achieved between sedimentation and abrasion; a velocity of about 10m/s is usually acceptable).

*Maximizing sediment throughflow* requires flow regulation during floods and/or flushing during a reservoir drawdown. Under certain conditions the sediment-laden inflow does not mix with the water in the reservoir but moves along the old river bed as a density current towards the dam, where it can be drawn off by suitably located and operated outlets. In principle, the development of density currents requires a significant difference between the density of the incoming flow and the water in the reservoir, a large reservoir depth, and favourable morphological conditions (steep, straight old river bed).

*The recovery of storage* can be achieved by flushing deposited sediment, a technique which is effective only when combined with a substantial reservoir drawdown, by siphoning or dredging; in the latter case either conventional methods, particularly a suction dredger with a bucket wheel, or special techniques (e.g. pneumatic or jet pumps) may be used.

c. Cavitations

Cavitations occur whenever the pressure in the flow of water drop to the value of the pressure of the saturated water vapor  $p_v$  (at the prevailing temperature); cavities filled by vapor, and partly by gases excluded from the water as a result of the low pressure are formed. When these "bubbles" are carried by the flow in to region of higher pressure, the vapor quickly condenses and implodes and the cavities suddenly filled by the surrounding water. This process is noisy with the disruption of the flow pattern. Importantly, if cavity implodes against a surface, the violet impact of the water particles acting in a quick succession at a very high pressure (1000 atmosphere) sustained over a period of time would cause a substantial damage to concrete or steel surface. This would ultimately lead in to the complete failure of the structure. Thus

cavitations corrosion (pitting) and the accompanying vibration should be taken in to account in the design of hydraulic structures, and prevented whenever possible.

Low pressures (below atmospheric pressure) will occur at points of separation of water flowing alongside fixed boundaries, particularly at high velocity. Thus, there two factors that influence the onset of cavitation.

- i) Pressure P
- ii) Velocity V

As the bubbles (cavities) move into a higher pressure zone, they collapse. The sudden collapse of the cavities generates intense pressure shock waves, which are responsible for the noise and eventual damage associated with psi cavitation. Although the pressure bursts may reach a million psi (thousands of Mpa), little damage can be expected from the collapse of individual cavities occurring a few millimeters away from the boundary.

For evaluation of the risk of cavitation, a cavitations number for the flow  $\sigma$  may be computed from the formula:

$$\sigma = \frac{\frac{p_a}{\gamma} - \frac{p_r}{\gamma} + h \cos \alpha + \frac{h}{g} \frac{V^2}{r}}{\frac{V^2}{2g}}$$

where:-  $p_a$ = atmospheric pressure

$\gamma$ =Specific weight of water

$p_r$ = vapor pressure

$\alpha$ = angle of the chute bottom to the horizontal

$g$ = acceleration of gravity

$V$ = water velocity

$r$ = radius of the vertical bend (positive if concave)

$h$ = water depth normal to the flow

This value of  $\sigma$  may be compared with the critical cavitations index  $\sigma_i$ . either the average velocity of the flow or the velocity at the height of the surface irregularity may be taken as reference. And due consideration should be given to that difference.

If the cavitations number of the flow falls below that limit aeration turns out to be the best alternative to prevent cavitations. A threshold velocity around 100 ft/sec (30.5 meters/sec) or cavitations number of about 0.25 gives the order of magnitude of the limit beyond which aeration of the flow is recommended. It is generally believed that the onset of cavitation occurs when  $P=P_v(\approx 0$  for normal water temperature, i.e., 10m below  $P_o/\rho g$ ).

When cavitation problems are eminent, then the design or the mode of operation of particular structures has to be changed or other safe guard has to be applied. The most frequent of these is the introduction of air at the endangered parts, i.e. the artificial aeration, preventing the occurrence of extremely low pressures. The use of epoxy mortars can also substantially postpone the onset of cavitation damage on concrete surfaces and can be adopted where the cavitation is not frequent or prolonged.

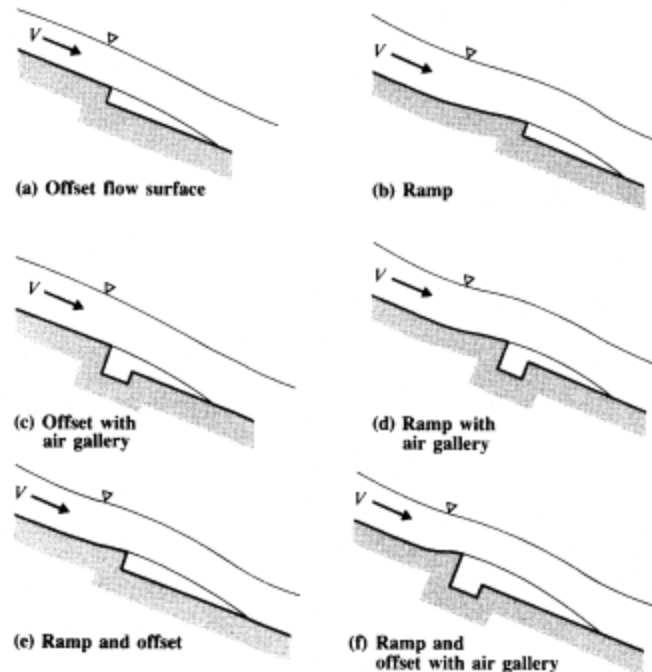
#### Basic Aerator Types

Ramps or steps inserted on the chute surface are the simplest and most practical devices used to cause natural aeration or water flow near a solid boundary. The sudden discontinuity in the bottom alignment creates an air-water interface along which the high-velocity water drags air in an intense mixing process.

Prototype tests of a few spillways have suggested that entrained air discharge could be grossly described by a simple formula

$$q_a = kVL$$

Coefficient k being a constant typical of the aerator. V the average velocity of flow, and L the length of the void under the water jet downstream from aerator.



Types of aeration facilities (after Cassidy and Elder, 1984)

#### d. Spillways

##### General

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 bypass flows exceeding those which are turned into the diversion system.

There are several spillway designs. The choice of design is a function of the nature of the site, the type of dam and the overall economics of the scheme. The importance of a safe spillway cannot be overemphasized; many failures of dams have been caused by improperly designed spillways or by spillways of insufficient capacity. Ample capacity is of paramount importance for earth fill and rock fill dams, which are likely to be destroyed if overtopped, whereas concrete dams may be able to withstand moderate overtopping. Usually, increase in cost is not directly proportional to increase in capacity very often the cost of a spillway of ample capacity will be only moderately higher than that of one which is obviously too small.

A spillway may be located either within the body of the dam or at one end of the dam or entirely away from the dam as an independent structure.

Spillway is the most important component of the dam which serves to release excess flood from a reservoir efficiently and safely. It is the most expensive of all the appurtenances structure. Its capacity is determined from the hydrological studies over the drainage area.

Spillway components include;

- a. Entrance channel: to minimize head loss and to obtain uniform distribution of flow over the spillway crest

- b. Control structure: to regulate and control the outflow. It may consist of a sill, weir, orifice, tube, or pipe.
- c. Discharge channel: to convey the discharge from the control structure to the terminal structure/stream bed. The conveyance structure may be the downstream face of a concrete dam, an open channel excavated along the ground surface, a closed cut-and-cover conduit placed through or under a dam, or a tunnel excavated through an abutment.
- d. Terminal structure: to dissipate excess energy of the flow in order to avoid scouring of the stream bed
- e. Outlet channel: to safely convey the flow from the terminal structure to the river channel.

Spillways may be classified:

- (i) According to their function (or based on the time when the spillway comes into operation) as
  - a. Service (or main) spillways: Designed for frequent use in conveying flood releases from the reservoir to a watercourse downstream from a dam. It is designed to pass the entire design flood.
  - b. Auxiliary Spillways: Designed for infrequent use and may sustain limited damages when used. Some damages of the structure from passage of infrequent flood is permissible. It is provided as a supplement to the main spillway and its crest is so located that it comes into operation only after the floods for which the main spillway is designed are exceeded. It is provided in conjunction with the main spillway. The total capacity of the spillway is then equal to the sum of the capacities of the main and auxiliary spillways.
  - c. Emergency spillways: Designed to provide a reserve protection against overtopping of a dam and are intended for use under extreme conditions, such as misoperation or malfunction of a service spillway or other emergency conditions. Under normal reservoir operation, emergency spillways are never required to function. The control crest is, therefore, placed at or above the designed maximum reservoir water surface.
  - ❖ Some of the situations, which may lead to emergency, are:
    - an enforced shut down of outlet works,
    - a malfunctioning of spillway gates,
    - The necessity for bypassing the regular spillway because of damage or failure of some part of that structure.
- (ii) Types of spillway taking the hydraulic as criteria are broadly (mode of control)
  - a. Controlled (Gated) spillway: a spillway having a certain means to control the outflow from the reservoir.
  - b. Uncontrolled (Ungated) spillway: is a spillway, the crest of which permits water to escape automatically, as the water level in the reservoir rises above the crest.
- (iii) Taking the most prominent feature as criteria, spillway types are
  - a. Free overfall (straight drop) spillway
  - b. Ogee (overflow) spillway

- c. Side channel spillway
- d. Siphon spillway
- e. Chute (open channel or trough) spillway
- f. Drop inlet (shaft or morning glory) spillway

### 1.1.1 Types of Spillway

#### A. Free overfall (straight drop) spillway

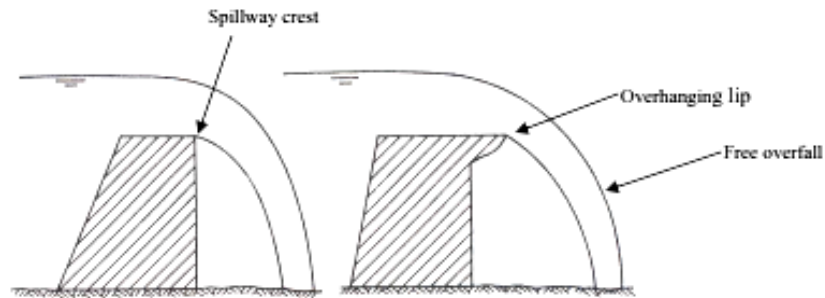
A free overfall spillway has a low height narrow crested weir as control structure and a vertical or nearly vertical downstream face. The overflowing water may be discharged as in the case of a sharp crested weir or it may be supported along the narrow section of the crest. However, in either case the water flowing over the crest of this spillway drops as a free jet clearly away from the downstream face of the spillway. Occasionally the crest of free overfall spillway is extended in the form of an overhanging lip to direct small discharges away from the downstream face of the overfall section. The underside of the nappe is ventilated sufficiently to prevent pulsating fluctuating jet.

If no artificial protection is provided on the downstream side of the overflow section, the falling jet usually cause the scouring of the streambed and will form a deep plunge pool. To protect the stream bed from scouring, an artificial pond may be created by constructing a low auxiliary dam downstream of the main structure or by excavating a basin which is then provided with a concrete apron. However, if tailwater depths are sufficient, a hydraulic jump will form when the jet falls freely from the crest, in which case a sufficiently long flat apron may be provided. In addition, floor blocks and an end sill may be provided in this case to help in the establishment of the jump and thus reduce the downstream scour.

The free overfall spillway is used:

- i. most commonly for low earth dams (or earthen bunds),
- ii. for thin arch dams,
- iii. or other dams having nearly vertical downstream face and would permit free fall of water, and
- iv. Where, in general, the hydraulic drops from head pool to tailwater are not in excess of about 6m.

However, free overfall spillways are not suitable for high drops on yielding foundations, because the apron will be subjected to large impact forces at the point of impingement. The impact force causes vibrations which may crack or displace the apron and may result in failure by piping or undermining.



### B. Ogee (overflow) spillway

The ogee spillway has a control weir which is ogee or S-shaped in profile. The profile is derived from the lower envelop of the overall nappe flowing over a high vertical rectangular notch with an approach velocity  $V_o \approx 0$  and a fully aerated space beneath the nappe ( $P = P_o$ ).

The following crest profile has been found to give good agreement with the prototype measurement by U.S. Waterways Experimental Station (WES). Such shapes are known as WES Standard Spillway shapes as shown in figure below.

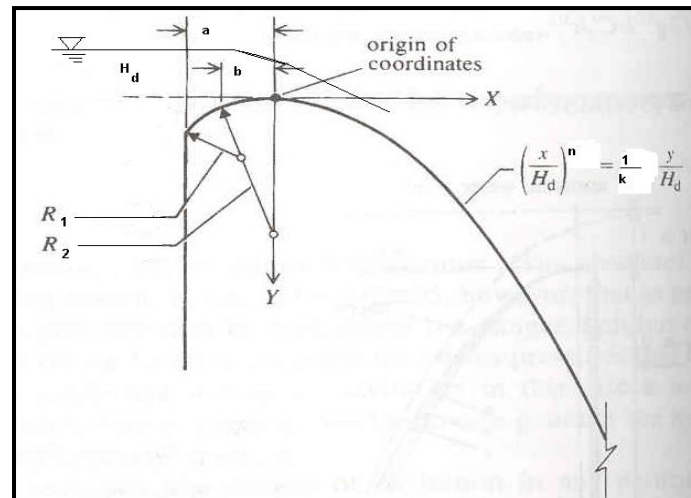


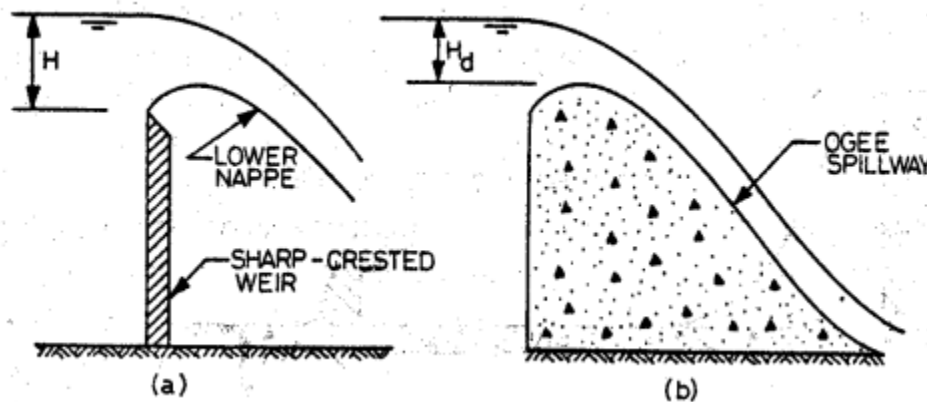
Figure: WES Standard Spillway Shapes

Table: Values of a, b, R<sub>1</sub>, R<sub>2</sub>, K and n for different U/S slope

| U/S slope | $a/H_d$ | $b/H_d$ | $R_1/H_d$ | $R_2/H_d$ | K     | n     | Crest equation                  |
|-----------|---------|---------|-----------|-----------|-------|-------|---------------------------------|
| 0H:3V     | 0.175   | 0.282   | 0.50      | 0.20      | 2     | 1.85  | $x^{1.85} = 2H_d^{0.85}y$       |
| 1H:3V     | 0.139   | 0.237   | 0.68      | 0.21      | 1.936 | 1.836 | $x^{1.836} = 1.936H_d^{0.836}y$ |
| 2H:3V     | 0.115   | 0.214   | 0.48      | 0.27      | 1.939 | 1.81  | $x^{1.810} = 1.939H_d^{0.810}y$ |

|         |   |       |      |          |       |       |                                 |
|---------|---|-------|------|----------|-------|-------|---------------------------------|
| $3H:3V$ | 0 | 0.199 | 0.45 | $\infty$ | 1.873 | 1.776 | $x^{1.776} = 1.873H_d^{0.776}y$ |
|---------|---|-------|------|----------|-------|-------|---------------------------------|

An ogee-shaped spillway is an improvement up on the free over flow spillways. The major difference between the free overflow spillway and the ogee-shaped spillway is that in the case of a free over fall spillway, the water flowing over the crest of the spillway drops vertically as a jet clear from the downstream side or face; where as in the case of an ogee-shaped spillway, the water flowing over the crest is guided smoothly over the crest and is made to glide over the downstream face 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. In this case no pressure is exerted on the spillway by the flowing water, as there will be atmospheric pressure along the contact surface between the flowing water and the spillway. At head less than the design head ( $H < H_d$ ) the overflowing water will remain in contact with the surface. The natural trajectory of the nappe falls below the profile of the spillway crest, then there will therefore be positive gage pressures over the crest, as the nappe tends to be depressed. In this case, as the spillway is supporting a sheet of flowing water backwater effect will be created and the discharge will be reduced.

