Outlet Work:

The major portion of the storage volume in most reservoirs is below the spillway crest. Outlet works must be provided in order that water can be drawn from the reservoir as needed. This water may be discharged into the channel below the dam or may be transported in pipes or canals to some distant point.

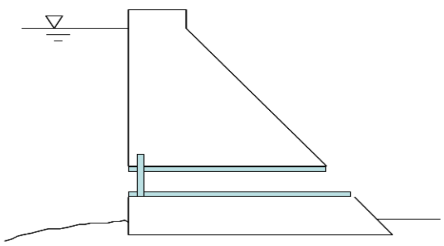
Functions of outlet works:

1. ***Flood Control***: Flood control outlets are designed for relatively large capacities where close regulation of flow is less important than are other requirements. When large discharges must he released under high heads, the design of gates, water passages, and energy dissipater should be carefully developed. Multilevel release provisions are often necessary for water quality purposes.
2. ***Navigation:*** Reservoirs that store water for subsequent release to downstream navigation usually discharge at lower capacity than flood control reservoirs, but the need for close regulation of the flow is more important. The navigation season often coincides with the season of low rainfall, and close regulation aids in the conservation of water. Outlet works that control discharges for navigation purposes are required to operate continuously over long periods of time. The designer should consider the greater operation and maintenance problems involved in continuous operation.
3. ***Irrigation***: The gates or valves for controlling irrigation flows are often basically different from those used for flood control due to the necessity for close regulation and conservation of water in arid regions. Irrigation discharge facilities are normally much smaller in size than flood regulation outlets.
4. ***Water Supply:*** Municipal water supply intakes are sometimes provided in dams built primarily for other purposes. Such problems as future water supply requirements and peak demands for a municipality or industry should be determined in cooperation with engineers representing local interests.
5. ***Power:*** For generation of hydropower, intake structures direct water from the reservoir into the penstock or power conduit. Gates or valves are used to shut off the flow of water to permit emergency unit shut-down or turbine and penstock maintenance. Racks or screens prevent trash and debris from entering the turbine units.
6. ***Low-Flow Requirements***: Continuous low-flow releases are required at some dams to satisfy environmental objectives, water supply, downstream water rights, etc. To meet these requirements multilevel intakes, skimmer weirs, or other provisions must be incorporated separately or in combination with other functions of the outlet works facility.
7. ***Diversion***: Flood control outlets may be used for total or partial diversion of the stream from its natural channel during construction of the dam. Such use is especially adaptable for earth dams.
8. ***Drawdown***: Requirements for low-level discharge facilities for drawdown of impoundments may also provide flexibility in future project operation for anticipated needs, such as major repairs of the structure, environmental controls, or changes in reservoir regulation.

1. Sluiceways:

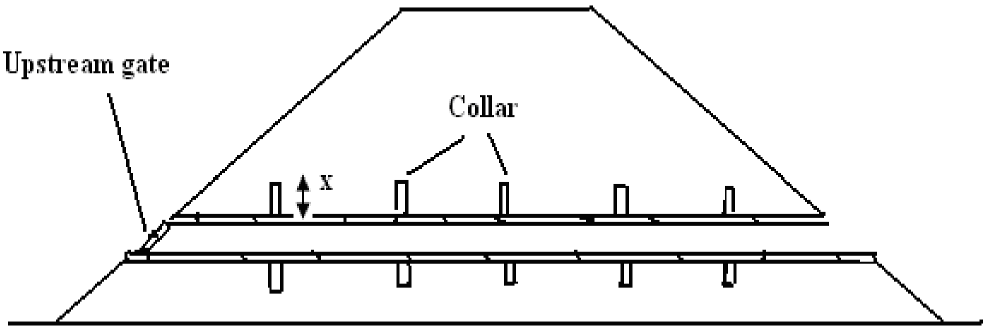
It is a pipe or tunnel that passes through a dam or the hillside at one end of the dam and discharges into the stream below.

Sluiceway for ***concrete dams*** generally passes through the dam.



Sluiceway for a gravity concrete dam

Sluiceways for earth or rock dams are preferably placed outside the limits of the embankment. If a sluiceway must pass through an earth dam, projecting collars should be provided to reduce seepage along outside of the conduit.



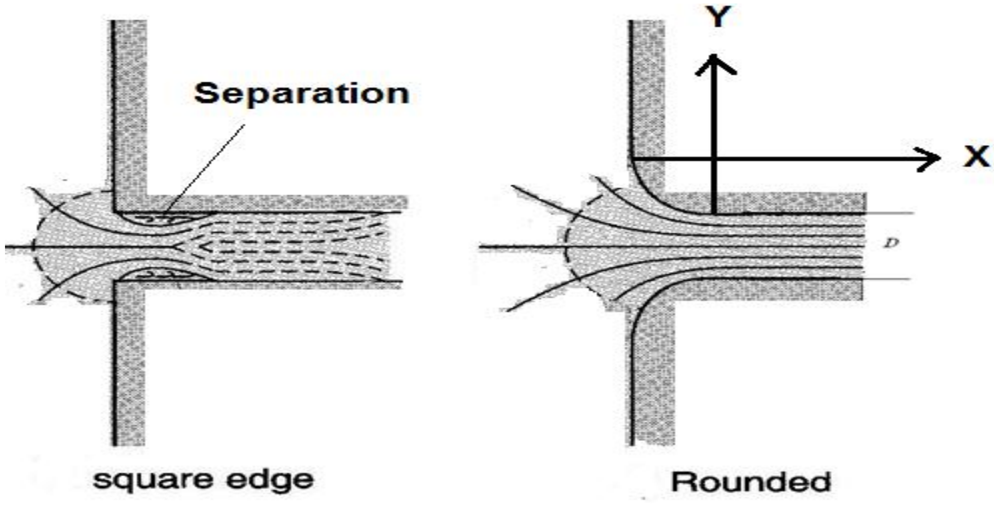
For design purpose:

$$2Nx > 0.25L$$

Where N is the number of collars, and x is the projection of the collars.

Example: determine the height of collars for an earth dam if the length of seepage path is 100m, (assume the available distance between two adjacent collars is 10m?
Sol:
N=(100/10)-1=9
X=(0.25*100)/(2*9)
=1.38m ≈ 1.5m
2(9*1.5) =27> (0.25*100) O.K.

- The outlets of most dams consist of one or more sluiceways with their inlets at about minimum reservoir level.
- Large dams may have sluiceways at various levels.
- In most cases a single large-capacity sluiceway may be structurally unsatisfactory, why?
- Sluiceways may be circular or rectangular,
- The interior should be smooth and without projections or cavities which might induce separation of flow boundary of the conduit and cause negative pressure and cavitation's.



The equation for circular conduits is:

$$4x^2 + 44.4y^2 = D^2$$

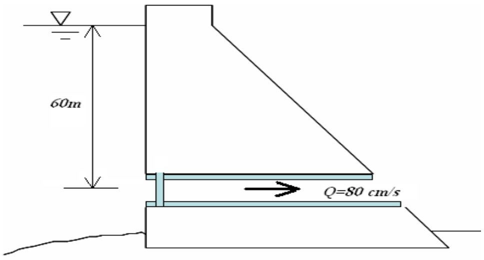
For rectangular conduits is:

$$x^2 + 10.4y^2 = D^2$$

D is the diameter of a circular conduit or the width or height of a rectangular conduit, depending on whether the side or top and bottom curves are being considered.

<u>Example:</u> the sluiceway for a dam discharge 80m³/s, if the coefficient of discharge from sluiceway is 0.9, and the head above the entrance is 60m, determine the profile of the entrance if:

1. **Circular conduit was used,**
2. **Rectangular conduit was used.**



<u>Sol:</u>

$$Q = C_d A(2gh)^{0.5} \Rightarrow 80 = 0.9 * A * (19.62 * 60)$$

$$A = \frac{80}{1059.48} = 0.076m^2$$

1- Circular conduit:

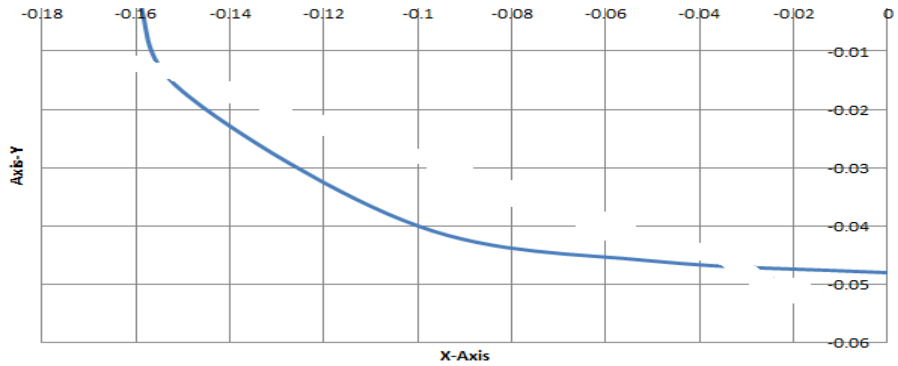
$$0.076 = \frac{d^2}{4}\pi \Rightarrow d = 0.31m \approx 0.32m$$

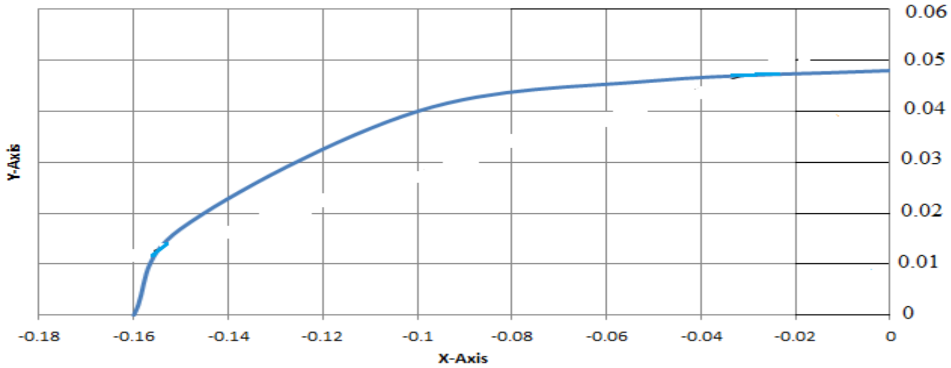
Use the following equation

$$4x^2 + 44.4y^2 = D^2$$

X	Y
0	**0.048**
0.05	**0.046**
0.10	**0.040**
0.15	**0.017**
0.16	**0**

X	Y
0	**-0.048**
-0.05	**-0.046**
-0.10	**-0.040**
-0.15	**-0.017**
-0.16	**0**





Hydraulics of Outlet Works:

- The outlet works of a dam must be designed to discharge water at rates dictated by the purposes of the dam,
- Head losses encountered in outlet conduits include those caused by the trash rack, conduit entrance, friction, gates, valves, and bends,
- Trash rack losses are low, approximately as indicated in table,

Velocity through rack (m/s)	Head losses (m)
0.2	0.01
0.4	0.05
0.5	0.09
0.6	0.13

- Head losses at entrance to a conduit depends on the shape of the entrance and varies from $0.04h_v$ for a bell mouth entrance to $0.5h_v$ for a square-edged opining, where hv is the velocity head in the conduit just downstream from the entrance,
- Head losses caused by conduit friction may be calculated by standard pipe-friction formulas,

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

- The discharge formula is,

$$Q = C_d A(2gh)^{0.5}$$

h=the total head at the valve.

Exercise: Find the discharge through a valve whose outlet diameter is 2m if the pressure just upstream of the valve is 200 kN/m^2 and Cd=0.68?

Energy Dissipation below Spillways:

Water flowing over a spillway has a very high kinetic energy because of the conversion of the entire potential energy to the kinetic energy. If the water flowing with such a high velocity is discharged directly into the channel downstream, serious scour of the channel bed may occur. If the scour is not properly controlled, it may extend backward and may endanger the spillway and the dam. In order to protect the channel bed against scour, the kinetic energy of the water should be dissipated before it is discharged into the *d/s* channel. The energy-dissipating devices can be broadly classified into two types:

1. Devices using a hydraulic jump for the dissipation of energy.
2. Devices using a bucket for the dissipation of energy.

The choice of the energy-dissipating device at a particular spillway is governed by the tail water depth and the characteristics of the hydraulic jump, if formed, at the toe. If the tail water depth at the site is not approximately equal to that required for a perfect hydraulic jump, a bucket-type energy dissipating device is usually provided. The characteristics of the hydraulic jump are discussed in the following section. The sequent depth (conjugate depth or post-jump depth) y_2 is determined for different values of the discharge, and a jump height curve (*JHC*) is plotted between the conjugate depth y_2 as ordinate and the discharge (Q) as abscissa. The tail water rating curve (*TWRC*) at the spillway site is determined by stream gauging, (Q) as abscissa. As discussed later, the correct choice of the energy-dissipating device is made after comparing the relative positions of the jump height curve (*JHC*) and the tail water rating curve (*TWRC*). For the design of spillways, the discharge per unit length (q) is usually taken as abscissa instead of Q. Different types of stilling basins have been developed which are quite effective for the formation of stable hydraulic jumps and for confining the hydraulic jump. Stilling basins are commonly used for spillways and other hydraulic structures, such as weir and barrages. In a stilling basin, chute blocks, basin blocks (baffle blocks) and an end sill are usually provided. Chute blocks are triangular blocks installed at the upstream end of the basin. An end sill is constructed at the downstream end of the basin. It may be a solid sill or a dentate sill. Baffle blocks are installed on the basin floor between the chute blocks and the end sill. These are also known as baffle blocks or baffle piers.

Characteristics of a Hydraulic Jump:

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

Types of jumps

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

$$F_1 = V_1/\sqrt{g y_1}$$

Where V_1 is the mean velocity of flow before the hydraulic jump, g is the acceleration due to gravity and y_1 is the pre-jump depth (or the initial depth of flow).

For the formation of a hydraulic jump, the initial Froude number F_1 should be greater than unity. Different types of hydraulic jump are as follows:

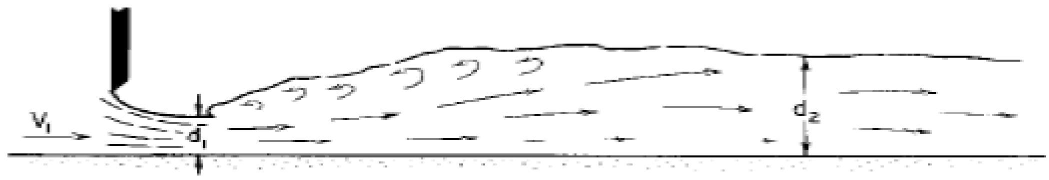
1. Undular Jump An undular jump is formed when F_1 = 1.0 to 1.70. In an undular jump, the water surface shows some undulation. The energy dissipation is about 5%.

2. Weak Jump When F_1 = 1.70 to 2.50, a weak hydraulic jump occurs. In this case, a series of small rollers develops on the surface of the jump, but the downstream water surface remains quite smooth. The velocity is uniform throughout. The energy dissipation is about 20%.

3. Oscillating Jump An oscillating hydraulic jump occurs when F_1 = 2.50 to 4.50. There is an oscillating jet entering the jump bottom to surface and back again without any periodicity. The energy dissipation is between 20 to 40 %.

4. Steady Jump A steady jump occurs when F_1 = 4.50 to 9.0. The jump is quite stable and balanced. This jump is not much sensitive to variations in the tail water depth. The steady jump has very good performance, and most of the hydraulic structures utilize this type of jump for the dissipation of energy. The energy dissipation is between 45 to 70 %.

