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$$Q = \left(\frac{K}{A}\right) A.H. \qquad ...(1)$$

For fine sand, $\frac{K}{A} = 0.5$ m³/hour per m² of area under unit depression head

H = depression head = 2.5 m.

Substituting values in (1), we get

$$10.8 = 0.5 \times A \times 2.5$$

$$\therefore \quad A = \frac{10.8}{0.5 \times 2.5} \text{ m}^2 = 8.64 \text{ m}^2$$

$$\therefore \quad \text{Well diameter} \quad d = \sqrt{\frac{4A}{\pi}} = \sqrt{\frac{4 \times 8.64}{\pi}} = 3.32 \text{ m} \quad \textbf{(say 3.4 m)}$$

Alternative Solution

Another method to find the well diameter is to use the discharge formula Eq. 5.20 in terms of percolation coefficient. From Eq. 5.20,

$$Q = C.A.H. \qquad ... (2)$$

From Table 5.2, percolation or permeability constant for fine sand can be taken equal to 0.0075 cm/sec. (average)

$$\therefore \quad C = 0.0075 \text{ cm/sec.} = 7.5 \times 10^{-5} \text{ m/sec.}$$

$$Q = 0.003 = 3 \times 10^{-3} \text{ m}^3\text{/sec.}$$

$$H = 2.5 \text{ m}$$

A = percolation area

$$\therefore \quad A = \frac{Q}{CH} = \frac{3 \times 10^{-3}}{7.5 \times 10^{-5} \times 2.5} = 16 \text{ m}^2$$

If no cavity is formed at the bottom, $A = \frac{\pi}{4} d^2$

If full hemispherical cavity is formed, $A = \frac{\pi d^2}{2}$

Usually the area of cavity is between the two values

$$\therefore \quad A = \frac{1}{2}\left(\frac{\pi}{4} d^2 + \frac{\pi d^2}{2}\right) = \frac{3}{8} \pi d^2 \simeq \frac{1}{3} \pi d^2.$$

Thus the area of percolation at the well bottom may be taken equal to 4/3 times the cross-sectional area of well bottom $\left(i.e., \frac{\pi}{3} d^2 = \frac{4}{3} \times \frac{\pi}{4} d^2\right)$

$$\therefore \quad \frac{\pi}{3} d^2 = 16$$

or

$$d = \sqrt{\frac{16 \times 3}{\pi}} = \mathbf{3.9\ m.}$$

Example 5.2. *During a recuperation test, the water in an open well was depressed by pumping by 2.5 metres and it recuperated 1.8 metres in 80 minutes. Find (a) yield from a well of 4 m diameter under a depression head of 3 metres, (b) the diameter of the well to yield 8 litres/second under a depression head of 2 metres.*

Solution

From Eq. 5.21, the specific yield is given by

$$\frac{K}{A} = \frac{2.303}{T} \log_{10} \frac{h_1}{h_2}$$

where T = time in hours $= \frac{80}{60} = \frac{4}{3}$ hours

$h_1 = 2.5$ m

$h_2 = 2.5 - 1.8 = 0.7$ m

$$\therefore \quad \frac{K}{A} = \frac{2.303 \times 3}{4} \log_{10} \frac{2.5}{0.7} = 0.955.$$

(*a*) Yield from the well of diameter 4 m :

$$A = \frac{\pi}{4}(4)^2 = 12.56 \text{ m}^2$$

$$Q = \left(\frac{K}{A}\right) A.H. = 0.955 \times 12.56 \times 3 = 36 \text{ m}^3\text{/hour} = \mathbf{10\ lit/sec.}$$

(*b*) Yield = 8 lit/sec = 28.8 m³/hour (since 1 lit/sec = 3.6 m³/hour)

$$\therefore \quad Q = \left(\frac{K}{A}\right) A.H.$$

$$\therefore \quad A = \frac{Q}{H} \times \left(\frac{A}{K}\right) = \frac{28.8}{2} \times \frac{1}{0.955} = 15.08 \text{ m}^2$$

$$\therefore \quad d = 4.38 = \mathbf{4.4\ m\ (say).}$$

Example 5.3. *A tube well of 30 cm diameter penetrates fully in an artesion aquifer. The strainer length is 15 m. Calculate the yield from the well under a drawdown of 3 m. The aquifer consists of sand of effective size of 0.2 mm having coefficient of permeability equal to 50 m/day. Assume radius of drawdown equal to 150 metres.*

Solution

From Eq. 5.6(*c*), the discharge is given by

$$Q = \frac{2.72\, b\, k\, s}{\log_{10} \frac{R}{r}}$$

where b = thickness of aquifer = length of strainer = 15 m

s = drawdown = 3 m

k = coefficient of permeability = 50 m/day

R = 150 m

r = 15 cm = 0.15 m

$$\therefore \quad Q = \frac{2.72 \times 15 \times 50 \times 3}{\log_{10} \frac{150}{0.15}} \text{ m}^3\text{/day}$$

$$= 2040 \text{ m}^3\text{/day} = 85 \text{ m}^3\text{/hour} = \mathbf{23.6\ lit/sec.}$$

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Example 5.4. *A tube well penetrates fully an unconfined aquifer. Calculate the discharge from the tube well under the following conditions :*

Diameter of the well	= *30 cm*
Drawdown	= *2 m*
Effective length of the strainer under the above drawdown	= *10 m*
Coefficient of permeability of aquifer	= *0.05 cm/sec.*
Radius of zero drawdown	= *300 m.*

Solution

From Eq. 5.5, discharge is given by

$$Q = \frac{2.72\,ks\left(L + \frac{s}{2}\right)}{\log_{10}\frac{R}{r}}$$

where $k = 0.05$ cm/sec $= \frac{0.05}{100}$ m/sec $= 5 \times 10^{-4}$ m/sec

$s =$ drawdown $= 2$ m

$L =$ effective length of strainer $= 10$ m

$R = 300$ m ; $r = 0.15$ m

$$\therefore \quad Q = \frac{2.72 \times 5 \times 10^{-4} \times 2(10+1)}{\log_{10}\frac{300}{0.15}} \text{ m}^3\text{/sec.} = 9.06 \times 10^{-3} \text{ m/sec}$$

$$= \mathbf{0.0091 \text{ cumec} = 9.06 \text{ lit/sec.}}$$

Example 5.5. *Design a tube well for the following data :*

(*i*)	*Yield required*	= *0.08 cumec*
(*ii*)	*Thickness of confined aquifer*	= *30 m*
(*iii*)	*Radius of circle of influence*	= *300 m*
(*iv*)	*Permeability coefficient*	= *60 m/day*
(*v*)	*Drawdown*	= *5 m.*

Solution

From Eq. 5.6(*c*), we have

$$Q = \frac{2.72\,bks}{\log_{10}\frac{R}{r}}$$

Here $Q = 0.08$ cumec $= 0.08$ m³/sec ; $b = 30$ m ; $s = 5$ m

$k = 60$ m/day $= \frac{60}{3600 \times 24}$ m/sec

$R = 300$; $r = ?$

Hence
$$0.08 = \frac{2.72 \times 30 \times 5}{\log_{10}\frac{R}{r}} \times \frac{60}{3600 \times 24}$$

$$\therefore \quad \log_{10}\frac{R}{r} = \frac{2.72 \times 30 \times 5 \times 60}{3600 \times 24 \times 0.08} = 3.5417$$

$$\therefore \quad \frac{R}{r} = \text{antilog } 3.5417 = 3480$$

$$\therefore \quad r = \frac{R}{3480} = \frac{3000}{3480} = 0.0862 \text{ m.}$$

Hence adopt well of 9 cm diameter.

Example 5.6. *An artesian tube well has a diameter of 20 cm. The thickness of aquifer is 30 m and its permeability is 38 m/day. Find its yield under a drawdown of 4 m at the well face. Use radius of influence as recommended by Sichardt.*

Solution

From Eq. 5.6, we have

$$Q = \frac{2.72\,bks}{\log_{10}\frac{R}{r}}$$

Here $b = 30$ m ; $s = 4$ m ; $r = 0.10$ m

$k = 36$ m/day $\left(= \frac{36}{24 \times 3600} \text{ m/sec}\right)$

$$R = 3000\ s\sqrt{k} = 3000 \times 4\sqrt{\frac{36}{24 \times 3600}} = 245 \text{ m.}$$

$$\therefore \quad Q = \frac{2.72 \times 30 \times 36 \times 4}{\log_{10}\frac{245}{0.1}}$$

$$= 3467 \text{ m}^3\text{/day} = \mathbf{144.5\ m^3/hour}$$

$$= \mathbf{40.1 \text{ lit/sec (or } 0.04 \text{ cumec)}}$$

Example 5.7. *A well penetrates fully a 10 m thick water bearing stratum of medium sand having coefficient of permeability of 0.005 m/sec. The well radius is 10 cm and is to be worked under a drawdown of 4 m at the well face. Calculate the discharge from the well. What will be the percentage increase in the discharge if the radius of the well is doubled ?*

Take $R = 300$ *m in each case.*

Solution

From Eq.5.6 (*c*), we have

$$Q = \frac{2.72\,bks}{\log_{10}\frac{R}{r}} = \frac{2.72 \times 10 \times 0.005 \times 4}{\log_{10}\frac{300}{r}} = \frac{0.544}{\log_{10}\frac{300}{r}}$$

(*i*) In the first case $r = 10$ cm $= 0.1$ m.

$$Q_1 = \frac{0.544}{\log_{10}\frac{300}{0.1}} = \frac{0.544}{3.477} = \mathbf{0.1565 \text{ cumec}}$$

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(*ii*) In the second case, $r = 20$ cm $= 0.2$ m.

$$Q_2 = \frac{0.544}{\log_{10}\frac{300}{0.2}} = \frac{0.544}{3.176} = 0.1713 \text{ cumec}$$

$$\therefore \text{ \% increase in discharge } = \frac{0.1713 - 0.1565}{0.1565} \times 100 = 9.5\%.$$

Thus the size of the well has very little influence on the discharge. By doubling the well diameter, the discharge has increased by 9.5% only.

Example 5.8. *In order to determine the field permeability of a free aquifer, pumping out test was performed and following observations were made :*

Diameter of well = 20 cm

Discharge from the well = 240 m^3/hour

R.L. of original water surface, before pumping started = 240.5 m

R.L. of water in the well at constant pumping = 235.6 m

R.L. of the impervious layer = 210 m

R.L. of water in observation well = 239.8 m.

Radial distance of observation well from the tube well = 50 m

Calculate k. Also calculate (i) the error in k if observations are not taken in the observation well, and radius of influnence is assumed to be 300 m, (ii) actual radius of influence based on the observations of observation well.

Solution. (Refer Fig. 5.7)

(*a*) From Eq. 5.4 (*b*), we have

$$Q = \frac{1.36\, k\,(h_2^2 - h_1^2)}{\log_{10}\frac{r_2}{r_1}}$$

Using suffix 1 for the well face, and suffix 2 for the observation well, we have

$r_1 = 10$ cm $= 0.1$ m ; $r_2 = 50$ m

$h_1 = 235.6 - 210 = 25.6$ m ; $h_2 = 239.8 - 210 = 29.8$ m

$\therefore$ $(h_1 + h_2) = 29.8 + 25.6 = 55.4$ m ; $(h_2 - h_1) = 29.8 - 25.6 = 4.2$ m.

$Q = 240$ m^3/hour.

$$\therefore \quad 240 = \frac{1.36 \times k\,(55.4 \times 4.2)}{\log_{10}\frac{50}{0.1}}$$

or

$$k = \frac{240}{1.36 \times 55.4 \times 4.2}\log_{10} 500 = 2.047 \text{ m/hour} = 49.13 \text{ m/day} \quad ...(a)$$

(*b*) If k is computed on the basis of assumed radius of influence we have, from Eq. 5.4,

$$Q = \frac{1.36\, k\,(H^2 - h^2)}{\log_{10}\frac{R}{r}}$$

Here $\frac{R}{r} = \frac{300}{0.1} = 3000$

$H = 240.5 - 210 = 30.5$ m ; $h = 235.6 - 210 = 25.6$ m

$\therefore$ $H + h = 30.5 + 25.6 = 56.1$ m ...(*b*)

$H - h = 30.5 - 25.6 = 4.9$ m

$$240 = \frac{1.36\, k(56.1)(4.9)}{\log_{10} 3000}$$

$$\therefore \quad k = \frac{240}{1.36 \times 56.1 \times 4.9}\log_{10} 3000 = 2.236 \text{ m/hour} = 53.67 \text{ m/day}$$

$$\therefore \quad \text{\% error } = \frac{53.67 - 49.15}{49.15} \times 100 = 9.1\ \%.$$

(*c*) Calculation of actual radius of influence :

From Eq. 5.4,

$$Q = \frac{1.36\, k(H^2 - h^2)}{\log_{10}\frac{R}{r}}$$

where $k = 2.047$ m/hour [From (*a*)]

$H + h = 56.1$ m [From (b)]

$H - h = 4.9$ m

$Q = 240$ m^3/hour.

$$\therefore \quad 240 = \frac{1.36 \times 2.047\,(56.1)(4.9)}{\log_{10}\frac{R}{r}}$$

$$\therefore \quad \log_{10}\frac{R}{r} = \frac{1.36 \times 2.047 \times 56.1 \times 4.9}{240} = 3.186$$

$$\therefore \quad \frac{R}{r} = 1544$$

$$\therefore \quad R = 1544 \times r = 1544 \times 0.1$$

or $R \triangleq$ **154 m.**

Example 5.9. *A tube well commands 20 hectares of land under rice. When the well is discharging at a constant rate, the water level in the well drops to 120.2 m. The level to the highest land is 129 m. The duty for rise is 800 hectares/cumec on the field. Taking the pump efficiency as 60 % calculate the input h.p. of the pump.*

Solution

Discharge required $= \frac{20}{800} = 0.025$ cumec.

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∴ Weight of water lifted per second = 0.025 × 1000 kg = 25 kg.

Minimum static lift of the pump = 129 – 120.2 = 8.8 m

Taking 0.2 m head required for the water to flow from the well to the field, design lift = 9 m (say).

∴ Work done by pump in lifting the water = 9 × 25 = 225 kg-m/sec.

∴ Now 1 h.p.(metric) = 75 kg-m/sec.

∴ Output h.p. of pump $= \frac{225}{75} = 3$

∴ Input h.p. pump $= \frac{3}{\eta \text{ of pump}} = \frac{3}{0.6} =$ **5 h.p.**

Example 5.10. *The discharge available from a tube well is 150 m³/hour. Assuming 4000 hours of working for a tube well in a year, estimate the culturable area that this tube well can command. The intensity of irrigation is 45 % and averge depth of Rabi and Kharif crop is 38 centimetres.*

Solution

Volume of water pumped = 150 × 4000 m³ = 60 × 10⁴ cubic metres.

Average depth of water = 38 cm = 0.38 m

∴ Area of irrigation $= \frac{60 \times 10^4}{0.38} = 1.58 \times 10^6 \text{ m}^2$

Culturable commanded area $= \frac{1.58 \times 10^6}{0.45} = 3.51 \times 10^6 \text{ m}^2 =$ **351 hectares.**

Example 5.11. *Two tube wells, each of 20 cm diameter are spaced at 100 m distance. Both the wells penetrate fully a confined aquifer of 12 m thickness. Calculate the discharge if only one well is discharging under a depression head of 3 m. What will be the percentage decrease in the discharge of the well if both the wells are discharging under the depression head of 3 m. Take radius of influence for each well equal to 250 metres, and coefficient of permeability of aquifer as 60 metres/day.*

Solution. (Refer Fig. 5.12)

In the first case, when only one well is discharging, the discharge is given by Eq. 5.6 (*c*),

$$Q = \frac{2.72\, b\, k\, s}{\log_{10} \frac{R}{r}}$$

where $b = 12$ m ; $s = 3$ m ; $k = 60$ m/day

$$\log_{10} \frac{R}{r} = \log_{10} \frac{250}{0.1} \approx 3.398$$

∴ $$Q = \frac{2.72 \times 12 \times 60 \times 3}{3.398} \text{ m}^3/\text{day}$$

$$= 1729 \text{ m}^3/\text{day} \simeq \mathbf{72.0\ m^3/hour.}$$

When both the wells are discharging, the discharge from each well is given by Eq. 5.4.

$$Q_1 = Q_2 = \frac{2\pi kb\,(H-h)}{\log_e \frac{R^2}{rB}}$$

$$= \frac{2.72\, k\, b\, s}{\log_{10} \frac{R^2}{rB}}$$

$$= \frac{2.72 \times 60 \times 12 \times 3}{\log_{10}\left(\frac{250 \times 250}{0.1 \times 100}\right)}$$

$$= \frac{2.72 \times 60 \times 12 \times 3}{3.796} = 1548 \text{ m}^3/\text{day}$$

$$= \mathbf{64.5\ m^3/hour.}$$

∴ Percentage decrease in the discharge

$$= \frac{72 - 64.5}{72} \times 100 = 10.43\ \%.$$

Example 5.12. *A gravity well has a diameter of 60 cm. The depth of water in the well is 40 metres before pumping is started. When pumping is being done at the rate of 2000 litres per minute, the drawdown in a well 10 metres away is 4 metres and in another well 20 metres away is 2 metres. Determine (a) radius of zero drawdown, (b) coefficient of permeability, (c) drawdown in the well, (d) specific capacity of the well, (e) maximum rate at which water can be pumped from the well.*

Solution

(*a*) For a tube well in unconfined aquifer, we have from Eq. 5.4,

$$Q = \frac{1.36\, k\,(H^2 - h^2)}{\log_{10} \frac{R}{r}}$$

Here, we have $H = 40$; At $r = 10$ m, $h = 40 - 4 = 36$ m.

At $r = 20$ m, $h = 40 - 2 = 38$ m.

Applying the above equation at these two locations, we have

$$\frac{40^2 - 36^2}{\log_{10} \frac{R}{10}} = \frac{(40^2 - 38^2)}{\log_{10} \frac{R}{20}}$$

or $$\log_{10} \frac{R}{20} = \frac{(40^2 - 38^2)}{40^2 - 36^2} \log_{10} \frac{R}{10} = 0.5132 \log_{10} \frac{R}{10}$$

or $$\log_{10} \frac{R}{20} = \log_{10} \left(\frac{R}{10}\right)^{0.5132}$$

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or $$\frac{R}{20} = \frac{R^{0.5132}}{3.2599}$$

$$\therefore \quad R^{1-0.5132} = \frac{20}{3.2599} = 6.1352$$

$$\therefore \quad R = (6.1352)^{\frac{1}{0.4868}} = (6.1352)^{2.0542}$$

$$\therefore \quad R = \mathbf{41.52\ m}$$

(*b*) $Q = 2000$ litres / min $= 2$ m³/min (since 1 litres/min = 0.001/m³/min).