At a head greater than the design head ( $H > H_d$ ), the nappe 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. The effect of negative pressure will be to increase the effective head and thereby increase the discharge. This may result in cavitation. However, in practice, this pressure reduction is not normally a serious problem unless  $H > 1.5 H_d$ . Indeed recent work suggests that separation will not occur until  $H$  approaches  $3H_d$ .

The ideal condition for an ogee spillway is when the head is equal to the design head ( $H_d$ ) for which the spillway has been shaped. At the design head, it attains nearly the maximum efficiency without any detrimental effect.

## ❖ Shape of the crest of the ogee or overflow spillway (outline)

The shaped of the ogee spillway depends on a number of factors among which includes:

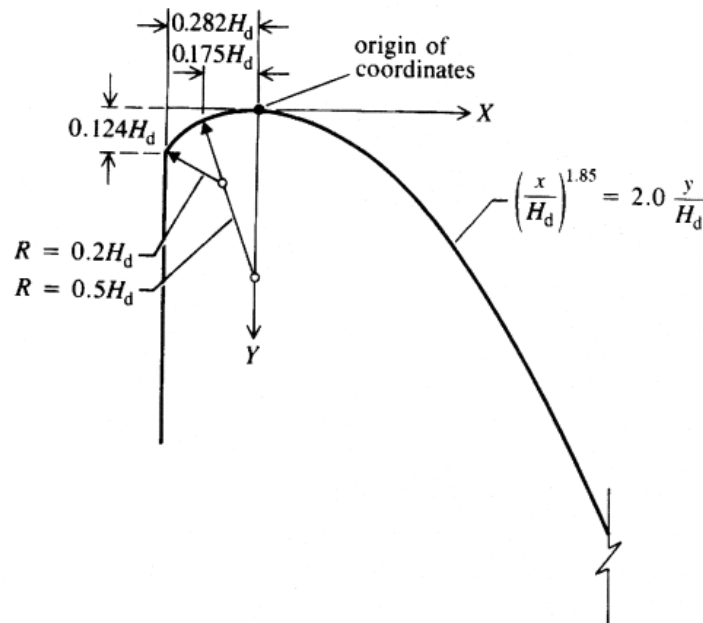
1. Head over the crest
2. Height of the spillway above the streambed or the bed of the entrance channel.
3. The inclination of the upstream face of the spillway

Thus, from research and experimental results, the following were arrived at

📌 Downstream profile: The downstream profile of the spillway can be represented by the following general equation.

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

Where x and y are coordinates of the point on the spillway surface, with the origin at the highest point 0 of the crest.  $H_d$  is the design head, excluding approaching velocity head. K and n are constants which depend upon the inclination of the upstream face of the spillway (see the figure below).



Standard spillway crest (after US Army Waterways Experimental Station, 1959)

The value of n and k for the vertical upstream face and three different inclinations are given in table of values (see table below).

Table: Values of k and n for vertical upstream face

| <i>U/S slope</i> | $b/H_d$ | $R_1/H_d$ | $R_2/H_d$ | $K$   | $n$   | <i>Crest equation</i>            |
|------------------|---------|-----------|-----------|-------|-------|----------------------------------|
| 0H:3V            | 0.282   | 0.50      | 0.20      | 2.00  | 1.85  | $x^{1.85} = 2H_d^{0.85} y$       |
| 1H:3V            | 0.237   | 0.68      | 0.21      | 1.936 | 1.836 | $x^{1.836} = 1.936H_d^{0.836} y$ |
| 2H:3V            | 0.214   | 0.48      | 0.27      | 1.939 | 1.81  | $x^{1.810} = 1.939H_d^{0.810} y$ |

Upstream profile of the crest

The profile of the upstream is either of vertical or sloping profile.

**(a)** Vertical upstream profile:

The upstream profile of the crest should be tangential to the vertical face and should have zero slope at the crest axis to ensure there is no discontinuity along the surface of flow. The upstream profile should conform to the following equation.

$$y = \frac{0.724(x + 0.270H_d)^{1.85}}{(H_d)^{0.85}} + 0.126H_d - 0.4315(H_d)^{0.375}(x + 0.270H_d)^{0.625}$$

**(b)** The sloping upstream face

Here the maximum permitted projection from the crest line is  $0.315H_d$  and the vertical depth of the maximum bulging is  $0.25H_d$ .

According to the latest analytical studies of the U.S Army, the upstream curve of the ogee shape has the following equation.

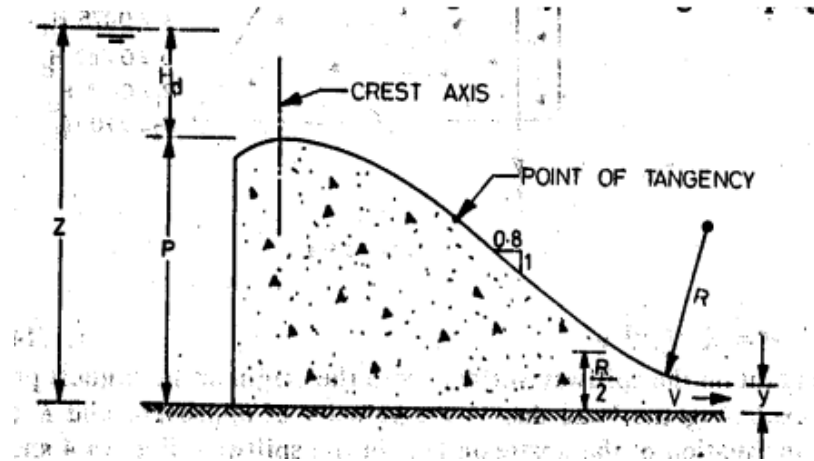
$$Y = \frac{0.724(x + 0.270H_d)^{1.85}}{H_d^{0.85}} + 0.126H_d - 0.4315H_d^{0.375}(x + 0.27H_d)^{0.625}$$

Where x and y are coordinates,  $H_d$ = design head.

- The radius of the bucket could be approximated by

$$R = \frac{P}{4}$$

Where P is the height of spillway crest above the bed.



The bucket radius could also be estimated from the following equation.

$$R = 10^{(V+6.4 H_d+4.88)/(3.6 H_d+19.52)}$$

Where  $V$  = velocity of flow at the toe of spillway m/s.

$H_d$  = the design head (m)

The relation may approximate the velocity of flow  $V$

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

Where  $Z$  = Total fall from the upstream water level to the flow level at the downstream toe.

$H_a$  = head due to velocity of approach.

$y$  = depth of flow at toe

$g$  = acceleration due to gravity

Discharge Equation

The discharge over an ogee shaped spillway is given by

$$Q = C_d L_e H_e^{3/2}$$

Where,  $C_d$  = Variable coefficient of discharge that depends on factors like depth of approach, relation of actual crest shape to the ideal nappe shape, upstream face slope, upstream apron interference and downstream interference. Value ranges from 2.1 to 2.5.

$L_e$  = Effective length of crest

$H_e$  = total head on crest, including velocity approach head.

*Coefficient of discharge,  $C_d$ , of Overflow spillway*

An overflow spillway has a relatively high coefficient of discharge 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:

- a) Depth of approach,  $p$
- b) Heads differing from design head
- c) Upstream face slope
- d) Downstream apron interference and downstream submergence

(a) Effect of Depth of Approach: With increase in the height of spillway the velocity of approach decreases and the coefficient of discharge increases. 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  $1.33 H_d$  ( $P \geq 1.33 H_d$ ).

(b) Effect of heads differing from the design head: Since for heads of flow higher than the design head higher will be the coefficient of discharge, if the spillway crest is designed by assuming a lower design head, for most of the range of heads of flow higher coefficient of discharge will be obtained.

However, the design head should not be less than about 80% of the maximum head in order to avoid the possibility of cavitation. Model tests have shown that for  $P > 1.33 H_d$  the head due to velocity of approach is negligible and when the total head of flow is equal to the design head, i.e.  $H = H_e$ , the coefficient of discharge is equal to 2.2.

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

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

Where,  $C'$  is 2.2, and  $H_e$  is the design head including velocity head.

(c) Effect of upstream face slope: For small values of the ratio ( $P/H_e$ ) a spillway with sloping upstream face has a higher coefficient of discharge than a spillway with vertical upstream face. However, for large values of the ratio ( $P/H_e$ ) the coefficient of discharge for spillways with sloping upstream face tends to decrease.

(d) Downstream apron interface and submergence effects: The coefficient of discharge is reduced due to submergence. When the tailwater level is such that the top of the weir is covered by it, such that the weir cannot discharge freely; the weir is then said to be submerged weir. 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  $\frac{h_d+d}{H_e}$  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 tailwater submergence.

The crest piers and abutments are shaped to cause side contractions of the overflow, the effective length,  $L_e$ , will be less than the net length of the crest. The effect of the end contraction may be taken into account by reducing the crest length as follows:

$$L_e = L' - 2(NK_p + K_a)(H + H_v)$$

Where:  $L'$  - net length of the crest

$N$ - Number of piers

$K_p$ - piers contraction coefficient

$K_a$ - abutment contraction coefficient

The pier contraction coefficient,  $K_p$ , is affected by the shape and location of the pier nose, the thickness of the pier, the head in relation to the design head, and the approach velocity. The average pier contraction coefficient may be assumed as follows:

| Pier condition                                                                              | $K_p$ |
|---------------------------------------------------------------------------------------------|-------|
| Square nosed pier with corners rounded on a radius equal to about 0.1 of the pier thickness | 0.02  |
| Rounded nosed piers                                                                         | 0.01  |
| Pointed nose piers                                                                          | 0     |

The abutment contraction coefficient is affected by the shape of the abutment, the angle between the upstream approach wall and the axis of flow, and the head in relation to the design head, and the approach velocity. The average abutment contraction coefficient may be assumed as follows:

| Abutment condition                                                                    | $K_a$ |
|---------------------------------------------------------------------------------------|-------|
| Square abutments with head wall at 90° to direction of flow                           | 0.20  |
| Rounded abutments with head wall at 90° to the direction flow                         | 0.10  |
| Rounded abutments with head wall placed at not more than 45° to the direction of flow | 0     |

- For the formula given below, there are three possibilities for the choice of the relationship between the design head  $H_d$  used for the derivation of the spillway shape and the maximum actual head  $H_{max}$ :

$$Q = \frac{2}{3} \sqrt{2g}^{1/2} b C_d H^{3/2}$$

For  $H_d = H_{max}$  the pressure is atmospheric and  $C_d = 0.745$ . For  $H_d > H_{max}$  the pressure on the spillway is greater than atmospheric and the coefficient of discharge will be  $0.578 < C_d < 0.745$ . The lower limit applies for broad crested weirs with  $C_d = 0.578$ , and is attained at very small values of  $H_{max}/H_d$  (say, 0.05). For  $H_d < H_{max}$  negative pressures result, reaching

cavitation level for  $H=2H_d$  with  $C_d=0.825$ . For safety it is recommended not to exceed the value  $H_{\max}=1.65H_d$  with  $C_d=0.81$ , in which case the intrusion of air on the spillway surface must be avoided, as otherwise the overfall jet may start to vibrate.

### C. Side channel spillway

Side channel spillways are mainly used when it is not possible or advisable to use a direct overfall spillway as, e.g., at earth and rock fill dams.

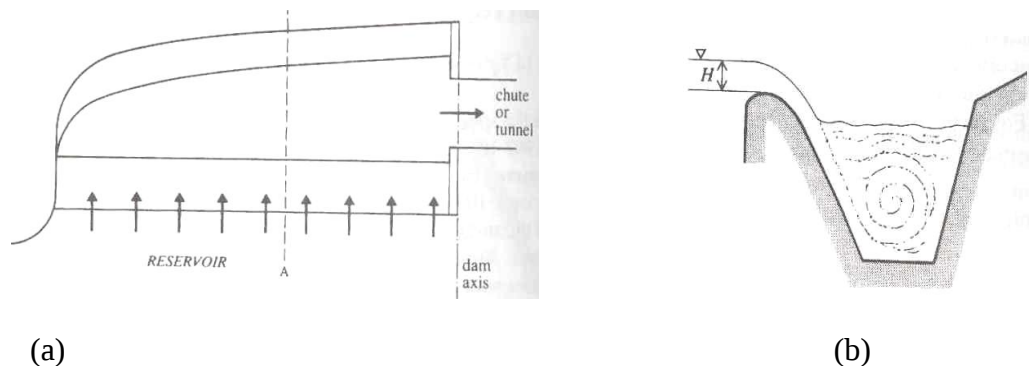


Figure: Side channel spillway: (a)Plan (b) section A-A, side view

They are placed on the side of the dam and have a spillway proper, the flume (channel) downstream of the spillway, followed by the chute or tunnel. The spillway proper is usually designed as a normal overfall spillway. The depth, width, and bed slope of the flume must be designed in such a way that even the maximum flood discharge passes with a free overfall over the entire horizontal spillway crest, so that the reservoir level is not influenced by the flow in the channel. The width of the flume may therefore increase in the direction of the flow. From the energy dissipation point of view, the deeper the channel and the steeper the side facing the spillway, the better; on the other hand, this shape is in most cases more expensive to construct than a shallow wide channel with a gently sloping side.

### D. Siphon spillway

Siphon spillways are closed conduits in the form of an inverted U with an inlet, short upper leg, throat (control section), lower leg, and outlet.

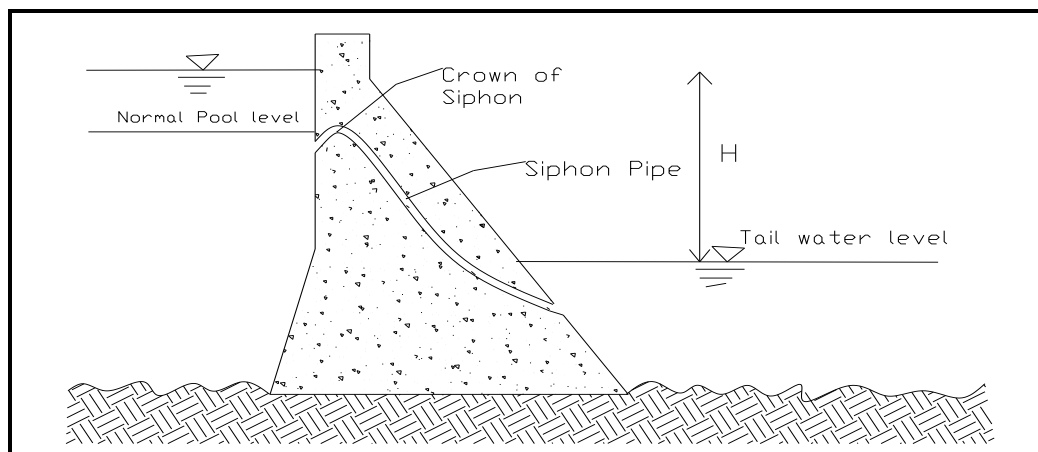


Figure: siphon pipe installed within the gravity dam

For very low flows a siphon spillway operates as a weir; as the flow increases, the upstream water level rises, the velocity in the siphon increases, and the flow in the lower leg begins to exhaust air from the top of the siphon until this primes and begins to flow full as a pipe, with the discharge given by

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

Where: A is the (throat) cross-section of the siphon, H is the difference between the upstream water level and siphon outlet or downstream water level if the outlet is submerged and

$$C_d = 1/(K_1 + K_2 + K_3 + K_4)^{1/2}$$

Where: K<sub>1</sub>, K<sub>2</sub>, K<sub>3</sub>, and K<sub>4</sub> are head loss coefficients for the entry, bend, exit, and friction losses in the siphon.