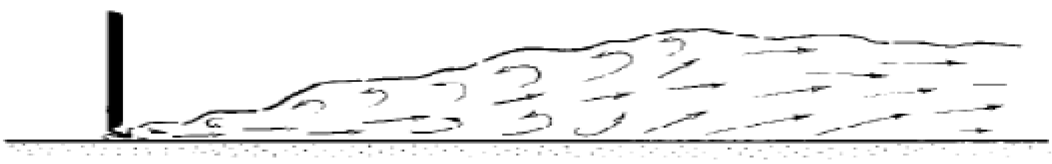
5. Strong Jump A strong jump occurs when $F_1 > 9.0$. The jump action is quite rough but effective. It causes a rough water surface with strong surface waves downstream. The energy dissipation is between 70 to 85 %. Because of rough action, a strong jump is avoided in spillways, as far as possible.



F₁ BETWEEN 1.7 and 2.5
FORM A-PREJUMP STAGE

F₁ BETWEEN 2.5 and 4.5
FORM B-TRANSITION STAGE

F₁ BETWEEN 4.5 and 9.0
FORM C-RANGE OF WELL-BALANCED JUMPS

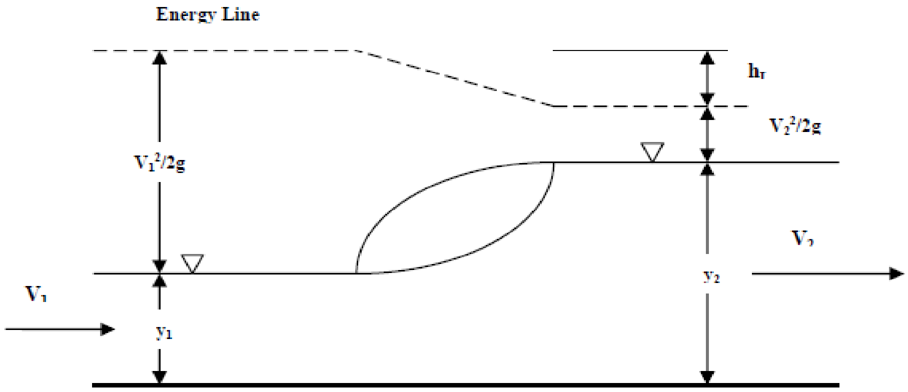


F₁ GREATER THAN 9.0
FORM D-EFFECTIVE JUMP BUT ROUGH
SURFACE DOWNSTREAM

Mathematical Derivation of Hydraulic Jump

In the mathematical derivation of hydraulic jump, the following assumptions are made,

- Rectangular channel with horizontal bottom slope,
- Before and after the hydraulic jump, velocity distributions are uniform and the pressure distribution over the cross sections are hydrostatic,
- Friction losses are neglected.



Hydraulic Jump

Momentum equation will be applied to the control volume taken at the hydraulic jump section for a unit width perpendicular to the control volume,

$$\frac{\gamma y_1^2}{2} - \frac{\gamma y_2^2}{2} = \rho Q V_2 - \rho Q V_1$$

$$q = y_1 V_1 = y_2 V_2 \Rightarrow V_1 = \frac{q}{y_1}, V_2 = \frac{q}{y_2}, \gamma = \rho g$$

$$\frac{\rho g}{2}(y_1^2 - y_2^2) = \rho(\frac{q^2}{y_2^2} y_2 - \frac{q^2}{y_1^2} y_1)$$

$$\frac{g}{2}(y_1 - y_2)(y_1 + y_2) = q^2(\frac{1}{y_2} - \frac{1}{y_1}) = \frac{q^2(y_1 - y_2)}{y_1 y_2}$$

$$y_1 y_2 (y_1 + y_2) = \frac{2q^2}{g} = \frac{2y_1^2 V_1^2}{g}$$

$$y_1^2 y_2 (1 + \frac{y_2}{y_1}) = \frac{2y_1^2 V_1^2}{g}$$

Multiplying both side of the above equation with ($1/y_1^3$) yields,

$$[y_1^2 y_2 (1 + \frac{y_2}{y_1}) = \frac{2y_1^2 V_1^2}{g}] * \frac{1}{y_1^3}$$

$$\frac{y_2}{y_1}(1 + \frac{y_2}{y_1}) = 2\frac{V_1^2}{g y_1}$$

Since for rectangular channels,

$$Fr = \frac{V}{\sqrt{gy}}$$

$$(\frac{y_2}{y_1})^2 + \frac{y_2}{y_1} - 2Fr_1^2 = 0$$

Solution of this equation and taking the positive sign of the square root gives,

$$\frac{y_2}{y_1} = \frac{1}{2}(\sqrt{1 + 8Fr_1^2} - 1)$$

The ratio of flow depths after and before the hydraulic jump (y_2/y_1) is a function of the Froude number of the subcritical flow before hydraulic jump.

Hydraulic Jump as an Energy Dissipater

If we write the difference of the specific energies before after the hydraulic jump,

$$\Delta E = E_1 - E_2 = (y_1 + \frac{V_1^2}{2g}) - (y_2 + \frac{V_2^2}{2g})$$

$$\Delta E = (y_1 - y_2) + (\frac{V_1^2}{2g} - \frac{V_2^2}{2g})$$

Since,

$$q = Vy \Rightarrow V_1 = \frac{q}{y_1}, V_2 = \frac{q}{y_2}$$

$$\Delta E = (y_1 - y_2) + \frac{q^2}{2g}(\frac{1}{y_1^2} - \frac{1}{y_2^2})$$

It has been derived that,

$$y_1 y_2 (y_1 + y_2) = \frac{2q^2}{g}$$

$$\frac{q^2}{2g} = \frac{1}{4} y_1 y_2 (y_1 + y_2)$$

Substituting above equation in equation of ΔE,

$$\Delta E = (y_1 - y_2) + \frac{1}{4} y_1 y_2 (y_1 + y_2)(\frac{y_2^2 - y_1^2}{y_1^2 y_2^2})$$

$$\Delta E = (y_1 - y_2) + \frac{1}{4} \frac{(y_2 - y_1)(y_1 + y_2)^2}{y_1 y_2}$$

$$\Delta E = \frac{4y_1 y_2 (y_1 - y_2) + (y_2 - y_1)(y_1 + y_2)^2}{4y_1 y_2}$$

$$\Delta E = \frac{(y_2 - y_1)[-4y_1 y_2 + (y_1 + y_2)^2]}{4y_1 y_2}$$

$$\Delta E = \frac{(y_2 - y_1)(y_2 - y_1)^2}{4y_1 y_2}$$

The analytical equation of the energy dissipated with the hydraulic jump is,

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

The power lost by hydraulic jump can be calculated by,

$$P = \gamma_w Q \Delta E$$

Where:
γ_w = Specific weight of water = 9.81 kN/m^3
Q = Discharge (m^3/sec)
ΔE = Energy dissipated as head (m)
P=Power dissipated (kW)

Length of Hydraulic Jump:

Some empirical equations were given to calculate the length of hydraulic jump as,

L = 5.2y_2	Safranez equation
L = 5($y_2 - y_1$)	Bakhmetef equation
L = 6($y_2 - y_1$)	Smetana equation
L = 5.6y_2	Page equation

Example: If the Froude number at the drop of a hydraulic jump pool is 6 and the water depth is 0.50 m, find out the length of the hydraulic jump. Calculate the power dissipated with the hydraulic jump if the discharge on the spillway is 1600 m³/sec.

Sol:

$$\frac{y_2}{y_1}=\frac{1}{2}(\sqrt{1+8Fr_1^2}-1)$$

$$\frac{y_2}{y_1}=\frac{1}{2}(\sqrt{1+8*6^2}-1)=8$$

$$y_2=y_1*8\Rightarrow y_2=0.5*8=4$$

The length of hydraulic jump by different equations,

L = 5.2y_2 =5.2*4=20.8m Safranez equation
L = 5(y_2 – y_1) =5(4-0.5)=17.5m Bakhmetef equation
L = 6(y_2 – y_1)=6(4-0.5)=21m Smetana equation
L = 5.6y_2 =5.6*4=22.4m Page equation

It is preferred to be on the safe side with the hydraulic structures. Therefore, the longest result will be chosen. The length of the hydraulic jump will be taken as L = 22.4 m for design purposes.

Energy dissipated as head,

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

$$\Delta E=\frac{(4-0.5)^3}{4*4*0.5}=5.36m$$

The power dissipated with the hydraulic jump,

$$P=\gamma_w Q\Delta E$$

$$P=9.81*1600*5.36$$

$$P=84131kW$$

Jump High Curve (JHC)

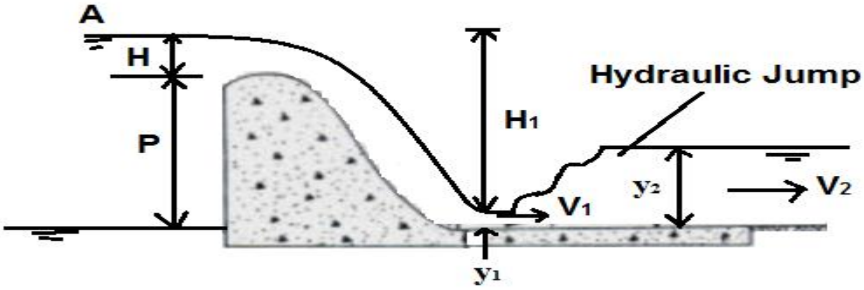
A hydraulic jump will occur in a rectangular open channel if the following equation between the initial depth y_1 and the sequent depth (post jump depth) y_1 is satisfied (See any text of Fluid Mechanics).

$$y_2=\frac{y_1}{2}[\sqrt{1+8Fr_1^2}-1................(1)$$

Where:

$$Fr_1=\frac{V_1}{\sqrt{gy_1}}$$

The mean velocity V_1 of the incoming flow for an ogee-shaped spillway can be determined by applying the Bernoulli equation to points A and 1 (Fig. below). Neglecting losses and the velocity of approach,



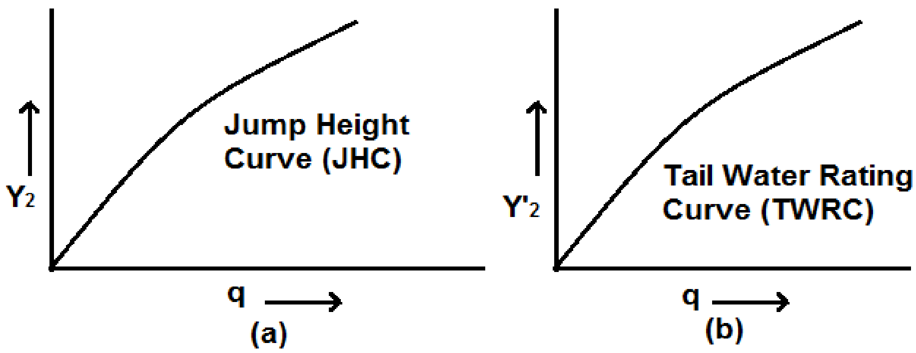
Characteristics of Hydraulic jump

$$P + H = y_1 + \frac{V_1^2}{2g} \text{.............(2)}$$

The mean velocity of flow V_1 at the toe of spillway is equal to (q/y_1). Therefore,

$$P + H = y_1 + \frac{(q/y_1)^2}{2g} \text{.............(3)}$$

By substituting the values of P,H, and q, the value of y_1 can be found from above equation. Thus the value of y_1 is determined for a given discharge intensity q over the spillway. The corresponding value of the sequent depth y_2 can be determined from Eq. b. Likewise, for different values of the discharge intensity; the values of the sequent depth y_2 can be computed. A plot is then made between the discharge intensity q as the abscissa and the corresponding value of the sequent depth y_2 as ordinate [Fig. below (a)]. The curve is known as the jump height curve (*JHC*) or jump rating curve (*JRC*).



Jump Rating Curve

Tail water rating curve

The tail water rating curve (TWRC) gives the relation between the tail water depth y_2' (i.e. the actual water depth in the river on the downstream) as ordinate and the discharge intensity q as abscissa [Fig. 2 (b)]. The actual tail water depth corresponding to any discharge intensity q depends upon the hydraulic characteristics of the river downstream. The values of y_2' corresponding to different values of q are obtained by actual stream gauging. If there is a suitable control somewhere downstream of the spillway where the depth of water and discharge can be accurately measured, the tail water depth y_2' at the spillway can also be determined by backwater computation. While plotting the tail-water rating curve, an allowance for channel retrogression, which is likely to occur, must be made.

Location of a Hydraulic Jump:

For a given discharge intensity (q), the sequent depth y_2 and the tail water depth y_2' are fixed. The location of hydraulic jump will depend upon the relative magnitudes of y_2 and y_2', and hence on the JHC and TWRC. There are five cases, depending upon the relative positions of JHC and TWRC, as discussed below.

Case-1 JHC and TWRC coincide throughout In this case; the JHC and TWRC curves coincide for all discharges [Fig. 3 (a)]. As the tail water depth y_2' is exactly equal to the sequent depth y_2 required for the formation of hydraulic jump, a perfect jump is formed just at the toe of the spillway as shown in Fig.1. However, this case indicates a highly idealized condition, which rarely occurs in practice.

Case-2 TWRC always lower than JHC In this case, the tail water rating curve (TWRC) is below the jump height curve JHC for all discharges [FIG. 3 (b)]. Such a condition occurs when the tail water is carried away quickly due to a rapid or a fall somewhere on the downstream of the spillway. In this case, the jump will be located at a point on the downstream of the toe of spillway. The high velocity jet would sweep down the toe and scour the river bed. Therefore, severe erosion may occur in the portion of the river between the spillway and the section where the hydraulic jump is formed.

Case-3 TWRC always higher than JHC In this case, the tail water rating curve is above the jump height curve for all discharges [Fig.3 (c)]. This condition usually occurs when the river cross-section on the downstream of the spillway is narrow and therefore the tail water backs up. The hydraulic jump in this case is located upstream of the toe on the spillway face. The hydraulic jump is drowned or submerged, and the high velocity jet dives under the tail water. The energy dissipation in a drowned hydraulic jump is not good.

Case-4 TWRC lower than JHC at low discharges, but higher at high discharges

In this case, the tail water rating curve is lower than the jump height curve at low discharges, but it becomes higher at a particular discharge and then remains higher than the jump height curve [Fig.3 (d)].

It is a combination of cases 2 and 3. The hydraulic jump is formed further downstream of the toe at low discharge, as in the case 2; but at higher discharges, it is drowned, as in the case 3.

Case-5 TWRC higher than JHC at low discharges, but lower at high discharges.

It is also combination of cases 3 and 2. However, in this case, at low discharges, the jump is drowned; whereas at high discharges, it is formed further downstream of the toe [Fig. 3 (e)].