$R = 41.52$ m ; $H = 40$ m,

$h = 36$ m ; $r = 10$ m

$$\therefore \quad Q = 2 = \frac{1.36\,k(40^2 - 36^2)}{\log_{10}\frac{41.52}{10}}$$

$$\therefore \quad k = \frac{2}{1.36\,(40^2 - 36^2)} \log_{10}\frac{41.52}{10} = 0.003 \text{ m/min} = \mathbf{4.31\ m/day.}$$

(*c*) Depth of water in the well is given by

$$2 = \frac{1.36 \times 0.003\,(40^2 - H_o^2)}{\log_{10}\frac{41.52}{0.30}}$$

$$\therefore \quad 1600 - H_o^2 = \frac{2}{1.36 \times 0.003} \times \log_{10}\frac{41.52}{0.30} = 1049.58.$$

$$\therefore \quad H_o = 23.46 \text{ m}$$

Hence drawdown at the well $= 40 - 23.46 = \mathbf{16.54\ m.}$

(*d*) ***Specific Capacity***. It is defined as the discharge per unit drawdown. Let it be designated by S_c. It is not contant, but decreases as the discharge increases. Let us assume that the yield is directly proportional to the drawdown or to the radius of zero-drawdown.

$$\therefore \quad Q \propto R$$

or $$Q = CR \text{ where } C \text{ is a constant.}$$

For the given data,

$$C = \frac{Q}{R} = \frac{2}{41.52} = 0.04817.$$

$$\therefore \quad R = \frac{Q}{C} = \frac{Q}{0.04817} \text{ in general.}$$

Now corresponding to drawdown of 1 m, discharge $Q = S_c$.

Hence radius of zero drawdown $= \frac{S_c}{C} = \frac{S}{0.04817}$

Hence $$Q = S_c = \frac{1.36k\,(40^2 - 39^2)}{\log_{10}\frac{R}{r}} = \frac{1.36 \times 0.003\,(40^2 - 39^2)}{\log_{10}\left(\frac{S_c}{0.04817 \times 0.3}\right)}$$

or $$S_c \log_{10}(61.2\ S_c) = 0.3223.$$

Solving this by trial and error, we get

$$S_c = 0.266 \text{ m}^2/\text{min.}$$

Hence, specific capacity of the well is **0.266 m³ / min / m** depression head.

(*e*) Maximum rate of discharge Q_m will be obtained when drawdown in the well is equal to H, *i.e.*, when H_o = zero.

$$Q_m = \frac{1.36 \times 0.003\,(40^2 - 0^2)}{\log_{10}\frac{Q_m}{0.04817 \times 0.3}}$$

or $$Q_m \log_{10} 69.2\ Q_m = 6.528.$$

Solving this by trial and error, we get

$$Q_m = \mathbf{2.85\ m^3/min.}$$

5.21. OTHER SOURCES OF UNDER GROUND WATER

The ground water can be tapped with the help of the following sources :

1. Wells.
2. Springs.
3. Infilteration galleries.
4. Karez.

Out of these, well have been discussed at length in the previous articles. We shall now discuss in brief the other sources of ground water.

Springs

Ground water which flows out at the surface constitutes a spring. A pervious layer, sandwiched between two impervious layers, give rise to a natural spring. A spring indicates the outcropping of the water table. Springs are of the following types :

1. Stratum spring.
2. Valley springs.
3. Fault springs.
4. Springs from solution channels.
5. Artesian spring (geyser).
6. Deep seated spring.

Stratum spring is formed when the downward passage of ground water in a permeable deposit is hindered by an underlying impervious layer. The ground water emerges as a spring at the contact of the two formations, and hence such spring is also known as *contact spring* (Fig. 5.35). Another kind of stratum spring may arise at the junction of two formations where permeable beds dip beneath an impermeable cover. This may be called an *overflow spring*, since the ground water literally overflows at the edge at the impervious stratum (Fig. 5.36).

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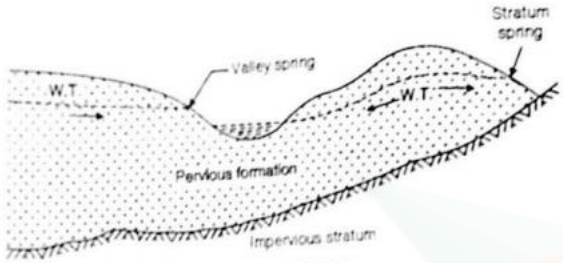

FIG. 5.35. SPRINGS

A ***Valley spring*** is due to the emergence of the water table where the topography falls below its level such as that shown in Fig. 5.35. Water flows out at the surface and feeds the surface stream.

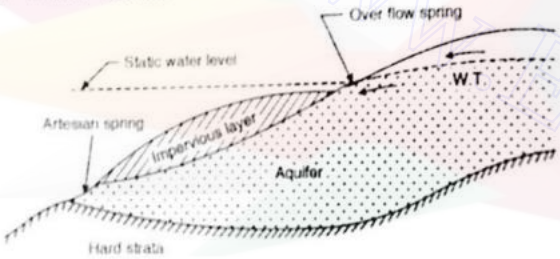

FIG. 5.36. SPRING.

When a permeable formation is brought against an impermeable one by faulting, ground water in the permeable bed may escape along the fault line and appear at the surface as a ***fault spring*** (Fig. 5.37).

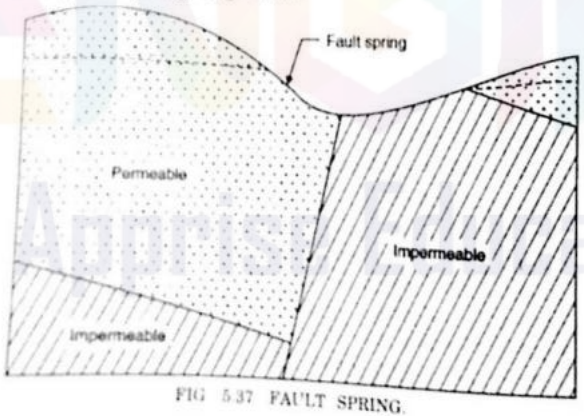

FIG. 5.37. FAULT SPRING.

In limestone district where there is much underground water in circulation, streams flow along bedding planes in channels enlarged by solutions, and may emerge as springs in valleys, to feed surface stream.

Artesian spring arise where water is confined in permeable rocks under artesian head beneath a cover of impervious layer (Fig. 5.36). If this cover is locally broken by fracturing, or if the artesian aquifer outcrops, the artesian water below emerges as an artesian spring. Artesian springs, also known as *geysers*, are of infrequent occurrence.

The ***deep seated springs*** are associated with vulcanicity and deep fractures, and yield juvenile water and not the rainwater.

Infiltration Galleries. It is a small tunnel in the form of pipes laid under the ground to tap up underground flow available at moderate depth. These tunnels could be of wood, stone or bricks. Their shape is generally circular, and they are covered

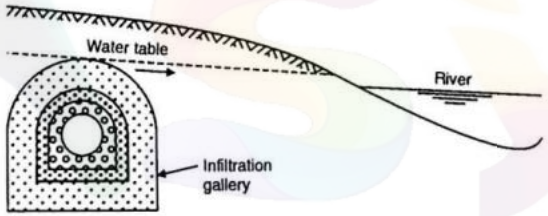

FIG. 5.38. INFILTERATION GALLERY.

by graded filters all round. The ground water can enter the tunnel through side and bottom. These galleries are often constructed parallel to a river bank to collect the inflow of ground water to river. Fig. 5.38 shows a typical infilteration gallery surrounded by a three layer graded filter consisting of :

(*i*) 16 cm ballast (25 to 50 mm size) around the pipe,

(*ii*) 16 cm large pebbles (12 to 24 mm) as intermediate layer and

(*iii*) 24 cm fine pebbles (3 to 10 mm size) as the outer layer.

Karez

A Karez is an underground tunnel driven into the hill side to tap water from the underground spring. The tunnel has certain bed inclination. Water from the Karez

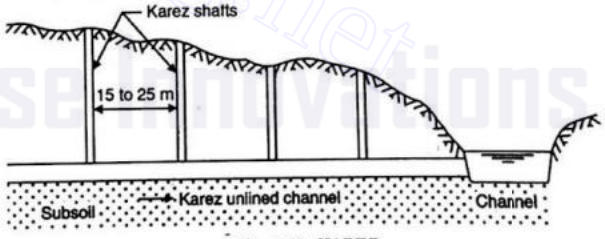

FIG. 5.39 KAREZ.

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is taken in an open channel. Such tunnels are in use in Baluchistan and North West Frontier provinces (West Pakistan).

5.22. UNSTEADY FLOW TOWARDS WELLS*

The analysis of flow towards well discussed earlier is based on the assumption that steady state of flow is developed immediately after pumping is started. Actually, the cone of depression fluctuates with time. The gradual approach of cone of depression towards a steady state is produced primarily by the removal of water from storage as cone deepens. Hence, a *storage coefficient* comes into play. The storage coefficient S is a dimensionless constant of the aquifer and may be interpreted as the amount of water in storage *released from storage* from a column of aquifer of unit cross-section under a unit decline of head.

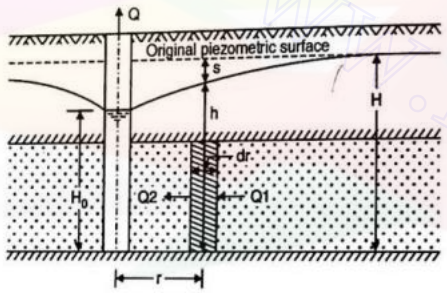

FIG. 5.40. UNSTEADY FLOW.

Equations developed for unsteady or transient well flow normally show how the drawdowns of the piezometric surface or water table is related to time of pumping the well. In Fig. 5.40, consider an annular cylinder of thickness dr and radius r. Due to unsteady flow conditions, water will be released from storage of this elementary cylinder. According to definition of storage coefficient S, the rate $\frac{\partial V}{\partial t}$ at which a certain volume V is released from storage of an aquifer of area A is given by

$$\frac{\partial V}{\partial t} = -\frac{\partial h}{\partial t} S.A \qquad ...(1)$$

where V = volume of water released per horizontal area A of aquifer.
h = height of piezometric surface or water table above lower boundary of aquifer.
S = storage co-efficient.
A = area of the aquifer to which $\frac{\partial V}{\partial t}$ applies.
t = time.

*Junior students may omit this article.

Since h decreases with time t, minus sign with $\partial h/\partial t$ has been used. For the elementary cylinder of the aquifer, $A = 2\pi r.dr$.

Hence
$$\frac{\partial V}{\partial t} = -2\pi r\, dr\,.\, S\frac{\partial h}{dt} \qquad ...(2)$$

Let Q_1 = discharge entering the outer face of the cylinder
Q_2 = discharge leaving the inner face of the cylinder.
$\therefore$ Change in discharge $\delta Q = Q_1 - Q_2$.

Since the rate of increase in Q can be expressed as $-\frac{\partial Q}{\partial r}$, the increase in discharge over the annular area $= -\frac{\partial Q}{\partial r}dr$. Substituting this for $\frac{\partial V}{\partial t}$ in (2), we get

$$-\frac{\partial Q}{\partial r}dr = -2\pi r\, dr\, S.\frac{\partial h}{\partial t} \text{ or } \frac{\partial Q}{\partial r} = 2\pi r S\frac{\partial h}{\partial t} \qquad ...(3)$$

For confined radial flow, we get from Darcy law,

$$Q = 2\pi r T\frac{\partial h}{\partial r}$$

$$\therefore \quad \frac{\partial Q}{\partial r} = 2\pi T\frac{\partial}{\partial r}\left(r\frac{\partial h}{\partial r}\right) = 2\pi T\left(\frac{\partial h}{\partial r} + r\frac{\partial^2 h}{\partial r^2}\right) \qquad ...(4)$$

Substituting in (3), we get

$$2\pi T\left(\frac{\partial h}{\partial r} + r\frac{\partial^2 h}{\partial r^2}\right) = 2\pi r.S\frac{\partial h}{\partial t}$$

which simplifies to
$$\frac{\partial^2 h}{\partial r^2} + \frac{1}{r}\frac{\partial h}{\partial r} = \frac{S}{T}\cdot\frac{\partial h}{\partial t} \qquad ...(5.24)$$

This is the basic equation for unsteady flow towards the well. Theis (1935) obtained a solution for this equation based on the analogy between ground water flow and heat conduction, by assuming that the well is replaced by a mathematical sink of constant strength such that $h = H$ before pumping begins and that $h \to H$, as $r \to \infty$ after pumping begins ($t \geq 0$). The solution is

$$H - h = \frac{Q}{4\pi T}\int_{u=\frac{r^2 S}{4Tt}}^{\infty}\frac{e^{-u}}{u}du \qquad ...[5.25(a)]$$

or
$$H - h = s = \frac{Q}{4\pi T}W(u) \qquad ...(5.25)$$

where
$$W(u) = \int_u^{\infty}\frac{e^{-u}}{u}du = \text{well function.}$$

Eq. 5.25 is known as the *non-equilibrium equation* or *Theis equation*. The integral in the above equation is a function of the lower limit, and is known as exponential integral. It can be expressed as a convergent series so that Eq. 5.25 can also be written as

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$$s = \frac{Q}{4\pi T}\left[-0.5772 - \log_e u + u - \frac{u^2}{2.2!} + \frac{u^3}{3.3!} \ldots\ldots\right] \quad \ldots[5.25(b)]$$

Wenzel tabulated the values of $W(u)$ for various values of u ranging from 10^{-15} to 9.9. Table 5.2 gives the values of $W(u)$ for u ranging from 10^{-15} to 9.

TABLE 5.2. VALUES OF WELL FUNCTION $W(u)$

$(u = N \times 10^n)$

$N =$ ↓ / $u =$ →	$N \times 10^{-15}$	$N \times 10^{-14}$	$N \times 10^{-13}$	$N \times 10^{-12}$	$N \times 10^{-11}$	$N \times 10^{-10}$	$N \times 10^{-9}$	$N \times 10^{-8}$
1	33.96	31.66	29.36	27.05	24.75	22.45	20.15	17.84
2	33.27	30.97	28.66	26.36	24.06	21.76	19.45	17.15
3	32.86	30.56	28.26	25.96	23.65	21.35	19.05	16.75
4	32.57	30.27	27.97	25.67	23.36	21.06	18.76	16.46
5	32.35	30.05	27.75	25.44	23.14	20.84	18.54	16.23
6	31.17	29.87	27.50	22.26	22.96	20.66	18.35	16.05
7	32.02	29.71	27.40	25.11	22.81	20.50	18.20	15.90
8	31.88	29.58	27.28	24.97	22.87	20.37	18.07	15.76
9	31.76	29.46	27.26	24.86	22.55	20.25	17.95	15.65

$(u = N \times 10^n)$

$N =$ ↓ / $u =$ →	$N \times 10^{-7}$	$N \times 10^{-6}$	$N \times 10^{-5}$	$N \times 10^{-4}$	$N \times 10^{-3}$	$N \times 10^{-2}$	$N \times 10^{-1}$	N
1	15.54	13.24	10.94	8.633	8.332	4.038	1.823	0.2194
2	14.85	12.55	10.24	7.940	5.639	3.355	1.223	0.04891
3	14.44	12.14	9.837	7.535	5.235	2.959	0.906	0.01315
4	14.15	11.85	9.550	7.247	4.948	2.681	0.702	0.003779
5	13.93	11.68	9.326	7.024	4.726	2.468	0.560	0.001148
6	13.75	11.45	9.144	6.842	4.545	2.295	0.454	0.000360
7	13.60	11.29	8.990	6.688	4.392	2.151	0.374	0.000116
8	13.46	11.17	8.856	6.555	4.259	2.027	0.311	0.0000377
9	13.34	11.04	8.739	6.437	4.142	1.919	0.260	0.0000125

From the Table,

When $u = 5$, $W(u) = 0.001148$; and when $u = 5 \times 10^{-3}$, $W(u) = 4.726$.

The values of *formation constant* S and T can be found by measuring drawdowns in observation wells when the well under study is pumped at a constant rate of discharge Q. There are several methods of determining S and T, but we will discuss here following three methods :

(*a*) Theis method,
(*b*) Jacob's method,
(*c*) Chow's method.

(*a*) THEIS METHOD

Theis proposed a curve-fitting method for finding formation constant S and T from a pumping out test. From Eq. 5.25, we observe that

$$s = \left[\frac{Q}{4\pi T}\right] W(u) \text{ or } \log W(u) = \left[\log \frac{4\pi T}{Q}\right] + \log s \quad \ldots[5.26(a)]$$

and
$$\frac{r^2}{t} = \left[\frac{4T}{S}\right] u \quad \text{or} \quad \log u = \left[\log \frac{S}{4T}\right] + \log \frac{r^2}{t} \quad \ldots[5.26(b)]$$

If a constant withdrawal rate (Q) is maintained, the bracketed portions of the above two equations are constant for a given pumping test. It is to be noted that s is related to r^2/t in a manner that is similar to the relation of $W(u)$ to u. Hence, if a plot of *data curve* is made between s and r^2/t on logarithmic co-ordinates tracing paper [Fig. 5.41(*a*)] to the same scale as the *type curve* $W(u)$ versus u [Fig. 5.41(*b*)] the data curve will be similar to the type curve.

Procedure

(*i*) In the *observation well* situated at a radial distance r from the main well, observe s and t.

(*ii*) Plot s versus r^2/t on a log-log tracing paper [Fig. 5.41(*a*)].

(*iii*) Plot $W(u)$ versus u on log-log graph paper [Fig. 5.41(*b*)].

(*iv*) Keep Fig. 5.41(*a*) on Fig. 5.41(*b*), and adjust in such a way that when the corresponding co-ordinate axes are held parallel, the data curve is oriented in position which represents the best fit of the field data to the type curve, [Fig. 5.41(*c*)].