#### E. Chute spillway

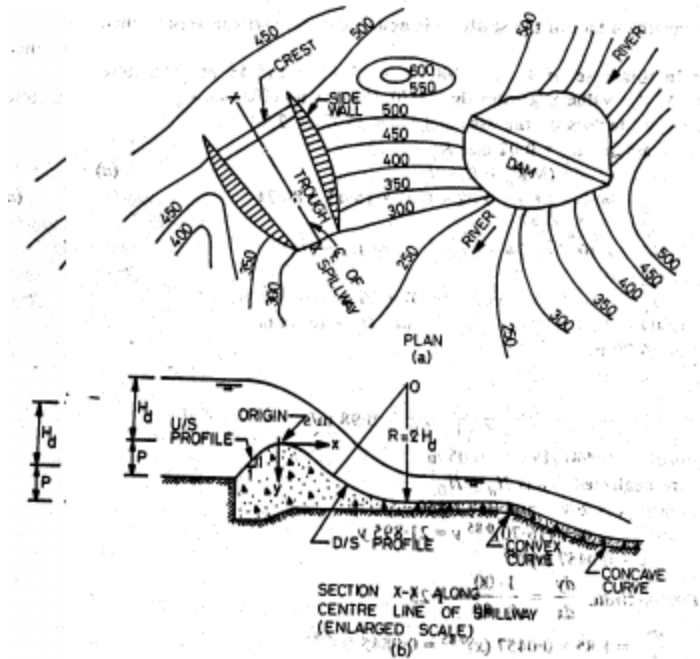
A chute spillway is a steep channel conveying the discharge from a low overfall, side channel, or special shape spillway over the valley side into the river downstream.

For earthen and rock fill dams, a separate spillway is generally constructed in a flank or a saddle, away from the main valley. Sometimes, even for gravity dams, a separate spillway is required because of the narrowness of the main valley. In all such circumstances, a separate spillway like chute could be provided.

A chute spillway essentially consists of a steeply sloping open channel, placed along a dam abutment or through a flank or a saddle. It leads the water from the reservoir to the downstream channel below.

The entire channel spillway can hence be divided into the following parts:

- I. Entrance channel
- II. Control structure (Low Ogee weir)
- III. Chute channel or discharge carrier
- IV. Energy dissipation arrangements at the bottom in the form of the stilling basin



#### F. Shaft spillway

A shaft ('morning glory') spillway consists of a funnel-shaped spillway, usually circular in plan, a vertical (sometimes sloping) shaft, a bend, and a tunnel terminating in an outflow as shown in the figure below.

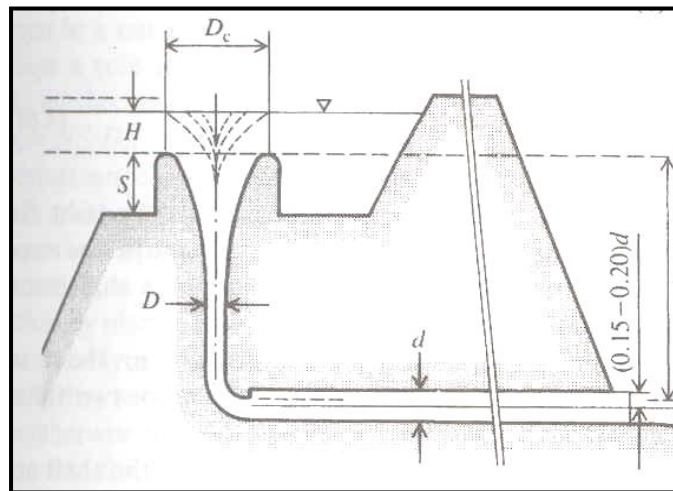


Figure: Shaft spillway

The shape of the shaft spillway is derived in a similar manner to the overfall spillway from the shape of the nappe flowing over a sharp-edged circular weir. Clearly, in this case the shape for atmospheric pressure on the spillway is a function of  $H_s/D_s$ , where  $H_s$  is the head above the notch crest of the diameter  $D_s$ . For ratios  $H_s/D_s < 0.225$  the spillway is free-flowing and for  $H_s/D_s > 0.5$  the overflow is completely drowned. For the free overfall the discharge is given by

$$Q = \frac{2}{3} C_d \pi D_c \sqrt{2g} H^{3/2}$$

And for the drowned (submerged) regime, the discharge is given by

$$Q = \frac{1}{4} C_{d1} \pi D^2 [2g(H + Z)]^{1/2}$$

Where:  $D$  is the shaft diameter,  $D_c$  is the crest diameter ( $D_c < D_s$ ),  $H$  is the head of the reservoir level above the crest ( $H < H_s$ ),  $Z$  is the height of the crest above the outflow from the shaft bend,  $C_d$  and  $C_{d1}$  are discharge coefficients.

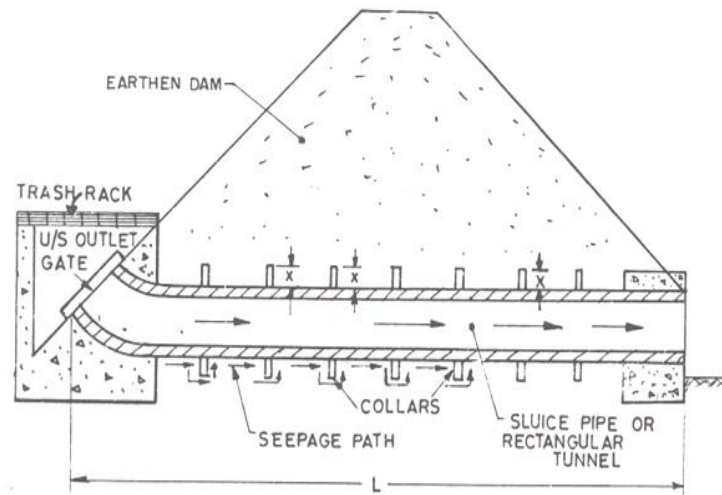
### 1.6 Bottom outlets

Bottom outlets are opening in the dam for several purposes among which are

- i) Filling of the reservoir control
- ii) Draw down of the reservoir
- iii) Flood and residual diversion.

According to the type of control gates (values) and the position of the outflow in relation to the tail water, they operate either under pressure or free the bottom outlets can also be used as compensating flow for a river stretch downstream of the dam where the flow would otherwise fall below acceptable limits. Large bottom outlet opening serve as submerged spillways.

A typical arrangement for a large bottom spillway is shown in figure below.



### Design principles

As a result of the high velocity  $V$  at the bottom outlet; cavitation, abrasion and aerated flow are among the problems. Additional concerns with bottom outlet are

- Sediment flow due to reservoir sedimentation
- Gate blocked due to floating debris or sediment deposits.
- Gate vibration due to the high velocity flow, and
- Sealing of funnel flow due to limited air access.

The velocity of the bottom outlet flow is nearly as large as given by Torricelli formula

$$V = (2gH_o)^{1/2}$$

Where  $H_o$  = Head of flow on the outlet.

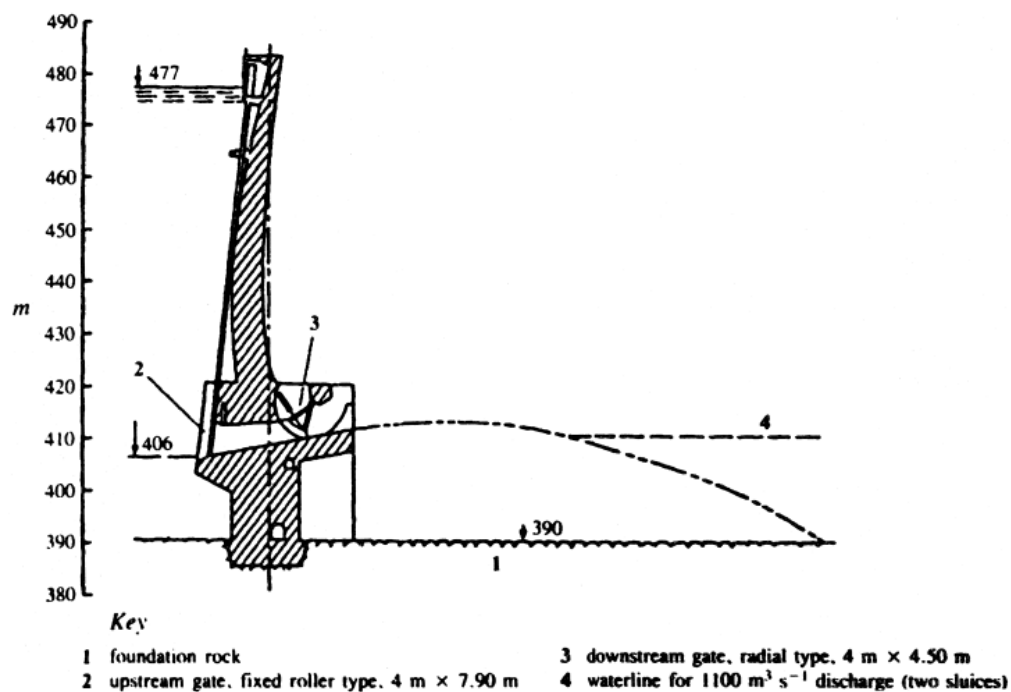
$g$  = Acceleration due to gravity.

Usually, bottom outlet structures are designed to operate under all conditions for which it was planned.

Two gates are often provided called outlet gates.

- i) Safety gate or guard gate either opened or closed.
- ii) Service gate or regulating gate with variable opening.

Bottom outlet should be provided for every dam of whatever size, particularly for emergency repair. A useful design is the combination of diversion tunnel and bottom outlet. However, for smaller dams or for arch dams, a culvert type bottom outlet may also be considered because of simplicity in design (See figure below).



Sainte-Croix arch dam bottom spillway (Thomas, 1976)

The technical requirements for a bottom outlet may be summarized as follows.

- Smooth flow for completely opened outlet structure.
- Excellent performance for all flows under partial opening.
- Effective energy dissipation at terminal outlets.
- Structure without leakage
- Simple and immediate application
- Easy access for maintenance and service
- Economic and useful design, and
- Long life span.

The inlet construction and shape is designed to reduce the head loss, the flared outflow section is to aid energy dissipation downstream of the outlet. The air vent is to protect the junction of the spillway and outlet against possible cavitation damage.

The head loss at an inlet is expressed as

$$h = \xi V^2 / 2g$$

$v$  = Velocity of flow

$g$  = acceleration due to gravity

$\xi$  = Coefficient ( $0.15 < \xi < 0.34$  for inlets with grooves and screens).

The lower values of the coefficient  $\xi$  is usually associated with inlets with curved walls, but sometimes the same result can be achieved by a transition formed by several plane surfaces at the expense of allowing small negative pressure to develop on them.

## 2. Energy Dissipation

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 arrangements are not made to dissipate this huge kinetic energy of the water, and 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. These arrangements are known as energy dissipation arrangements or energy dissipaters.

For the dissipation of the excessive kinetic energy possessed by the water the two common methods adopted are:

- i. By converting the supercritical flow into subcritical flow by hydraulic jump.
- ii. By using different types of buckets, i.e. by directing the flow of water into air and then making it falls away from the toe of the structure.

### 2.1. Jump Height and Tailwater 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 ( $y_2$ ).

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

Where  $y_1$  = pre-jump (initial) depth

$y_2$  = post- jump (sequent) depth

$F_{r1}$  = Froude number of the incoming flow

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, (see the figure below).

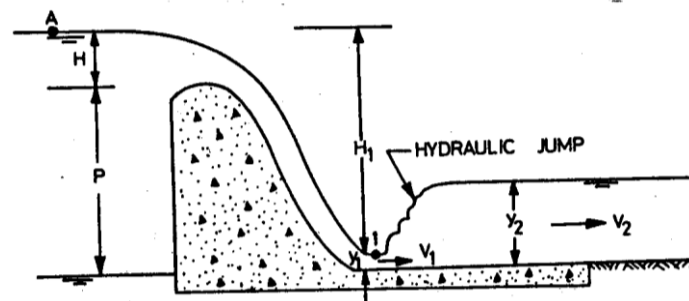


Fig: Hydraulic  
a spillway

jump at the toe of

Hence, for a given discharge intensity and given height of spillway,  $y_1$  is fixed and thus  $y_2$  is also fixed. But the availability of a depth equal to  $y_2$  in the channel on the downstream cannot be guaranteed as it depends upon the tail water level  $y_0$  which depends on the hydraulic conditions of the river channel on the downstream side. The values of  $y_0$  corresponding to different values of  $q$  may be obtained by actual gauge discharge observations and plot of  $y_0$  versus  $q$  prepared, known as Tailwater Rating curve (T.W.R.C.). The post-jump depth ( $y_2$ ) for all those discharges, are also computed from the hydraulic jump equation above and a plot of  $y_2$  versus  $q$  may be made which is known as jump height curve (J.H.C.). 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.

1. T.W.R.C. ( $y_2'$ ) coinciding with  $y_2$  curve for all discharges
2. T.W.R.C. ( $y_2'$ ) lying above the  $y_2$  curve for all discharges
3. T.W.R.C. ( $y_2'$ ) lying below the  $y_2$  curve for all discharges
4. T.W.R.C. ( $y_2'$ ) lying below the  $y_2$  curve for smaller discharges and lying above  $y_2$  curve for larger discharges
5. T.W.R.C. ( $y_2'$ ) lying above the  $y_2$  curve for smaller discharges and lying below the  $y_2$  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.

*Condition 1:* In this case for the entire discharges jump will develop close to the toe of the spillway. In such a case, a simple horizontal concrete apron may be provided whose length is equal to the length of the jump corresponding to the maximum discharge over the spillway.

- Provide a horizontal concrete apron and stilling basin

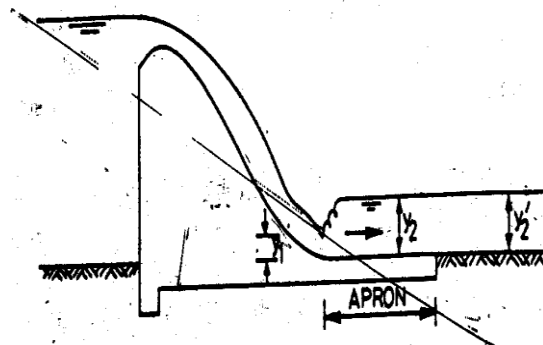
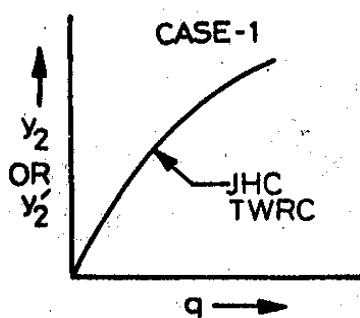


Fig: Condition 1

*Condition 2:* The jump forming at toe will be drowned out by tailwater, and little energy will be dissipated.