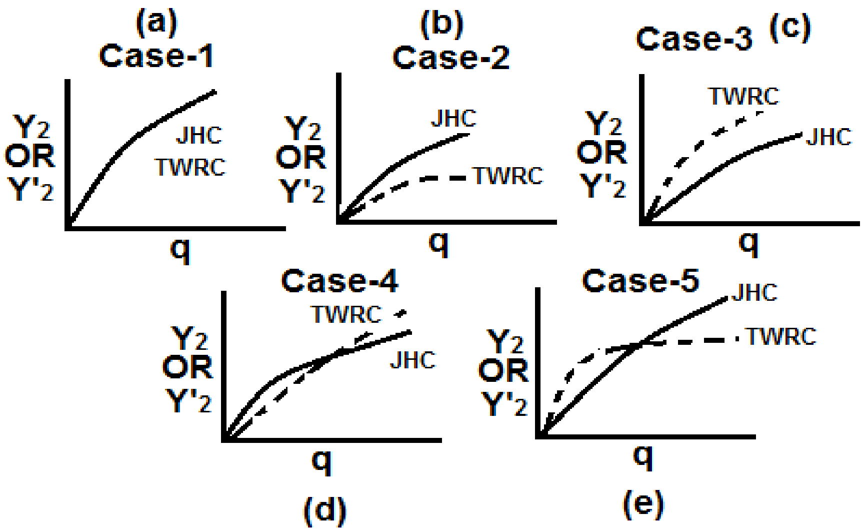


Fig.3

Measure Adopted For Dissipation of Energy:

Various measures are adopted at or near the toe of the spillway so that a perfect jump is formed for the dissipation of energy. The measures adopted will depend upon the relative positions of the tail water rating curve (TWRC) and the jump height curve (JHC). Measures are discussed separately for all the five cases discussed in the preceding section.

Case-1 In this case, the tail water rating curve and jump height curve concide for all discharges. There is no need of any special measure for the formation of hydraulic jump, as a perfect jump will always form at the toe. A horizontal apron is however provided on the downstream of the toe for the protection of the river bed (Fig.4). The length of a horizontal apron is taken equal to the maximum length of the hydraulic jump. Sometimes, baffle blocks are also constructed on the horizontal apron for dissipation of energy. However, if the baffle blocks are placed too near the toe, they may be subjected to cavitation and abrasion.

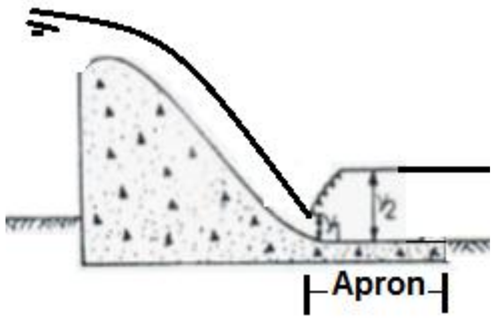


Fig.4

If may be noted that the case-1 rarely occurs in practice. However, by suitably choosing the length of the spillway, the TWRC and JHC may be made to coincide to some extent.
Case –2 As the tail water rating curve is lower than the hydraulic jump curve, the hydraulic jump forms at a certain section downstream of the toe. The following measures are adopted:

- A depressed horizontal apron is formed by excavating the river bed on the downstream of the toe of the spillway to increase the tail water depth [Fig.5 (a)]. The length and depth of the apron should be such that, for all discharges, the jump is confined to the apron. Sometimes, the depressed apron is made sloping instead of horizontal.

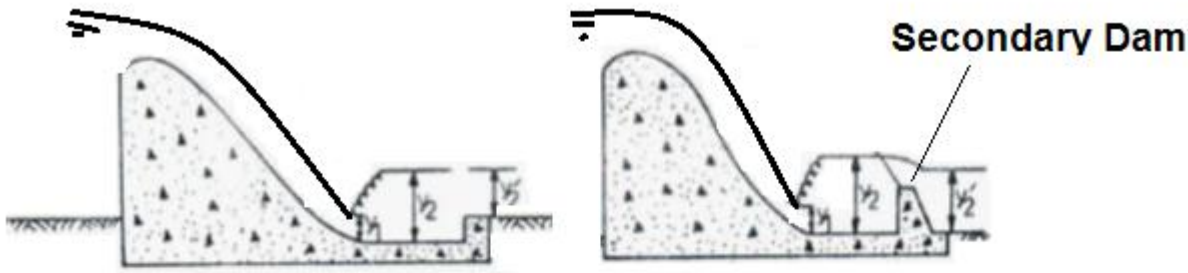


Fig.5

- (ii) A low secondary weir (or dam) is constructed downstream of toe to raise the tail water [Fig.5(b)].
- A stilling basin is formed on the downstream of toe and a sill or baffle wall is provided at the end of the stilling basin. The length and depth of the stilling basin should be sufficient to contain the hydraulic jump for all discharges.
- If the river bed consists of solid rock, a ski jump bucket can be provided which throws the water up so that it strikes the bed at a safe distance away from the toe.

Case-3 In this case, the tail water rating curve is higher than the jump height curve and the hydraulic jump is drowned, the following measures are adopted.

- A sloping apron is constructed above the river bed level extending from the spillway surface to the toe [Fig.6 (a)]. The sloping apron raises the level of the point where the hydraulic jump is formed. The slope of the apron should be such that a perfect jump will form somewhere on the sloping apron for all discharges. A large quantity of concrete is however required for the construction of the sloping apron.

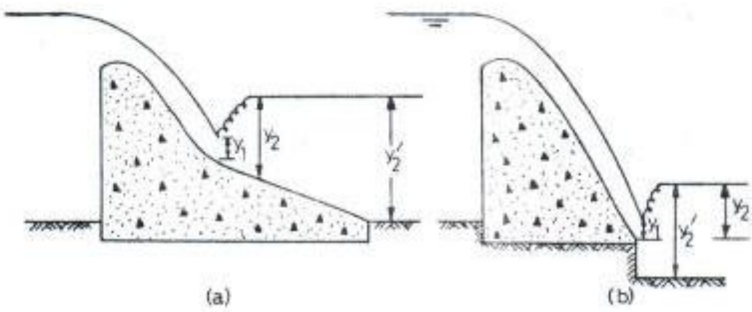


Fig.6

- The river bed may be excavated to provide a drop in the river bed to lower the tail water [Fig.6(b)].
- A roller bucket is provided near the toe, which forms rollers for the dissipation energy.

Case-4 In this case, the tail water rating curve is lower than the jump height curve at low discharges but higher at high discharges. Thus at low discharges, the hydraulic jump is shifted to a downstream point; but for high discharge, it is shifted upstream of the toe and the jump is drowned.

The following measures are adopted.

- A sloping apron is provided which lies partly above and partly below the river bed level so that a perfect jump will form in the lower portion of the apron at low discharges and in the higher portion of the apron at high discharges (Fig.7).

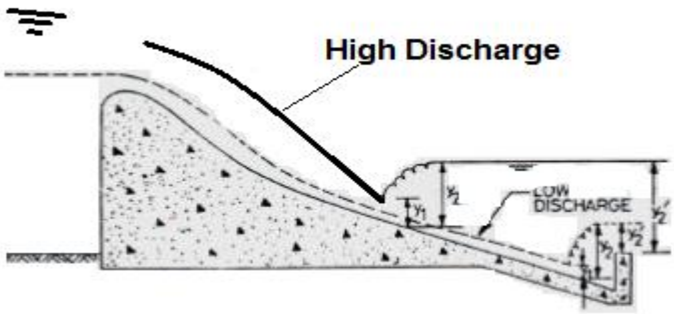


Fig.7

- A low secondary dam (or a sill) with a stilling basin is provided downstream of the toe to raise the tail water level at low discharges. This arrangement is combined with a sloping apron at a higher level for developing a jump at high discharges (Fig.8). It is found in practice that the low secondary dam has negligible effect at high discharges.
- If the velocity is not greater than 15 m/s, baffle blocks or dentated sills may be constructed to break up the jet and raise tail water level at low discharges to assist jump formation. At high discharges, the high velocity jet dives under the tail water and breaks up and the energy is dissipated in internal turbulence, though jump is not formed.

Case-5 In this case, the tail water depth is higher than jump height curve at low discharges, but lower at higher discharges. The case is similar to case 4 but the range of discharge is different. The following measures are usually adopted.

- A sloping apron is provided which is partly above the river bed level and partly below the river bed level, as in Fig. 2.7. In this case, the jump will form in the upper portion of the apron at low discharges, and in the lower portion, at high discharges.
- A low secondary dam (or a sill) with a stilling basin is provided to increase the depth at high discharges as in Fig.8. However, at low discharges, this arrangement will further increase the tail water depth, which is already quite high. Therefore, at low discharges, the jump will be more drowned and consequently, there will be less dissipation of energy. If this arrangement is not likely to cause much scour, if may be acceptable.

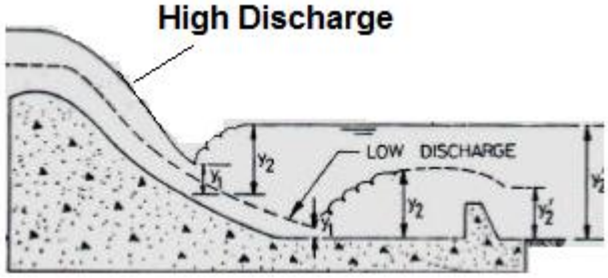


Fig.8

Stilling Basins:

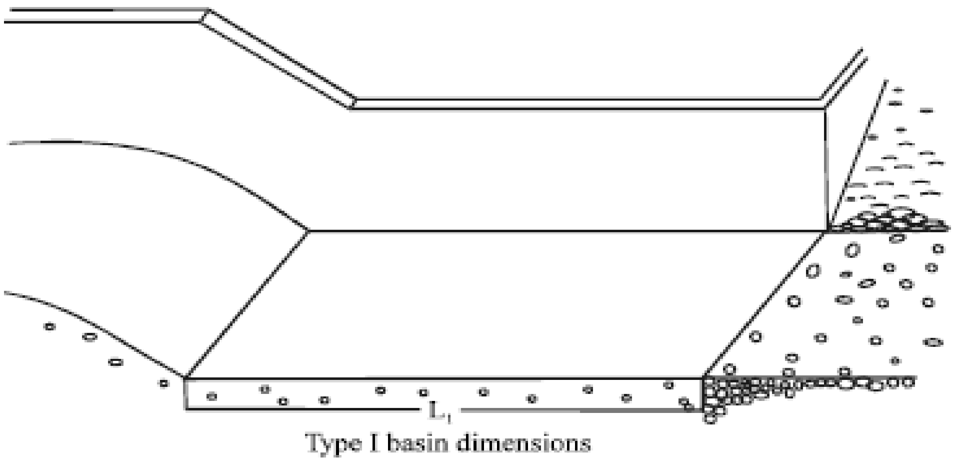
A stilling basin is a basin-like structure in which all or a part of the energy is dissipated. The positioning of a hydraulic jump on an unobstructed horizontal surface is very sensitive to the close match of sequent depths. If the downstream depth matches the sequent depth y_2, the hydraulic jump will occur as desired on the apron. If the downstream depth is less than y_2, $y_3<y_2$, the jump will occur downstream from the apron (a swept-out jump), and the river will become exposed to high scouring velocities. If the downstream depth is greater than y_2, $y_3>y_2$, the jump will be submerged. Although a submerged jump is preferable to a swept-out jump, much of the initial kinetic energy remains in the form of a submerged jet, which alone can result in considerable scour. A carefully designed stilling basin will not only improve the dissipation characteristics of a hydraulic jump, it will shorten its length and stabilize the position of the jump so that it is not sensitive to fluctuations in tailwater levels. In a stilling basin, the kinetic energy causes turbulence and it is ultimately lost as heat energy. The stilling basins commonly used for spillways are of the hydraulic jump type, in which dissipation of energy is accomplished by a hydraulic jump. A hydraulic jump can be stabilized in stilling basin by using appurtenances (or accessories such as chute blocks, basin blocks and end sill.

Types of Stilling Basin:

Because stilling basin block arrangements are difficult to design analytically, their design must be based on experimental methods. Standard designs have been developed through both observations of existing installations and a systematic series of model studies. Four types of stilling basins are developed by the U.S. Bureau of Reclamation are explained in the following:

Type I ($1.7<Fr_1<2.5$)

It is a rectangular stilling basin with a horizontal bottom, no chutes, no baffles or sills, which include a classical hydraulic jump. Because of high costs that come from the basin length is large as well as the hydraulic jump is sensitive to downstream level variation and effects on safety, it is not recommended.



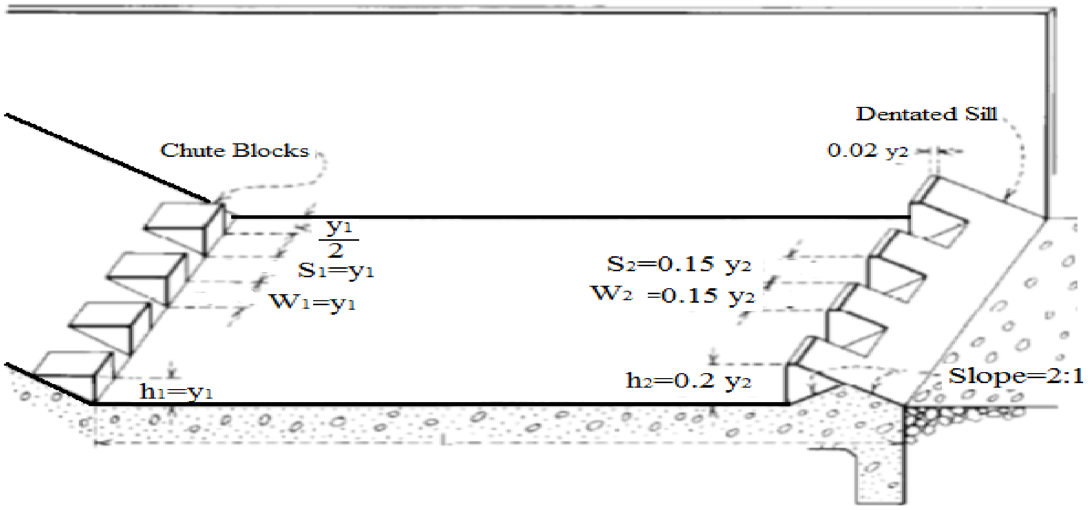
Stilling Basin Type I

Type II ($Fr_1 > 4.5$; $V_1 > 18$ m/s)

The Type II basin is designed for use on high dams, earth dam spillways for Froude numbers greater than 4.5. The chute blocks and end sill help reduce the basin length by 33%. The design includes blocks and dentate end sill. The end sill has a stabilizing effect and dissipative reason. The length of stilling basin calculates from the following equations:

$L_{II} = y_2[4.0 + 0.055(Fr_1 - 4.5)]$ for $4.5 < Fr_1 < 10$

$L_{II} = 4.35 y_2$ for $Fr_1 > 10$



Type II Stilling Basin

Type III ($Fr_1 > 4.5$; $V_1 < 18$ m/s)

The Type III basin reduces the length by 45-60 % with the addition of chute blocks, baffle piers, and an end sill. This structure is also used for Froude numbers greater than 4.5, but its use is restricted to small spillways where the upstream velocity is less than 15-18 m/sec. It was developed for gravity dam, earth dam spillways. With the inclusion of baffles, the inflow velocities are restricted to avoid cavitation damage to the concrete surface and reduce the impact force to the blocks. The length of stilling basin can also calculate from the following equation:

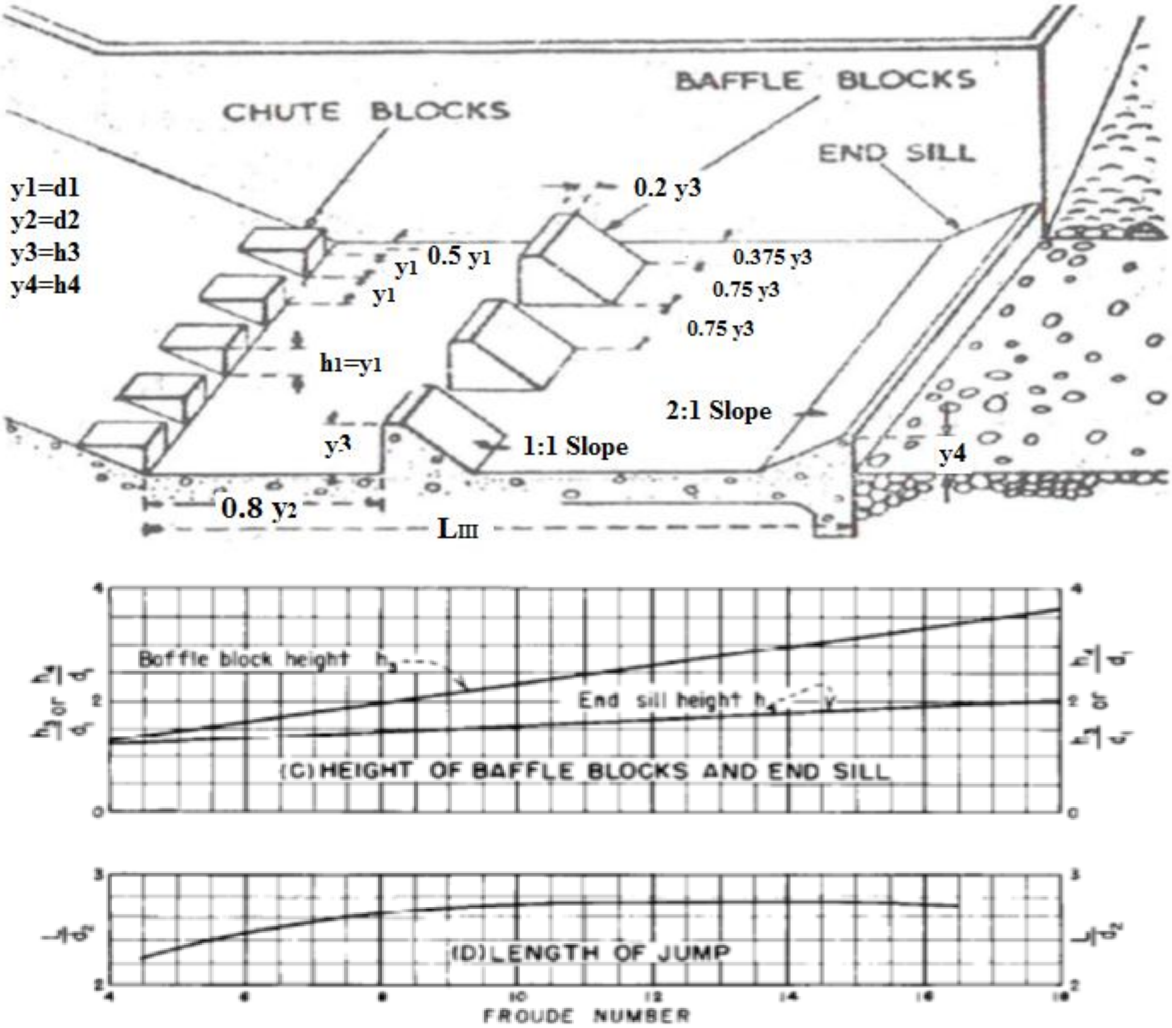
$L_{III} = y_2[2.4 + 0.073(Fr_1 - 4.5)]$ for $4.5 < Fr_1 < 10$

$L_{III} = 2.8 y_2$ for $Fr_1 > 10$

For the Type III stilling basin, the dimensions h3(y3) and h4(y4) are given by,

$h_3 = y_1[1.3 + 0.164(Fr_1 - 4.0)]$

$h_4 = y_1[1.25 + 0.056(Fr_1 - 4.0)]$

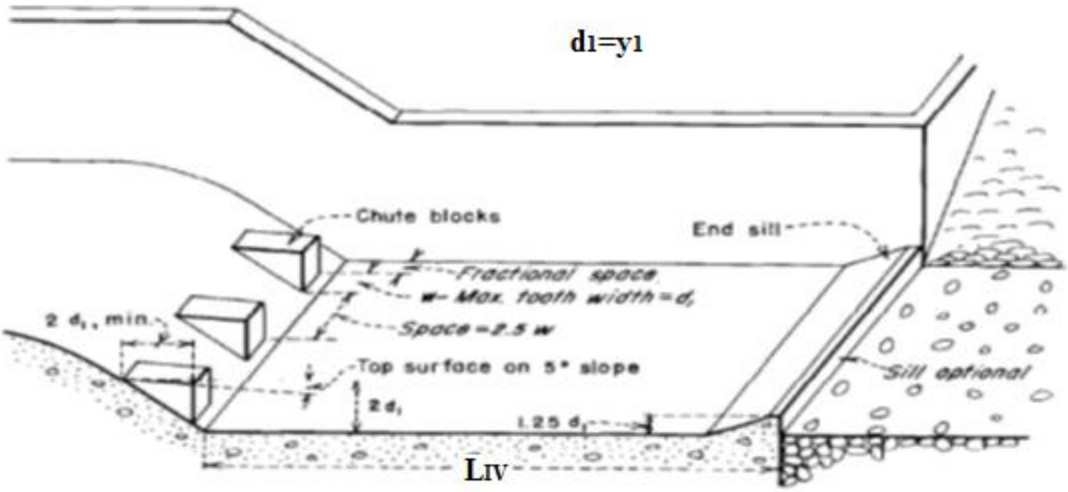


Characteristics of Stilling Basin III

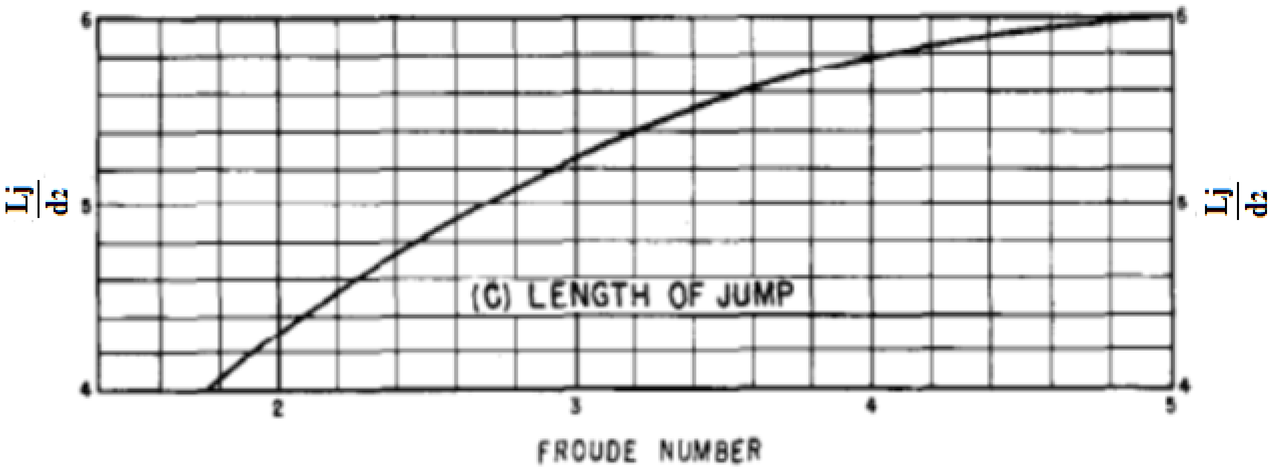
Type IV (2.5<Fr₁ <4.5)

The Type IV basin is used for Froude numbers 2.5-4.5, and thus is used primarily for oscillating hydraulic jump on canal structures and diversion canals. It includes chute deflector blocks to reduce the instability of the oscillating jump and continuous end sill. The length of the stilling basin calculates from the following equation:

$$L_{IV} = y_2[5.2 + 0.40(Fr_1 - 2.5)]$$



Type IV Basin Dimension



Characteristics of Stilling Basin IV

Problems

Q (1): A horizontal rectangular stilling basin is used at the outlet of a spillway to dissipate energy. The spillway discharges 13m^3/s and has a uniform width of 12m. At the point where the water enters the basin, the velocity is 10m/s, compute:

- **The sequent depth of the hydraulic jump,**
- **The length of the stilling basin,**
- **The energy loss in the jump,**
- **The efficiency of the jump (the ratio of specific energy after to the specific energy before the hydraulic jump,**
- **If the thickness of the basin is 0.75m, and the sieve analysis for the site soil indicate the d50 is 0.05mm, are the requirements of scour design satisfied or not?**

Q (2): A spillway carries a discharge of 22.5m^3/s with the outlet velocity of 15m/s at a depth of 0.2m. Design the stilling basin and determine the jump efficiency?

Q (3): An increase in discharge through the spillway in Q (2) to 45m^3/s will increase the outlet depth to 0.25m. Design the stilling basin and determine the jump efficiency?

River Diversion:

Whenever a dam is to build a cross an existing river channel, the river must be diverted so that construction can be done.

The manner in which the diversion is accomplished depends on:

- Kinds of dam being constructed.
- The character of the site,
- The characteristics of stream flow.

Gravity Dams:

In the construction of this type, there are two stages:

First stage

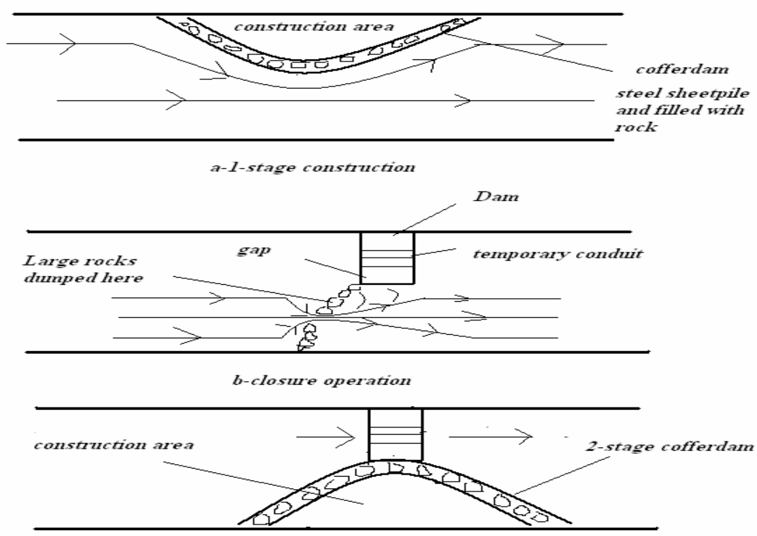
- A coffer dam is constructed a cross about half the channel and the flow is diverted to the other half.
- The area inside the cofferdam is dewatered and part of the dam is built inside this cofferdam.

A temporary diversion tunnel also may be including in this stage construction through which flow can be diverted in the second stage.

The second stage:

- The first stage cofferdam is removed and construction of part of the second stage is started,
- The flow is restricted to small opening in the second stage cofferdam.

For the large river, cutting off the flow is a critical operation (2-stage) because the flow velocity through the opening in the already constructed cofferdam frequently will be large and will increase as the opening because smaller. The process of stopping this flow is called the closure.



After construction is complete the 2-stage the cofferdam is removed and flow is diverted through the part of the dam (spillway or powerhouse).

Scheduling the different phases of construction of the cofferdam must be synchronize with the normally expected variation in stream flow. For example, the first cofferdam will normally be built during a low-flow period and then the stream flow will be diverted to the completed structure during another low flow season.

The depending on the stream characteristics, the designer may have to think in term of 1-year time increments between low flow seasons.

The designer also will have to decide how high to design the cofferdams. Normally, they are not made so high that they would never be overtopped by the design flood for the dam. The designer must balance the added cost of a very high cofferdam against the damage might include the costs due to work stoppage and cleanup. For major dam, designing the cofferdams so that it will withstand a 20-year flood is common.

Arch Dams:

The two stage diversion described for gravity dam may not be suitable because of the limited working space in the canyon.

A common procedure is to excavate a tunnel through one abutment and then build cofferdams upstream and downstream dam sites. When the cofferdams are closed, the water is diverted through the tunnel, and construction of the dam can take place inside the cofferdam.

In designing arch dams, it is fairly common to include a tunnel as part of the spillway. In this case, part of the diversion tunnel can usually be used as part of the spillway tunnel.

Diversions for Earth Dams:

The diversion scheme is much more important for earth dams than for concrete dams, if cofferdams for an earth dam were overtopped. It is possible than all the work done to that date could be wiped out.

That is, the damage infected by overtopped a cofferdam protecting an earth embankment is usually much more than that for a concrete dam. Therefore the direction scheme for earth dam must be designed to accommodate very large floods.

Problems

Q (1): The following data represent the flow measurement of a river, which is achieved at the location of a construction dam site, design the diversion of the river if the type of dam is: v=3m/s

- **Gravity dam,**
- **Arch dam,**
- **Buttress dam,**
- **Earth dam**

Time	**Elevation of Water**	**Q(m^3/s)**
1	**112**	**1500**
2	**111.7**	**1300**
3	**111.5**	**1150**
4	**111.35**	**1000**
5	**111.1**	**980**
6	**111**	**970**
7	**110.95**	**963**
8	**110.9**	**954**
9	**110.25**	**700**
10	**110.15**	**690**
11	**110.01**	**614**
-	**-**	
50	**-**	

Dams Operation

Dams are built for a difference purpose;

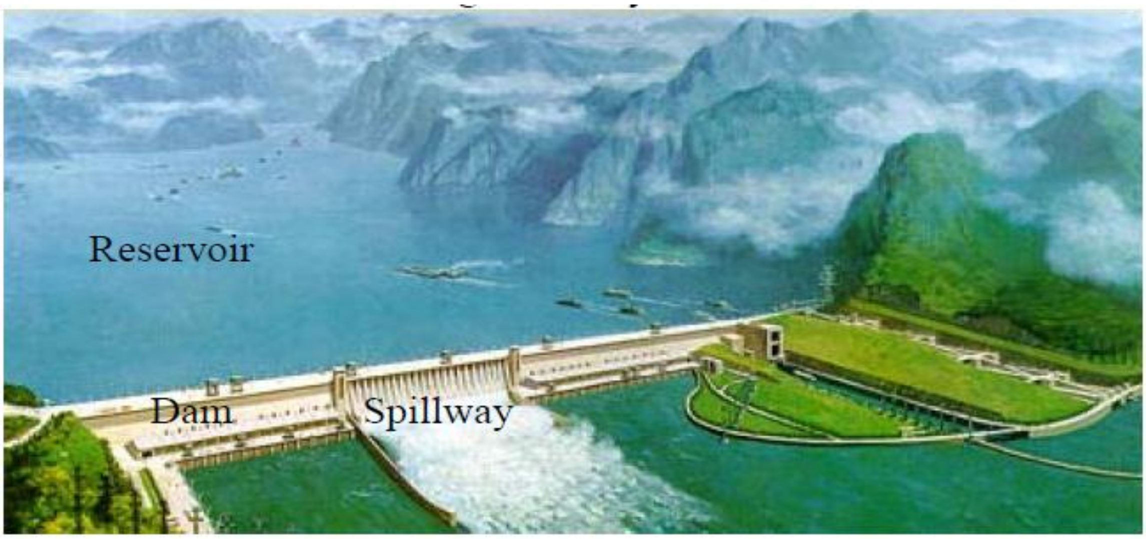
- Irrigation
- Flood control
- Power Supply
- Water demand in downstream region.
- Tourist.

To satisfy these purposes we need high accuracy to operate and control of dam, for example to supply power the storage in the reservoir must be at high level, this cause the problem for flood control purpose which needs the reservoir must be empty or at low level. Also, the operation for irrigation purpose needs we must fill the reservoir quickly to discharge water according to agriculture demand, while the power demand needs the storage at high level along the year.