(*v*) With both graph sheets at the best match position, an arbitrary point P is selected on the type curve and pricked through.

(*vi*) The co-ordinates (a, b) and (a_1, b_1) of the match point are noted from both the curves.

Thus, the pair of values $[s, W(u)]$ and $(r^2/t, u)$ are known. The aquifer constants are then calculated from the relation

$$T = \frac{Q}{4\pi s} W(u) \quad \ldots[5.27(a)]$$

and
$$S = \frac{4uT}{r^2/t} \quad \ldots[5.27(b)]$$

(*b*) JACOB'S METHOD

Jacob suggested method which completely avoids curve fitting. He observed that when r is *small* and t is large, u may be small and hence all terms after 2nd term of the series expansion [Eq. 5.25(*b*)] of $W(u)$ may be neglected.

Thus
$$W(u) = -0.5772 - \log_e u$$

$$\therefore \quad s = \frac{Q}{4\pi T}\left[-0.5772 - \log_e u\right]$$

or
$$s = \frac{Q}{4\pi T}\left[\log_e \frac{4Tt}{r^2 S} - 0.5772\right] \quad \ldots(5.28)$$

For the same observation well, if s_1 and s_2 are the observed drawdowns at times t_1 and t_2 since pumping started, we have

$$\Delta s = s_2 - s_1 = \frac{2.303\,Q}{4\pi T} \log_{10} \frac{t_2}{t_1} \quad \ldots(5.29)$$

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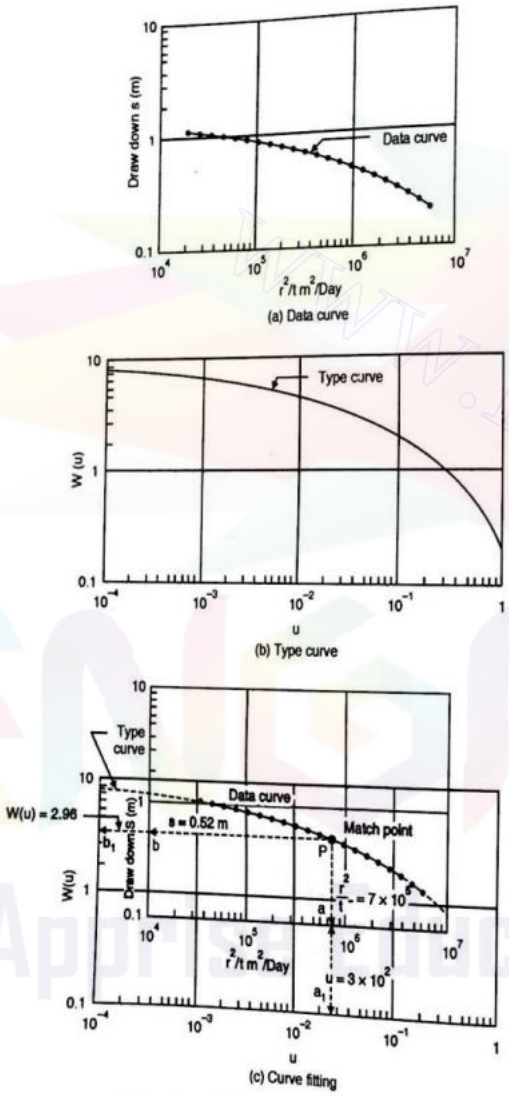

FIG. 5.41. THEIS METHOD.

If a plot is made between s and $\log_{10} t$ (Fig. 5.42) then for one log cycle of time difference (*i.e.*, $t_2 = 10\ t_1$), we get

$$\Delta s = \frac{2.303\ Q}{4\pi\ T}$$

$$\therefore \quad T = \frac{2.303\ Q}{4\pi\ \Delta s} \qquad ...(5.30)$$

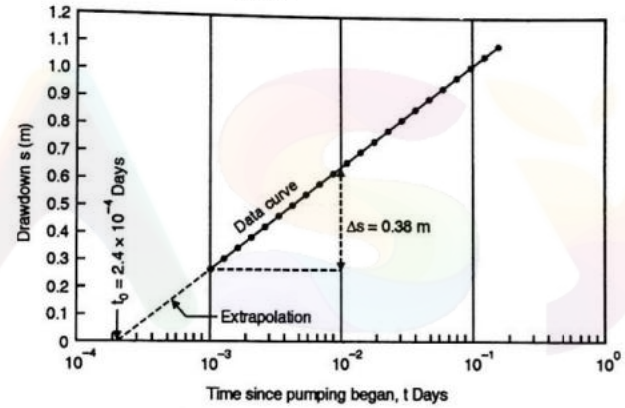

FIG. 5.42. JACOB'S METHOD.

Extrapolating the straight line of the curve to intersect with the zero drawdown axis permits the calculation for S. Let t_o be the time (since pumping started), at which the drawdown in observation well is still zero, we have from Eq. 5.28 :

$$s = 0 = \frac{Q}{4\pi\ T}\left[\log_e \frac{4T\ t_0}{r^2\ S} - 0.5772\right]$$

or $$\log_e \frac{4T\ t_0}{r^2\ S} = 0.5772$$

or $$\frac{4T\ t_0}{r^2\ S} = e^{0.5772}$$

which gives $$S = \frac{2.25\ T\ t_0}{r^2} \qquad ...(5.31)$$

Thus storage-coefficient S can be calculated from Eq. 5.31. *The straight line approximation for this method should be restricted to values of u less than about 0.01 to avoid large errors.*

(c) CHOW'S METHOD

Chow gave a method of solution which avoids curve fitting. He introduced a function $F(u)$ given by the relation :

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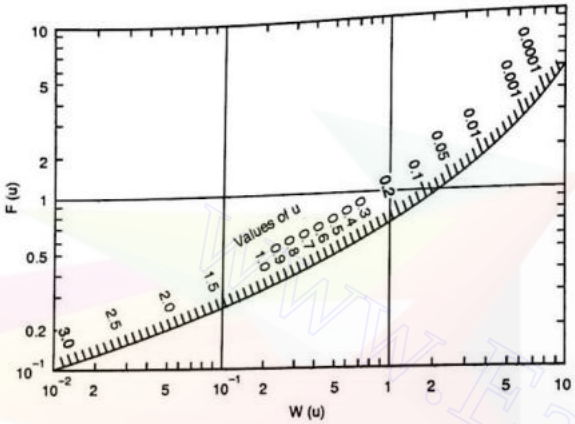

FIG. 5.43. FUNCTION $F(u)$.

$$F(u) = \frac{W(u) \cdot e^{u}}{2.303} \qquad ...(5.32)$$

The relation between $F(u)$, $W(u)$ and u is shown in Fig. 5.43 with the u scale on the curve. The function $F(u)$ is introduced to relate $W(u)$ and u to a certain combination of s and t. The procedure is as follows :

1. Select an observation well near the pumped well and observe the drawdown (s) at all times.
2. Plot a graph between s and $\log_{10} t$ as shown in Fig. 5.44. Join the points by a smooth curve.
3. On the plotted graph, choose an arbitary point P and note its co-ordinates s and t.
4. Draw the tangent of curve at the chosen point P and determine the drawdown difference Δs per log cycle of time.
5. Compute the function $F(u)$ by the relation

$$F(u) = \frac{s}{\Delta s} \qquad ...(5.33)$$

6. Knowing $F(u)$ and using Fig. 5.43, find u and $W(u)$.
7. Compute S and T from the following relations :

$$T = \frac{Q}{4\pi s} W(u) \qquad \text{and} \qquad S = \frac{4\,u\,T\,t}{r^2}$$

Note. For small values of u, we have from Jacob's method :

$$\Delta s = \frac{2.303\,Q}{4\pi T} \log_{10} \frac{t_2}{t_1} = \frac{2.303\,Q}{4\pi T} \quad (\text{when } t_2 = 10\ t_1\ )$$

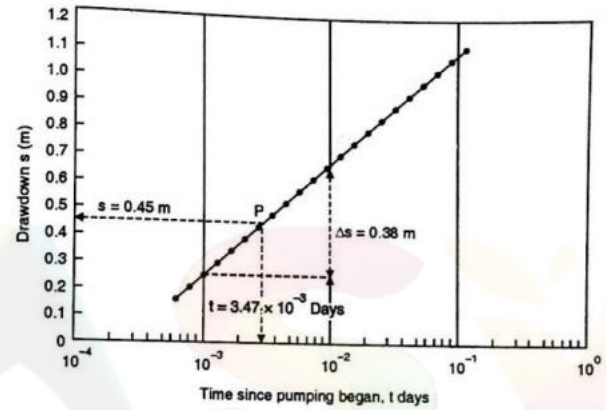

FIG. 5.44. CHOW'S METHOD.

Also $$s = \frac{Q}{4\pi T} W(u)$$

$$\therefore \qquad F(u) = \frac{s}{\Delta s} = \frac{W(u)}{2.303} \qquad ...[5.32(a)]$$

For $F(u) < 2$, the above equation can safely be used. For $F(u) > 2$, u becomes large. Hence Eq. 5.32 should be used.

Example 5.12. ***Theis Method.*** *A well fully penetrating a confined aquifer is pumped at a uniform rate of 2500 litres per minute. The drawdown in an observation well situated at 60 m away are given in Table 5.3. Using Theis method, determine the formation constants of the aquifer (Adapted from data from U.S. Geological Survey).*

TABLE 5.3

Time since Pumping began t (min)	*Drawdown s (m)*	*t (min)*	*s (m)*	*t (min)*	*s (m)*
0	0.00	12	0.59	150	1.03
1.0	0.20	14	0.62	180	1.05
1.5	0.26	18	0.66	210	1.08
2.0	0.30	24	0.71	240	1.10
2.5	0.33	30	0.75		
3	0.36	40	0.80		
4	0.41	50	0.83		
5	0..45	60	0.86		
6	0.48	80	0.91		
8	0.53	100	0.95		
10	0.56	120	0.98		

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Solution

The values of t, s and r^2/t are tabulated in Table 5.4. A plot is made between s and r^2/t as shown in Fig. 5.41 (a) on a log-log tracing paper. The type curve [$W(u)$ versus u] is shown in Fig. 5.41 (b), prepared from data of Table 5.2.

TABLE 5.4. COMPUTATION FOR DATA CURVE (r = 60 m)

Time t		r^2/t m^2/day	s (m)	Time t		r^2/t m^2/day	s (m)
min.	days			min	days		
0	0	∞	0.0	18	12.5×10^{-2}	2.88×10^5	0.66
1.0	6.94×10^{-4}	5.18×10^6	0.20	24	1.67×10^{-2}	2.16×10^5	0.71
1.5	1.04×10^{-3}	3.46×10^6	0.26	30	2.08×10^{-2}	1.73×10^5	0.75
2.0	1.39×10^{-3}	2.59×10^6	0.30	40	2.78×10^{-2}	1.30×10^5	0.80
2.5	1.74×10^{-3}	2.07×10^6	0.33	50	3.47×10^{-2}	1.04×10^5	0.83
3.0	2.08×10^{-3}	1.73×10^6	0.36	60	4.17×10^{-2}	8.64×10^4	0.86
4	2.78×10^{-3}	1.30×10^6	0.41	80	5.56×10^{-2}	6.48×10^4	0.91
5	3.47×10^{-3}	1.04×10^6	0.45	100	6.94×10^{-2}	5.18×10^4	0.95
6	4.17×10^{-3}	8.64×10^5	0.48	120	8.33×10^{-2}	4.32×10^4	0.98
8	5.56×10^{-3}	6.48×10^5	0.53	150	1.04×10^{-2}	3.46×10^4	1.03
10	6.94×10^{-3}	5.18×10^5	0.56	180	1.25×10^{-1}	2.88×10^4	1.05
12	8.33×10^{-3}	4.32×10^5	0.59	210	1.46×10^{-1}	2.47×10^4	1.08
14	9.72×10^{-3}	3.70×10^5	0.62	240	1.67×10^{-1}	2.16×10^4	1.10

The data curve of Fig. 5.41(a) is superimposed on the type curve and properly oriented to get the best possible fit. A match point P is pricked [Fig. 5.41(c)] and its co-ordinates are found as follows :

$$s = 0.52 \text{ cm} \; ; \quad W(u) = 2.96$$

$$\frac{r^2}{t} = 7 \times 10^5 \; ; \qquad u = 3 \times 10^{-2}$$

Discharge $\quad Q = 2500$ litres/min $= 2.5$ m³/min. $= 3600$ m³/day

$$\therefore \quad T = \frac{Q}{4\pi s} W(u) = \frac{3600}{4 \times \pi \times 0.52} \times 2.96 = \mathbf{1631 \ m^3/day/m}$$

and

$$S = \frac{4u\,T}{r^2/t} = \frac{4 \times 3 \times 10^{-2}}{7 \times 10^5} \times 1631 = \mathbf{0.00028.}$$

Example 5.13. *Jacob's Method.* *Solve example 5.12 by Jacob's method.*

Solution

Fig. 5.42 shows the plot between s and t. Most of the points lie on a straight line. The drawdown Δs for one log cycle of time difference comes out to be 0.38 m. Hence from Eq. 5.30,

$$T = \frac{2.303\,Q}{4\pi\,\Delta s} = \frac{2.303 \times 3600}{4\pi \times 0.58} = \mathbf{1738 \ m^3/day/m.}$$

Extending the straight line so as to cut the time axis, we get

$$t_0 = 2.4 \times 10^{-4} \text{ days}$$

$$\therefore \quad S = \frac{2.25\,T\,t_0}{r^2} = \frac{2.25 \times 1738 \times 2.4 \times 10^{-4}}{(60)^2} = \mathbf{0.00026.}$$

Example 5.14. *Chow's Method.* *Solve Example 5.12 by Chow's mehtod.*

Solution

The plot between s and t is shown in Fig 5.44. Choose any point P whose co-ordinates are $s = 0.45$ m and $t = 3.47 \times 10^{-3}$ days. Since the curve is a straight line, the tangent to the curve at P will coincide with the straight line itself. For one log cycle of time, we get $\quad \Delta s = 0.38.$

$$\therefore \quad F(u) = \frac{s}{\Delta s} = \frac{0.45}{0.38} = 1.184$$

From Fig. 5.34, corresponding to $F(u) = 1.184$, we get

$$W(u) = 2.7 \text{ and } u \simeq 0.04$$

Alternatively, from Eq. 5.32(a)

$$W(u) = 2.303F(u) = 2.303 \times 1.184 = 2.727,$$

and from Table 5.2, $\quad u = 0.035$

Hence

$$T = \frac{Q}{4\pi s} W(u) = \frac{3600}{4\pi \times 0.45} \times 2.727 = \mathbf{1736 \ m^3/day/m}$$

$$S = \frac{4utT}{r^2} = \frac{4 \times 0.035 \times 3.47 \times 10^{-3}}{3600} \times 1736 = \mathbf{0.000234.}$$

5.23. EXAMPLES FROM COMPETITIVE EXAMINATIONS

Example 5.15. *A 30 cm diameter well penetrates 25 m below the static water table. After 24 hours of pumping @5400 litres/minute, the water level in a test well at 90 m is lowered by 0.53 m, and in a well 30 m away the drawdown is 1.11 m. (a) what is the transmissibility of the aquifer ? (b) Also determine the draw down in the main wall.*

(Engg. Services Exam. 1968)

Solution :

For the present case of well penetrating an unconfined aquifer, we have :

$$Q = \frac{\pi\,k\,(h_2^2 - h_1^2)}{\log_e r_2/r_1} \qquad \text{...(5.4)}$$

Here, depth of static water level $= H = 25$ m

At $\quad r = r_1 = 30$ m, $s_1 = 1.11$ m. Hence $h_1 = H - s_1 = 25 - 1.11 = 23.89$ m

Also, $\quad$ at $r_2 = 90$ m, $s_2 = 0.53$ m. Hence $h_2 = H - s_2 = 25 - 0.53 = 24.47$ m

$$Q = 5400 \text{ lpm} = 5400 \times 10^{-3}/60 \text{ m}^3/\text{s} = 0.09 \text{ m}^3/\text{s}.$$

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$$k = \frac{Q \log_e r_2/r_1}{\pi (h_2^2 - h_1^2)} = \frac{0.09 \log_e (90/30)}{\pi [(24.47)^2 - (23.89)^2]} = \mathbf{1.1221 \times 10^{-3}\ m/s}$$

Hence $T = k H = 1.1221 \times 10^{-3} \times 25 = 0.0281\ m^3/s$

Applying Eq. 5.4. at the well face, we have $r_1 = r_w$ and $h_1 = h_w$

$$Q = \frac{\pi k (h_2^2 - h_w^2)}{\log_e r_2/r_w} \quad \text{where } r_w = 0.3/2 = 0.15 \text{ m}$$

$$\therefore \quad h_2^2 - h_w^2 = \frac{Q \log_e r_2/r_w}{\pi k} = \frac{0.09 \log_e (90/0.15)}{\pi \times 1.1221 \times 10^{-3}} = 163.317$$

or $h_w^2 = h_2^2 - 163.317 = (24.47)^2 - 163.317 = 435.46$

or $h_w = 20.87$ m

Hence draw down at well face, $s_w = H - h_w = 25 - 20.87 = \mathbf{4.13\ m}$

Example 5.16. *A well penetrates into an unconfined aquifer having a saturated depth of 100 metres. The discharge is 250 litres per minute at 12 metres drawdown. Assuming equilibrium flow conditions and a homogenous aquifer, estimate the discharge at 18 metres drawdown. The distance from the well where the drawdwon influences are not appreciable may be taken equal for both the cases.*

(Engg. Services Exam. 1969)

Solution :

For an unconfined aquifer, we have from Eq. 5.4

$$Q = \frac{\pi k (H^2 - h^2)}{\log_e R/r}$$

In the first case, $Q = Q_1 = 250$ lpm, $h_1 = H - s_1 = 100 - 12 = 88$ m

$$\therefore \quad \frac{\log_e R/r}{\pi k} = \frac{(H^2 - h_1^2)}{Q_1} = \frac{(100)^2 - (88)^2}{250} = 9.024 \text{ units.}$$

In the second case, $Q = Q_2 = \dfrac{\pi k}{\log_e R/r}(H^2 - h_2^2) = \dfrac{[(100)^2 - (82)^2]}{9.024} = \mathbf{363\ lpm}$

Example 5.17. *A horizontal aquifer is of 10 m thickness and infinite areal extent with its top level 25 m below ground level. Static piezometric surface is 10 m below ground level. During steady rate of pumping from the well at 5000 m^3/day, the steady drawdown at the well is 12 m. Permeability of aquifer formation is 48 m/day. Assume the radius of influence as 500 m. Find the effective well diameter, in metres.*

(Gate, 1988)

Solution

For a confined aquifer, we have, from Eq. 5.6

$$Q = \frac{2 \pi b k s}{\log_e R/r}$$

Here $Q = 5000\ m^3/\text{day}$; $b = 10$ m ; $k = 48$ m/day ; $R = 500$ m ; $s = 12$ m

Substituting the values, we get

$$\log_e R/r = \frac{2 \pi b k s}{Q} = \frac{2 \pi (10 \times 48 \times 12)}{5000} = 7.2383$$

$$\therefore \quad R/r = 1391.63$$

$$\therefore \text{ Dia. of well} = D_w = 2r = \frac{2R}{1391.63} = \frac{2 \times 500}{1391.63} \simeq \mathbf{0.72\ m}$$

Example 5.18. *A well is located in a 30 m thick confined aquifer of permeability 35 m/day and storage coefficient of 0.004. If the well is pumped at the rate of 1500 litres per minutes, calculate the drawdown at a distance of 40 m from the well after 20 hours of pumping.* ***(Civil Services Exam. 1991)***

Solution

This is the case of unsteady flow towards a well in confined aquifer. The draw down (s) in an observation well at $r = 40$ m and $t = 20$ hours $= 20 \times 3600$ seconds after pumping started is given by Jacob's equation

$$s = \frac{Q}{4 \pi T}\left[\log_e \frac{4 T t}{r^2 S} - 0.5772\right] \qquad ...(5.28)$$

where $Q = 1500$ lpm $= 1500 \times 10^{-3}/60\ m^3/s = 0.025\ m^3/s$

S = storage coefficient $= 0.004$

$$T = bk = 30 \times 35\ m^2/\text{day} = \frac{30 \times 35}{60 \times 60 \times 24}\ m^2/s = 0.01215\ m^2/s$$

$t = 20$ hours $= 20 \times 3600$ seconds $= 72000$ seconds

Substituting the values in eq. 5.28. we get

$$s = \frac{0.025}{4 \pi (0.01215)}\left[\log_e \frac{4 \times 0.01215 \times 72000}{(40)^2 \times 0.004} - 0.5772\right] = \mathbf{0.94\ m}$$

Example 5.19. *During a recuperation test, the water in an open well was depressed by pumping by 2.5 m and it recuperated 1.8 m in 80 minutes. Calculate the yield from a well of 4 m diameter under a depression head of 3 m.*

(Engg Services Exam. 1993)

Solution : ***This is exactly the same as example 5.2.***

For solution, refer Example 5.2, part (a).