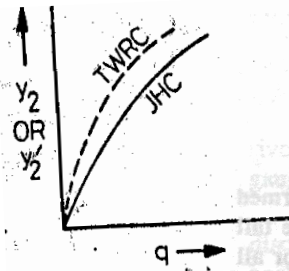


Fig a: Condition 2

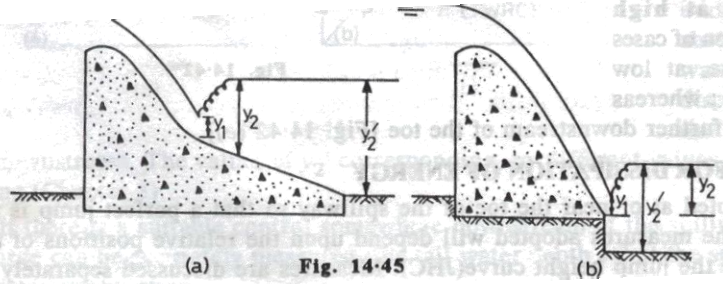


Fig b: Hydraulic jump on a sloping apron

Fig C: drop provision

Water may continue to flow at high velocity along the channel bottom for a considerable distance. The problem can be solved:

- i) By constructing a sloping apron over the riverbed extending from the downstream surface of the spillway. The jump will form on the sloping apron where depth equal to  $y_2$  (less than the tailwater depth at toe) is available.

The slope of the apron is made in such a way that proper conditions for a jump to occur somewhere on the apron at all discharges.

- ii) By providing a roller bucket type energy dissipater.

Also a drop provided in the riverbed to lower the TWL can be used to dissipate the energy (Fig C)

- iii) Construction of a secondary dam

*Condition 3:* 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.

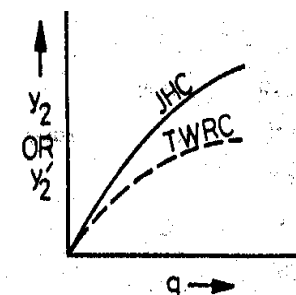
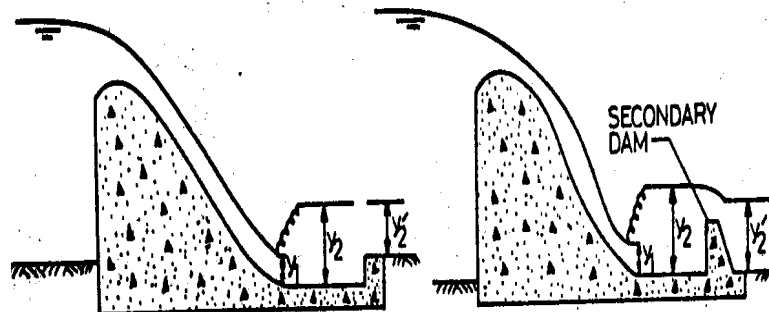


Fig: a. Condition 3,



b. stilling basin with baffle piers, c. stilling basin with a low secondary dam

This means that the depth of flow in the river in particular section is *insufficient* for all discharges for formation of jump at the toe of structure. The jump will try to sweep across the apron at a high velocity and attack the bed downstream. The energy dissipation can be achieved in any of the following ways:

(a) Lowering the floor level D/S of the spillway so as to make the tail water depth in the stilling basin equal to jump height curve for all discharges. This may lead to three cases:

- (i) Horizontal floor but depressed below the river bed level
- (ii) A depressed floor but rising towards the Downstream end
- (iii) A depressed floor but sloping away from the toe of spillway

(b) Stilling basin with baffles or sills at river bed level

(c) Stilling basin with a low subsidiary dam downstream.

(d) Bucket type structure - If under the conditions of low tail water depth there is a bed of solid rock which can withstand the impact of water, Ski Jump bucket energy dissipater may be adopted. Such a device will throw the high velocity flow passing over the spillway upwards so it travels some distance from the toe of the spillway before it falls back and strikes the river bed. Here the energy is dissipated by the aeration of the jet and impact of the water on the river bottom. Though some scour takes place, it is too small or too far from the dam to endanger it.

*Condition 4:* In this case the following measures may be taken to develop jump close to the spillway.

- i) Provide a stilling basin with an end sill or low secondary dam for developing a jump at low discharges and combine the basin with a sloping apron for developing a jump at high discharges.
- ii) 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.

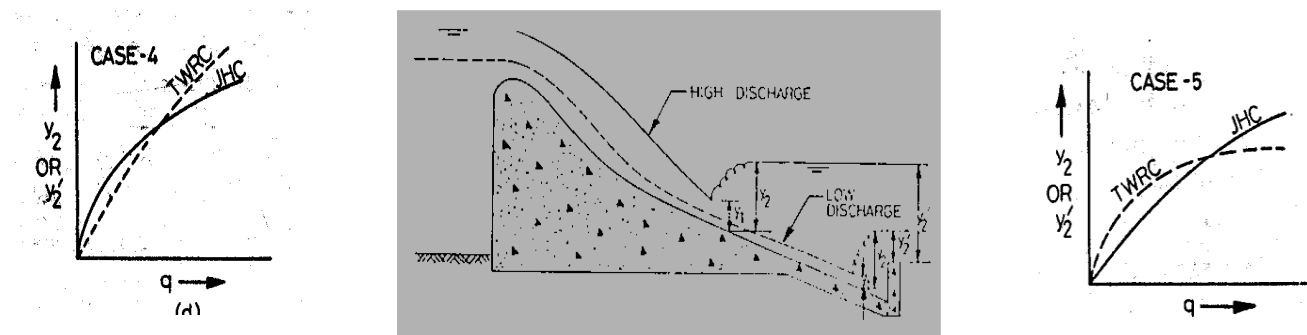


Fig: Condition 4

In this case, the tail water depth is insufficient at low discharges and is greater at high discharges.

A horizontal apron in river level in this case provides an insufficient depth at low discharges and extra depth for high discharges to form a suitable Hydraulic Jump. The solution therefore, lies in creating artificially enough water depth to make the jump form on apron at low discharges.

The following may be adopted:

- (a) Stilling basin with a low secondary dam
- (b) Stilling basin with baffle piers or some form of dentated sill

*Condition 5:* This condition is just the reverse of condition (4) and the same arrangement that was made for condition (4) will serve the purpose (see the figure above).

## 2.2. Energy Dissipation On Spillway Surface

The energy loss on the spillway surface may be expressed as

$$e = \zeta \alpha \frac{V'^2}{2g}$$

Where  $V'$  = the (supercritical) velocity at the end of the spillway

$\alpha$  = Coriolis coefficient (energy coefficient)

$\zeta$  = head loss coefficient.

The total energy,  $E$ , can be expressed as

$$E = \zeta \alpha \frac{V'^2}{2g} + \alpha \frac{V'^2}{2g}$$

And, taking the velocity coefficient,  $\phi = \frac{\text{actual velocity}}{\text{theoretical velocity}}$

Head loss coefficient is related to the velocity coefficient  $\phi$  (the ratio of actual to theoretical velocity) as;  $\frac{1}{\phi^2} = 1 + \zeta$

The ratio of the energy loss,  $e$ , to the total energy  $E$  (i.e. relative energy loss) is

$$\frac{e}{E} = \frac{\zeta V'^2}{2g} \left/ \left( \frac{V'^2}{2g} + \zeta \frac{V'^2}{2g} \right) \right. = \frac{\zeta}{1 + \zeta} = 1 - \phi^2$$

For the ratio of the height  $P$  of the spillway crest above its ending and the overflow head  $H$ , with  $P/H < 30$ , and smooth spillways (Novak & Cabelka, 1981),

$$\phi \cong 1 - 0.0155P/H$$

For a given  $P$ ,  $\phi$  increases as  $H$  increases, i.e., if for a given discharge  $Q$  the spillway width  $b$  decreases and thus  $q$  increases.

Thus, for  $P/H = 5$ ,  $\phi = 0.92$  and the relative head loss ( $e/E$ ) is 15%, where as for  $P/H = 25$ ,  $\phi = 0.61$  and relative head loss is 62 %.

The value of head loss coefficient ( $\zeta$ ) could be increased (and  $\phi$  decreased) by using a rough spillway or by placing baffles on the spillway surface. However, unless aeration is provided at these protrusions, the increased energy dissipation may be achieved only by providing an opportunity for cavitation damage.

Stepped spillways *may* provide an opportunity for additional energy dissipation (when compared with smooth spillways) pending on the value of the unit discharge ( $q$ ).

### 2.3. Energy Dissipation in the Stilling Basin

Referring to the notation in the figure below and to the equations  $e = \zeta \alpha \frac{V'^2}{2g}$  and  $\frac{1}{\phi^2} = 1 + \zeta$  we can write as:

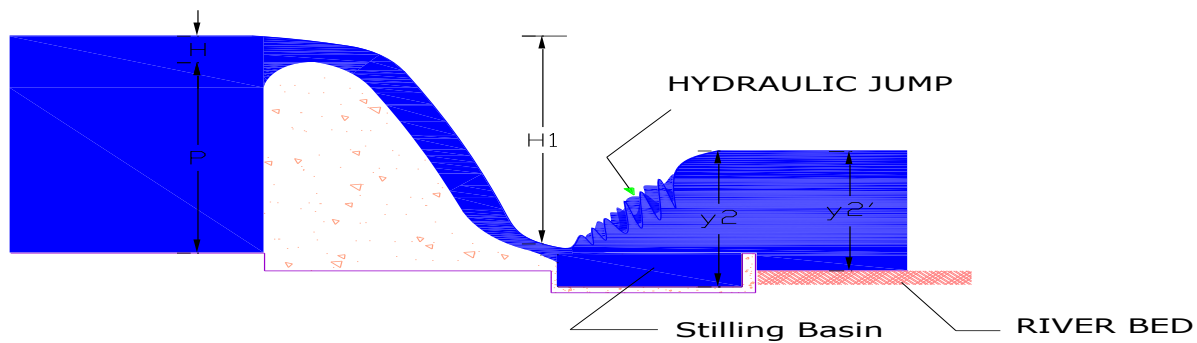


Fig: Definition sketch for hydraulic jump stilling basin

$$E = y_1 + \frac{\alpha q^2}{2g\phi^2 y_1^2} \dots\dots\dots(a)$$

$$y_2 = \frac{y_1}{2} \left[ -1 + \left( 1 + \frac{8q^2}{g y_1^3} \right)^{1/2} \right] \dots\dots\dots(b)$$

The stilling basin depth is then given by

$$y' = \sigma' y_2 - y'_2 \dots\dots\dots(c)$$

and the length of the stilling basin is given by

$$L = k(y_2 - y_1) \dots\dots\dots(d)$$

Where  $\sigma'$  and  $k$  are coefficients derived from laboratory and field experiments.

According to Novak and Cabelka Coefficients  $\sigma'$  and  $k$  can be taken as  $1.1 < \sigma' < 1.25$  and  $4.5 < k < 5.5$ , where the lower value of  $k$  applies for  $F_{r1} > 10$  and the higher for  $F_{r1} \leq 3$ .

When applying the equations above we start with a known discharge  $q$  and the corresponding downstream depth  $y_0$  and apply the iterative procedure, which follows:

- 1) Take the energy (reference) datum at downstream riverbed level, and compute  $E$  assuming an initial value of  $y' = 0$ ;
- 2) Choose a suitable value of  $\phi$ ; ( $\phi = 1 - 0.0155P/H$ );
- 3) Compute  $y_1$  for  $q_{\max}$  from equation (a);  $y_2$  from equation (b); and  $y'$  from equation (c) (From a chosen value of safety coefficient,  $\sigma'$ );
- 4) Compute  $y_0$  (from uniform flow equation – Manning, Chezy) and compare it with  $y_2$ .
- 5) If  $y_2 < y_0$ , no stilling basin is required; if  $y_2 \geq y_0$  stilling basin is required and therefore compute  $y'$  with  $1.1 < \sigma' < 1.2$  ( $\approx 1.2$ ) from equation (c);
- 6) Take new reference datum at basin bed level; and calculate new  $E$  and repeat steps 2-4 to check that  $\sigma' \geq 1.1$ .
- 7) Repeat the above steps at least for one smaller  $q$  to check whether the designed stilling basin is adequate for lower discharges as well.

Note: Equations (b) and (d), and thus the design under discussion, apply to basins with a horizontal floor only.

### 2.3.1. Additional Considerations in Stilling Basin Design

The hydraulic jump entrains a substantial amount of air additional to any incoming aerated flow. The main significance of the presence of air in the jump region is the requirement of higher stilling basin sidewalls due to higher depth of flow.

The major problems in spillway stilling basin are cavitation, uplift, and abrasion.

The highly turbulent nature of the flow in the hydraulic jump induces large pressure fluctuations and is the cause of cavitation. Cavitation number can be expressed as

$$\sigma = P' / \left( \frac{1}{2} \rho V_1^2 \right)$$

Where,  $P'$  is the deviation of the instantaneous pressure  $P$  from the time average pressure. If  $\sigma$  falls below a critical value,  $\sigma_c$ , then cavitation occurs.

Another serious structural problem in hydraulic jump stilling basins is the effect of uplift pressures due to the dam drainage system or the tailwater level or the water table in the basin bank, which is aggravated by the macro-turbulent pressure fluctuations underneath the jump. Therefore, it is sensible to design the floor slab for the full downstream uplift pressure applied over the whole area of the floor with the basin empty or the uplift pressure head equal to the root mean square value of pressure fluctuations of the order of  $0.12V_1^2/2g$  ( $V_1$ = inlet supercritical velocity) applied under the whole full basin. Furthermore, all contraction joints should be sealed, no drain openings should be provided, and the floor slab should be as large as possible and connected by dowels and reinforcement (ICOLD, 1986).

Abrasion of concrete in the basin could take place if this is also used for bottom outlets carrying abrasive sediments (unlikely to happen for  $V < 10\text{m/s}$ ), or from sediment drawn into the basin from downstream either by bad design or operation. The basin should be self-cleaning to flush out any trapped sediment.

The prevention of vibration of basin elements (due to turbulence of the flow) also requires massive slabs, pinned to the foundation when possible.

### 2.3.2. Standard Stilling Basins

Although the stilling basin based purely on a simple hydraulic jump works well and relatively efficiently, in certain conditions other types of basins may produce savings in construction costs. Certain accessories such as chute blocks, baffle blocks (or floor blocks), and end sills (or baffles) are usually provided in the stilling basins to reduce the length of the jump and thus to reduce the length and the cost of the stilling basin. Moreover, these accessories also improve the dissipation action of the basin and stabilize the jump.

The type of stilling basin to be provided depends on the type of jump, which in turn depends on the Froude number  $F_{r1}$  of the incoming flow.