The engineer must be balance among different purposes; any mistake in filled or empty the reservoir may be causes a considerable effect on dam operation.
The engineer may be make a mistake when filled or empty the malti-reservoir these cause the results are not good.
When he (engineer) didn't fill the reservoir quickly will cause the shortage of water demand for irrigation and power supply. While will case a catastrophic for agriculture and cities in downstream region when he fill the reservoir and then the flood occur.

Reservoirs:

A water-supply, irrigation, or hydroelectric project drawing water directly from a stream may be unable to satisfy the demands of its consumers during low flows. This stream, which may carry little or no water during portions of the year, often becomes a raging torrent after heavy rain and a hazard to all activities along its banks.
A storage reservoir can retain such excess water from periods of high flow for use during periods of drought. In addition to conserving water for later use, the storage of floodwater may also reduce flood damage below the reservoir.
Whatever the size of a reservoir or the ultimate use of the water, ***the main function of a reservoir is to stabilize the flow of water, either by regulating a varying supply in a nature stream or by satisfying a varying demand by the ultimate consumers.***



Types of Reservoirs

If a reservoir serves only one purpose, it is called a ***single-purpose reservoir***. On the other hand, if it serves more than one purpose, it is termed a ***multipurpose reservoir***. Because in most of the cases, a single purpose reservoir is ***not economically feasible***, it is the general practice in Iraq to develop multipurpose reservoirs. The various purposes served by a multipurpose reservoir include

- irrigation
- municipal and industrial water supply,
- flood control
- hydropower,
- navigation,
- recreation,
- development of fish and wild life,
- soil conservation
- Pollution control.

Depending upon the purpose served, the reservoirs may be broadly classified into five types:
(1) Storage (or conservation) reservoirs,
(2) Flood control reservoirs,
(3) Multipurpose reservoirs,
(4) Distribution reservoirs, and
(5) Balancing reservoirs.

1. **Storage reservoirs:**

Storage reservoirs are also called ***conservation reservoirs*** because they are used to ***conserve water***. Storage reservoirs are constructed to store the water in the rainy season and to release it later when the river flow is low. Storage reservoirs in Iraq are usually constructed for:

- Irrigation,
- The municipal water supply and
- Hydropower.

Although the storage reservoirs are constructed for storing water for various purposes, incidentally they also help in ***moderating the floods and reducing the flood damage to some extent on the downstream***. However, they are ***not designed as flood control reservoir***.

2. **Flood control reservoirs:**

A flood control reservoir is constructed ***for the purpose of flood control***.

Functions:

- It protects the areas lying on its downstream side from the damages due to flood. However, absolute protection from extreme floods is not economically feasible. It is also known as the flood-mitigation reservoir. Sometimes, it is called flood protection reservoir.
- A flood control reservoir is designed to moderate the flood and not to conserve water. However, incidentally some storage is also done during the period of floods.
- Flood control reservoirs have relatively large sluice-way capacity to permit rapid drawdown before or after the occurrence of a flood.

In a flood control reservoir, the flood water is discharged downstream till the ***outflow reaches the safe capacity of the channel downstream***. When the discharge exceeds the safe capacity, the excess water is stored in the reservoir. The stored water is subsequently released when the inflow to reservoir decreases. Care is, however, taken that the discharge in the channel downstream, including local inflow, does not exceed its safe capacity:

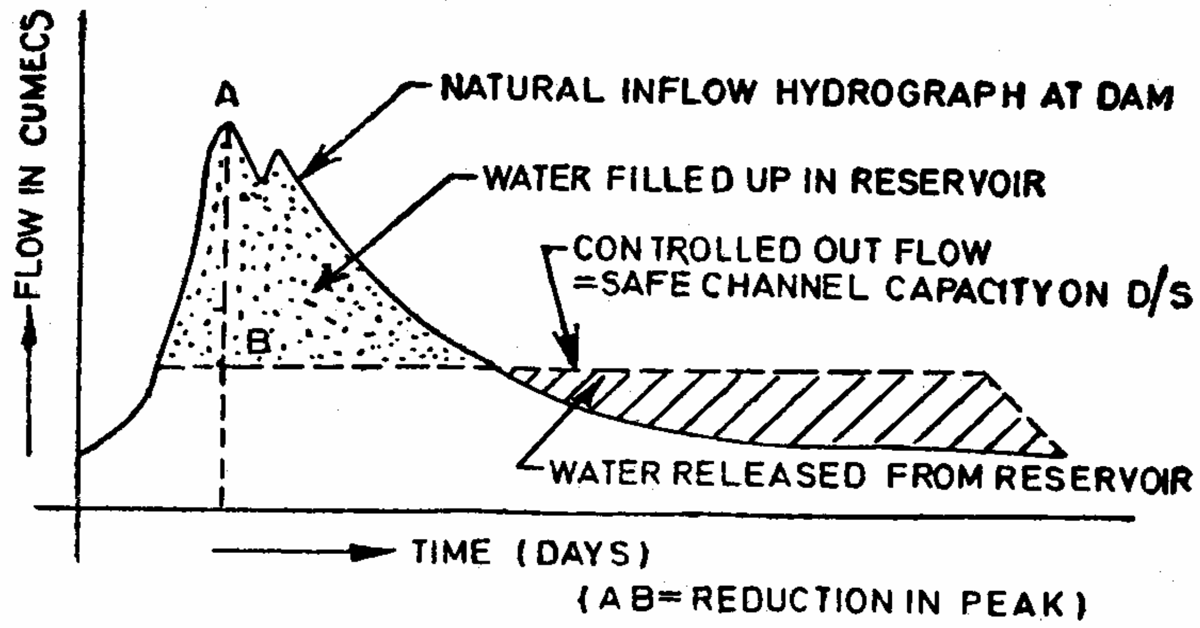
The flood control reservoirs are of two types:
(i) Detention reservoirs and
(ii) Retarding reservoirs.

i. Detention reservoirs:

Function:

- Stores excess water during floods and releases it after the flood.
- It is similar to a storage reservoir but is provided with large gated spillways and sluiceways to permit flexibility of operation.

The discharge from a detention reservoir to the downstream channel is ***regulated by gates***. In the earlier stages of a flood, the gates are left open and the water is released subjected to the safe carrying capacity B of the channel downstream. In the later stages of the flood when the discharge downstream exceeds the maximum capacity of the downstream channel, the gates are kept partially closed.



There is basically no difference between the detention reservoir and a storage reservoir except that the former has a larger spillway capacity and sluiceway capacity to permit rapid drawdown just before or after a flood. The reservoir is quickly emptied and thus the full reservoir capacity is made available again for moderating a subsequent flood after a short interval. In this manner, the available capacity is more effectively used. When the natural inflow is greater than controlled outflow rate (B), the excess water is stored in the reservoir. The volume of stored water is indicated by dotted area. When the discharge is less than B the stored water is released. The volume of released water is shown by hatched area. Because of detention reservoir, the flood peak is reduced from A to B. ***Thus the effect of reservoir on a flood is to reduce the peak discharge*** by absorbing a volume of flood water when the flood is rising, and releasing the same later gradually when the flood is receding.

Advantages

(1) The detention reservoirs provide more flexibility of operation and better control of outflow than retarding reservoirs. Large reservoirs are usually detention reservoirs.
(2) The discharge from various detention reservoirs on different tributaries of a river can be adjusted according to the carrying capacity of the d/s channel.

Disadvantages

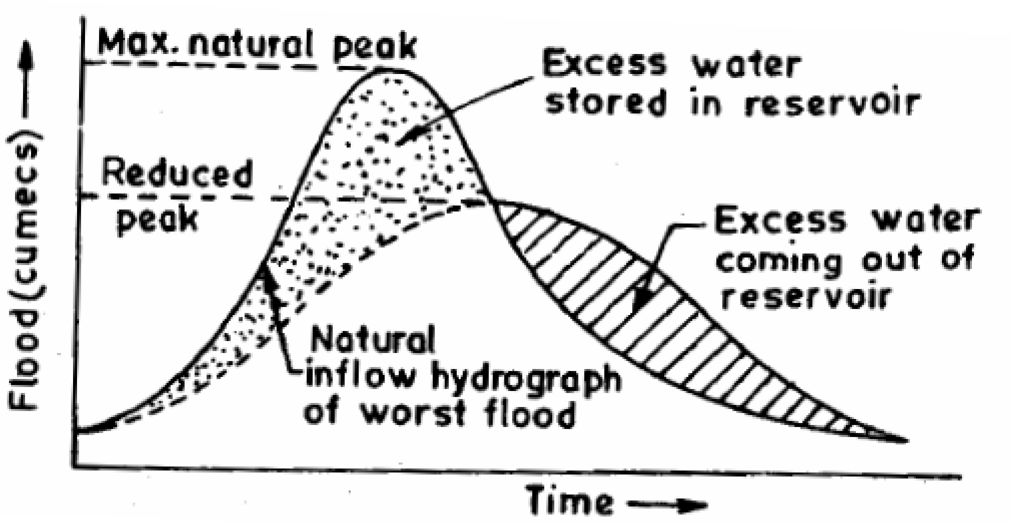
(1) The detention reservoirs are more expensive than the retarding reservoirs because of high initial cost and maintenance cost of gates and the lifting machinery.
(2) Due to the possibility of human error or negligence, a disaster can occur.

ii. Retarding Reservoirs:

Function

- A retarding reservoir is provided with spillways and sluiceways which are un-gated.
- The maximum combined discharging ***capacity of all spillways and sluiceways is limited to the safe-carrying capacity of the channel downstream***.
- The retarding reservoir stores a portion of the flood when the flood is rising and releases it later when the flood is receding. However, in this case, the discharge downstream cannot be controlled because there are no gates.
- There is an automatic release of water, depending upon the level of water in the reservoir.

As the flood occurs, the reservoir gets filled and at the same time, the discharge from the spillways and sluiceways occurs. When the elevation of water in the reservoir increases, the discharge through spillways and sluiceways also increases.



The water level in the reservoir goes on rising until the flood starts receding when the inflow is reduced and it becomes equal to or less than the outflow. After that stage has reached, the water level in the reservoir starts falling and it continues till the stored water has been completely discharged and the water level has reached the lowest sluiceway level. Fig. shows the peak reduction mechanism of a retarding reservoir. The stored water in the reservoir (dotted), which is later released, is shown hatched.

Location of Retarding Reservoirs:

- A favorable location for a retarding reservoir is just above the area or the city to be protected against floods.
- A retarding reservoir is also usually located on a tributary of a river just upstream of its confluence to protect the area downstream of it.

Advantages

(1) The retarding reservoirs are relatively less expensive than detention reservoirs.
(2) As the outflow is automatic, there is no possibility of a disaster due to human error or negligence.

Disadvantages

(1) The retarding reservoirs do not provide any flexibility of operation as the outflow is automatic.
(2) The discharge from retarding reservoirs on different tributaries of a river may coincide and cause heavy flood in the river downstream.

3. **Multipurpose Reservoirs:**

A multipurpose reservoir is designed and constructed to serve two or more purposes. In Iraq, most of the reservoirs are designed as multipurpose reservoirs to store water for irrigation and hydropower, and also to effect flood control.

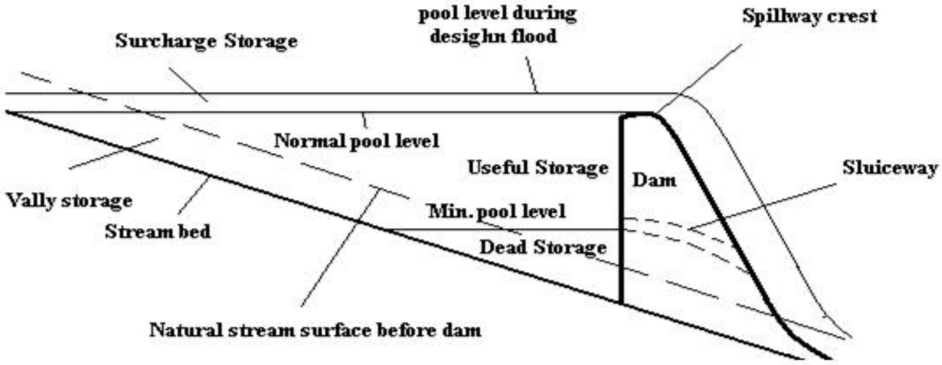
4. **Distribution Reservoir:**

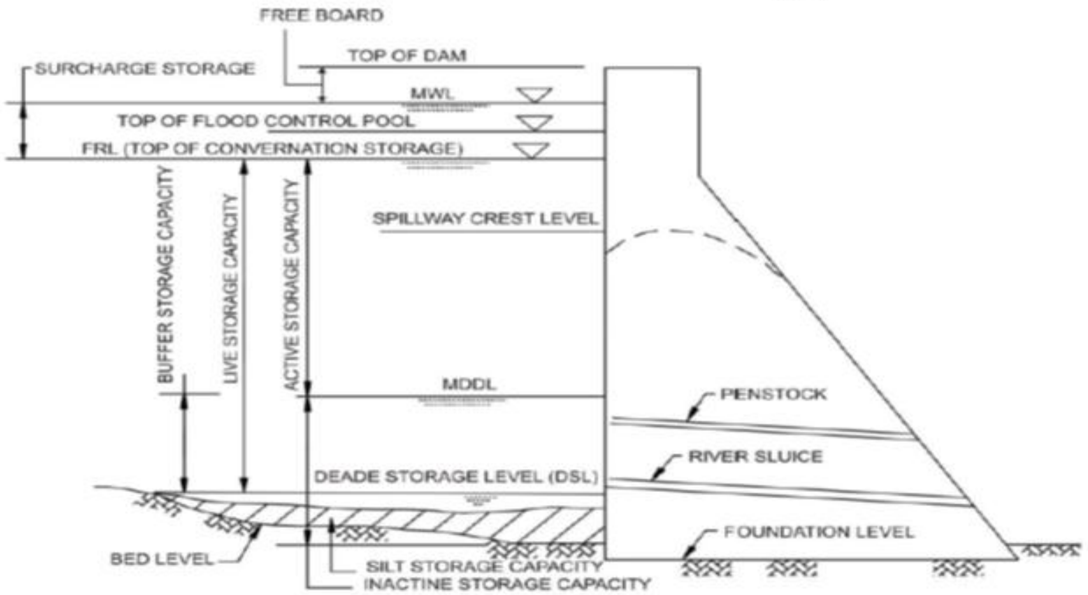
A distribution reservoir is a small storage reservoir to tide over the peak demand of water for municipal water supply or irrigation. The distribution reservoir is helpful in permitting the pumps to work at a uniform rate. It stores water during the period of lean demand and supplies the same during the period of high demand. As the storage is limited, it merely helps in distribution of water as per demand for a day or so and not for storing it for a long period. Water is pumped from a water source at a uniform rate throughout the day for 24 hours but the demand varies from time to time. During the period when the demand of water is less than the pumping rate, the water is stored in the distribution reservoir. On the other hand, when the demand of water is more than the pumping rate, the distribution reservoir is used for supplying water at rates greater than the pumping rate. Distribution reservoirs are rarely used for the supply of water for irrigation. These are mainly used for municipal water supply.

5. **Balancing reservoir:**

A balancing reservoir is a small reservoir constructed d/s of the main reservoir for holding water released from the main reservoir.

Zones of Storage:





Normal pool level: **is the maximum elevation to which the reservoir surface will rise during ordinary operating conditions. It is determined by the elevation of the spillway crest or the top of the spillway gates.**

Minimum pool level: **is the lowest elevation to which the pool is to be drawn under normal conditions. This level may be fixed by the elevation of lowest outlet in the dam or, in the case of hydroelectric reservoirs, by conditions of operating efficiency for turbines.**

Useful Storage: is the storage volume between the minimum and normal pool levels.
For multipurpose reservoirs in accordance with adopted plan of operation, the useful storage may be subdivided into:

- **Conservation storage,**
- **Flood mitigation storage.**

During floods, discharge over the spillway may cause the water level to rise above normal pool level. This *surcharge storage* is normally uncontrolled, i.e., it exists only while a flood is occurring and cannot be retained for later use.