Example 5.20. *A 30 cm well completely penetrates an unconfined aquifer of depth 40 m. After a long period of pumping at a steady rate of 1500 lpm, the drawdown in two observation wells 25 m and 75 m from the pumping well were found to be 3.5 m and 2.0 m respectively. Determine the transmissibility of the acquifer. What is the drawdown at the pumping well ?* ***(Civil Services Exam. 1994)***

Solution : From Eq. 5.4, we have

$$Q = \frac{\pi k (h_2^2 - h_1^2)}{\log_e r_2/r_1}$$

If s_1 and s_2 are the drawdowns in the two wells, and if H (= 40 m) is the depth of water before pumping we have

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$$h_1 = H - s_1 = 40 - 3.5 = 36.5 \text{ m} \quad \text{and} \quad h_2 = H - s_2 = 40 - 2 = 38 \text{ m}$$

Also, $r_1 = 25$ m, $r_2 = 75$ m and $Q = 1500 \times 10^{-3} \times 60 = 0.025 \text{ m}^2/\text{s}$.

Substituting the values, we get

$$k = \frac{Q \log_e r_2/r_1}{\pi (h_2^2 - h_1^2)} = \frac{0.025 \log_e 75/25}{\pi [(38)^2 - (36.5)^2]} = 7.82 \times 10^{-5} \text{ m/sec.}$$

$$\therefore \quad T = bk = 40 \times 7.82 \times 10^{-5} = \mathbf{3.129 \times 10^{-3} \text{ m}^2/\text{sec}}$$

Again, applying Eq. 5.4 at the well face, making $h_1 = h_w$ and $r_2 = 0.15$ m, we get

$$Q = \frac{\pi k (h_2^2 - h_w^2)}{\log_e r_2/r_w}$$

Substituting the values, we get

$$h_2^2 - h_w^2 = \frac{Q \log_e r_2/r_w}{\pi k} = \frac{0.025 \log_e (75/0.15)}{\pi (7.82 \times 10^{-5})} = 632.4$$

$$\therefore \quad h_w^2 = h_2^2 - 632.4 = (38)^2 - 632.4 = 811.59$$

From which $h_w = 28.48$ m

$\therefore$ Drawdown at well face, $s_w = H - h_w = 40 - 28.48 = \mathbf{11.52 \text{ m}}$

Example 5.21. *An aquifer of 20 m average thickness is overlain by an impermeable layer of 30 m thickness. A test well of 0.5 m diameter and two observation wells at a distance of 10 m and 60 m from the test-well are drilled through the aquifer. After pumping at a rate of 0.1 m³/sec for a long time, the following drawdowns are stabilised in these wells : First observation well, 4 m; second observation well, 3 m. Show the arrangement in a diagram. Determine the coefficient of permeability and drawdown in the test well.*

(Engg. Services Exam. 1995)

Solution : The arrangements is shown in Fig. 5.45. From Eq. 5.6 (*a*)

$$Q = \frac{2.72\, kb\, (h_2 - h_1)}{\log_{10} \frac{r_2}{r_1}}$$

$$h_2 = H - s_2 \quad \text{and} \quad h_1 = H - s_1$$

$$\therefore \quad h_2 - h_1 = (H - s_2) - (H - s_1)$$

$$= s_1 - s_2$$

Hence
$$Q = \frac{2.72\, bk\, (s_1 - s_2)}{\log_{10} \frac{r_2}{r_1}} \quad \text{...(5.6 c)}$$

Substituting the values, we get

$$k = \frac{Q \log_{10} r_2/r_1}{2.72\, b\, (s_1 - s_2)} = \frac{0.1 \log_{10} (60/10)}{2.72 \times 20\, (4 - 3)}$$

$$= \mathbf{1.43 \times 10^{-3} \text{ m/sec}}$$

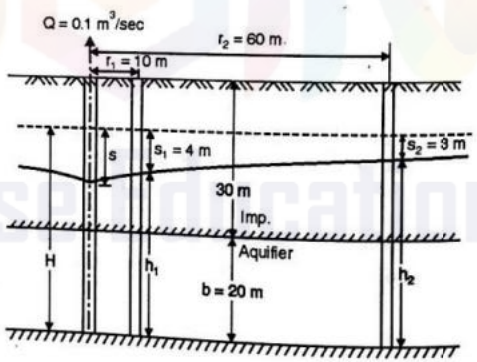

FIG. 5.45

Again, making $s_1 = s =$ draw down at well face and $r_1 = r =$ radius of well, Eq. 5.6 (c) reduces to the form

$$Q = \frac{2.72\, bk\, (s - s_2)}{\log_{10} \frac{r_2}{r}}$$

$$s - s_2 = \frac{Q \log_{10} r_2/r}{2.72\, bk} = \frac{0.1 \log_{10} (60/0.25)}{2.72 \times 20 \times 1.43 \times 10^{-3}} \simeq 3.06$$

$$\therefore \quad s = 3.06 + s_2 = 3.06 + 3 = \mathbf{6.06 \text{ m}}$$

Example 5.22. *During a recuperation test, the water in an open well was depressed, by pumping, by 2.1 m and it recuperated 1.6 m in 90 minutes. Find the diameter of well to yield 10 litres/sec. under a depression head of 2 m.*

(Engg. Services Exam. 1996)

Solution : *This problem is similar to Example 5.2.*

The specific yield (K/A) of the well is given by Eq. 5.21

$$\frac{K}{A} = \frac{1}{T} \log_e \frac{h_1}{h_2}$$

where T = time in hours = 90/60 = 1.5 h .; $h_1 = 2.1$ m ; $h_2 = 2.1 - 1.6 = 0.5$ m

$$\therefore \quad \frac{K}{A} = \frac{1}{1.5} \log_e \frac{2.1}{0.5} = 0.9567$$

Required yield, $Q = 10$ litres/sec $= \frac{10 \times 3600}{1000} = 36 \text{ m}^3/\text{hour}$

Now $\quad Q = \left(\frac{K}{A}\right) A H$, where $H = 2$ m

From which $\quad A = \frac{Q}{(K/A)\, H} = \frac{36}{0.9567 \times 2} = 18.8147 \text{ m}^2$

$$\therefore \quad d = \left(\frac{4}{\pi} A\right)^{1/2} = \left(\frac{4}{\pi} \times 18.8147\right)^{1/2} = 4.89 \simeq \mathbf{4.9 \text{ m}}$$

Example 5.23. *During a recuperation test, the water level in an open well was depressed by pumping upto 2.5 m. The water level was raised by 1 m within 60 minutes just after stopping the pumping. Determine the yield from the well of 2 m diameter when the depression head is 3 m.* ***(Engg. Services Exam. 2001)***

Solution : From Eq. 5.21, we have

$$\frac{K}{A} = \frac{1}{T} \log_e \frac{h_1}{h_2}$$

Here $\quad T = 60$ min. $= 1$ h ; $h_1 = 2.5$ m ; $h_2 = 2.5 - 1 = 1.5$ m

$$\therefore \quad \frac{K}{A} = \frac{1}{1} \log_e \frac{2.5}{1.5} = 0.5108$$

Now $\quad Q = KH = \left(\frac{K}{A}\right) AH = 0.5108 \left(\frac{\pi}{4} \times 2^2\right) \times 3 = \mathbf{4.814 \text{ m}^3/\text{hr}}$

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PROBLEMS

1. Define the following terms :
Aquifer, aquiclude, specific yield, peizometric surface, water table, perched aquifer.

2. Show, with the help of sketches, various types of wells.

3. Explain the terms 'storage cofficent' and 'coefficient of transmissibility'.

4. State and discuss assumptions and limitations of Dupuit's theory.

5. Derive an expression for discharge from a well in unconfined aquifer. The well fully penetrates it.

6. Derive an expression for discharge from a well fully penetrating a confined aquifer.

7. Explain the method of determining the coefficient of transmissibility of a confined aquifer by pumping out test. How can this method be extended for unconfined aquifer ?

8. Write notes on the following :
(*i*) well loss, (*ii*) specific capacity of well, (*iii*) spherical flow in well, (*iv*) interference among wells.

9. Describe various types of tube wells.

10 Describe with the help of sketches, some of the common types of strainers used in tube wells.

11. Explain the percussion method of drilling a tube well.

12. What do you understand by well shrouding.?

13. Describe in brief various methods of developing a tube well.

14. What do you understand by recuperation test ? Derive the equations used in the test.

15. Distinguish clearly between a shallow well and a deep well. How does a deep well differ from a tube well in confined aquifer ?

16. The following observations were recorded during a pumping out test on a tube well penetrating fully in a aquifer :
Well diameter : 25 cm
Discharge from the the well : 300 m^3/hour
R.L. of original water surface, before pumping started = 122.0 m
R.L. of water in the well at constant pumping =117.1 m
R.L. of water in the observation well = 121.3 m
R.L. of impervious layer = 92.0 m
Radial distance of observation well from the tube well = 50 m
Determine : (*a*) the field permeabilty coefficient of the free aquifer and (*b*) radius of zero drawdown. [**Ans.** (*a*) 60.7 m/day, (*b*)157 m]

17. A tube well commands 40 hectares of land under wheat. When the well is discharging at constant rate, the water level in the well drops to R.L. 160.2 m. The R.L. of highest land is 166.5 m. The duty for wheat is 1600 hectares/cumec. Taking the pump effciency as 75% calculate the input H.P. of the pump. [**Ans.** 2.89 H.P.]

18. Design a tube well for the following data :
(*i*) Yield required = 0.2 cumec
(*ii*) Thickness of confined aquifer = 40 m

(*iii*) Radius of circle of influence = 300 m
(*iv*) Permeability coefficient = 80 m/day
(*v*) Drawdown = 6 m. [**Ans.** 28 cm, or say 30 cm]

19. During a recuperation test, the water in open well was depressed by pumping by 2 m and it recuperated 1.5 m in 1 hour. Estimate the yield from a well of 2 m diameter under a depression head of 2 m situated in the same area. Derive the expression you use. [**Ans.** 8.7 m^3/hour]

20. A tube well penetrates fully a 8 m thick water bearing stratum (confined) of medium sand having coefficient of permeability of 0.004 m/sec. The well radius is 15 cm and is to be worked under a drawdown of 3 m at the well face. Calculate the discharge from the well. What will be the percentage increase in the discharge if the radius of the well is doubled ? Take radius of zero drawdown equal to 400 m in each. [**Ans.** (*i*) 275 m^3/hour, (*ii*) 9.6%]

21. Design an open well in fine sand to give a discharge of 0.005 cumec when worked under a depression head of 3 metres. Take the value of the specific yield for fine sand as 0.5 m^3/hour per square metre of area, under unit depression head. [**Ans.** Dia. 3.9 m]

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6

Reservoir Planning

6.1. INTRODUCTION

Dams are constructed across the rivers and streams to create an artificial lake or reservoir behind it. Dams and reservoirs are the most important and expensive elements in multi-purpose river basin development. They require very careful planning, design, and operation. A number of problems arise in design, construction and operation of dams and reservoirs ; for example, selection of site, the relative merits of different types of dams, storage capacity and optimum yield and co-ordinated use of storage for different purposes.

Storage works are constructed to serve many purposes, which include :

1. Storage and control of water for irrigation
2. Storage and diversion of water for domestic uses
3. Water supplies for industrial uses
4. Development of hydroelectric power
5. Increasing water depths for navigation
6. Storage space for flood control
7. Reclamation of low-lying lands
8. Debris control
9. Preservation and cultivation of useful aquatic life
10. Recreation.

Depending upon the purposes served, resevoirs may be classified as under :

(*i*) Storage or conservation reservoirs
(*ii*) Flood protection reservoirs
(*iii*) Distribution reservoirs
(*iv*) Multipurpose reservoirs.

(*i*) Storage or Conservation Reservoir : Storage reservoirs are primarily used for water supplies for irrigation, hydroelectric developement, domestic and industrial supplies. A river does not carry the same quantity of water throughout the year, and may carry large quantities in the other part of the year. A storage reservoir is constructed to store the excess water during the period of large supplies, and release it gradually as and when it is needed.

(*ii*) Flood Control Reservoirs : Flood control or flood protection reservoirs are those which store water during flood and release it gradually at a safe rate when the flood reduces. By the provision of artificial storage during the floods, flood damage downstream is reduced. There are two pillars in flood prevention planning : river bed stabilizing planning for stabilizing the river channel from the upper stream down to the estuary and flood disposition planning for causing the flood water to flow down without doing any damage. Flood control by means of dam is a part of the flood disposition planning covering the whole river system and, sometimes it shares part of river-bed stabilization planning.

Thus, in Fig. 6.1 *ABC* is the natural hydrograph at the dam site, having a maximum flood discharge Q_1. By the construction of the dam, the natural hydrograph is moderated by the reservoir, as shown by the dotted lines (*AB'C*). Thus, the flood discharge is reduced from Q_1 to Q_2. The area shown hatched represents the storage to be provided in the reservoir. The portion marked by dots represents the excess volume released later.

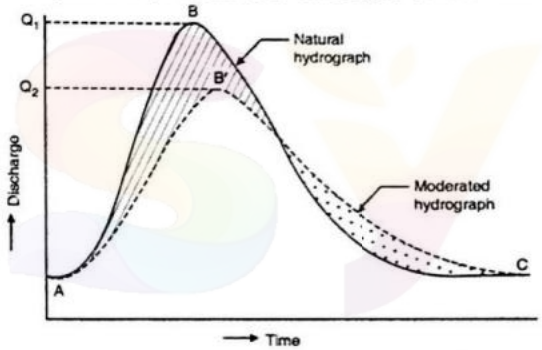

FIG. 6.1. HYDROGRAPH MODERATED BY THE RESERVOIR.

(*iii*) Distribution Reservoir : A distribution reservoir is a small storage reservoir used for water supply in a city . A distribution reservoir accounts for the varying rate of water during the day. Such distribution reservoir permits the pumping plants and water-treatment works etc., to operate at a constant rate. The varying demand rate, exceeding to constant pumping rate, is met from the distribution reservoir.

(*iv*) Multipurpose Reservoir : A multipurpose reservoir is that which serves more than one purpose. For example a reservoir designed to protect the down stream area from floods, and to store water for irrigation and hydroelectric purposes is a multipurpose reservoir.

6.2. INVESTIGATIONS FOR RESERVOIR PLANNING

The following investigations are required for reservoir planning :

1. Engineering surveys. 2. Geological investigations.
3. Hydrological investigations.

1. ENGINEERING SURVEYS

The area at the dam site is surveyed in detail and a contour plan is perpared. From the plan, the following physical characteristics are prepared :

(*a*) Area-elevation curve. (*b*) Storage-elevation curve.
(*c*) Map of the area to indicate the land property to be surveyed

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(*d*) Suitable site selection for the dam.

Area-elevation and Storage-elevation Curves.

Fig. 6.2 shows a typical contour plan at the reservoir site. The hatched area shows the water spread area. The area A_1, A_2, A_3 enclosed by the successive contours can be determined with a planimeter. A close observation of Fig. 6.2 shows that as the value of a contour (*i.e.*, elevation) increases, its area increases. A curve, such as curve *AB* shown in Fig. 6.3, may be drawn between elevation and area.

The reservoir capacity, or the volume of storage, corresponding to a given water level in the reservoir may be calculated either by trapezoidal formula or by prismoidal formula. Thus, if *V* is the storage volume and *h* is the contour interval, the formulae are:

FIG. 6.2. A TYPICAL CONTOUR PLAN.