#### U.S.B.R. Stilling Basins

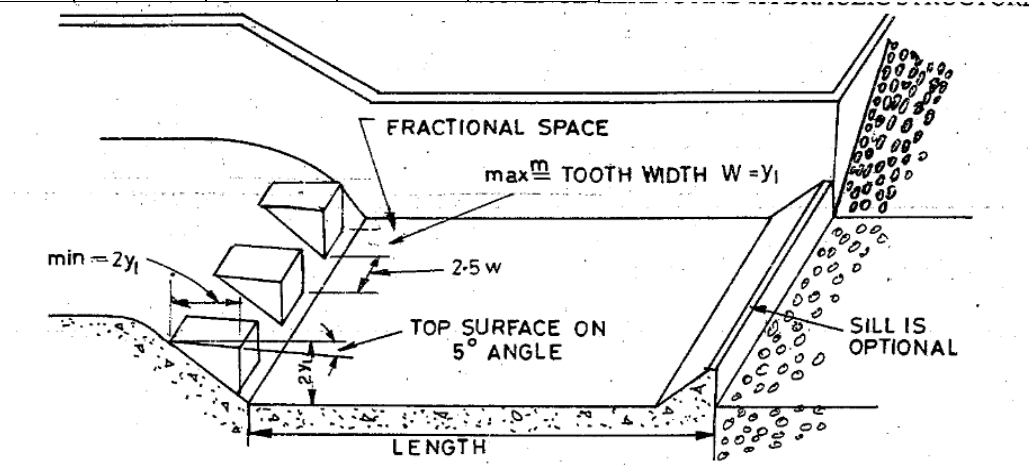
- (i) Stilling basin for  $1.7 < F_{r1} < 2.5$  (Type I)

Only horizontal apron needs to be provided. The flow does not have much turbulence and hence no accessions are required. However, the apron must be sufficiently long to contain the entire jump. The length of the apron should be the length of the jump (i.e.  $5y_2 = L$ , and  $(L \geq 4y_2)$  where  $y_2 =$  sequent depth).

(ii) Stilling Basin for  $2.5 < Fr_1 < 4.5$  (Type IV)

Type IV stilling basin is found effective. It is provided with chute blocks and end sill is optional. The length  $L$  of the stilling basing may be obtained from the following table.

|         |     |     |     |   |
|---------|-----|-----|-----|---|
| $Fr_1$  | 2   | 3   | 4   | 5 |
| $L/y_2$ | 4.3 | 5.3 | 5.8 | 6 |

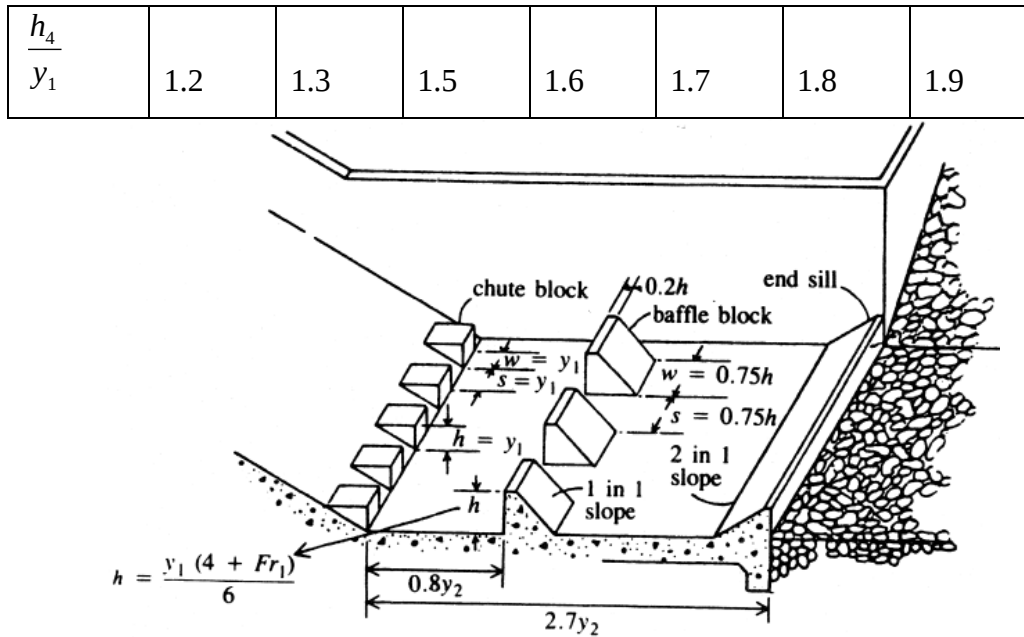


(iii) Stilling Basins for  $Fr_1 > 4.5$

True hydraulic jump will form. Depending on the incoming velocity of flow two types of basins are developed:

- a)  $V_1 < 15$  m/s: *Type III stilling basin may be adopted.* This basin utilizes chute blocks, baffle blocks and end sill (the size, spacing and location of the chute and baffle blocks are shown in the Figure). The length of the stilling basin and the height  $h_3$  and  $h_4$  of the baffle blocks and the end sill may be obtained for different values of  $Fr_1$  as follows:

|                   |     |     |     |     |     |     |     |
|-------------------|-----|-----|-----|-----|-----|-----|-----|
| $Fr_1$            | 5   | 6   | 8   | 10  | 12  | 14  | 16  |
| $\frac{L}{y_2}$   | 2.3 | 2.5 | 2.6 | 2.7 | 2.8 | 2.8 | 2.8 |
| $\frac{h_3}{y_1}$ | 1.5 | 1.7 | 2.0 | 2.3 | 2.7 | 3.0 | 3.3 |



Stilling basin with chute blocks and baffles, USBR Type III

The use of chute blocks, impact baffle blocks, and end sill shortens the jump length and the stilling basin. This basin relies on dissipation of energy by the impact blocks and on the turbulence of the jump phenomena for its effectiveness. Because of the large impact forces to which the baffles are subjected by the impingement of high incoming velocities and because of the possibilities of cavitation along the surfaces of the blocks and floor (due to downstream suction), the use of this basin should be limited to heads where the velocities do not exceed 15 m/s.

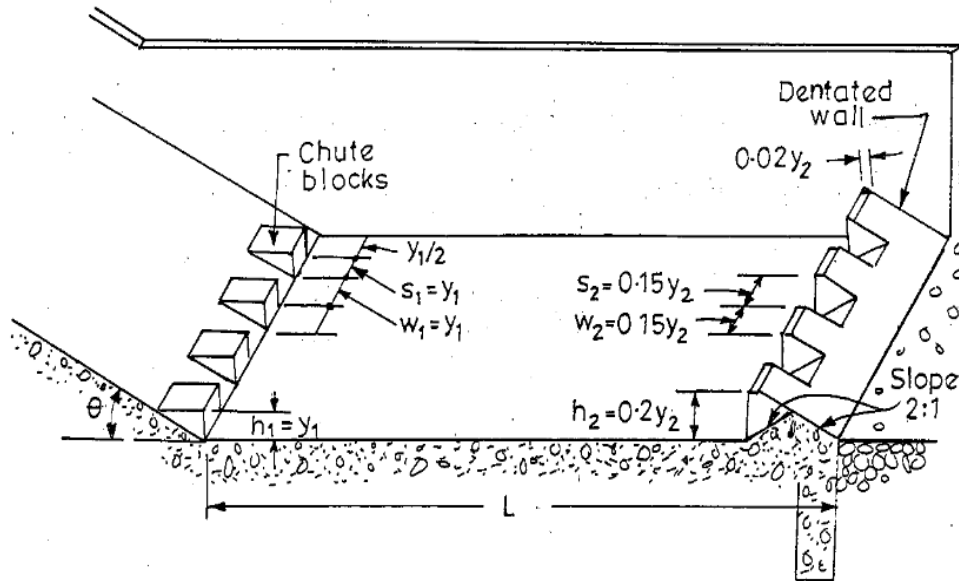
**b)**  $V_1 > 15$  m/s: Here impact baffle blocks are not employed and they are designated as *Type II stilling basin*. Because the dissipation is mainly accomplished by hydraulic jump action, the basin length will be greater than that indicated for type III basin.

However, the chute blocks and dentated end sill (instead of solid end sill) will still be effective in reducing the length from that which would be necessary if they were not used. In this basin baffle blocks are not provided because.

- i) due to the high velocities of incoming flows these blocks will be subjected to excessively large impact forces, and
- ii) There is a possibility of cavitation along the downstream face of these blocks and the adjacent floor of the basin due to large negative pressure being developed in this region.

The length  $L$  of Type II stilling basin may be obtained for different values of  $F_{r1}$  from the following table

|          |      |     |     |     |     |     |
|----------|------|-----|-----|-----|-----|-----|
| $F_{r1}$ | 5    | 6   | 8   | 10  | 12  | 14  |
| $L/y_2$  | 3.85 | 4.0 | 4.2 | 4.3 | 4.3 | 4.3 |



## 2.4. Plunge pools

Downstream of free-falling jets (jet, ski-jump spillways, flip buckets) energy dissipation takes place in stilling basins, or more frequently, in plunge pools, usually excavated fully or partially in the stream bed during dam construction, but sometimes only scoured by the action of the jet itself.

The general form of the scour,  $y_s$ , measured from the tailwater surface  $y'_s = y_s + y_o$  is

$$y'_s = Cq^x H_*^y \beta^w / d^z$$

Where  $C$  is a coefficient,  $\beta$  is the angle of the flip bucket with the horizontal;  $d$  is the particle size (mm),  $H_*$  is the difference between upstream and downstream water levels (m),  $y_o$  is the tailwater depth (m),  $q$  is the specific discharge ( $m^2/s$ ), and  $d_{90}$  is the 90% grain size of sediment forming the river bed (mm).

$y_s$  is the scour depth below the river bed (m), given by

$$y_s = 0.55[6H_*^{0.25} q^{0.5} (y_o/d_{90})^{1/3} - y_o]$$

The range of the coefficient  $C$  and exponents  $x, y, z$  is  $0.65 < C < 4.7$ ,  $0.5 < x < 0.67$ ,  $0.1 < y < 0.5$ ,  $0 < z < 0.3$ ,  $0 < w < 0.1$ .

- A simplified equation by with  $x=0.6, y=0.1, w=z=0$  and  $C=1.5$  can be used:

$$y'_s = 1.5q^{0.6}H_*^{0.1}$$

$H_*$  is the drop from the reservoir level to the flip bucket lip.

- For flip bucket spillways the equation proposed by Tarajmovich (1978) also gives reasonable results:

$$y_s = 6y_{cr} \tan \beta_1$$

Where  $y_s = y'_s - y_o$ ,  $y_{cr}$  is the critical depth, and  $\beta_1$  is the upstream angle of the scour hole, which is a function of the flip bucket exit angle but does not vary widely ( $14^\circ$  to  $24^\circ$  for  $10^\circ$  to  $40^\circ$ ). The pressure fluctuations on the floor of a plunge pool underneath a plunging jet can be very considerable; it will be a function of plunge length, pool depth, jet size and shape, and can reach 40% of the head  $H_*$ .

## 2.5. Energy dissipation at bottom outlet

The flow from outlets occurs most frequently in a concentrated stream of high velocity. The outlet may terminate below or above the downstream water level, with or without an outlet regulating valve at its end. These variations in design are also reflected in the methods of energy dissipation.

The two main design trends are either to disperse artificially and to aerate the outflow jets (outflow above tailwater with or without control gate at its end) or to reduce the specific discharge at entry into the stilling basin.

This basin may be a common one with the spillway – the best solution when feasible or a separate one. The reduction in specific discharge for a high-velocity stream can be achieved either by depressing the soffit of the outlet simultaneously with its widening, or by using blocks and sills or guide walls just downstream of the outlet and before the entry into the stilling basin, or in the basin itself, or by a combination of the various methods. The first method has the advantage of avoiding cavitation and/or abrasion and is particularly effective when used in

conjunction with a spillway stilling basin because the stream from the outlet can be suitably directed into the basin.

When *submerged deflectors* are used as, for example, in outflows from tunnels, care must be taken in shaping the end of the deflector to avoid cavitation.

For the design of a gradually widening transition for the free supercritical outflow from an outlet which terminates in a separate stilling basin, Smith (1978), recommends for an initial width  $B_0$  (at the outlet), final width  $B_1$  (at the entry into a hydraulic jump basin), and straight side walls diverging at an angle  $\theta$  from outflow axis, the equations:

$$B_1 = 1.1Q^{1/2},$$

$$\tan \theta = \left( \frac{B_1}{B_0} - 1 \right)^{1/3} / (4.5 + 2Fr_0)$$

Where  $Fr_0 = \frac{V_0}{(gB_0)^{1/2}}$  for square conduit section or,

$$Fr_0 = \frac{V_0}{(gD)^{1/2}} \text{ for circular conduit section}$$

For small-capacity outlets (5–10m<sup>3</sup>/s) under high heads, vertical stilling wells provide a compact means of energy dissipation.

Sudden expansion energy dissipator which utilize the principle of energy loss at a sudden enlargement are a fairly recent development. Although almost inevitably associated with cavitation, this occurs away from the boundaries without undue danger to the structure. As an example of this type of structure, the section of one of the three circular cross-section expansion chambers of the New Don Pedro dam, designed to pass a total of 200m<sup>3</sup>/s at a gross head of 170m, is shown in the figure below. The chambers dissipate about 45m of head and the remainder is dissipated in pipe resistance and the 9.14m diameter tunnel downstream of the gates.

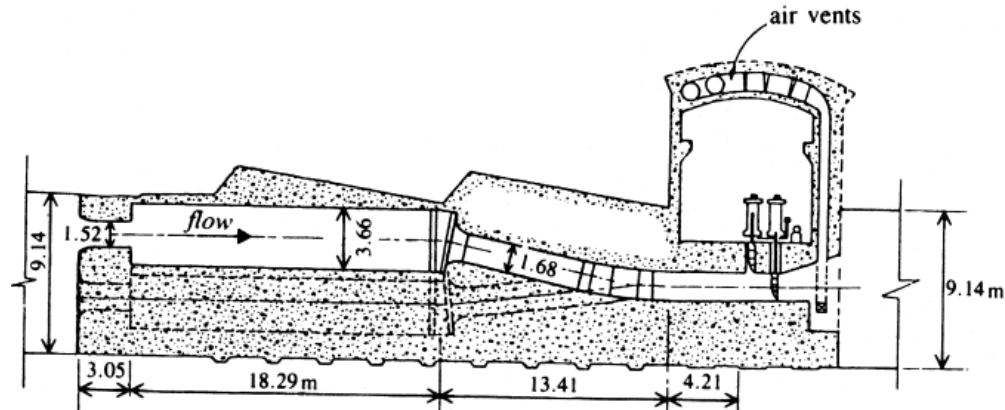


Fig: Sudden expansion dissipator, New Don Pedro dam; dimensions in meters

## 2.6. Submerged bucket dissipaters

When the tail water depth is too great for the formation of a hydraulic jump (i.e. when TW depth are too large as compared to the sequent depths required for the formation of hydraulic jump), dissipation of the high energy of flow can be effected by the use of submerged bucket deflector.

They are of two types, viz.

- i) Solid roller bucket
- ii) Slotted roller bucket

**Solid Roller Bucket:** Consists of a bucket like apron with a concave circular profile of large radius and a deflector lip. When water flows over the bucket the entire sheet of water leaving the bucket is deflected upwards by the bucket lip and two rollers are developed. One of the rollers, called bucket roller, moves in counterclockwise direction and is developed on the surface of the bucket. The other roller moving in clockwise direction, called ground roller, is developed on the ground surface immediately downstream of the bucket. The movement of the rollers, also with the intermingling of the incoming flows, causes the dissipation of energy.

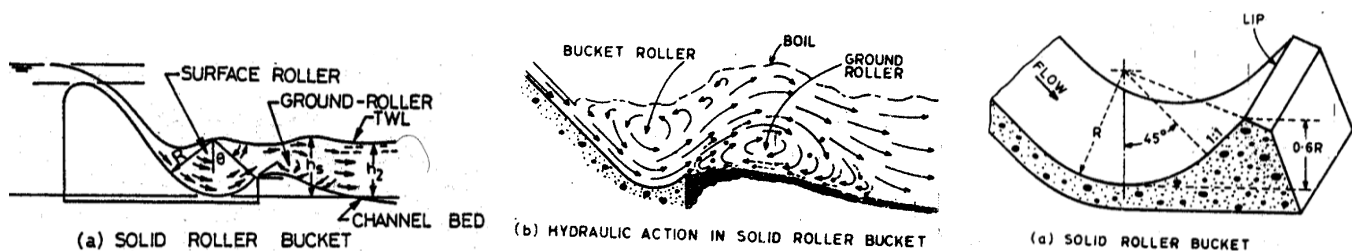


Fig: Solid roller bucket

Slotted Roller Bucket: Consists of a bucket like apron with a concave circular profile of large radius and a slotted or dentated deflector lip. Its action is, in general, same as solid roller buckets. The two rollers are also developed in this case. However, in this case water leaves the lip at a flatter angle and only a part of it is deflected upwards. Thus surface boil is considerably reduced and less violent ground roller occurs which results in a smoother flow on the downstream side.