Dead Storage: is the water held below minimum pool level.

Reservoir banks are usually permeable, and water enters the soil when the reservoir fills and drains out as the water level is lowered.

This bank storage increases the capacity of the reservoir above that indicated by the *elevation-storage curve.* This storage depends on:

- **Geological conditions,**
- **Reservoir volume.**

The water in a natural stream channel occupies a variable volume of *valley storage*.

The net increase in storage capacity resulting from the construction of a reservoir is the total capacity less the natural valley storage.

This distinction is of no important for conservation reservoirs, but from the viewpoint of flood mitigation the effective storage in the reservoir is the useful storage plus the surcharge storage less the natural valley storage corresponding to the rate of inflow to the reservoir.

Reservoir Yield:

Probably the most important aspect of storage-reservoir design is an analysis of the relation between yield and capacity.

Yield: is the amount of water that can be supplied from the reservoir during a specified interval of time. The time interval may vary from a day for small distribution reservoir to a year or more for a large storage reservoir.

Yield is dependent on inflow and will vary from year to year.

***The safe or firm, yield*:** is the maximum quantity of water that can be guaranteed during a critical dry period.

In practice, ***the critical period is often taken as the period of lowest natural flow on record for stream***, there is a finite probability that a drier period may occur, with a yield even less than the safe yield.

Firm yield can never be determined with certainty, it is better to treat yield in probabilistic terms. ***The maximum possible yield*** during a given time interval ***equal the mean inflow less evaporation and seepage losses during that interval***. If the flow were ***absolutely constant,*** **no reservoir would be required;** but, as variability of the flow increases, the required reservoir capacity increases.

Water available in excess of safe yield during periods of high flow is called *secondary yield*.

Hydroelectric energy developed from secondary water may be sold to large industries on a (when available) basis. Energy commitments to domestic users must be on a firm basis and should not exceed the energy that can be produced with the firm yield unless thermal energy (steam or diesel) is available to support the hydroelectric energy. The decision is an economic one based on costs and benefits for various levels of design.

Selection of Distribution-Reservoir Capacity for a Given Yield:

Often project design requires the determination of the reservoir capacity required to meet a specific demand. Since the yield (outflow) is equal to inflow plus or minus an increment of storage, the determination of the capacity to supply a given yield is based on the storage equation:

$$I\Delta t - \Delta s = O\Delta t$$

Where I and O are the average rates of inflow and outflow for the time interval Δt and Δs is the change in volume water. A simple problem involving the selection of distribution reservoir capacity is given in following examples:

Example (1): The water supply for a city is pumped from well to a distribution reservoir. The estimated hourly water requirements for a maximum day are as in table below. If the pumps are to operate at a uniform rate, what distribution capacity is required? Also determine the yield

Hour ending	Demand(m^3/hr)	Pumping rate (m^3/hr)	Required from reservoir (m^3)
1	273	529.3	0
2	206	529.3	0
3	256	529.3	0
4	237	529.3	0
5	257	529.3	0
6	312	529.3	0
7	438	529.3	0
8	627	529.3	98
9	817	529.3	288
10	875	529.3	346
11	820	529.3	291
12	773	529.3	244
13	759	529.3	230
14	764	529.3	235
15	729	529.3	200
16	671	529.3	142
17	670	529.3	141
18	657	529.3	128
19	612	529.3	83
20	525	529.3	0
21	423	529.3	0
22	365	529.3	0
23	328	529.3	0
24	309	529.3	0
Total	12703	12703	2426

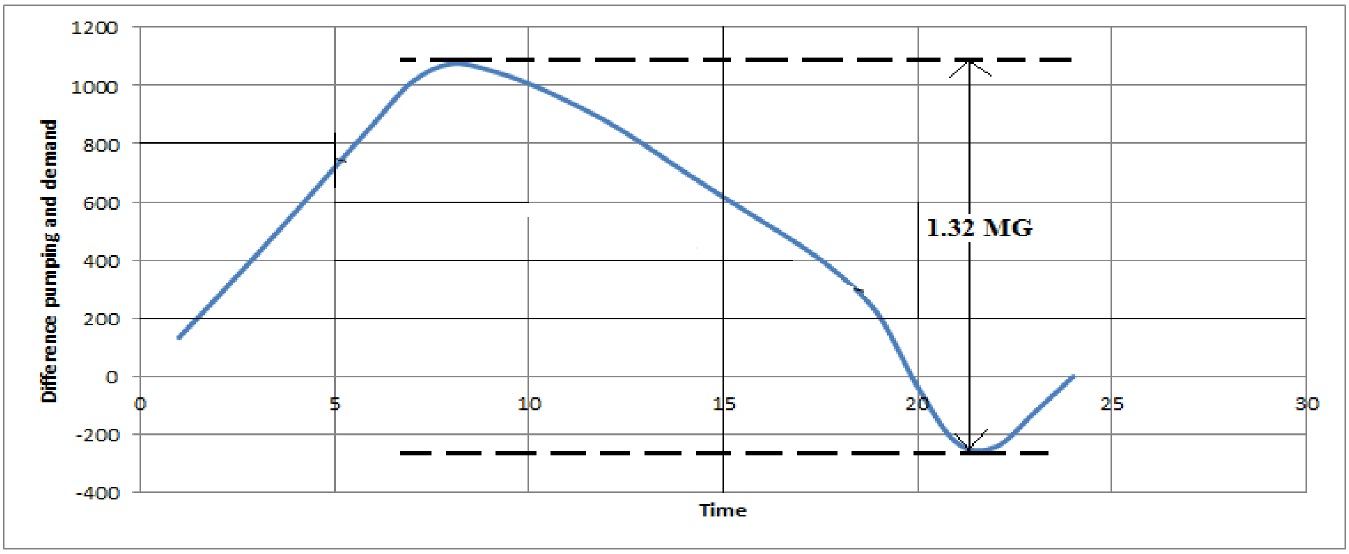
Example (2): for the data in table below, determine the storage capacity for a reservoir if the pumping rate of the pump from well to reservoir is (257.6 *1000) gal/hr, when:

1. **24-pumping per day,**
2. **12-pumping per day.**

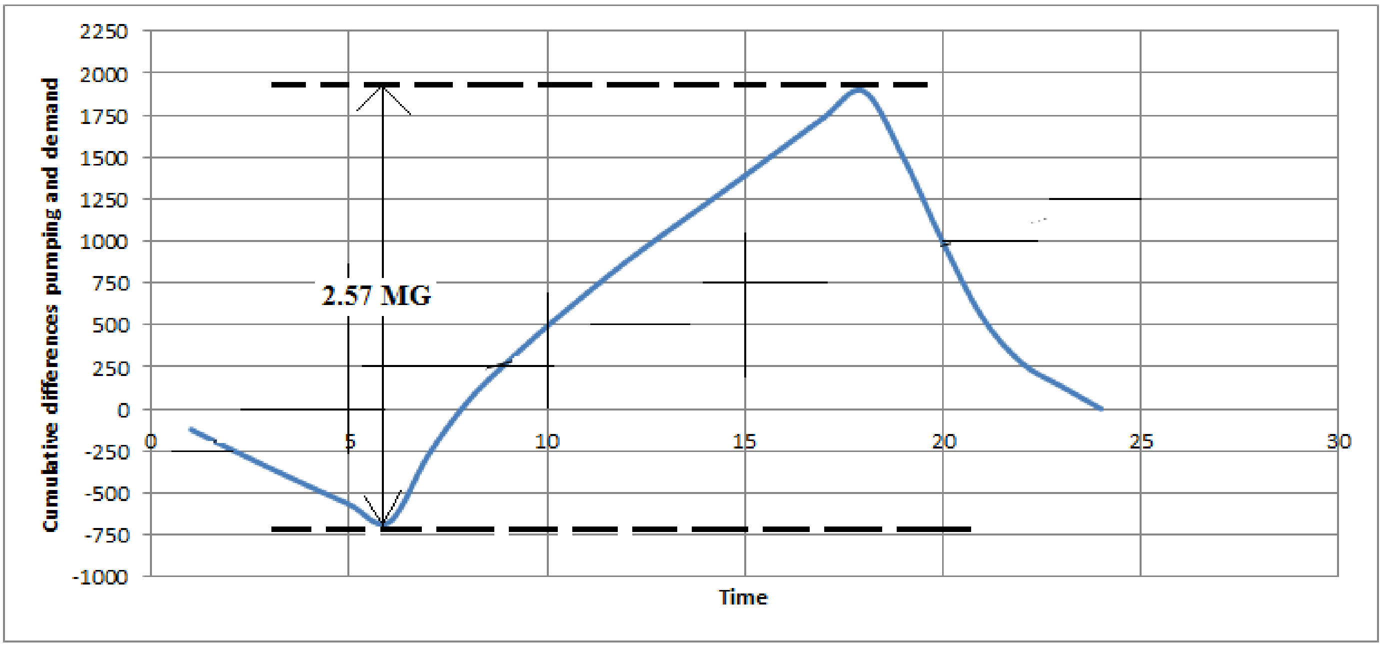
Time	**Hourly Demand Rate (gpm)**
12-midnight	2061
1am	1953
2	1890
3	1818
4	1773
5	1782
6	1872
7	3267
8	4671
9	5058
10	5310
11	5436
12	5688
1pm	5796
2	5733
3	5688
4	5706
5	5976
6	6588
7	8400
8	7488
9	4545
10	2313
11	2223

Sol:

Time	Hourly Demand Rate (gpm)	Hourly demand (gal*1000)	Cumulative hourly demand (gal*1000)	24-pumping		12-pumping	
				Cumulative 24-Pumping (gal*1000)	Cumulative Difference Col5-Col4 (gal*1000)	Cumulative 12-Pumping (gal*1000)	Cumulative Difference Col7-Col4 (gal*1000)
12-midnight	2061	123.7	123.7	257.6	133.9	0	-123.7
1am	1953	117.2	240.9	515.2	274.3	0	-240.9
2	1890	113.4	354.3	772.8	418.5	0	-354.3
3	1818	109.1	463.4	1030.4	567	0	-463.4
4	1773	106.4	569.8	1288	718.2	0	-569.8
5	1782	106.9	676.7	1545.6	868.9	0	-676.7
6	1872	112.3	789	1803.2	1014.2	515.2	-273.8
7	3267	196	985	2060.8	1075.8	1030.4	45.4
8	4671	280.3	1265.3	2318.4	1053.1	1545.6	280.3
9	5058	303.5	1568.8	2576	1007.2	2060.8	492
10	5310	318.6	1887.4	2833.6	946.2	2576	688.6
11	5436	326.2	2213.6	3091.2	877.6	3091.2	877.6
12	5688	341.3	2554.9	3348.8	793.9	3606.4	1051.5
1pm	5796	347.8	2902.7	3606.4	703.7	4121.6	1218.9
2	5733	344	3246.7	3864	617.3	4636.8	1390.1
3	5688	341.3	3588	4121.6	533.6	5152	1564
4	5706	342.4	3930.4	4379.2	448.8	5667.2	1736.8
5	5976	358.6	4289	4636.8	347.8	6182.5	1893.5
6	6588	395.3	4684.3	4894.4	210.1	6182.5	1498.2
7	8400	504	5188.3	5152	-36.3	6182.5	994.2
8	7488	449.3	5637.6	5409.6	-228	6182.5	544.9
9	4545	272.7	5910.3	5667.2	-243.1	6182.5	272.2
10	2313	138.8	6049.1	5924.8	-124.3	6182.5	133.4
11	2223	133.4	6182.5	6182.5	0	6182.5	0
total		**6182.5**					



Pumping 24-hr



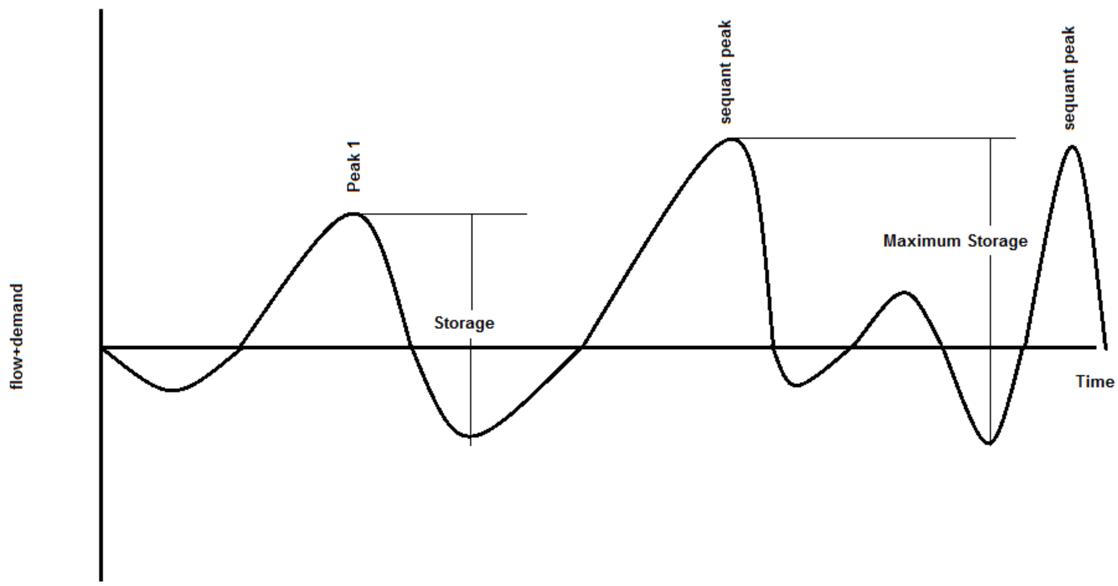
Pumping 12-hr

Selection of Capacity for a River Reservoir:

The determination of required capacity for river reservoir is usually called an ***operation study*** and is essentially a simulation of the reservoir operation for a period of time in accord with an adopted set of rule.
Operation study may analyze:

- Only a selected "critical period" of very low flow (include no more than define the capacity required during the selected drought.
- Modern practice favors the use of a long synthetic record (synthetic data it is possible to estimate the reliability of reservoirs of various capacity.
- An operation study may be performing with annual, monthly, or daily time intervals. Monthly data are most commonly used, but for large reservoirs that carry over storage for many years, annual intervals are satisfactory.