(1) $V = \Sigma \frac{h}{2}(A_1 + A_2) \ldots$ (Trapezoidal formula)

$$= \left\{\frac{A_1 + A_n}{2} + A_2 + A_3 \ldots\ldots\ldots\ldots A_{n-1}\right\} \quad \ldots(6.1)$$

(2) $V = \Sigma \frac{h}{3}(A_1 + A_2 + \sqrt{A_1 A_2}\ )$ .....(Cone formula) ...(6.2)

(3) $V = \frac{h}{3}[(A_1 + A_n) + 4(A_2 + A_4 + ..) + 2(A_3 + A_5 + ...)]$ ...(Prismoidal formula)...(6.3)

where A_n is the area of the contour corresponding to the water surface elevation in the proposed reservoir. The volumes of storage corresponding to various water-surface elevation may be calculated and a curve, such as *CD* in Fig. 6.3, may be plotted between elevation and storage.

The contour plan (Fig. 6.2) also indicates the water spread corresponding to the reservoir elevation and enables us to determine the area under submergence and the compensation to be paid to the owners.

2. GEOLOGICAL INVESTIGATIONS

In almost all major civil engineering projects, geological advice is the most essential. Geological investigations cost very little in comparison to the total cost of the project; in a number of cases in recent years it has amounted to beween 1/4 to 1 percent of the whole. This relatively small amount represents a valuable insurance against expensive difficulties, otherwise unforeseen, which might arise during constuction but

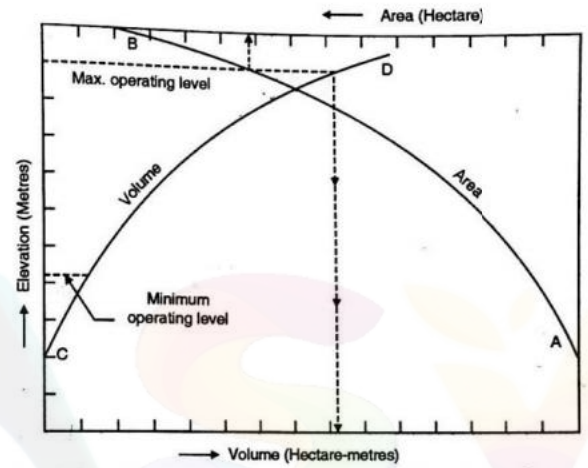

FIG. 6.3. ELEVATION AREA AND ELEVATION-STORAGE CURVES.

which it may be possible to predict from a study of the geological factors involved. Geological investigations are required to give detailed information about the following items :

1. Water tightness of reservoir basin.
2. Suitability of foundations for the dam.
3. Geological and structural features, such as folds, faults, fissures etc. of the rocks basin.
4. Type and depth of over burden (superficial deposits).
5. Location of permeable and soluble rocks, if any.
6. Ground water conditions in the region.
7. Location of quarry sites for materials required for the dam construction and quantities available from them.

The geology of the catchment area should also be studied, since it affects the proportions of run-off percolation. The special requirement for the geology of the resevoir site is that there should be no danger of serious leakage when the ground is under pressure from the full head of water in the reservoir. The geology of the dam site is important from the point of view of secure foundation for the dam. The nature of sub-surface geology should be explored by trial bores or various means of geophysical exploration.

3. HYDROLOGICAL INVESTIGATIONS

The hydrological investigation is a very important aspect of reservoir planning. The capacity of the irrigation canals and/or the installed capacity of the power houses will depend upon the available supplies from the reservoir. The hydrological investigations may be divided into two heads :

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1. Study of run-off pattern at the proposed dam site, to determine the storage capacity corresponding to a given demand, and

2. Determination of the hydrograph of the worst flood, to determine the spillway capacity and design.

6.3. SELECTION OF SITE FOR A RESERVOIR

The final selection of site for a reservoir depends upon the following factors :

1. The geological condition of the *catchment area* should be such that percolation losses are minimum and maximum run-off is obtained.

2. The reservoir site should be such that quantity of leakage through it is a minimum. Reservoir site having the presence of highly permeable rocks reduce the water tightness of the reservoir. Rocks which are not likely to allow passage of water include shales and slates, schists, gneisses, and crystalline igneous rocks such as granite.

3. Suitable dam site must exist. The dam should be founded on sound watertight rock base, and percolation below the dam should be minimum. The cost of the dam is often a controlling factor in selection of a site.

4. The reservoir basin should have narrow opening in the valley so that the length of the dam is less.

5. The cost of real estate for the reservoir, including road, rail road, dwelling re-location etc. must be as less as possible.

6. The topography of the reservoir site should be such that it has adequate capacity without submerging excessive land and other properties.

7. The site should be such that a deep reservoir is formed. A deep reservoir is preferable to a shallow one because of (*i*) lower cost of land submerged per unit of capacity, (*ii*) less evaporation losses because of reduction in the water spread area, and (*iii*) less likelihood of weed growth.

8. The reservoir site should be such that it avoids or excludes water form those tributaries which carry a high percentage of silt in water.

9. The reservoir site should be such that the water stored in it is suitable for the purpose for which the project is undertaken. The soil and rock mass at the reservoir site must not contain any objectionable minerals and salts.

6.4. ZONES OF STORAGE IN A RESERVOIR

The following are various zones of storage in reservoir :

(1) Useful storage. (2) Surcharge storage.

(3) Dead storage. (4) Bank storage.

(5) Valley storage.

The maximum level to which the water will rise in the reservoir during ordinary operation condition is called *normal pool level*. The normal pool level is corresponding to either the level of the spillway crest, or to the top level of the spillway gates. The level to which water rises during the design flood is known as the *maximum pool level*. The *lowest* elevation to which the water in the reservoir is to be drawn under ordinary operating conditions is known as the *minimum pool level*.

The volume of water stored between the normal pool level and the minimum pool level is known as the *useful storage*. The volume of water below the minimum pool level is known as the *dead storage* and is not useful under ordinary operating conditions. The volume of water stored between the normal pool level and the maximum level corresponding to a flood is called *surcharge storage*, and is usually uncontrolled. The terms *bank storage* and *valley storage* are referred to the volume of water stored in the pervious formations of the river banks and the soil above it. Such storage depends upon the geological conditions of river banks. The bank storage effectively increases the capacity of the reservoir above that indicated by the elevation-storage curve.

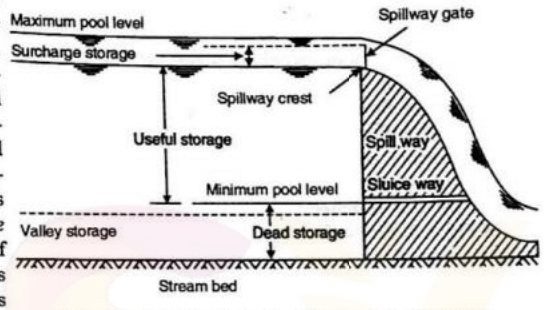

FIG. 6.4. ZONES OF STORAGE IN A RESERVOIR.

6.5. STORAGE CAPACITY AND YIELD

Yield. Yield is the amount of water that can be supplied from the reservoir in a specified interval of time. The interval of time chosen for the design varies from a day for small distribution reservoirs to a year for large conservation reservoirs. For example, if 25,000 cubic metres of water is supplied from a reservoir in one year, its yield is 25,000 cubic metres/year or 2.5 hectare-metres per year.

Safe yield or firm yield. The maximum quantity of water that can be guaranteed during a critical dry period is known as the safe yield or firm yield.

Secondary yield. Secondary yield is the quantity of water available in excess of safe yield during periods of high flood.

Average yield. The arithmetic average of the firm and the secondary yield over a long period of time is called average yield.

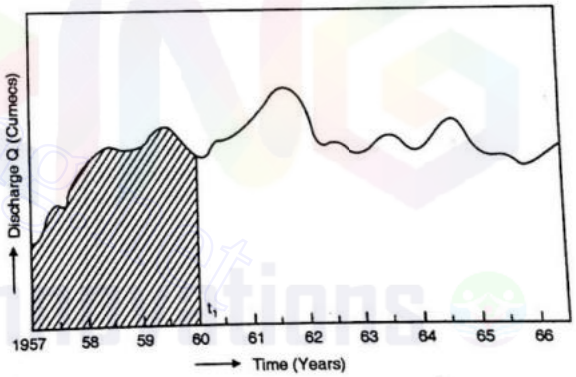

FIG. 6.5. FLOOD HYDROGRAPH OF INFLOW.

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Mass inflow curve. The reservoir capacity corresponding to a specified yield is determined with the help of mass inflow curve and the demand curve. A *mass inflow curve* is a plot between the cumulative inflow in the reservoir with time.

Fig. 6.5 shows a flood hydrograph of inflow for several years. Fig. 6.6 shows a mass inflow curve prepared from the flood hydrograph of Fig.6.5. Taking the starting year (*i.e.*1957) as the base, the total quantity of water from 1957 to a time t_1 (say 1960) that has flown through the river is the volume represented by the hatched area. In the mass inflow curve (Fig. 6.6), the corresponding ordinate at time t_1 (ordinate *AB*) will, therfore, be equal to the volume of water indicated by the hatched area of Fig. 6.5. Similarly, the ordinates of the mass inflow curve corresponding to other years can be computed from Fig. 6.5 and plotted. A mass inflow curve continuously rises as it shows *accumulated inflow*. If there is no inflow during certain period, the mass inflow curve will be horizontal during that period. The mass inflow curve will rise very sharply during the period of high flood. Thus the steepness of the mass inflow curve shows the rate of inflow at that time interval. The hollows on the mass inflow curve show relatively dry periods.

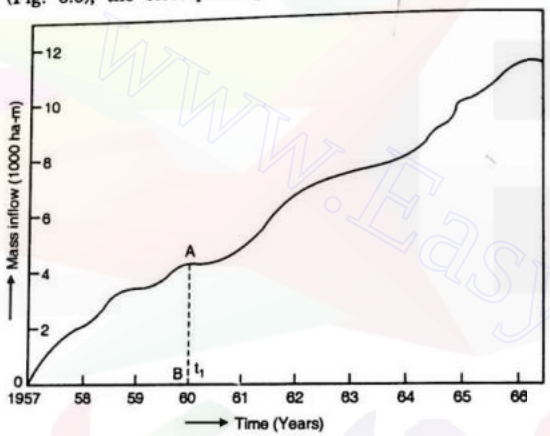

FIG. 6.6. MASS INFLOW CURVE.

Demand curve. A *demand curve* (Fig. 6.7) is a plot between accumulated demand with time. The demand curve representing a uniform rate of demand is a straight

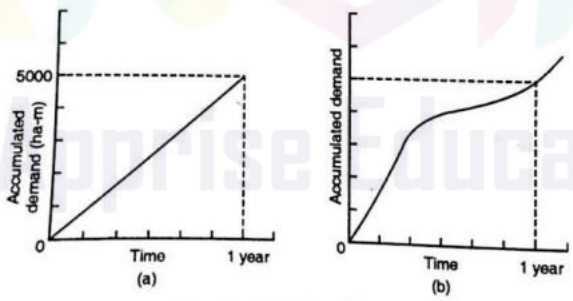

FIG. 6.7. DEMAND CURVES.

ine having the slope equal to the demand rate. A demand curve may be curved also, indicating variable rate of demand.

6.6. CALCULATION OF RESERVOIR CAPACITY FOR A SPECIFIED YIELD, FROM THE MASS INFLOW CURVE

Procedure (Fig. 6.8) :

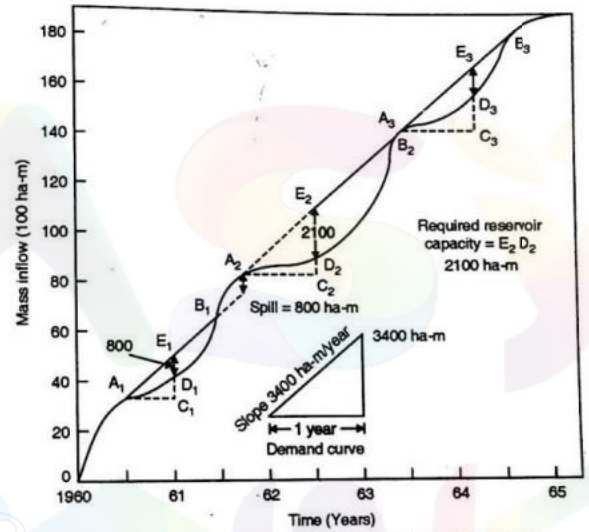

FIG. 6.8. DETERMINATION OF RESERVOIR CAPACITY.

1. From the flood hydrograph of inflow for several years, prepare the mass inflow curve. Also prepare the mass curve of demand on the same scale.
2. From the apices A_1, A_2, A_3..... etc. of the mass inflow curve, draw tangents parallel to the demand curve.
3. Measure the maximum vertical intercepts E_1D_1, E_2D_2, E_3D_3, etc., between the tangent and the mass inflow curve. The vertical intercepts indicate the volume by which the inflow falls short of demand. For example, for a period corresponding to points A_1 and C_1, C_1D_1 represents the net inflow while C_1E_1 represents the demand. Hence, the volume E_1D_1 has to be provided from the reservoir storage.
4. The biggest of the vertical ordinates amongst E_1D_1, E_2D_2, E_3D_3 etc. represent the required reservoir capacity.

It should be noted that the vertical distance between successive tangents represents water wasted over the spillway. The spillway must have sufficient capacity to discharge this flood volume,

Corresponding to the numerical figures indicated in Fig. 6.8, we observe that:

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(1) The required reservoir capacity is 2100 ha-m.

(2) Assuming the reservoir to be full at A_1 it is depleted to (2100 – 800) = 1300 ha-m at at D_1 and is again full at B_1.

(3) Assuming the reservoir to be full at A_2 it is empty at D_2 and is again full at B_2

(4) The resevoir is full between B_1 and A_2 and the quantity of spill is 800 ha-m.

6.7. DETERMINATION OF SAFE YIELD FROM A RESERVOIR OF A GIVEN CAPACITY

The following is the procedure of determining the safe yield from a reservoir of a given storage capacity, with the help of a mass inflow curve :

(1) Prepare the mass inflow curve. On the same diagram, draw straight lines, from a common origin, representing demands at various rates, say varying from 0 to 5000 ha-m per year.

(2) From the apices A_1, A_2, A_3 etc. of the mass inflow curve, draw tangents in such a way that their maximum departure from the mass inflow curve does not exceed the specified reservoir capacity. Thus, in Fig. 6.9, the ordinates E_1D_1, E_2D_2, E_3D_3, etc. are all equal to the reservoir capacity (say 1500 ha-m).

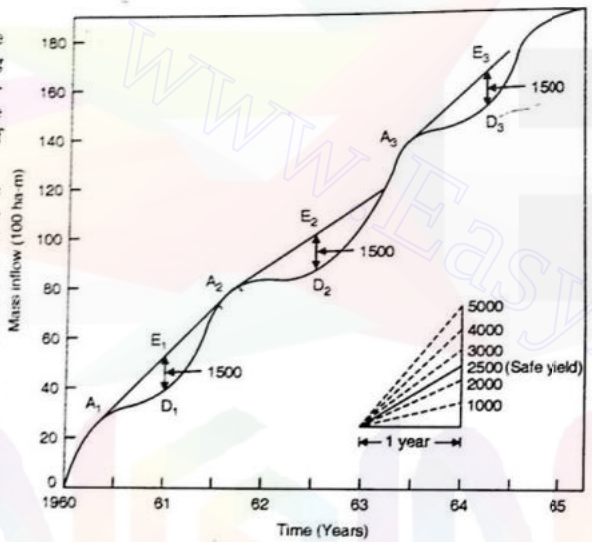

FIG. 6.9. DETERMINATION OF YIELD FROM RESERVOIR OF SPECIFIED CAPACITY

(3) Measure the slopes of each of these tangents. The slopes indicate the yield which can be attained in each year from the reservoir of given capacity. *The slope of flattest demand line is the firm yield* or *safe yield.*

6.8. SEDIMENT FLOW IN STREAMS : RESERVOIR SEDIMENTATION

All the rivers carry certain amount of silt eroded from the catchment area during heavy rains. The extent of erosion, and hence the silt load in the stream depends upon the following factors :

(i) Nature of soil of the catchment area (ii) Topography of the catchment area

(iii) Vegetation cover (iv) Intensity of rainfall.

The nature of the soil of the cathment area is an important factor. If the soil is soft, there is always a possibility of *sheet erosion.* The tributaries collecting water of the catchment area containing hard soil carry lesser silt. Steep slopes give rise to high velocities and erode the surface soil easily. Similarly higher intensity of rainfall causes greater run-off and more erosion. If the catchment area has sufficient vegetation cover the higher velocities are checked and erosion is very much reduced. Area having poor or practically no vegetal cover are productive of more silt. The rivers or tributaries passing through such areas carry more silt load with it, causing quick silting of the reservoir.

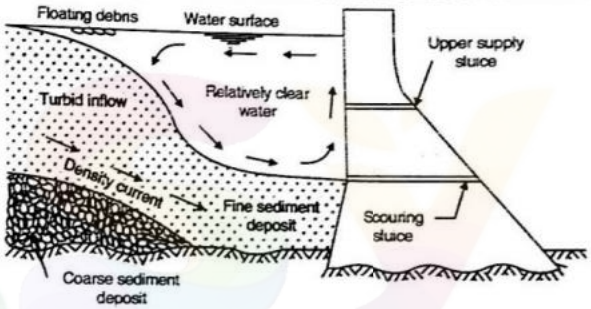

FIG. 6.10. RESERVOIR SEDIMENTATION.

The sediment transported by the river can be divided into two heads : (a) *Bed load*, and (b) *Suspended load.* The bed load is dragged along the bed of the stream. The suspended load is kept in suspension because of the vertical component of the eddies formed due to friction of flowing water against the bed. The bed load is generally much smaller — 10 to 15% of the suspended load. When the stream approaches the reservoir, the velocity is very much reduced. Due to this reduction, the coarser particles settle in the head reaches of the reservoir, while the finer particles are kept in suspension for sufficient time till they settle just to the upstream side of the dam, as shown in Fig. 6.10. Some fine particles may pass through sluice ways, turbines or spillways.

Density Currents : Density current may be defined as a gravity flow of fluid under another fluid of approximately equal density. In case of reservoirs the water stored is usually clear and the inflow during floods is generally muddy. The two fluids have, therefore, different densities and the heavy turbid water flows along the channel bottom towards the dam under the influence of gravity as shown in Fig 6.10. This is known as density current. The rate of silting in case of reservoirs reduces if the density currents are vented by proper location and operation of outlet and sluice gates.