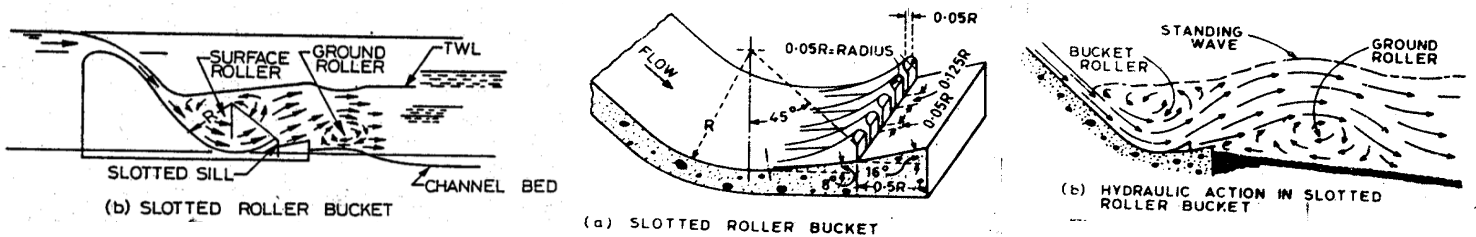


Fig: Ground and surface rollers

### 2.7. Ski – Jump /Deflector Bucket

This type of dissipater has a longitudinal profile, which resembles the submerged bucket. However, the deflector is elevated above the tail water level, so a jet of water is thrown clear off the dam and falls into the stream well clear off the toe of the dam. Spillways may be arranged in pairs, and then the jets made to angle inwards so that they converge and collide in mid – air. This breaks up the jets, and is very effective means of energy dissipation.

The ski – jump bucket may be used where the tail water depth is less than the sequent depth required for the formation of hydraulic jump and the riverbed is composed of stiff rock. In this case, the energy is dissipated by air resistance, breaking up of the jet into bubbles and the impact of the falling jet against the riverbed and tail water.

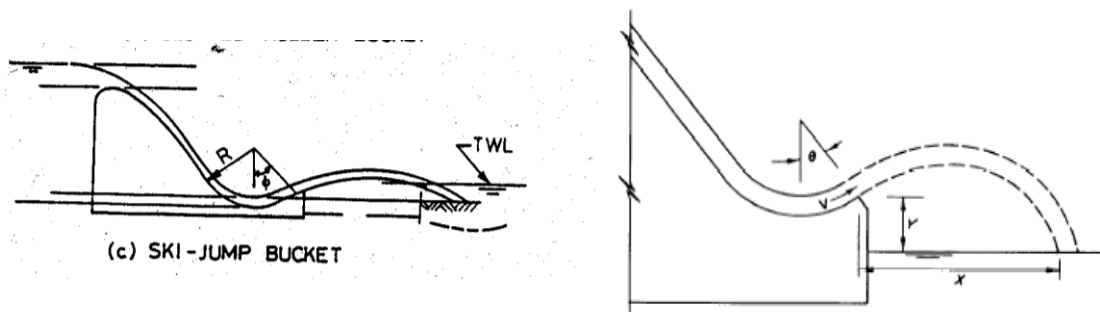


Fig: Sky jump bucket

The use of ski-jump brings substantial economies where geological and morphological conditions are favorable, and particularly where the spillway can be placed over the power station. The head loss in the jet itself, whether solid or disintegrated, is only about 12% (Novak,

1996). But if the jet is split into several streams, which collide, substantial energy will be dissipated. The main benefit for energy dissipation from jet spillways is in the impact into the downstream pool. The major amount of energy dissipation occurs in the region where the jet plunges into the tailwater.

The key parameters for flip-bucket (ski-jump bucket) design are:

- The approach flow velocity and depth
- The radius R of the bucket, and
- The lip angle,  $\theta$ .

At low flow, the bucket acts like a stilling basin with water flowing over the lip and the downstream face; the foundation of the bucket has, therefore, to be protected against erosion.

As the flow increases, a 'sweep-out' discharge is attained at which point the flip-bucket starts to operate properly with a jet. Here, the impact zone of the jet has to be as far away as possible from the bucket to protect the structure against retrogressive erosion. The jet trajectory is hardly affected by air resistance for  $v < 20$  m/s, but for velocities of 40 m/s the throw distance can be reduced by as much as 30% from the theoretical value given by  $(V^2/g) \sin 2\theta$ .

The throw distance  $x$  can also be computed from

$$\frac{x}{H_v} = \sin 2\theta + 2 \cos \theta \sqrt{\sin^2 \theta + \frac{y}{H_v}}$$

Where  $x$  = throw distance, m

$y$  = vertical drop from lip to tail water surface, m

$H_v = V^2/2g$  = velocity head of jet at bucket lip, m

$\theta$  = bucket lip angle

Factors affecting horizontal throw distance include:

1. Initial velocity of jet
2. Bucket lip angle
3. Difference in elevation between lip and tail water

### **3. Intake structures**

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.

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, discharge requirements, climatic conditions etc.

#### **Site selection of intake structures**

There are certain factors which affects the site selection of intakes. They are listed below:

##### **Location**

- The intake should be constructed in the upstream side.
- The intake should never be located in the curves in river.
- The intake should never be constructed near the navigation channel.
- The intake should be constructed such that it is accessible during flood.
- The site must be well connected by good approach of roads.
- The location of intake regarding the sources of pollution need to be considered.

##### **Quantity**

- The intake should be constructed such that sufficient withdrawal of water is permitted to meet the demand of the population or other requirements e.g. for power production or irrigation
- The intake must be capable to fulfill the expansion water works.

##### **Quality**

- Purer zone of the source must be selected for intake construction.

##### **Economy**

- For the reduction in system cost the intake site is selected near the treatment plant/ power house or other irrigation facilities.

## Types of Intake Structures

Intakes are classified under three categories:

### Category 1:

1. Submerged intake/Bottom intake
2. Exposed intake/surface intake

### Category 2:

3. Wet intake
4. Dry intake

### Category 3:

5. River intake
6. Reservoir intake
7. Lake intake
8. Canal intake

### 3.1. Bottom intake

**Simple submerged intake:** it consists of a simple concrete blocks or a rock filled timber crib supporting the starting end of the withdrawal pipes as shown below.

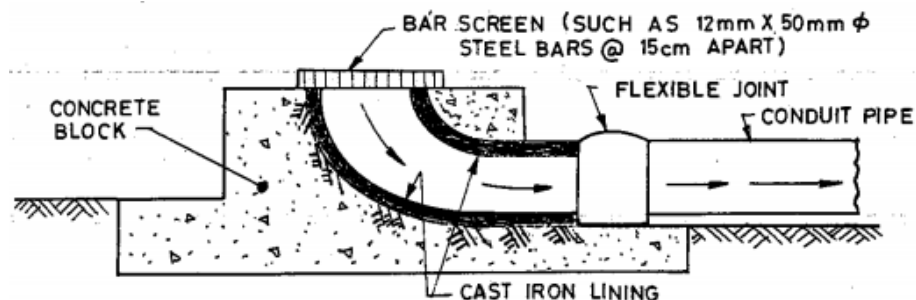


Fig.: Simple concrete block submerged intake

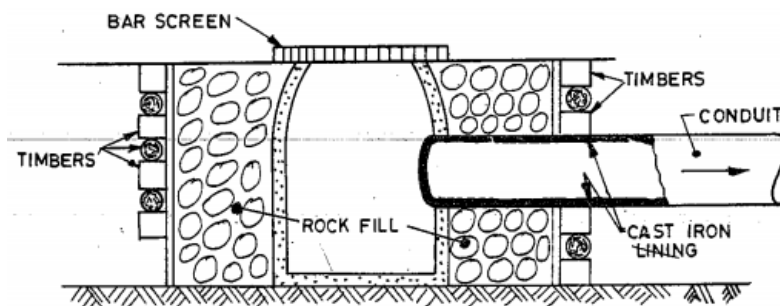


Fig.: Rock-filled timber-crib submerged intake

Such intake structures should be in the river or the reservoir at a place where they may not get buried under the sediment. If the capacity of a project is not much and if it is constructed for a

single purpose, then such type of intake devices may be used to justify the cost benefit analysis. This type of intake is particularly suitable for water supply intake from small rivers. They are not used on bigger projects as their main disadvantage is the fact that they are not easily accessible for repairing of their gates etc.

### 3.2.Surface intake

#### 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

**Wet- intake tower:** A typical section of a wet intake tower shown below, 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 wet-intake tower.

#### Dry intake tower

The essential difference between a wet intake tower and a dry intake tower is that, in the case of a wet intake tower the water enters from the entry ports in to the tower and it enters in into the conduit pipe through separate gate controlled openings. Whereas in a dry intake tower the water is directly drawn in to the withdrawal conduit through the gated entry ports as shown in the figure.

A dry intake tower will therefore have no water inside the tower if its gates are closed, whereas the wet intake tower will be full of water even if the gates are closed.

When the entry ports are closed a dry intake tower will subjected to additional buoyant force & hence must be of heavier construction than the wet intake towers. However, the dry intake towers are useful and beneficial in the sense that water can be withdrawn from any selected level of the reservoir by opening the port at that level.

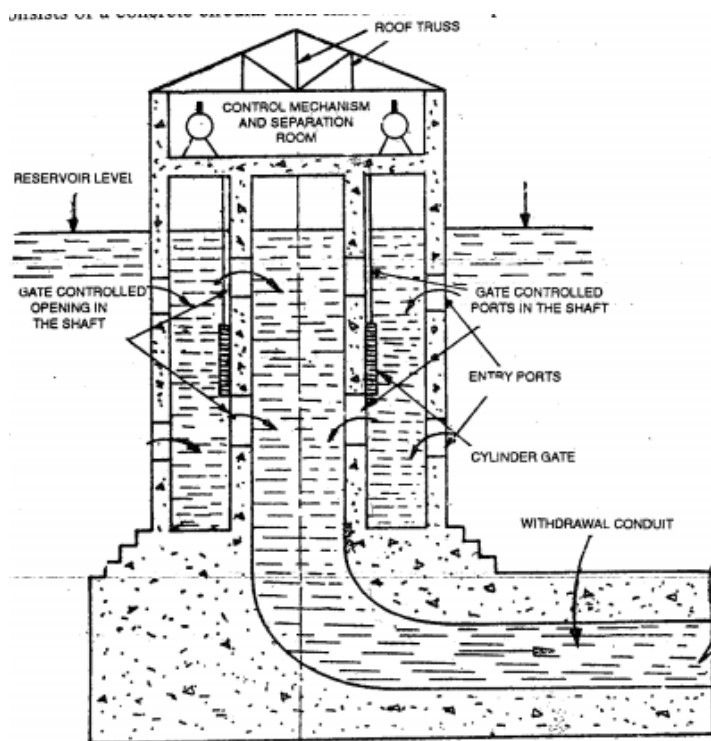


Fig.: Wet- intake tower

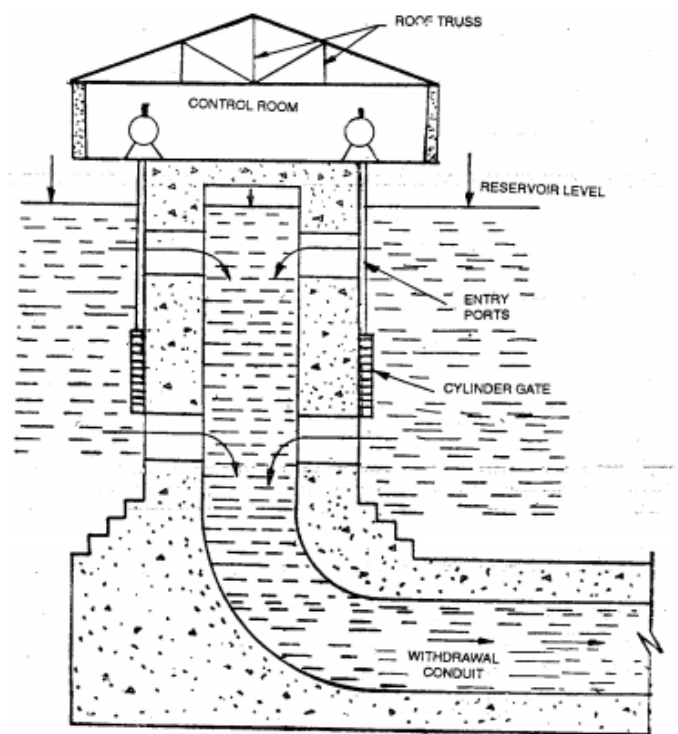


Fig.: dry intake tower

### 3.3.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.

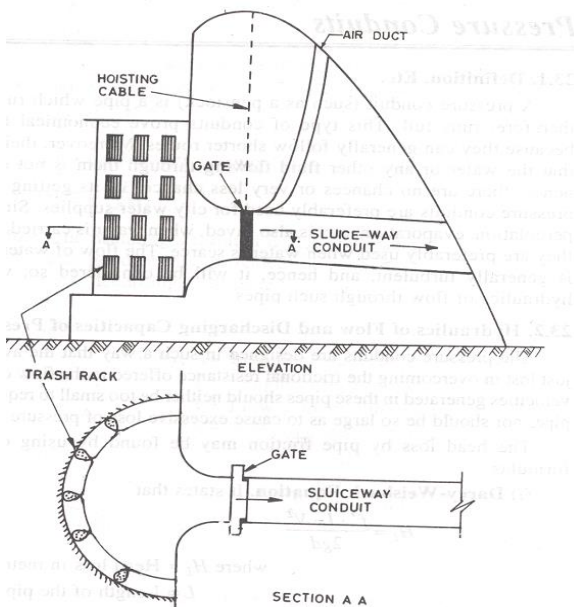


Fig.: Half Cylinder-trash Rack and Tractor Gate installation

## 4 GATES AND VALVES

The main operational requirements for gates are failure-free performance (requiring routine testing/operation), water tightness, rapidity of operation, minimum hoist capacity, and convenience in installation and maintenance.

Gates may be classified as follows:

- Position in the dam – crest gates and high-head (submerged) gates and valves;
- Function – service, bulkhead (maintenance) and emergency gates;
- Material – gates made of steel, aluminium alloys, reinforced concrete, wood, rubber, nylon and other synthetic materials;
- Pressure transmission – to piers or abutments, to the gate sill, to the sill and piers and to the whole structure;
- Mode of operation – regulating and non-regulating gates and valves;
- Type of motion – translatory, rotary, rolling, floating gates, gates moving along or across the flow;
- Moving mechanisms – gates powered electrically, mechanically, hydraulically, automatically by water pressure or by hand.

Because of the multitude of their functions and sizes there is great scope for innovation in gate and valve design both in details (e.g. seals, trunnions), as well as in conceptual design (automatic level control by hinged flap gates, gates and valves used in water distribution systems, ‘hydrostatic’ gates etc.).

### 4.1 Crest gates

#### 4.1.1 Pressure transmission

The basic feature of the structural design of crest gates is the method used for transferring the pressure acting on them.