When lengthy synthetic data are to be analyzed, computer analysis is indicated and the ***sequent- peak algorithm*** **(see fig. below)** is commonly used**.**

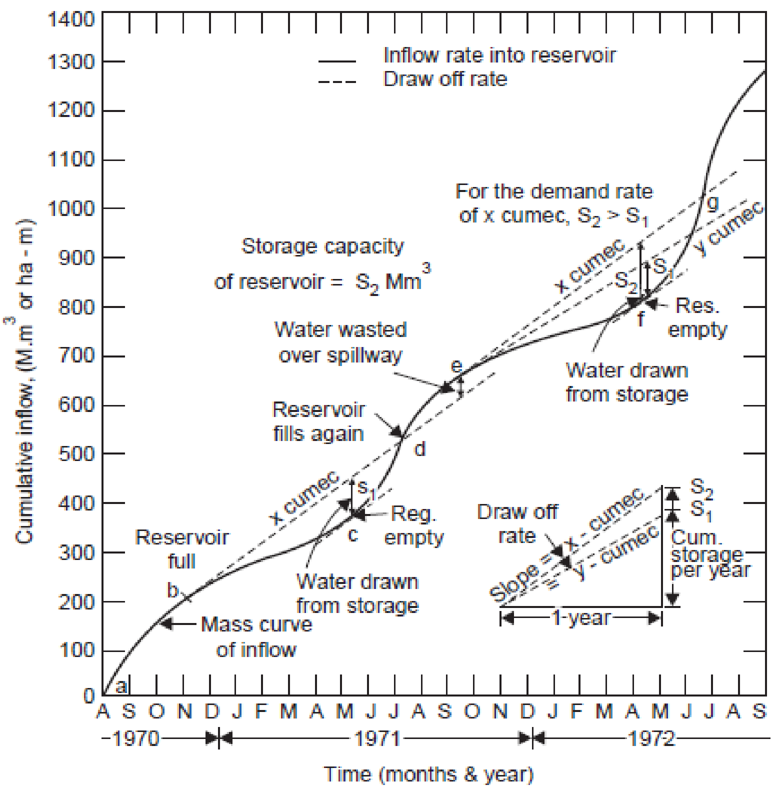


Sequent- peak algorithm

In evaluating storage requirements a hydrologist would use various hydrological tools such as cumulative mass curves, runoff, estimation of flood design, flood routing and other factors.

Reservoir Mass Curve and Storage:

During high flows, water flowing in a river has to be stored so that a uniform supply of water can be assured, for water resources utilisation like irrigation, water supply, power generation, etc. during periods of low flows of the river. A mass diagram is a graphical representation of cumulative inflow into the reservoir versus time which may be monthly or yearly. A mass curve is shown in Fig. below for a 2-year period. The slope of the mass curve at any point is a measure of the inflow rate at that time.
Required rates of draw off from the reservoir are marked by drawing tangents, having slopes equal to the demand rates, at the highest points of the mass curve. The maximum departure between the demand line and the mass curve represents the storage capacity of the reservoir required to meet the demand. A demand line must intersect the mass curve when extended forward, otherwise the reservoir is not going to refill. The vertical distance between the successive tangents represents the water wasted over the spillway. The salient features in the mass curve of flow in Fig. below are:
a-b: inflow rate exceeds the demand rate of *x* cumec and reservoir is overflowing
b: inflow rate equals demand rate and the reservoir is just full
b-c: inflow rate is less than the demand rate and the water is drawn from storage
c: inflow rate equals demand rate and *S*1 is the draw off from the reservoir (Mm3)
c-d: inflow rate exceeds demand rate and the reservoir is filling
d: reservoir is full again
d-e: same as *a-b*
e: similar to *b*
e-f: similar to *b-c*
f: inflow rate equals demand rate and *S*2 is the draw off from the reservoir
f-g: similar to *c-d*
To meet the demand rate of *x* cumec the departure $S2 > S1$; hence, the storage capacity of the reservoir is *S*2 Mm^3. If the storage capacity of the reservoir, from economic considerations, is kept as *S*1 Mm^3, the demand rate of *x* cumec cannot be maintained during the time *e-f* and it can be at a lesser rate of *y* cumec ($y < x$).



Storage capacity of reservoir from mass curve

The use of mass curve is to determine:

(*i*) The storage capacity of the reservoir required to meet a particular withdrawal rate.

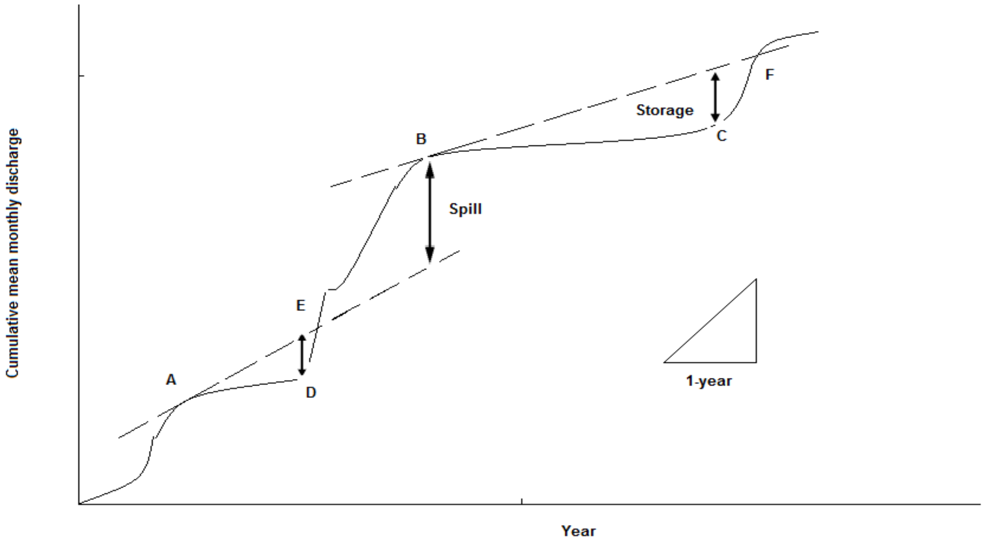
(*ii*) The possible rate of withdrawal from a reservoir of specified storage capacity.

The observed inflow rates have to be adjusted for the monthly evaporation from the reservoir surface, precipitation, seepage through the dam, inflow from adjacent basins, required releases for downstream users, sediment inflow, etc. while calculating the storage capacity of the reservoir.

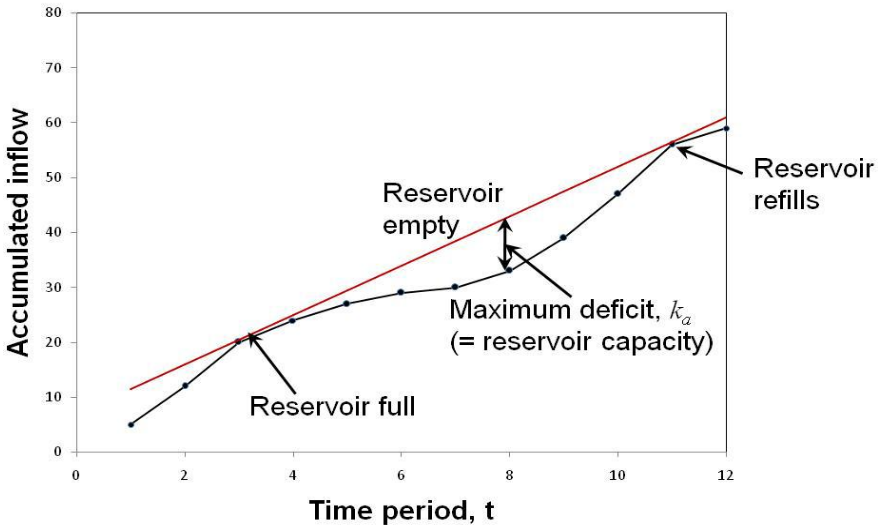
Conclusions:

A mass curve (or Rippl diagram) is a cumulative of net reservoir inflow, the figure below shows a typical mass curve.

- The slope of the mass curve at any time is a measure of the inflow at that time.
- Demand lines drawn tangent to the high points of the mass curve represent rates of withdrawal from reservoir.
- Maximum departure between the demand line and the mass curve represent the reservoir capacity required to satisfy the demand.
- The vertical distance between successive tangents represents water wasted over the spillway.



Typical mass curve



Typical mass curve

Example (1): **Find the required storage for the following stream?**

Month	Inflow (supply) m^3/month	Outflow (demand) m^3/month	Losses m^3/month
1	**45**	**7**	**1**
2	**45**	**8**	**1**
3	**14**	**8**	**1**
4	**10**	**8**	**1**
5	**8**	**9**	**2**
6	**5**	**11**	**3**
7	**2**	**11**	**3**
8	**2**	**11**	**2**
9	**2**	**11**	**2**
10	**2**	**11**	**2**
11	**1**	**11**	**1**
12	**1**	**11**	**1**

Sol:

Month	Inflow (supply) m^3/month	Outflow (demand) m^3/month	Losses m^3/month	Storage m^3/month
1	**45**	**7**	**1**	**37**
2	**45**	**8**	**1**	**73**
3	**14**	**8**	**1**	**78**
4	**10**	**8**	**1**	**79**
5	**8**	**9**	**2**	**76**
6	**5**	**11**	**3**	**67**
7	**2**	**11**	**3**	**55**
8	**2**	**11**	**2**	**44**
9	**2**	**11**	**2**	**33**
10	**2**	**11**	**2**	**22**
11	**1**	**11**	**1**	**11**
12	**1**	**11**	**1**	**0**
	137	**117**	**20**	

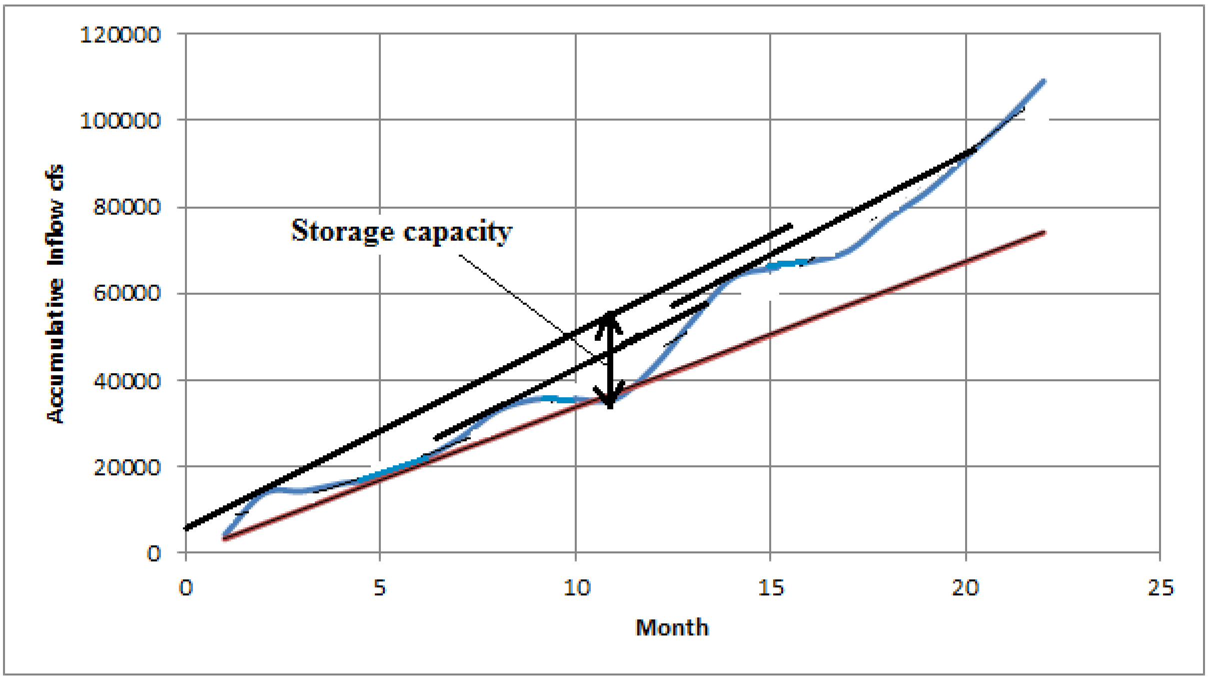
Example (2): determine the storage capacity for a reservoir according to the data in table, the uniform release is 18 cfs (0.52) and average evaporation is 2 cfs (0.1) (use TP or mass curve method)?

Month	Inflow (cfs)	Outflow (cfs)
Apr.	141 (4)	90 (2.55)
May	310 (8.8)	92 (2.61)
Jun	18 (0.51)	92 (2.61)
Jul	56 (1.59)	93 (2.63)
Aug	40 (1.13)	90 (2.55)
Sep	135 (3.82)	90 (2.55)
Oct	160 (4.53)	90 (2.55)
Nov	221 (6.26)	89 (2.52)
Dec	85 (2.41)	89 (2.52)
Jan	0	89 (2.52)
Feb	0	91 (2.58)
Mar	241 (6.82)	90 (2.55)
Apr	359 (10.17)	90 (2.55)
May	312 (8.83)	92 (2.61)
Jun	75 (2.12)	92 (2.61)
Jul	50 (1.42)	93 (2.63)
Aug	82 (2.32)	93 (2.63)
Sep	247 (7)	90 (2.55)
Oct	198 (5.61)	90 (2.55)
Nov	268 (7.6)	90 (2.55)
Dec	266 (7.5)	89 (2.52)
Jan	305 (8.64)	89 (2.52)

Sol:

Month	Inflow (cfs)	Outflow (cfs)	total outflow (cfs)	Inflow volume, It(cfs-day)	Outflow volume Ot(cfs-day)	Cumulative inflow ∑It (cfs-day)	Cumulative outflow ∑Ot (cfs-day)	Difference ∑It- ∑Ot (cfs-day)
Apr.	141	90	110	4230	3300	4230	3300	930
May	310	92	112	9610	3472	13840	6772	7068
Jun	18	92	112	540	3360	14380	10132	4248
Jul	56	93	113	1736	3503	16116	13635	2481
Aug	40	90	110	1240	3410	17356	17045	311
Sep	135	90	110	4050	3300	21406	20345	1061
Oct	160	90	110	4960	3410	26366	23755	2611
Nov	221	89	109	6630	3270	32996	27025	5341
Dec	85	89	109	2635	3379	35631	30404	5227
Jan	0	89	109	0	3379	35631	33783	1848
Feb	0	91	111	0	3108	35631	36891	-1260
Mar	241	90	110	7471	3410	43102	40301	2801
Apr	359	90	110	10770	3300	53872	43601	10271
May	312	92	112	9672	3472	63544	47073	16471
Jun	75	92	112	2250	3360	65794	50433	15361
Jul	50	93	113	1550	3503	67344	53936	13408
Aug	82	93	113	2542	3390	69886	57326	12560
Sep	247	90	110	7410	3300	77296	60626	16670
Oct	198	90	110	6138	3410	83434	64036	19398
Nov	268	90	110	8040	3300	91474	67336	24138
Dec	266	89	109	8246	3379	99720	70715	29005
Jan	305	89	109	9455	3379	109175	74094	35081

Uniform release + evaporation=20 cfs



Storage Capacity=7068-(-1260)=8328cfs.day or 16490 acre.ft

Example (3): The following is a record of the mean monthly discharges of a river in a dry year. The available fall is 80 m. Determine

(1) The minimum capacity of a reservoir if the entire annual inflow is to be drawn off at a uniform rate (with no flow going into waste over the spillway).

(2) The amount of water must be initially stored to maintain the uniform draw off.

(3) The uniform power output assuming a plant efficiency of 70%.

(4) If the amount of water initially stored is 125 Mm3, the maximum possible draw off rate and the amount of water wasted over the spillway (assuming the same reservoir capacity determined in (i) above.

(5) If the largest reservoir that can be economically constructed is of capacity 125 Mm3, the maximum possible output and the amount of water wasted over the spillway.

(6) The capacity of the reservoir to produce 22.5 megawatts continuously throughout the year.