Measurement of Sediment load : The amount of silt or the sediment load carried by a stream is determined by taking the samples of water carrying silt, at various depths. The samples are then filtered and the sediment is removed and dried. The sediment load measured in the units of 'parts per million part of water' (ppm). There are no accurate devices to measure the bed load, which is estimated to about 15% of the suspended load.

Life of Reservoir : *The ultimate destiny of a reservoir is to be filled with silt deposits.* To allow for silting, a certain percentage of the total storage is usually left unutilised, and is called 'dead storge'. However, as the time passes on, more and more silting takes place and the 'live' or 'effective storage ' is gradually reduced. The

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useful life of reservoir is terminated when its capacity is reduced to 20 % of the designed capacity. The reservoir planning must, therefore, include the consideration of probable rate of silting so that the useful life of the reservoir may be determined.

The reservoir sedimentation is measured in terms of its *trap efficiency* (η). Trap efficiency of a reservoir is the percent of inflowing sediment which is retained in reservoir. Detailed observations show that the trap efficiency is a function of the ratio of reservoir capacity to the total inflow, *i.e.*

$$\eta = f\left(\frac{\text{Capacity}}{\text{Inflow}}\right).$$

Fig. 6.11 shows a plot between trap efficiency and capacity inflow ratio on the basis of the existing reservoirs. It is clear from the above curve that for a given inflow rate, the trap efficiency decreases with the reduction in reservoir capacity due to sediment deposit. Hence the rate of silting is higher in the initial stages, and it decreases as silting take place. Thus the complete filling of a reservoir with silt may take a very long time.

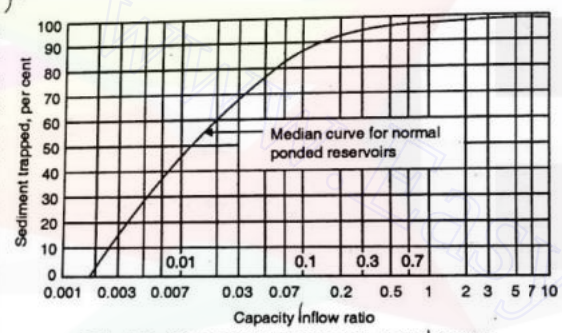

FIG. 6.11 TRAP EFFICIENCY OF RESERVOIRS

At the same time, a small reservoir (having small capacity) on a large stream (having large inflow rates) has a very small $\left(\text{Capacity / Inflow}\right)$ ratio. The trap efficiency for such a reservoir is extremely small and the stream passes most of its inflow so quickly that the finer sediments do not settle but are discharged downstream. On the other hand, a large reservoir constructed on small stream (having less inflow rates) has greater $\left(\text{Capacity / Inflow}\right)$ ratio. Such a reservoir has a greater trap efficiency. Such a reservoir may retain water for several years and permit almost complete deposition of the sediment.

Procedure for Calculation of Life of a Reservoir :

(1) The required reservoir capacity is determined from the considerations discussed in the previous article. Knowing the inflow rate calculate the $\left(\text{Capacity / Inflow}\right)$ ratio and obtain the trap efficiency from the curve (Fig. 6.11) for the full capacity of the reservoir.

(2) Divide the total capacity into any suitable interval, say 10%. Assuming that 10 % capacity has been reduced due to sediment deposit, find the trap efficiency for the reduced capacity (*i.e.*, 90 % of the original) and the inflow ratio.

(3) For this interval of 10 % capacity, find the average trap efficiency by taking the average of η found in steps (2) and (3).

(4) Determine the sediment inflow rate by taking water samples and drying the sediment. Multiply the total annual sediment transported by the trap efficiency found in step (3). Convert this sediment deposited into hectare-metre (volumetric) units deposited in one year.

(5) Divide the volume interval (*i.e.*,10 % of the capacity) by the sediment deposited (determined in step 4) to get the number of years to fill this volume interval of 10 % capacity.

(6) Repeat this procedure for further intervals, *i.e.* at 80%, 70%....., 20% of the capacity. The total life of the reservoir will be equal to the total of the number of years required to fill each of the volume intervals.

The procedure is illustrated in Example 6.3.

6.9. RESERVOIR SEDIMENT CONTROL

Following are some of the methods used for the control of silting of reservoirs:

(1) *Proper Selection of Reservoir Site*

The silt transported in the system depends upon the nature of the catchment area. A stream collecting water from catchment area having soft or loose soil, and having steep slopes, may carry more sediment load. Hence, the reservoir site should be such that it excludes runoff from easily erodable catchment area. If a certain tributary of the main stream carries more silt, the dam should be constructed to the upstream of that tributary.

(2) *Control of Sediment Inflow*

This is a preventive measure. Small check dams may be constructed across those tributaries which carry more silt. Increase of vegetal cover over the catchment area also decreases the soil erosion and hence sediment inflow is reduced.

(3) *Proper Designing and Reservoir Planning*

The sediment trapped in the reservoir also depends upon the reservoir planning. As discussed earlier, a small reservoir on a big river has lesser trap efficiency. Hence, if the dam is constructed lower in the first instance, and is being raised in stages, the life of the reservoir will be very much increased. During the floods, the sediment carried by the stream is the maximum. Hence, sufficient outlets should be provided in the dam at various elevations, so that the floods can be discharged to the downstream without much silt deposit.

(4) *Control of Sediment Deposit*

The sediment deposit in the reservoir can be controlled by proper operation of the gates of scouring sluices and the head regulators of the canals etc. These should be so designed and operated that selective removal of silt is affected. During the floods, when sediment inflow is higher, the scouring sluices must be opened to discharge the silt downstream.

(5) *Removal of Sediment Deposit*

This is a curative measure. The under-sluices may be used to scour the deposited silt. However, it has been observed that when the scouring sluices are operated, they do not remove the bulk of the silt. A narrow channel is scooped along its line and

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the rest of silt remains undisturbed. The silt deposit can also be removed by excavating and dredging processes, though this proposal is very costly. The best way, therefore, is to first disturb the deposited silt by mechanical means so that the silt is loosened and pushed towards the sluices, and then opening the sluices so that most of the silt is discharged downstream. Another useful way is to let most of the flood water pass through the scouring sluices with minimum detention in the reservoir. This will require strong structural components of the sluice gates.

(6) *Erosion Control in the Catchment Area*

Under this category fall the various methods of soil conservation, such as provision of control bunds, checking gully formation by providing small embankments, afforestation, regrassing and control of grazing etc. Provision of vegetation screen helps in reducing the 'sheet erosion'.

6.10. SINGLE PURPOSE FLOOD CONTROL RESERVOIRS

Flood control reservoirs are adopted to prevent loss of life and valuable property due to floods. Most often it is provided to protect the economic development in an area. There are two types of single purpose flood control reservoirs, namely :

(1) Retarding Reservoirs. (2) Detention Basins.

1. Retarding Reservoirs

A retarding reservoir is the one in which the spillway and out-lets are not controlled by gates or valves. Instead, they moderate the flood peak by passing discharge downstream, depending upon reservoir elevation. It has uncontrolled outlets and maximum discharge through the outlets do not exceed the safe carrying capacity of downstream river during the highest flood.

Advantages of a Retarding Reservoir

(1) There are no gates at the outlets and hence the possibility of human error in reservoir operation is eliminated.

(2) The cost of expensive gate installation and operation is saved.

Disadvantages

A serious disadvantage of retarding reservoirs is that automatic regulation may cause coincidence of flood crest farther downstream where two or more channels taking off from retarding reservoirs join together.

An ideal *location* of such reservoirs is immediately above a city or above the area to be protected from flood furiosity.

2. Detention Basins

A flood control reservoir with gate installations at the spillway and outlets is termed as a detention basin. The gate installation provide more flexibility of operation and thus increase the usefulness of reservoir. Gated outlets permit the release of any amount of discharge upto maximum. The reservoir storage can, therefore, be utilized more effectively by passing maximum safe discharge in downstream channel at every stage of flood. Also, the reservoir can be emptied more rapidly during early floods to prepare it to receive the subsequent floods.

The disadvantages of detention basins arise out of the possibility of human error during operation and high initial cost to be incurred for gate installation and subsequent maintenance. But the advantage that the flood crest in the downstream channel can be regulated properly so as not to cause their coincidence far outweighs its disadvantages. Detention should be adopted where the protection of large areas is needed and where the area extends for some distance downstream of reservoir site.

Maximum capacity of reservoir should be equal to the design inflow minus safe release in the downstream channel. However, reservoir size should be fixed by an economic analysis. The economic analysis involves the determination of benefit and cost ratio for different capacities of the reservoir. The capacity which gives maximum benefit cost ratio — which should be greater than one — should be adopted for design.

6.11. MULTIPURPOSE RESERVOIRS

The term 'mutlipurpose reservoir' includes all reservoirs actually designed and operated to serve more than one function and that it excludes those whose design and operation are controlled by a single function, even though other benefits accrue as by-products.

There can be several purposes for which a reservoir may be made. These are listed in § 6.1. If some of these purposes are combined, there will be more effective utilisation of water and economical construction of a reservoir. Preferable combinations for a multipurpose reservoir are :

(1) Reservoir for Irrigation and Power.
(2) Reservoir for Irrigation, Power and Navigation.
(3) Reservoir for Irrigation, Power and Water supply.
(4) Reservoir for Recreation, Fisheries and Wild life.
(5) Reservoir for Flood control and Water supply.
(6) Reservoir for Power and Water supply.
(7) Reservoir for Flood Control, Irrigation, Power and Water Supply. This is the most common combination.

Planning of a Multipurpose Reservoir

Multipurpose planning involves substantially more than simple combination of single purpose elements, if effective utilisation of water resources has to be brought about. None of the possible uses of a reservoir are entirely compatible with any other use, although under many circumstances it is possible to bring them into good agreement. The unique feature in the multipurpose design, therefore, is an operation plan which is an effective compromise among the various uses.

There are two possible extremes in allocation of reservoir storage :

(1) To assume that all storage is jointly used.

(2) To assume that no storage is jointly used.

In the second case, storage requirements for all functions are pyramided to create a large total storage requirement which can be economically obtained only when unit cost of storage is constant or decreases as total storage increases. The other extreme results in maximum economy since the required storage is not greater than that necessary

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for any one of the several purposes. Such a situation is relatively rare and the usual multipurpose reservoir is designed to fall somewhere between these two extremes.

For Indian conditions the usual combination is irrigation, hydroelectric power and flood control. Planning of such a reservoir requires a detailed analysis of past records of run-off and other hydrological data. The requirements of operation for such a reservoir are :

(1) to provide storage for flood control either by permanent allocation of space exclusively for flood control or by seasonal allocation of space for flood storage.

(2) to provide a steady discharge during April, May and June for 'Kharif' irrigation and a large discharge from mid-November to mid-February for 'Rabi' irrigation.

(3) to provide a steady discharge for running of turbines to generate maximum firm power.

Supposing the reservoir is located on an ice fed river, it will have lowest water level, sometimes upto dead storage level during February and March. This is because of the low supplies in the river and heavy draw of water for Rabi irrigation during preceeding months. During the months of April, May and June, the supplies in the river steadily increases as the snow starts melting. Reservoir has to supply water for power and Kharif irrigation during these months. Therefore, depending upon inflow and outflow, the reservoir surface may increase or decrease slightly at a slow rate. This goes on and just before monsoon, a large reservoir capacity is available for dual purpose of conservation and flood control. During monsoon months demand for irrigation is small. There is only a constant release of water needed for power and inflow in the river is very large. The reservoir surface rises and it is designed to rise upto maximum conservation level at the end of normal monsoon period, say middle October. Even at this time some flood control storage is available between conservation level and maximum reservoir level. Actual data by which the reservoir should be allowed to be filled upto maximum conservation level depends upon run-off pattern of past years and many a time upon wise descretion of operating authorities, who have studied the behaviour of flood for many years. During the month of October and November, the supplies in the river go down and the outflow increases to meet the demands for 'Rabi' irrigation and, therefore, reservoir surface goes down. During the months from November to February, draw-down from reservoir is heavy and the reservoir surface goes down rapidly except for some winter freshets in December and January.

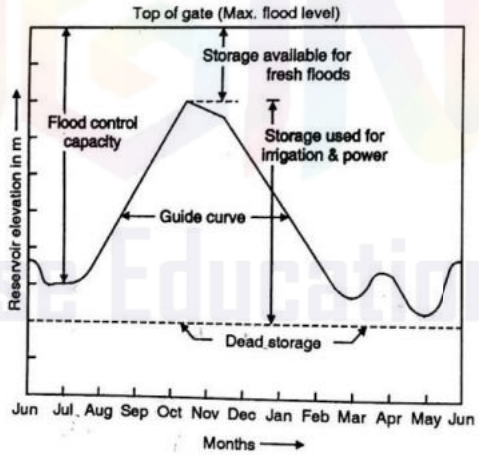

FIG. 6.12.

Seasonal allocation to flood control is done only where a definite flood season occurs such as the case in North India. However, a small portion need always be reserved for unusual floods.

Various features of the operation are usually expressed in the form of chart known as *rule curve*. The rule curve shows the amount of space to be left vacant on any date and dates during which the reservoir is to be used. Fig. 6.12 shows the operation plan for a reservoir intended to serve flood control, irrigation and power. The operation plan is usually a guide based on the analysis of past records to help the operators.

A good example of multipurpose reservoir is Damodar Valley Corporation (D.V.C.) on tributaries of river Damodar in Bihar and Bengal. It envisages 7 dams in Bihar and Bengal with a total of 490 sq km of reservoir area. It produces 200,000 kW of hydroelectric power, irrigates 0.4 million hectare of land, supplies 96 cusecs of drinking water, stores 60 ha-m of flood, besides doing navigation, soil conservation and eradication of malaria.

6.12. APPORTIONMENT OF TOTAL COST OF A MULTIPURPOSE RESERVOIR

Apportionment of total cost of a multipurpose reservoir means the sub-division of cost for various uses for which the reservoir is planned. Various methods of apportionment of total cost are as follows :

(1) Equal apportionment method. (2) Use of facilities method.
(3) Alternative justifiable expenditure method. (4) Remaining benefits method.

There is no really satisfactory method of cost allocation, which is equally applicable to all projects and which may yield unquestionable correct results. Any method of allocation must first set aside the *separable costs* which are chargeable to a single project function directly, such as the cost of a power-house structure and its equipments in case of a hydroelectric project, the cost of navigation lock in case of a navigation project and the cost of canal head works in case of irrigation project etc. The term *joint cost* used hereinafter, is the difference of total cost and separable cost.

1. Equal Apportionment Method

The joint cost of the project is equally divided into different purposes, irrespective of separate cost of benefit. The method is not based on convincing logic and is used only when data available are so meagre that other methods cannot be used.

2. Use of Facilities Method

The joint cost is divided in the ratio of storage volume provided for each purpose. The method is simple in use but the cost is not directly proportional to storage. In fact the cost per unit volume goes on decreasing as the storage increases and hence the cost of each unit storage is not the same.

3. Alternative Justifiable Expenditure Method : In this method, the joint cost is distributed in the ratio of difference between the separable cost and the estimated cost of a single-purpose project which would provide equivalent services and would itself be economically justifiable. The greatest difficulty lies in use of this method is the collection of data for estimation of alternative justifiable cost of a single purpose project.

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4. ***Remaining Benefit Method***

In this method, the joint cost is assumed to be distributed in accordance with the difference between separable cost and the estimated benefit of each function or its alternative justifiable cost whichever is lower. This is the best of all the methods and is sufficiently accurate, though here also it is difficult to find out the benefits or its alternative cost.

6.13. FLOOD ROUTING

It is the process of calculating water levels in reservoir, the storage quantities and outflow rates corresponding to a particular inflow hydrograph at various instants. Flood routing is carried out in a reservoir to determine what will be the maximum rise in its water surface and what will be the discharge in the downstream channel when particular flood passes through it. Flood routing is an important technique necessary for the complete solution of a flood control problem and for the satisfactory operation of a flood-prediction service.

Modification of a inflow flood hydrograph due to flood routing process is shown in Fig. 6.1. It can be seen therein that when a flood passes through the reservoir, its peak reduces, its hydrograph acquires a bigger time base and a great basin lag is introduced in it.

METHODS OF FLOOD ROUTING

A large number of methods are in use for flood routing process. Some of the important methods which have a more practical bearing are :

(1) Calculus method

(2) Step by step methods : (*a*) Graphical method (*b*) Trial and error method.

The calculus method is beyond the scope of present study.

Step by Step Methods

The relationship governing the inflow and outflow is essentially simple. The flood entering a given instant of time is party stored in the reservoir and is partly let out by the spillway and other outlets. The process is, however, complicated by the fact that neither the flood inflow hydrograph nor the storage and outlet discharge can be expressed by simple algebraic equations. A step by step procedure is, therefore, followed which can be done either graphically or by trial and error.

(*a*) GRAPHICAL METHOD : Inflow-Storage-Discharge (ISD) Curves Method

The governing relation between inflow, outflow and change in storage is

$$\bar{I} - \bar{O} = \Delta s \qquad ...(1) \qquad ...(6.4)$$

where $\bar{I}$ is average inflow during a given time period ; $\bar{O}$ is the average outflow during given time period, and Δs is volume of water stored in the reservoir during the same period. Equation (1) can also be written as,

$$\frac{I_1 + I_2}{2}\, t - \frac{O_1 + O_2}{2}\, t = s_2 - s_1 \qquad ...(2) \qquad ...(6.5)$$

where I_1 and I_2 are the inflow rates, O_1 and O_2 are the outflow rates at the beginning and end of the time interval t and s_1 and s_2 are the storage volumes in the beginning

and end of the time interval. Equation (2) can be written as,

$$(I_1 + I_2) + \left(\frac{2\,s_1}{t} - O_1\right) = \left(\frac{2\,s_2}{t} + O_2\right) \qquad ...(3) \qquad ...(6.6)$$

Following graphs are required to proceed further with the analysis.

1. *Inflow flood hydrograph* from which ordinates I_1, I_2 etc. can be found at choosen interval of time period. This is shown in Fig. 6.13 (Graph I).

2. *Elevation-outflow curve* of the reservoir, as shown in Fig. 6.14 (Graph II).

3. *Elevation-storage* curve of the reservoir, as shown in Fig. 6.14 (Graph III).