1. Pressure transmission to piers and abutments is used by plain vertical lift gates and stop-logs, radial gates and roller gates; the gates may be designed for flow over or under them or for a combined flow condition.
2. Pressure transmission to the gate sill is used, for example, by sector (drum) gates (with upstream or downstream hinge), roof (bear-trap) gates, pivot-leaf (flap) gates, roll-out gates, and inflatable gates.
3. Pressure transmission both to piers and sill, e.g. some types of flap gates and floating (pontoon) gates.

#### 4.1.2 Plain gates

Plain (vertical lift) gates, designed as a lattice, box girder, a grid of horizontal and vertical beams and stiffeners, or a single slab steel plate, may consist of single or double section (or even more parts can be involved in the closure of very high openings); in the case of flow over the top of the gate it may be provided with an additional flap gate. The gates can have slide or wheeled support as in the figure below. In the latter case fixed wheels (most frequent type), caterpillar or a roller

train (Stoney gate) may be used; for fixed wheels their spacing is reduced near the bottom. The gate seals are of timber, skin plate, or specially formed rubber as shown in the **figure b** below.

The gate weight,  $G$ , is related to its span,  $B(\text{m})$ , and the load,  $P(\text{kN})$ , by:

$$G = k(PB)^n$$

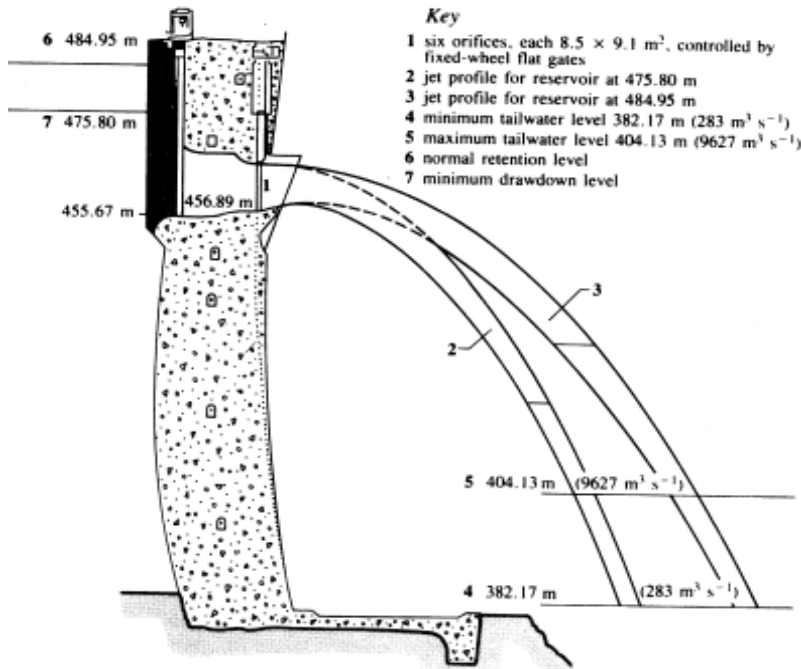


Fig. a: slide gates in Kariba dam spillway, Zimbabwe

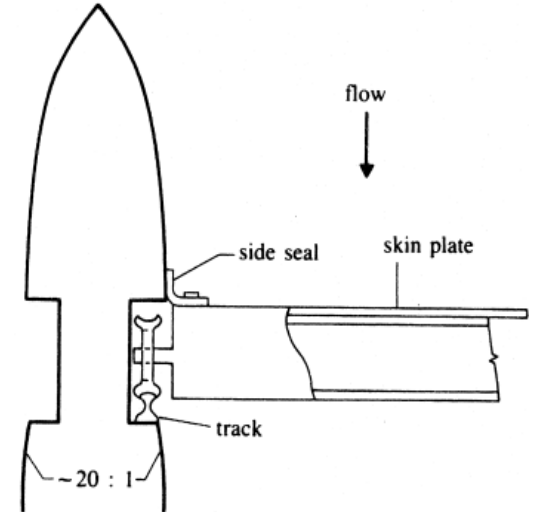


Fig b.: Vertical lift gate: gate track

For slide gates with  $PB > 200 \text{ kNm}$ ,  $k = 0.12$  and  $n = 0.71$ ; for wheeled gates and  $PB > 270 \text{ kNm}$ ,  $k = 0.09$  and  $n = 0.73$ . The usual range of heads for single plain gates is  $1 < H(\text{m}) < 15$ , and of spans  $4 < B(\text{m}) < 45$ . The product of span  $B$  and head at the bottom of the gate  $H$  is usually kept below  $200 \text{ m}^2$ .

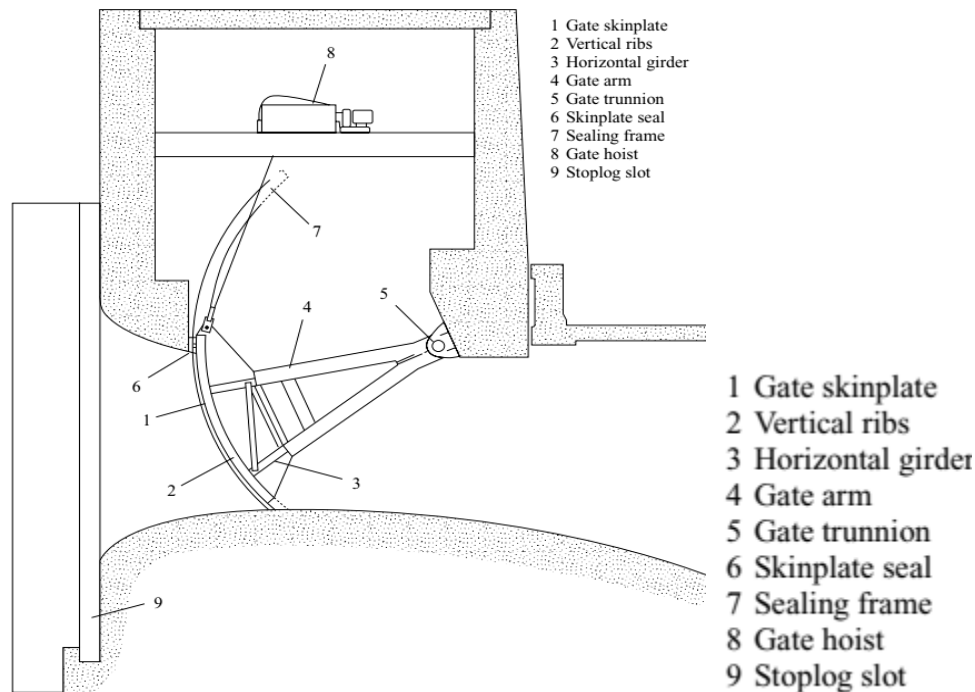
#### 4.1.3 Radial gates

Radial (Tainter) gates are usually constructed as portals with cross-bars and arms (straight, radial, or inclined), but could also be cantilevered over the arms. Their support hinges are usually downstream but (for low heads) could also be upstream, resulting in shorter piers.

For the above equation in radial gates  $0.11 < k < 0.15$  and  $n = 0.07$  for  $PB > 150 \text{ kNm}$ . The usual range of heads and spans for radial gates is  $2 < H(\text{m}) < 20$  and  $3 < B(\text{m}) < 55$ , with  $(BH)_{\text{max}} < 550 \text{ m}^2$ . Radial gates may be designed for more than  $20 \text{ MN}$  per bearing.

The advantages of radial over plain gates are smaller hoist, increased speed of raising, higher stiffness, lower piers, absence of gate slots, easier automation and better winter performance; on the other hand, radial gates require longer and thicker piers, and there may be difficulties with

the bulkhead gate installation. The gate is usually hoisted by cables fixed to each end to prevent it from twisting and jamming as shown in the figure below. If the cables are connected to the bottom of the gate its top can be raised above the level of the hoist itself.

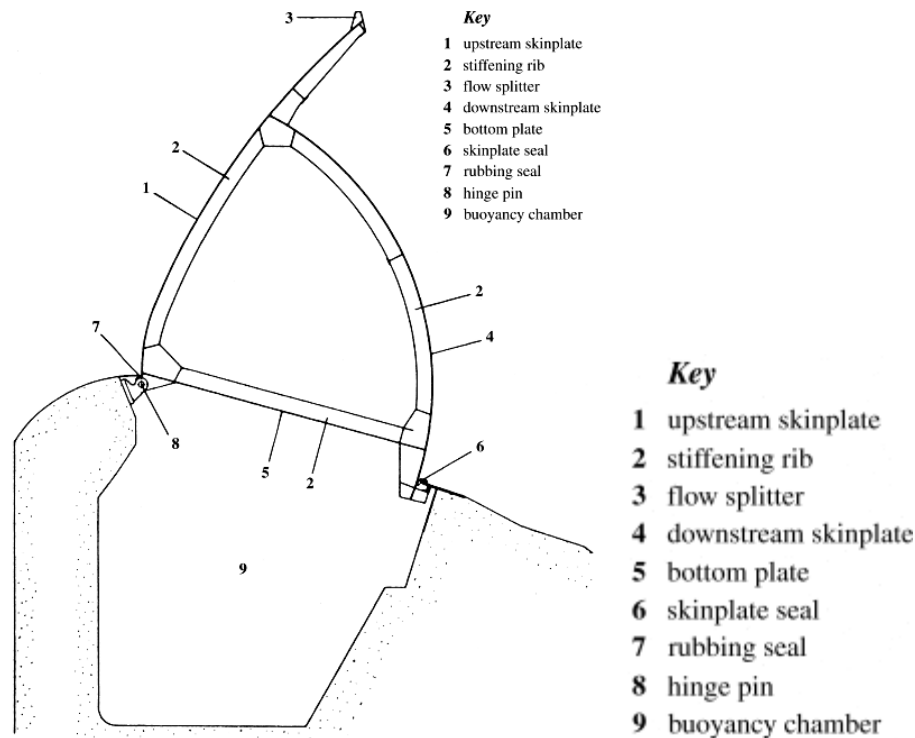


**Fig.: Radial gate (after Thomas, 1976)**

#### 4.1.4 Sector gates

Sector (drum) gates may be hinged upstream (as in the figure below) or downstream; in the latter case the hinge is usually below the spillway crest by about  $0.1H$  to  $0.2H$ . Sector gates on dam crests are usually of the upstream hinge type, with the hinge about  $0.25H$  above the downstream gate sill and a radius of curvature of  $r \approx H$ . The heads can be as high as 10m and the spans 65m.

Sector gates are difficult to install, and require careful maintenance and heating in winter conditions; their main advantages are ease of automation and absence of lifting gear, fast movement, accuracy of regulation, ease of passing of ice and debris, and low piers.



**Fig.: Drum gate (after Thomas, 1976)**

#### 4.1.5 Flap gates

Flap gates (bottom-hinged gates) are one of the simplest and most frequently used types of regulating gates used on dam crests and weirs, either on their own or in conjunction with plain lift gates. They were developed as a replacement for wooden flashboards, originally as a steel-edged girder flap, which was later replaced by a torsion-rigid pipe; further development was achieved by placing the pipe along the axis of the flap bearing, with the skin plate transmitting the water pressure to cantilevered ribs fixed to the pipe. Next in use were torsion-rigid gate bodies with curved downstream sides (fish-belly gates), with torsion-rigid structures using prism-shaped sections being the latest development in flap gates.

The heads for which flap gates (on their own) are being used may be as high as 6m and the span up to about 30m; for larger spans several flaps connected to each other, but with each actuated by its own hydraulic hoist, may be used.

Flap gates provide fine level regulation, easy flushing of debris and ice, and are cost effective and often environmentally more acceptable than other types of gates; they require protection against freezing and are particularly sensitive to aeration demand and vibration, which may be prevented, however, by the use of splitters at the edge of the flap.

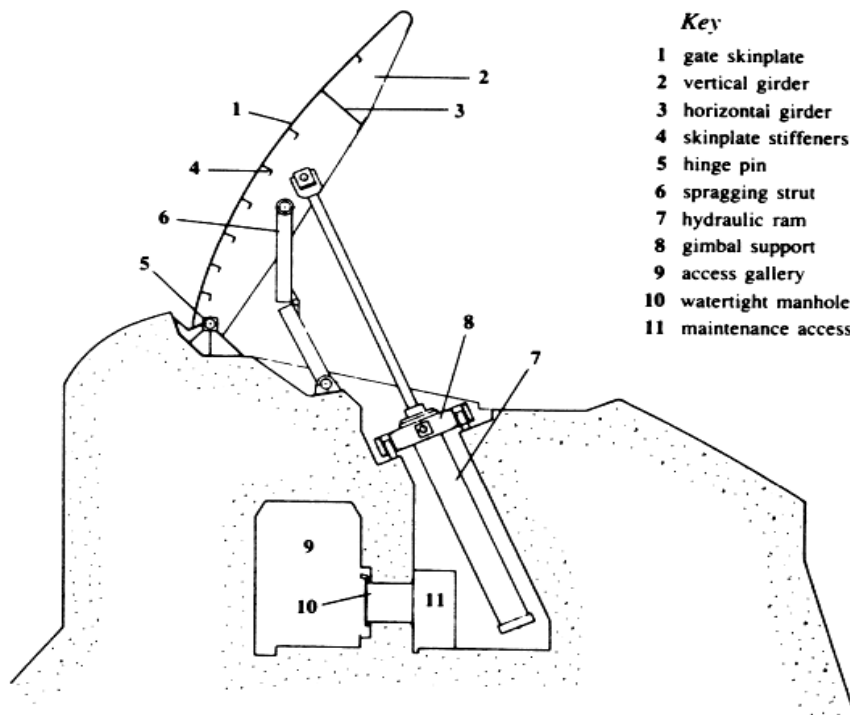


Fig.: Flap gate (after Thomas, 1976)

#### 4.1.6 Roller gates

A roller gate consists of a hollow steel cylinder, usually of a diameter somewhat smaller than the damming height; the difference is covered by a steel attachment, most frequently located at the bottom of the cylinder (in the closed position). The gate is operated by rolling it on an inclined track (see figure below). Because of the great stiffness of the gate, large spans (of up to 50m) may be used, but roller gates require substantial piers with large recesses.

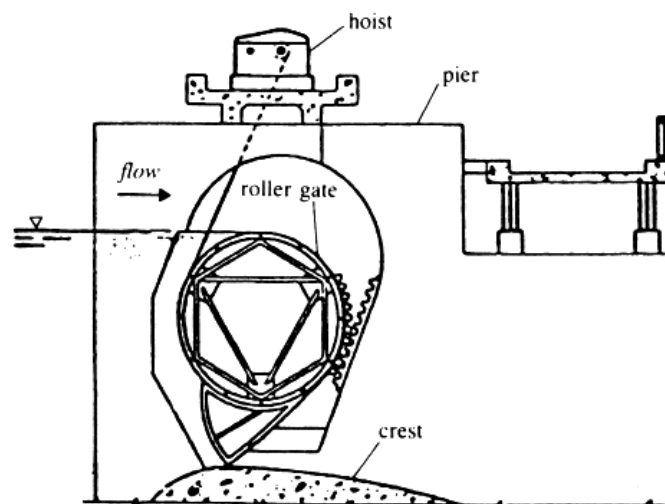


Fig.: Roller gate (after US Army Waterways Experimental Station, 1959)

## 4.2 High-head gates and valves

High-head (submerged) gates and valves transmit the load to the surrounding structure either directly through their support, e.g. plain (vertical lift), radial, or ring follower gates, or through the shell encasing the valve. The most common valves of the latter type are non-regulating disc valves (turning about a horizontal or vertical axis), cylindrical or ball valves and the regulating needle, tube, hollow-jet and Howell–Bunger (fixed-cone dispersion) valves.