Month	*Mean flow (cumec)*	*Month*	*Mean flow (cumec)*
Jan.	29.7	July	68.0
Feb.	75.3	Aug.	50.2
March	66.8	Sept.	74.5
April	57.2	Oct.	66.8
May	23.2	Nov.	40.5
June	26.3	Dec.	26.3

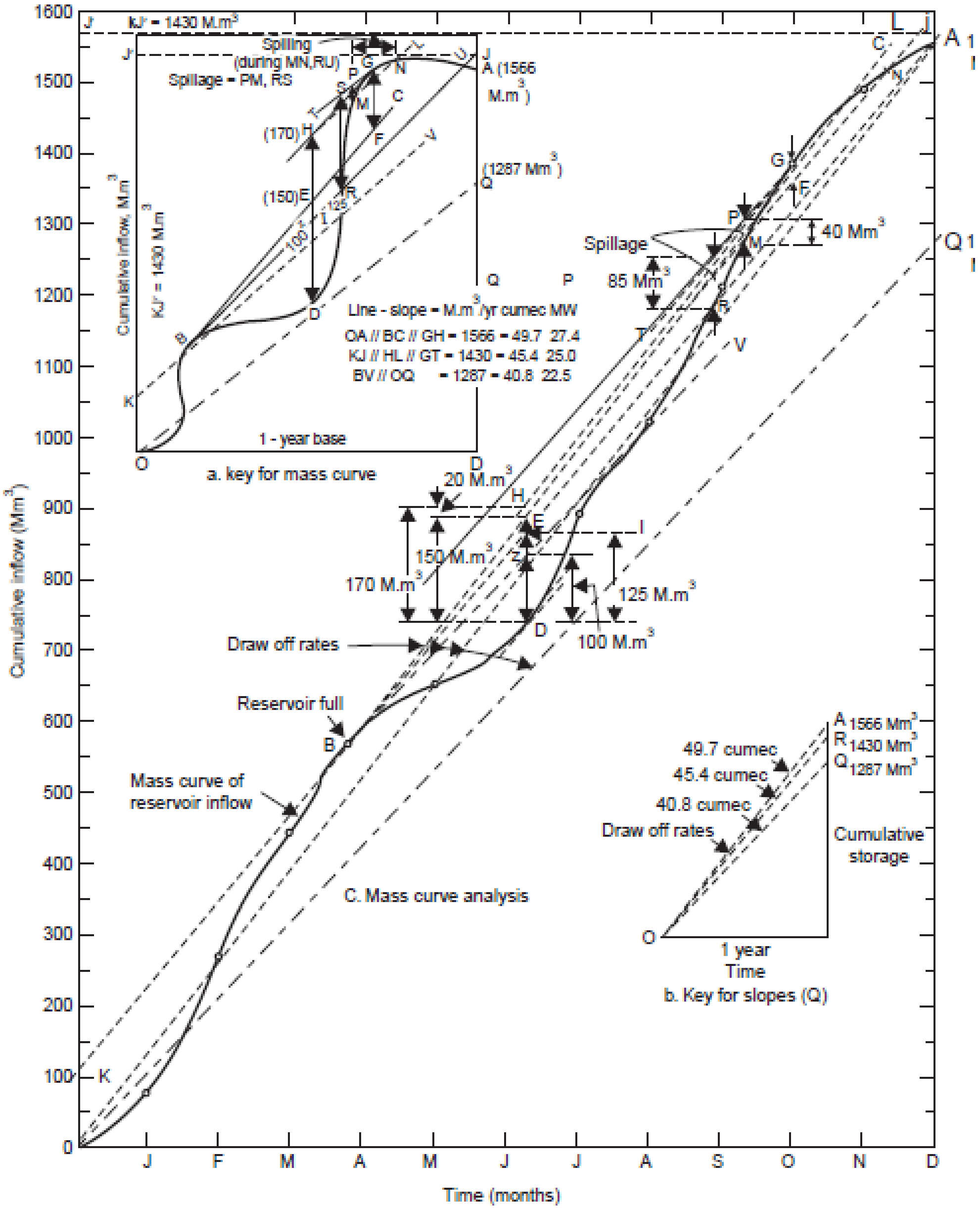
<u>Sol:</u>

Take each month as 30 days for convenience; 1 month = 30 days × 86400 sec = 2.592 × 106 sec. Inflow volume in each month = monthly discharge × 2.592 Mm3; and monthly inflow and cumulative inflow are tabulated in Table below:

Cumulative inflow into reservoir

Month	*Mean flow (cumec)*	*Inflow volume (Mm^3)*	*cumulative inflow (Mm^3)*	*Month*	*Mean flow (cumec)*	*Inflow volume (Mm^3)*	*cumulative inflow (Mm^3)*
Jan.	29.7	77	77	July	68.0	176	897
Feb.	75.3	195	272	Aug.	50.2	130	1027
Mar.	66.8	173	445	Sept.	74.5	193	1220
April	57.2	148	593	Oct.	66.8	173	1393
May	23.2	60	653	Nov.	40.5	105	1498
June	26.3	68	721	Dec.	26.3	68	1566

Plot the mass curve of flow as cumulative inflow vs month as shown in Fig. below.



Mass curve studies in reservoir design (Example 1)

1. Join *OA* by a straight line; the slope of *OA*, *i.e.*, 1566 Mm3/yr or (1566 × 106) m^3/ (365 × 86400) sec = 49.7 cumec is the uniform draw off throughout the year with no spill over the spillway. Draw *BC* || *OA*, *GH* || *OA*, *B*, *G* being the crests of the mass curve; *EH* = *FG*
   Minimum capacity of reservoir = *DE* + *EH* = 150 + 20 = **170 Mm3**

Note: If the capacity is less than this, some water will be wasted and if it is more than this, the reservoir will never get filled up.

2. Amount of water to be initially stored for the uniform draw off of 49.7 cumec = DE = 150 Mm^3
3. Continuous uniform power output in kW, $P = \frac{\rho_w g Q H}{1000} * \eta_o$

Where
ρw = mass density of water, 1000 kg/m3
Q = discharge into turbines
H = head on turbines (≈ available fall)
η0 = overall or plant efficiency

$$P = \frac{\rho_w g Q H}{1000} * \eta_o \Rightarrow P = \frac{1000 * 9.81 * 49.7 * 80}{1000}$$
$$= 27400 kW$$
$$= 27.4 MW$$

4. If the amount of water initially stored is only 125 $M.m^3$, measure *DI* = 125 $M.m^3$, join *BI* and produce to *J*. The slope of the line *BJ* is the maximum possible draw off rate. Let the line *BJ* intersect the ordinate through *O* (*i.e.*, the cumulative inflow axis) at *K*. The vertical intercept *KJ′* = 1430 Mm^3 and the slope of this line = 1430 Mm^3/yr = 45.4 cumec which is the maximum possible draw off rate. To maintain the same reservoir capacity of 170 $M.m^3$, draw the straight line HL || KJ intersecting the mass curve of flow at M and N. Draw the straight line GT || HL. The vertical intercept PM gives the amount of water wasted over the spillway (during the time period MN) which is 40 Mm^3.

5. If the reservoir capacity is limited to 125 M.m3 from economic considerations, the line KJ intersects the mass curve of flow at R. Let the vertical at R meet the line GT (GT || KJ) at S. In this case the amount of water wasted over the spillway = RS = 85 Mm3. The maximum possible output in this case for a uniform draw off rate of 45.4 cumec is
$$P' = 27.4 \frac{45.4}{49.7} = 25 MW$$
6. For a continuous power output of 22.5 MW the uniform draw off rate can be determined from the equation
$$22500 kW = \frac{1000 * 9.81 * Q * 80}{1000} * 0.7$$
$$= 40.8 cumec$$

Which can also be calculated as 49.7 ×(22.5/27.4)= 40.8 cumec = 40.8 (365 × 86400 sec) = 1287 Mm^3/yr. On the 1-year base, draw the ordinate at the end of December = 1287 M.m3 and join the line OQ (dashed line). The slope of this line gives the required draw off rate (40.8 cumec) to produce a uniform power output of 22.5 MW. Through B and D, i.e., the crest and the trough draw tangents parallel to the dashed line OQ (BV || OQ). The vertical intercept between the two tangents DZ gives the required capacity of the reservoir as 100 Mm3.

Reservoir Water Level:

- For short, deep reservoir, the reservoir water surface is level representing a reasonable assumption.
- If flow is passing the dam, there must be some slope to the water surface to cause this flow. When the cross sectional area of the reservoir is large compared with the rate of flow, the velocity will be small and the slope of the hydraulic grade line will be very flat.
- The computation of the water surface profile is an important part of reservoir design since it provides information on the water level at various points along the length of the reservoir from which the land requirements for the reservoir can be determined.

Acquisition of land or flowage rights over the land is necessary before the reservoir can be built. Docks, houses, storm drain outlets, roads, and bridge along the bank of the reservoir must be located above the maximum water level expected in the reservoir.

Recently, computer programs are widely used in reservoir and river modeling such as HEC-RAS, MIKE11, and MIKE21 as well as the geographic information system (GIS).

Storage in reservoir subject to marked backwater effects cannot be related to water surface elevation alone. A second parameter such as inflow rate or water surface elevation on a gage near upper end of the reservoir must also be used. Storage volume under each profile can be computed from cross sections by the methods used for earthwork computations.

Sensors

Are devices that are installed in dams for the purpose of observing the dam during operation and to evaluate in situ conditions for the dam and the surrounding areas:

1. Investigation stage and preliminary designs for the bridge: The devices are used to identify the nature of the bases, and cold, spring water monitoring and groundwater.
2. The implementation phase. Devices to measure movements Caldstovrat. Pore Pressure measuring devices.
3. Operating / hardware erected stage in his shoulders and the body of the dam and the surrounding areas the dam, to inspect movements that occur, changes the pressure and stress, colds.

Types of Sensors Depending on The Purpose of Monitoring Changes

Sensors to guide us to any risks affecting the stability of the dam, comparing reading zero with subsequent readings, and some devices where criteria specified by the designer, for example, any rise in piezometers levels gives indications of a malfunction. There are many types of sensors:

- Monitoring points
- Seepage Meters to measure the amount of seepage
- Piezometers the used to measure groundwater levels in the bases, under the foundations, and the surrounding facilities. They are attached to the dam usually concentrated in selected sections of the dam or expected malfunction in which areas.

Problems of Piezometers Readings:

1-Slow response piezometer / reasons:

- Blockage of the drill.
- Entry of sand from the upper nozzle tube because of the closed well.
- The position of the drill at low permeability clay layer.
- Piezometer away from the tank.
- Defect in the reading device

2- Fixed reading piezometer / reasons:

- Blockage in whole or in part of the drill pipe.
- Blockage of the main pipeline because of stones falling.
- Not change in the level of leaching and this phenomenon is good.

3-Dry piezometer:

- The piezometer on the wrong level or at impermeable layer.
- Clogged pipe because of large stone falling from the nozzle tube

4-Sudden change in the piezometric water level:

- This is important evidence of a change in the seepage paths.
- Piezometer filled with water after heavy rain especially if it has not closed its crater or that he has not placed under the correct specifications.
- Defect in the filters of dam, in many sites wet spots or green will appear downstream of the dam.
- Stop the pump of water from the drainages and flooded closed area.
- Defect in the reading device or pressure sensor.

- Trocar or dig a hole deep close to the site and this hole attracts seepage water and reduce attributed to rapid and substantial.

5- Piezometer influenced by acting work of extra grouting:
The moulder of grouting curtain means increased water seepage through which passers-by. This leads to a rise in the water level in the piezometers downstream and decrease in the levels of piezometers upstream. The efficiency of the grouting curtain depend on many factors, it causes the curtain of a difference in the water level in the piezometers downstream and upstream.

Pore Pressure Cells:

Water pressure that is generated in the foundations certainly affects the pressure of compacted soil. Pore pressure cells are placed at the intersection of the foundations points with compacted soil, it is buried in foundations of dam and end monitor the development of pore pressure during the execution of the dam and after the implementation of the dam, and be connected to wires out of the dam and its peak and readings recorded device go back to the tables recorded readings by device type and company.

Pore Pressure Cells

The pore pressure cells are very important for dam operating monitoring especially in the first storage stages. The reading of pore pressure cells can explain as following:

1. **Cell always read negative read**: usually produces an error in the calibration of the monument for the first time.
2. **Cell to stop reading suddenly and remains constant**: due to the sudden damage because of the cell pressure exceeds the design pressure or cut the wire carrier for any reason, or bad setup.
3. **Cell erected in the area of thick and impermeable layer**. This phenomenon may be a sign of quality clay core in the dam or that the implementation of the core of dam was very well led to prevent the seepage from this region.

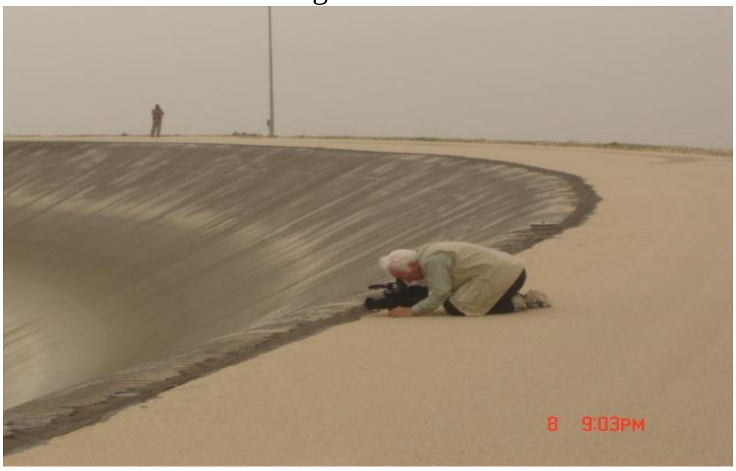
Stress Meter

It is used to measure various stresses inflicted on earth fill and rocky foundations through the construction of the dam in the first years of operation. Common type of stress meter is known as distofor which usually is setup inside a pilot hole (borehole) to monitor any cracks in the foundation.

Seismic Monitoring Devices:

They are used for recording seismic events, which lies near the dam area. There are two types of these devices:

1. ***High sensitivity*** it is used to measure and record the exact events of seismic devices (micro seismic), which is possible to occur before the creation of the dam and the associated seismic events in the region. Then measure the possible seismic events occurring after the creation of the dam and spelled.
2. ***Strong motion:*** It is used to record events seismic strength of the medium and strong levels when they occur within the limits of the ground accelerating on-site. It is setup in different locations of the dam, at the crest, in the foundations, and in the outlet. It is integral system, record and sends the events that are recorded to a central recording station.



Seismic Monitoring Device Setup

Earthquake seismic structural monitors

Internal Deformation Meter:

To measure the internal landing slips in foundations (measuring device slips inclinometer).

Automatic multi-purpose dam deformation monitoring system

Joint Meter:

Measuring cracks in the joints of the concrete and other movements that occur.

Inverted Pendulum:

It is used for the purpose of measuring the deviation in the origin column to the right or to the left.

Sequent Peak Algorithm

This algorithm computes the cumulative sum of differences between the inflows and reservoir releases for all period's t over the time interval [0, T]. Let Kt be the maximum total storage requirement needed for periods 1 through period t and Rt be the required release in period t, and Qt be the inflow in that period. Setting $K0$ equal to 0, the procedure involves calculating Kt using equation below for upto twice the total length of record. Algebraically,

$$K_t = \begin{cases} R_t - Q_t + K_{t-1} \Rightarrow posative \\ 0 \Rightarrow otherwise \end{cases}$$

The maximum of all Kt is the required storage capacity for the specified releases Rt and inflows, Qt.

Formulation of reservoir sizing using LP

Linear Programming can be used to obtain reservoir capacity more elegantly by considering variable demands and evaporation rates. The optimization problem is Minimize Ka where Ka is the active storage capacity Subject to

(1) Hydraulic constraints as defined by the reservoir continuity equation

$$S_{t+1} = S_t + I_t - EV_t - R_t - Q_t \quad \text{For all t}$$

(2) Reservoir capacity

$$S_t \le K_a \qquad \text{For all t}$$

$$S_{T+1} = S_t \quad \text{Where T is the last period}$$

(3) Target demands

$$R_t \ge D_t \quad \text{For all t}$$