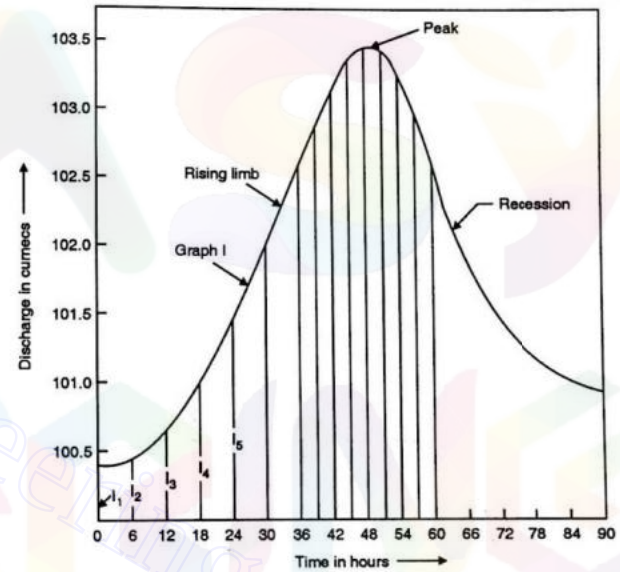

FIG. 6.13. INFLOW FLOOD HYDROGRAPH

Before starting the actual routing operations by the graphical method, the curves of $\left(\frac{2s}{t} \pm O\right)$ *v/s* outflow O are derived, as shown in Fig. 6.15. To do this, the values of s and O are taken, from Fig. 6.14, for various values of reservoir elevations. Knowing the corresponding values of s and O, values of $\left(\frac{2s}{t} \pm O\right)$ are computed and plotted against the corresponding outflow rates O (Fig. 6.15). In this, t is any choosen time interval.

Now, at the beginning of the routing period, both s_1 and O_1 are zero. Substituting these in Eq. (3), we get $(I_1 + I_2) = \frac{2s_2}{t} + O_2$.

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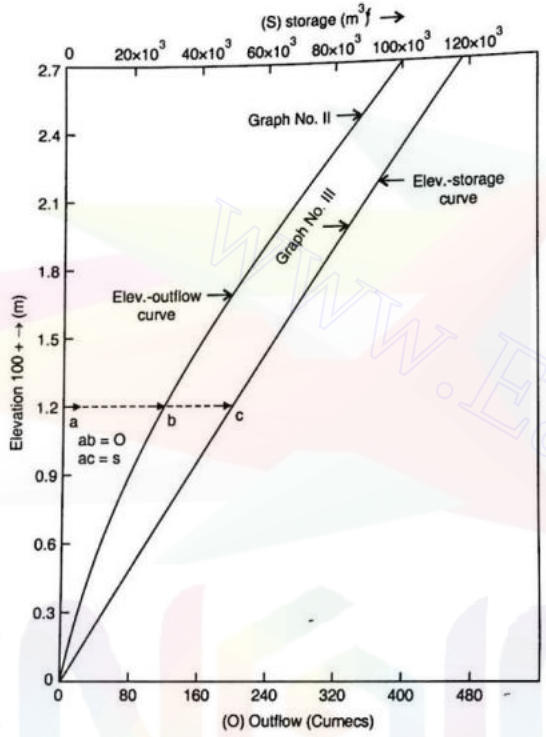

FIG. 6.14 ELEVATION OUTFLOW AND ELEVATION STORAGE CURVES.

For the first time interval t, the inflow rates I_1 and I_2 at its beginning and end are known from Fig. 6.13. Hence, $\left(\frac{2s_2}{t}+O_2\right)$ is known. In Fig. 6.15, enter this value of $\left(\frac{2s_2}{t}+O_2\right)$ and get O_2. For this value of O_2 also find $\left(\frac{2s_2}{t}-O_2\right)$ from the same figure. Now applying Eq. (3) for the next time interval, we get

$$(I_2+I_3)+\left(\frac{2\,s_2}{t}-O_2\right)=\left(\frac{2s_3}{t}+O_3\right)$$

In this equation, I_2 and I_3 are known from Fig. 6.13, and $\left(\frac{2s_2}{t}-O_2\right)$ is also known as explained above. Hence, $\left(\frac{2s_3}{t}+O_3\right)$ is known. Hence, from Fig. 6.15, find the value

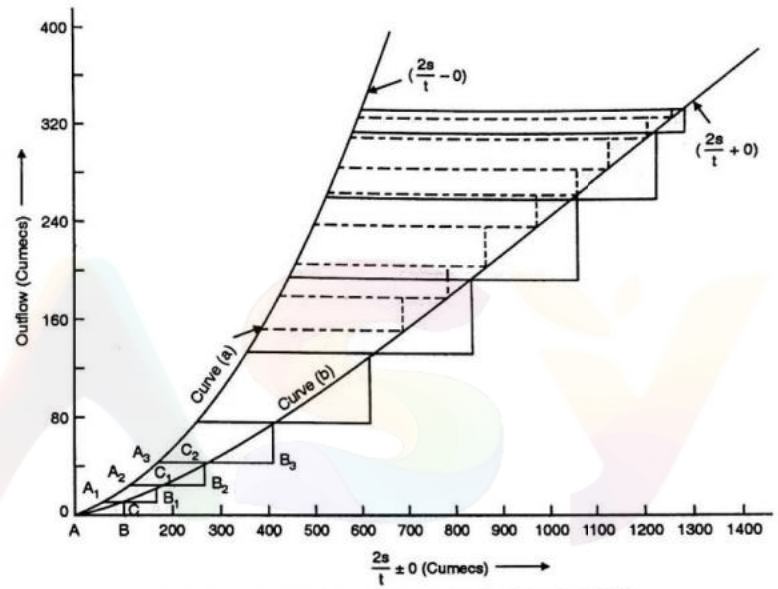

FIG. 6.15. FLOOD ROUTING BY GRAPHICAL METHOD.

of O_3 corresponding the value of $\left(\frac{2s_3}{t}+O_3\right)$. Also find $\left(\frac{2s_3}{t}-O_3\right)$ for this value of O_3. Similarly, applying Eq. (3) for the next time interval, we get

$$(I_3+I_4)+\left(\frac{2\,s_3}{t}-O_3\right)=\left(\frac{2s_4}{t}+O_4\right)$$

From which $\left(\frac{2s_4}{t}+O_4\right)$ is known and the process is repeated for the complete period of flood.

The graphical method can thus be summarised by the following steps:

(*i*) Calculate the total inflow from Fig. 6.13 for the time interval t and enter it on Fig. 6.15 as the value AB.

(*ii*) Draw vertical line from B to meet the curve of $\left(\frac{2s}{t}+O\right)$ in a point C. Point C gives the value of outflow at the end of interval.

(*iii*) Draw a horizontal line through C to cut the curve of $\left(\frac{2s}{t}-O\right)$ in point A_1.

(*iv*) Calculate the total inflow during the next period from inflow flood hydrograph and mark and measure it as $A_1\,C\,B_1$.

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(*v*) Erect a perpendicular from B_1 to meet the curve of $\left(\frac{2s}{t}+O\right)$ in a point C_1 and repeat the same procedure as outlined in (*iii*) and (*iv*) until the entire flood is routed. The outflow discharge at any time interval is given by the total vertical ordinate. The biggest of these ordinates will give the value of the peak outflow rate for which the spillway is to be designed.

For example, the peak outflow rate obtained in Fig. 6.15 is 334 cumecs. When flood routing is continued, the vertical ordinates decrease, representing the receding flood.

(*vi*) The outflow discharge at various time intervals having been determined as described above, the reservoir water surface elevation for these can be determined from Fig. 6.14, Graph No.II.

Semi-graphical method : Instead of drawing the vertical ordinates and doing it purely graplicalhy, the values can be tabulated, as illustrated in Table 6.4 of Example 6.1.

(*b*) **Modified Puls' method**

The modified Puls' method is simpler than the conventional graphical method (ISD method), since it uses lesser number of graphs for computations. The basic equation used for flood routing is the same i.e. Eq. 6.6 :

$$I_1+I_2+\left(\frac{2s_1}{t}-O_1\right)=\left(\frac{2s_2}{t}+O_2\right) \quad ...(6.6)$$

Procedure

1. Choose a suitable time interval t and prepare a curve between elevation as ordinate and the function $\left(\frac{2s}{t}+O\right)$ as abscissa, as shown in Fig. 6.16.

2. At the beginning of the routing process, s_1 and O_1 are zero and hence all the terms on the L.H.S. of of Eq. 6.6 are known. Hence compute the value of the function $\left(\frac{2s_2}{t}+O_2\right)$.

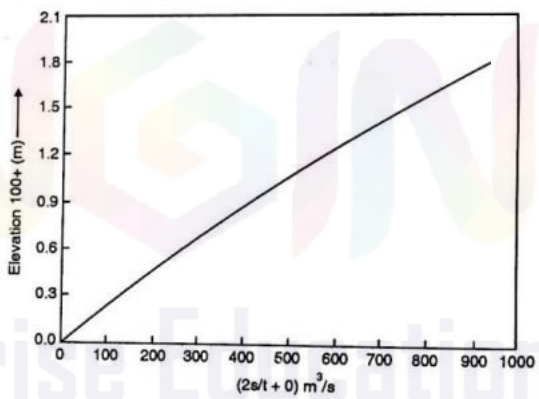

FIG. 6.16

3. For the value of function $\left(\frac{2s_2}{t}+O_2\right)$ determined in step 2 above, find the value of elevation from Fig. 6.16 and outflow rate O_2 from Fig. 6.14.

4. Compute the value of the function $\left(\frac{2s_2}{t}-O_2\right)$ from the known value of the function $\left(\frac{2s_2}{t}+O_2\right)$ and O_2, using the following equation :

$$\left(\frac{2s_2}{t}-O_2\right)=\left(\frac{2s_2}{t}+O_2\right)-2O_2 \quad ...(6.7)$$

5. Next, compute the value of $\left(\frac{2s_3}{t}+O_3\right)$, using Eq. 6.6 modified as under

$$\left(\frac{2s_3}{t}+O_3\right)=(I_2+I_3)+\left(\frac{2s_2}{t}-O_2\right)$$

In the above equation all the terms in the R.H.S. are completely known. Thus the value of $\left(\frac{2s_3}{t}+O_3\right)$ is known, corresponding to which elevation is found from Fig. 6.16 and O_3 is found from Fig. 6.14.

6. Repeat the above procedure till the entire inflow hydrograph is routed.

7. Determine the maximum water level and the maximum outflow rate from the values obtained in the above procedure.

(*c*) **TRIAL AND ERROR METHOD**

This is a more commonly used method. The steps required for flood routing are described below :

(*i*) Divide the inflow flood hydrograph of Fig. 6.13 into various small intervals of time.

(*ii*) Begin with the first period and calculate the total inflow during the interval by multiplying the average inflow rate at the beginning and end of the period with the time interval $\left(=\frac{I_1+I_2}{2}t\right)$.

(*iii*) The water surface elevation at the beginning of flood is known. Asume a trial value of water surface elevation in the reservoir at the end of period t.

(*iv*) From the *elevation-outflow curve* [Fig. 6.14] find out the outflow at the beginning and end of period for the corresponding water surface elevations. The average value of outflow multiplied by the time interval represents the net outflow during the period $\left(=\frac{O_1+O_2}{2}t\right)$.

(*v*) From the *elevation-storage curve* [Fig. 6.14] find out the storage at the beginning and end of the period as the corresponding water surface elevations are known. Find out their difference (s_2-s_1) which represents the amount of flood stored in the reservoir during the period.

(*vi*) Add the value of outflow obtained in step (*iv*) to the value of storage obtained in step (*v*). This value should be equal to the total inflow during the periods as calculated in step (*ii*). If this is so the assumed trial elevation is correct, otherwise another value

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of trial elevation should be assumed and the trial repeated so as to bring the coincidence between the total inflow with the sum of outflow and storage.

(*vii*) Repeat steps (*ii*) to (*vi*) for other time intervals, till the flood is routed.

A typical example illustrating trial and error procedure follows.

Example 6.1. *The following data (Table 6.1) pertains to an inflow flood hydrograph whose flows in 100 cumecs have been recorded at 6 hours interval starting from 0.00 hours on June 1, 1959 on a certain stream.*

TABLE 6.1

0.42	0.45	0.57	0.88	1.47	2.10
2.72	3.40	3.50	3.38	3.14	2.88
2.63	2.40	1.98	1.70	1.43	1.20

This flood approachs a reservoir with uncontrolled spillway with elevation area and elevation outflow data as shown in Table 6.2.

TABLE 6.2

Elevation (m)	100.00	100.3	100.6	100.9	101.2	101.5	101.8	102.1	102.4	102.7
Area (ha)	405	412	420	425	428	436	445	453	460	469
Outflow (cumecs)	0	14.9	42.2	77.3	119	169	217	272	334	405

The water level just reaches the crest level (elevation 100.00 m) of the spillway at 4 hours on June 1, 1959.

Determine the maximum reservoir level and maximum discharge over the spillway. Draw inflow and routed hydrographs indicating the reduction in peak flow and peak lag introduced due to routing.

(a) *Solution by semi-graphical method*

1. Draw the inflow hydrograph as shown in Fig. 6.13 Graph-I.
2. Draw elevation-outflow curve, as shown in Fig. 6.14 Graph II.
3. Draw elevation storage curve, as shown in Fig. 6.14 Graph III. The starting elevation for this spillway is crest level. For plotting this curve, volume between different contours is found by cone formula, or by any other formula.
4. The values of $\frac{2s}{t} \pm O$ are calculated in the Table 6.3.

The values of column (7) and column (8) are plotted against column (2), as shown in Fig. 6.15.

The routing procedure is exactly the same as described under the graphical method. Instead of drawing the vertical ordinates and doing it purely graphically, the values can be tabulated in the following table (Table 6.4). The values of $\left(\frac{2s}{t} - O\right)$ and O are read each time from Fig. 6.15 for different values of $\frac{2s}{t} + O$. The tabulated values are given in Table 6.4. Assume a time interval of 6 hours.

TABLE 6.3

Elevation (m)	Outflow (cumces) O	Area enclosed by each contour (A) sq m × 10^4	Storage between contours (s)= H/3 × [$A_1 + A_2 + \sqrt{A_1 A_2}$]	Cumulative storage above spillway crest	$\frac{2s}{t}$ for t = 6 hr = $\frac{s}{10800}$	$\frac{2s}{t} - O$	$\frac{2s}{t} + O$
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
100.0	0	405					
100.3	14.9	412	122.4	122.4	113.3	98.4	128.2
100.6	42.2	420	124.8	247.2	228.5	186.3	270.7
100.9	77.3	425	126.8	374.0	346.0	268.7	428.3
101.2	119	428	128.0	502.0	465.0	348.0	584.0
101.5	167	436	129.6	631.9	585.0	418.0	752.0
101.8	217	445	132.2	763.8	706.0	489.0	923.0
102.1	272	453	134.8	898.6	831.5	559.5	1103.5
102.4	334	460	137.0	1035.6	957.5	623.5	1291.5
102.7	405	469	139.4	1175.0	1088.0	683.0	1493.0

The method of drawing vertical ordinates and corresponding horizontal ordinates are shown in Fig. 6.15.

TABLE 6.4

Time (Starting from June 1, 1959) (1)	Inflow cumecs I (2)	$I_1 + I_2$ (3)	2s/t–O Values read from curve a of Fig. 6.15 (4)	$\frac{2s}{t} + O$ (5)	Outflow O [Read from Fig. 6.15 for Col. (5)] (6)	Reservoir Elevation [Read from Fig. 6.14 for Col. (6)] (7)
hours 0	42					
6	45	87	0	87	10	100.20
12	57	102	60	162	24	100.39
18	88	145	122	267	42	100.58
24	147	235	185	420	74	100.86
30	210	357	266	623	130	101.26
36	272	482	362	844	194	101.65
42	340	612	455	1067	260	102.03
48	350	690	545	1235	316	102.31
54	338	688	605	1293	334*	102.40*
60	314	652	623	1275	328	102.37
66	288	602	620	1222	312	102.30
72	263	551	600	1151	286	102.18
78	240	503	575	1078	264	102.06
84	198	438	550	988	236	101.90
90	170	368	515	883	204	101.72
96	143	313	470	783	177	101.56
102	120	263	430	693	150	102.40

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Results.

*1. Maximum reservoir level, from column (7) = 102.4 m
2. Maximum discharge over spillway, from column (6) = 334 cumecs.
3. Reduction in peak discharge = 350 – 334
(= Maximum inflow – Maximum outflow) = 16 cumecs
4. Peak lag = (Time for maximum inflow) – (Time for maximum outflow)
= 54 – 48 = 6 hours.

The inflow flood hydrograph and outflow flood hydrograph have been plotted together in Fig. 6.17.

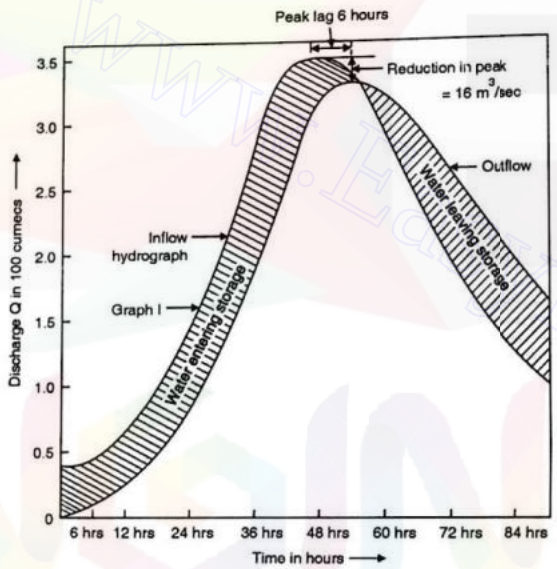

FIG. 6.17.