### 4.2.1 High-pressure gates

*Plain (vertical lift)* gates are sliding, wheeled or moving on rollers or caterpillars. The load on them may reach 3000–4000kN/m with heads up to 200m and gate areas up to 100m<sup>2</sup>. For optimum conditions it is best to contract the pressure conduit upstream of the gate and to provide deflectors downstream to aid aeration as an anti-cavitation measure. The conduit face downstream of the gate slots should be protected against cavitation in the same way as for crest gates. Even better protection is provided for non-regulating lift gates by ring followers, which close the gate grooves and ensure a smooth passage for flow through the fully opened gate.

Radial gates are normally hinged downstream (see figure below), but are sometimes used in the reversed position with arms inside or, more frequently, outside the conduit at the end of which the regulating gate is installed.

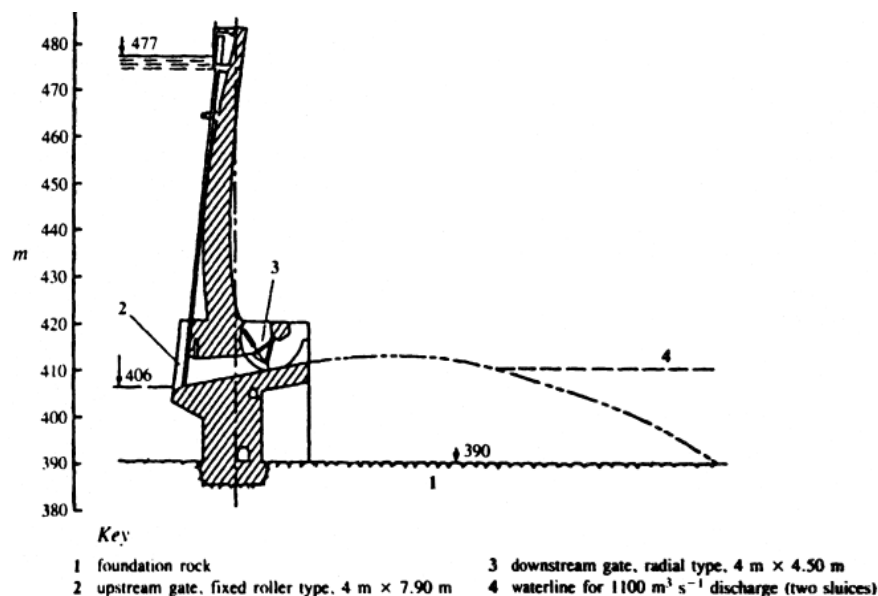


Fig.: Radial gate: Hinged downstream

A special feature of some high-head Tainter gates is the use of eccentric trunnions which permit a gap to be formed between the seals and the seal seats before opening the gate so that only moments caused by the gate weight and hinge friction have to be overcome.

Cylinder gates with all hydraulic forces counterbalanced are frequently used in tower type intakes; gate diameters up to 7m with maximum heads about 70m have been used.

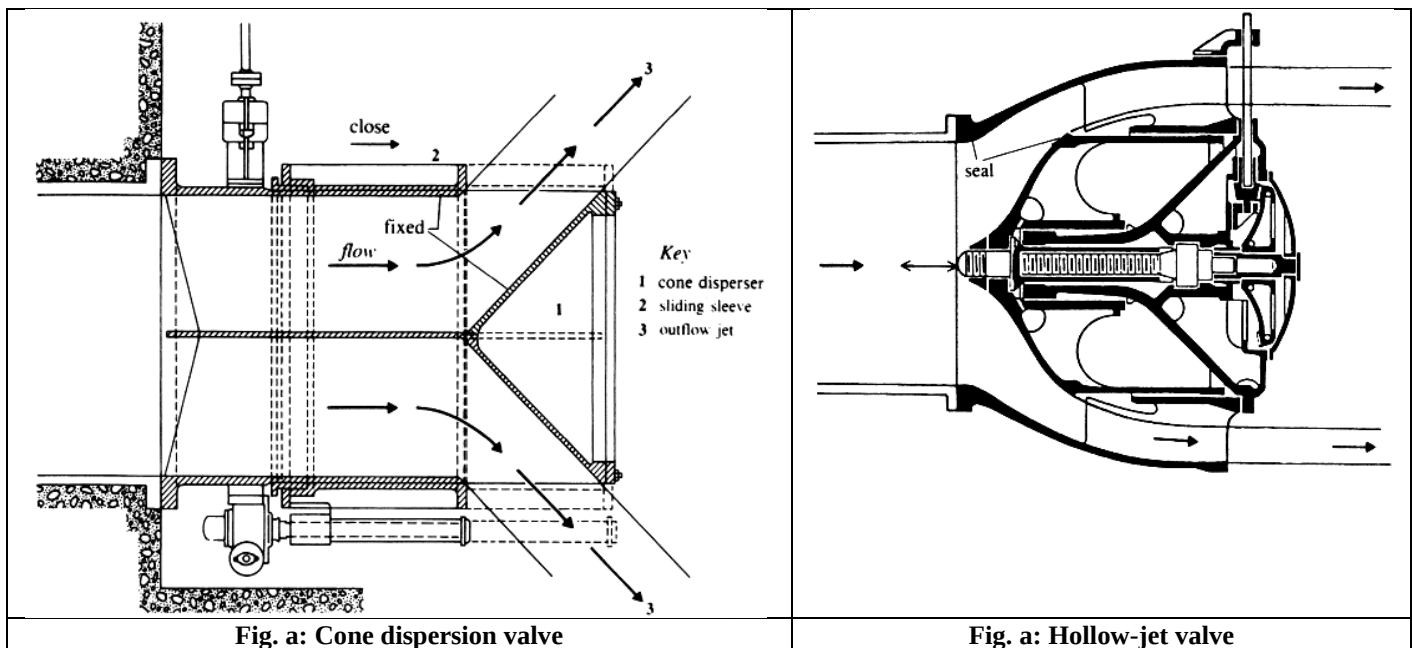
#### 4.2.2 High-head valves

The *cone dispersion* (Howell–Bunger) valve (below in the figure) is probably the most frequently used type of regulating valve installed at the end of outlets discharging into the atmosphere. It consists of a fixed 90° cone disperser, upstream of which is the opening covered by a sliding cylindrical sleeve. It has been used for heads of up to 250m, and when fully open its discharge coefficient is 0.85–0.9. The fully opened valve area is about 0.8 of the conduit area.

The Howell–Bunger valve is cheaper but less robust than the needle or hollow-jet valves, and should really be used in dam design only when discharging into the atmosphere. The fine spray associated with the operation of the valve may be undesirable, particularly in cold weather; sometimes, therefore, a fixed large hollow cylinder is placed at the end of the valve downstream of the cone, resulting in a ring jet valve. The discharge coefficient is reduced in this case to about 0.75–0.80.

The *needle* valve, or its variation the *tube* valve, has a bulb-shaped fixed steel jacket, with the valve closing against the casing in the downstream direction. When open, the valves produce solid circular jets and can also be used in submerged conditions. The valves may suffer from cavitation damage and produce unstable jets at small openings, and are expensive as they have to withstand full reservoir pressures.

Most of those disadvantages are overcome in the *hollow-jet* valve, which closes in the upstream direction (when closed the valve body is at atmospheric pressure); because of this the valve is, of course, not suitable for use in submerged conditions. The discharge coefficient of the fully open valve is about 0.7.



#### 4.3 Tidal barrage and surge protection gates

Due to the large spans and heads to be controlled the design of tidal barrages often built for flood control and surge protection purposes provides a special challenge in the design, installation and operation of gates.

The following text gives just some parameters of gates used in a few of these works.

The main storm-surge protection barrage of the Delta works on the Western Scheldt in the Netherlands completed in 1986 has 62 lift gates of heights 5.9–11.9m and 42m span. The storm surge Maeslant barrier in the navigation way (Nieuwe Waterweg) to Rotterdam, completed in 1997 and forming part of the same overall scheme has two floating horizontal radial/sector gates capable of closing the 360m wide channel. Each gate is 21.5m high, 8m wide and 210m long; the two horizontal arms of each gate are 220m long. The gates have a unique ball (pivot) joint 10m in diameter. Under normal conditions the gates are ‘parked’ in two dry docks (one on each bank). To operate the gates the docks are flooded, the gates are floated and swivelled into position in the waterway. When contact between the gates is established, they are flooded and sink to the bed closing the channel. After the flood danger has passed the whole process is reversed and the gates are towed back to their ‘parking’ position.

The protection of Venice against flooding is a great engineering challenge. To avoid piers in the navigation channels connecting the Venice Lagoon and the Adriatic it is proposed to use four barriers each 400 m wide, with bottom hinged buoyant 20m wide and 6–15m long flap gates recessed in closed position in reinforced concrete caissons with a hydraulic sediment ejector system (i.e. altogether 79 gates). An experimental fullscale module was built and operated 1988–92 (Bandarin, 1994).

#### **4.4 Hydrodynamic forces acting on gates**

In their closed position gates and valves are subjected to hydrostatic forces determined by standard procedures for forces acting on plane and curved surfaces. For radial or sector gates the vector of the resultant force must pass through the gate pivot and thus its moment will be zero; equally it can be converted into one tending to open or close the gate by placing the pivot above or below the centre of curvature of the skin plate.

The most important factors on which flow over and under the gates will depend are the geometry of the upstream and downstream water passage, the geometry and position of the gate and its accessories such as seals, supports, etc., the upstream head, whether the flow through the gate is free with atmospheric pressure downstream of it or submerged, the flow Froude and Reynolds numbers, the degree of turbulence of the incoming flow and aeration of the space downstream of the gates.

Generally the gate design is a difficult area partly because of the complexity of hydraulic conditions indicated above and partly because the design has to satisfy conflicting demands: vibration damping may conflict with keeping the forces for the gate operation to a minimum; the need to avoid vibrations may conflict with the optimum shape and strength required by the flow

conditions and loading; the optimum shape of the edges and seals may conflict with water tightness of the closed gate, etc.

- For gates with overflow the discharge coefficient can be expressed by the same equation as for an overfall spillway, i.e.

$$C_d = Q / \left( \frac{2}{3} (2g)^{1/2} b h^{3/2} \right)$$

- Where  $h$  is the head of the upstream water level above the crest (or top edge) of the gate.
  - Very broadly, the range of  $C_d$  is similar to that for overfall spillways, i.e.  $0.55 < C_d < 0.8$
- For gates with an **underflow** the discharge and flow field will depend primarily whether the outflow is *free* (modular) or *drowned*. For free flow the form of equation given below may be used:

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

- ( $a$  is the gate opening and  $H$  the upstream head), where the coefficients of discharge  $C_d$  and contraction  $C_c$  are related by:
- $$C_d = C_c / \left( 1 + C_c \frac{a}{H} \right)^{1/2}$$
- For a vertical gate the coefficient of contraction  $C_c$  varies slightly between 0.6 and 0.61
- For gates with planar and cylindrically shaped skin plates (Tainter gates) with free flow and a downstream horizontal apron and the outflow edge of the gate inclined at  $\theta$  to the flow ( $\theta < 90^\circ$  and varies according to the gate position). Toch (1955) suggested for  $C_c$  the equation

$$C_c = 1 - 0.75 \frac{\theta}{90} + 0.36 \left( \frac{\theta}{90} \right)^2$$

In *high-head* vertical lift gates, the hoist lifting force has to be dimensioned to overcome the gate weight, frictional resistance and, most importantly, the *downpull forces* resulting from the fact that during the gate operation the pressure along the bottom edge of the gate is reduced (to atmospheric or even smaller pressures), whereas the pressure acting on the top of the gate is practically the same as under static conditions (i.e. full reservoir pressure). This condition applies both to gates located within a conduit or at the upstream face of the dam or an intake (it should be noted that the seals for the gates at the intakes have to be on the downstream side and for gates in conduits are usually in the same position). The downpull (or uplift) forces acting on a gate with a given housing and seal geometry and the gate vibrations have to be analyzed at various operating conditions using theoretical considerations and experience from field trials, as well as model experiments, if necessary.

The lifting force of a gate with underflow (crest or submerged gate) will also be influenced by the geometry of the bottom edge and the seal. In order to avoid negative pressures and downpull there should be no separation of the flow until the downstream edge of the gate is reached.

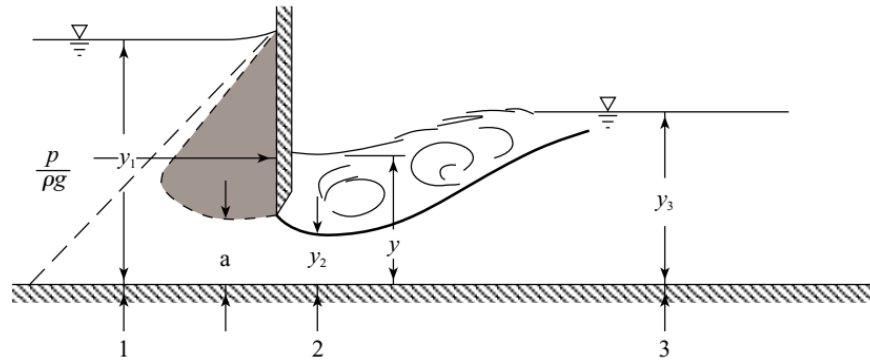


Fig.: Flow under a vertical gate

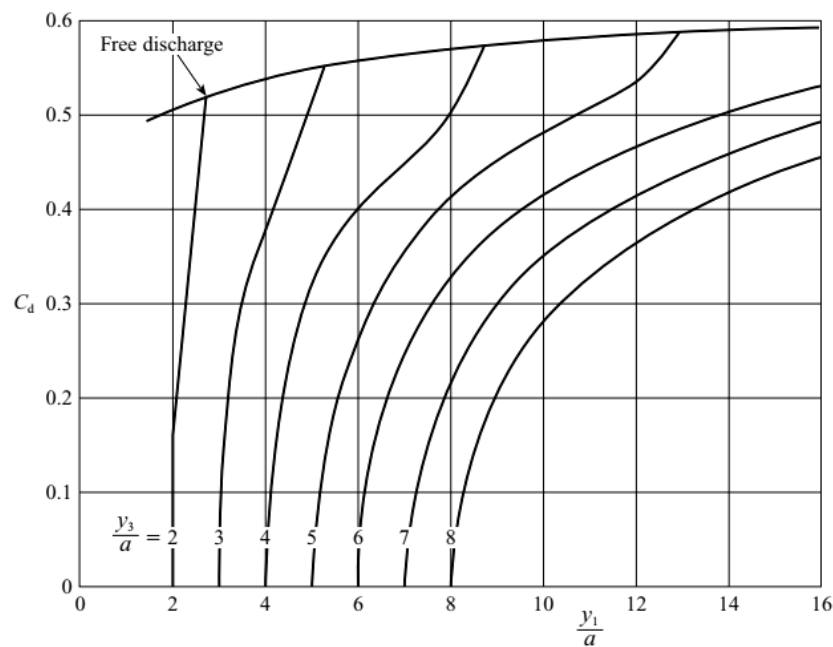


Fig.:

The design of *air vents* for gates is important as, together with the hydraulic parameters, they determine underneath the nappe for overfall head gates.

$$C_d = \phi\left(\frac{y_1}{a}, \frac{y_3}{a}\right)$$

the pressure downstream of the gate crest gates, or in the conduit for high

$$D = C \frac{H^{1.82} b^{0.5}}{p^{0.82}}$$

- Where D is an air vent diameter, and for a flow over the gate only  $C=0.004$  for all parameters in meters ( $p$  is the negative head underneath the overfall jet)