(b) Solution by Trial and Error Method

Make a table having 12 columns (Table 6.5). Choose a time interval of 6 hours in the beginning of flood and 3 hours during the period the flood is near its peak, and enter into column (1). Begin routing the flood from spillway crest onward and enter the reservoir elevation at the beginning of the period in column (2). Calculate the inflow flood volume from flood inflow hydrograph (plotted in Fig. 6.13) for the time interval under consideration and enter it into column (3). Find the spillway discharge for the reservoir elevation at the beginning of period from elevation outflow curve of Fig. 6.14 and enter it into column (4). Assume a trial reservoir elevation, and enter it into column (5). For this trial reservoir elevation, calculate the spillway discharge and enter into column (6). Find out the mean outflow rate at the beginning and end of period and enter it into column (7). Find the outflow volume from the mean outflow rate $\left(\frac{O_1+O_2}{2}\,t\right)$ and enter into column (8). The outflow is less than inflow volume so long as the peak of flood has not been routed. Once the peak of flood has been routed, the outflow volume exceeds the inflow volume and the maximum condition may be taken to have occurred. From the storage elevation curve (Fig. 6.14) find the storage capacity between the resevoir elevation at the beginning

and end of period and enter it into column (9). Find out the sum of outflow volume [column(8)] and storage capacity [column(9)] and enter it into column (10). Find out the difference between column (9) and column (10) and enter it into column (11). This differnce should be negligible, otherwise the trial should be repeated. . Mention may be made about the trials made and maximum conditions for the outflow flood in the remarks column (12).

TABLE 6.5. FLOOD-ROUTING BY TRIAL AND ERROR METHOD

Time period hours beginning from June 1	*Reservoir Elevation : beginning of period (metres)*	*Inflow Volume (million cubic metres)*	*Spillway discharge : beginning of period (cumecs)*	*Trial Reservoir Elevation : end of period (metres)*	*Spillway discharge : end of period (cumecs)*	*Mean outflow rate (cumecs)*	*Outflow Volume (million cubic metres)*	*Storage capacity between Reservoir Elevation in beginning and end of period (million cubic metres)*	*Outflow Volume + Storage capacity (million cubic metres)*	*Col. (3) – Col. (10)*	*Remarks*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*	*(8)*	*(9)*	*(10)*	*(11)*	*(12)*
0–6	100.00	0.940	0	100.20	10	5.0	0.108	0.832	0.940	0.000	
6–12	100.20	1.100	10	100.39	24	17.0	0.367	0.740	1.107	– 0.007	
12–18	100.39	1.565	24	100.58	42	33.0	0.712	0.850	1.562	0.003	
18–24	100.58	2.540	42	100.86	74	58.0	1.250	1.290	2.540	0.000	
24–30	100.86	3.860	74	101.26	130	102.0	2.200	1.665	3.865	0.005	
30–36	100.26	5.200	130	101.65	194	162.0	3.500	1.700	5.200	0.000	
36–39	101.65	3.300	194	101.83	224	209.0	2.260	1.045	3.305	0.005	
39–42	101.83	3.490	224	102.02	260	242.0	2.610	0.880	3.490	0.000	
42–45	102.02	3.725	260	102.165	284	272.0	2.940	0.780	3.720	0.005	
45–48	102.165	3.755	284	102.31	316	300.0	3.240	0.515	3.755	0.000	*Max.
48–51	102.31	3.750	316	102.355	325	320.5	3.460	0.290	3.750	0.000	condition
51–54	102.355	3.680	325	102.40*	334*	329.5	3.506	0.180	3.686	0.006	
54–57	102.40	3.508	334	102.385	331	332.5	3.590	–0.080	3.510	0.002	

*Maximum reservoir elevation = **102.40 m** *Maximum discharge over spillway = **334 cumecs**

Example 6.2. *Table 6.6 gives the monthly inflows during the critical low water period at the site of a proposed dam, the corresponding monthly evaporation and precipitation at a nearby station and the estimated monthly demand for water. Prior water rights require the release of full natural flow or 10 ha-m per month, whichever is least. Assume that 30 percent of the rainfall on the land area to be flooded by the reservoir has reached the stream in the past. Using a net increased pool area of 500 hectares, find the required useful storage. Use a pan evaporation coefficient of 0.72.*

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TABLE 6.6

Months	Inflow at the proposed site (ha-m)	Pan evaporation (cm)	Precipitation (cm)	Demand (ha-m)
January	8.6	2.2	0.8	14.5
February	2.2	2.3	1.2	15.8
March	1.8	3.1	0.0	16.2
April	0.0	8.6	0.0	16.8
May	0.0	12.8	0.0	17.5
June	13.5	15.6	4.8	18.0
July	280.6	12.3	12.2	18.0
August	510.2	10.6	18.6	17.0
September	136.0	10.0	8.6	16.5
October	52.5	8.6	1.5	16.0
November	20.6	5.8	0.0	15.8
December	12.3	3.0	0.0	15.0

Solution . Computations are arranged in Table 6.7 in which the first five colums are the same as given in Table 6.6. Column 6 gives the downstream requirements due to water right flow and the entries in this column are equal to the river flow or 10 hectare metres whichever is the minimum.

TABLE 6.7

Month	Inflow (ha-m)	Pan Evap. (cm)	Precipitation (cm)	Demand (ha-m) (D)	D / S Require-ments (ha-m)	Evaporation E= 3.6 ×col.(3) (ha-m)	Precipitation P = 3.5 × col. (4) (ha-m)	Adjusted Inflow I (ha-m) col. (2)– col. (6) – col. (7) + col. (8)	Water reqd.from storage S = col. (5) – col. (9) (ha-m)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
January	8.6	2.2	0.8	14.5	8.6	7.92	2.8	– 5.12	19.62
February	2.2	2.3	1.2	15.8	2.2	8.28	4.2	– 4.08	19.88
March	1.8	3.1	0.0	16.2	1.8	11.16	0.0	– 11.16	27.36
April	0.0	8.6	0.0	16.8	0.0	30.96	0.0	– 30.96	47.76
May	0.0	12.8	0.0	17.5	0.0	46.08	0.0	– 46.08	63.58
June	13.5	15.6	4.8	18.0	10.0	56.16	16.8	– 35.86	53.86
July	280.6	12.3	12.2	18.0	10.0	44.28	42.7	+ 269.02	Nil
August	510.2	10.6	18.6	17.0	10.0	38.16	65.1	+ 527.14	Nil
September	136.0	10.0	8.6	16.5	10.0	36.00	30.1	+ 120.10	Nil
October	52.5	8.2	1.5	16.0	10.0	29.52	5.25	+ 18.23	Nil
November	20.6	5.8	0.0	15.8	10.0	20.88	0.0	– 10.28	26.08
December	12.3	3.0	0.0	15.0	10.0	10.80	0.0	– 8.50	23.50
Sum	1038.3	94.5	47.7	197.1	82.6	340.20	166.95	782.45	281.64

Column 7 gives the evaporation over the whole of the reservoir area. Its value E is calculated from the relation :

$$E = \frac{\text{Reservoir area} \times \text{Pan evaporation} \times \text{Pan evaporation coefficient}}{100}$$

$$= \frac{500 \times \text{Column (3)}}{100} \times 0.72 \text{ hectare-metres} = 3.6 \times \text{Column (3) hectare-metres.}$$

Column 8 gives the precipitiaion in hectare-metres, falling over the reservoir area. Since 30 % of the precipitation is already reaching, and has been included in the inflow (column 2), only 70% of the precipitaion has been included in the computation. Thus the precipitation P is calculated from the ralation

$$P = \frac{\text{Reservoir area} \times \text{Column(4)}}{100} \times 0.7 = \frac{500 \times \text{Column (4)}}{100} \times 0.7$$

$$= 3.5 \times \text{Column (4) hectare-metres.}$$

Column 9 gives the *adjusted Inflow* (I) computed from the relation

$$I = \text{Column (2)} - \text{Column (6)} - \text{Column (7)} + \text{Column(8)}$$

Column (10) gives the water required from storage, *(i.e.S)* and as computed from the following relation :

$$S = D - I$$

or Column (10) = Column (5) – Column (9).

In the above relation, only *positive values* are to be included. When column(9). *i.e.* adjusted inflow is more than column (5), zero values are to be written in column (10) indicating that no water is required from the storage since demand is much less than the adjusted inflow . The required storage is the sum of the monthly increments of demand in excess of stream flow. The required storage capacity in this case works out to be 281.64 ha-m.

Example 6.3. *Table 6.8 gives the details about the average seasonal discharges of a river for 12 years. Determine the storage capacity required to maintain a flow of 475 cumecs throughout the year.*

TABLE 6.8

Year	16th June to 30 Sept. (cumecs)	1st Oct-31 st March (cumecs)	1st April -15th June (cumecs)
1960-61	1050	300	50
1961-62	3000	250	40
1962-63	3500	370	90
1963-64	2000	150	120
1964-65	1200	350	65
1965-66	1400	400	100
1966-67	3600	200	80
1967-68	3000	150	120
1968-69	700	210	50
1969-70	800	120	80
1970-71	2400	320	120
1971-72	3200	280	80

Solution. Let the periods from 16th June to 30th Sept., 1st Oct. to 31st March and 1st April to 15th June be designated as M, W and S respectively. The data for preparing mass inflow curve are tabulated in Table 6.9. The mass inflow curve is shown in Fig. 6.18.

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TABLE 6.9. DATA FOR MASS-INFLOW CURVE

S.N.	Year	Season	No of days	Av. discharge (cumecs)	Volume (million ha-m)	Cumulative Volume (million ha-m)
1.	1960-61	M	107	1050	0.9707	0.9707
		W	182	300	0.4717	1.4424
		S	76	50	0.0328	1.4752
2.	1961-62	M	107	3000	2.7734	4.2486
		W	182	250	0.3981	4.6417
		S	76	40	0.0263	4.6680
3.	1962-63	M	107	3500	3.2357	7.9037
		W	182	370	0.5818	8.4855
		S	76	90	0.0591	8.5436
4.	1963-64	M	107	2000	1.8490	10.3935
		W	182	150	0.2356	10.6293
		S	76	120	0.0788	10.7083
5.	1964-65	M	107	1200	1.1094	11.8177
		W	182	350	0.5504	12.3681
		S	76	65	0.0427	12.4108
6.	1965-66	M	107	1400	1.2943	13.7051
		W	182	400	0.6290	14.3341
		S	76	100	0.0657	14.3998
7.	1966-67	M	107	3600	3.3281	17.7279
		W	182	200	0.3145	18.0424
		S	76	80	0.0525	18.0949
8.	1967-68	M	107	3000	2.7734	20.8673
		W	182	150	0.2359	21.1032
		S	76	120	0.0788	21.1820
9.	1968-69	M	107	700	0.6441	21.8291
		W	182	210	0.3302	22.1593
		S	76	50	0.0328	22.1921
10.	1969-70	M	107	800	0.7396	22.9317
		W	182	120	0.1887	23.1204
		S	76	80	0.0525	23.1729
11.	1970-71	M	107	2400	2.2188	25.3917
		W	182	320	0.5032	25.8949
		S	76	120	0.0788	25.9737
12	1971-72	M	107	3200	2.9583	28.9320
		W	182	280	0.4403	28.3723
		S	76	80	0.0525	29.4248

Yearly demand = 475×365 cumec-days $= 475 \times 365 \times 8.64 = 1{,}497{,}960$ ha-m

$\simeq$ 1.5 millon ha-m.

In Fig. 6.18, T_1, T_2, T_3, T_4, T_5, etc. are drawn tangent to the apexes, and parallel to the demand curve which has a slope of 1.5 milion ha-m in 1 year. The ordinates $O_1, O_2, \ldots O_5$, indicate the deficiencies during the dry periods, assuming that the reservoir was full at the beginning of the period. The maximum of these ordinates (*i.e.* O_5 =1.6 m. ha-m) gives the desired reservoir capacity.

Fig. 6.19 shows the enlarged view of the curve from period 1967 to 1971 during which maximum storage is requird. Line *AB* is drawn parallel to the demand curve

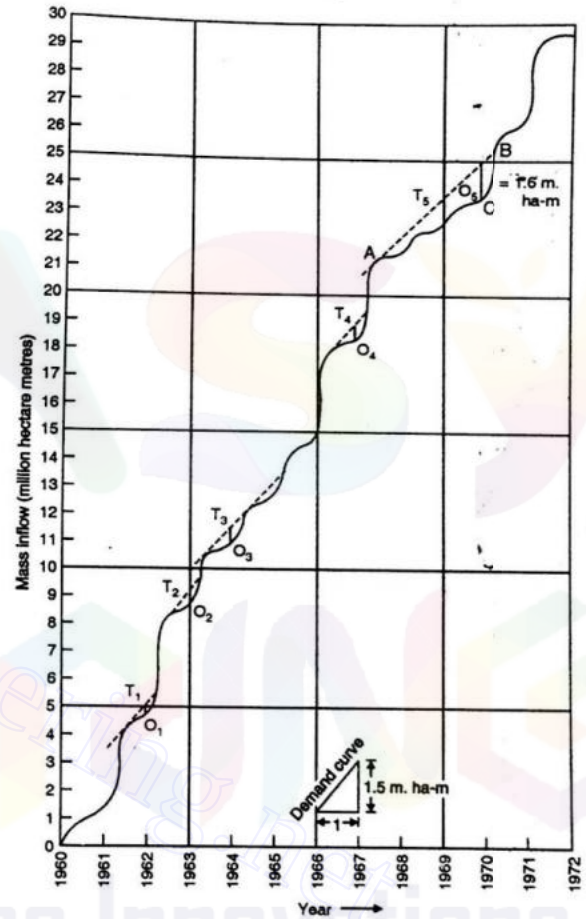

FIG. 6.18. DETERMINATION OF RESERVOIR CAPACITY.

and tangential to the mass inflow curves at point *A*. At point *C* of curve, a storage capacity of 1.6 million hectare metres is required. It is essential that the demand line *AB* should meet the inflow curve at point *B*, so that reservoir becomes full at *B*, otherwise it will never be full. Similarly, if a line *CD* is drawn parallel to the demand curve, and tangential to the mass-inflow curve at *C*, then it should intersect the curve at *D* so that the reservoir becomes full at the start of the dry period.

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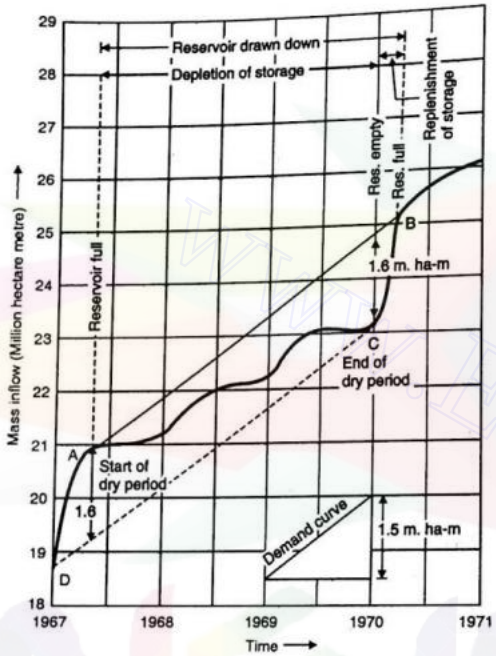

FIG. 6.19

Example 6.4. *The following information is available regarding the relationship between trap efficiency and capacity-inflow ratio for a reservoir :*

$\frac{\text{Capacity}}{\text{inflow}}$ ratio	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
Trap efficiency (η %)	87	93	95	95.5	96	96.5	97	97.2	97.3	97.5

Find the probable life of the reservoir with an initial reservoir capcity of 30 million cubic metres, if the annual flood inflow is 60 million cubic metres and the average annual sediment inflow is 3600,000 kN. Assume a specific weight of sediment equal to 12 kN/m^3. The useful life of the reservoir will terminate when 80% of initial capacity is filled with sediment.

Solution

Annual sediment inflow = 3600,000 kN = 3.6×10^6 kN.

$\therefore$ Volume of sediment inflow $= \frac{3.6 \times 10^6}{12} = 0.3$ million cubic metres.

Initial reservoir capacity = 30 m.c.m.

Annual flood inflow = 60 m.c.m.

$\therefore$ Initial capacity/inflow ratio = 30 / 60 = 0.5.

This ratio goes on decreasing as the sedimentation takes place, and hence trap efficiency goes on decreasing. The computations for the life of the reservoir are done in Table 6.10. The volume interval chosen in the Table is 6 m.c.m. (*i.e.* 20% of initial capacity).

TABLE 6.10

Capacity %	Capacity Volume $\times 10^6$ m^3)	Capacity Inflow Ratio	Trap efficiency η	Average η for the Interval	Annual Sediment, trapped, S_t (m.c.m)	Year to fill $= \frac{6}{S_t}$
100	30	0.5	0.96			
				0.9575	0.2872	20.9
80	24	0.4	0.955			
				0.9525	0.2857	21.0
60	18	0.3	0.95			
				0.9400	0.2820	21.3
40	12	0.2	0.93			
				0.9000	0.2700	22.2
20	6	0.1	0.87			
					Total	85.4 years.

Hence the useful life of the reservoir is $\simeq$ **85 years.**

Example 6.5. *A small reservoir has a water spread area given by $A = (30 + 6\,h)$ M . m^2, when h is the head of water over spillway crest. The spillway discharge is given by $q = 300\,h^{3/2}$ cumecs. The inflow hydrograph due to a certain storm is as under.*

Time from start (hours)	0	3	6	9	12	15	18	21	24
Inflow (m^3/s)	60	480	900	470	270	160	110	80	60

Compute the maximum outflow discharge over the spillway and the corresponding maximum level of water above the spillway crest, if the reservoir level was at the spillway crest at the start.

Solution

Let us solve the problem analytically by proceeding step by step and using the basic routing equation. Let us assume that the head over the spillway crest at the end of each 3-hour interval be $h_1, h_2, h_3 \ldots\ldots h_n$ metres.

1. For first interval of 0 to 3 hours

Let h_1 be the head over the spillway crest at the end of the interval.

Inflow : $I_1 = 60$ cumecs ; $I_2 = 480$ cumecs.

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$\therefore$ Total inflow $= \frac{1}{2}(I_1 + I_2)\,t = \frac{1}{2}(60 + 480) \times 3 \times 3600 = 2.916$ M . m^3

Outflow : $O_1 = 0;\ O_2 = 300\,h_1^{3/2}$

$\therefore$ Total outflow $= \frac{1}{2}(O_1 + O_2)\,t = \frac{1}{2}(0 + 300\,h_1^{3/2})\,3 \times 3600 = 1.62\,h_1^{3/2}$ M . m^3

Storage : $s_1 = 0$

Water spread area at spillway crest $= 30$ M . m^2

Water spread area at height h_1 above crest $= 30 + 6\,h_1$

$\therefore$ $s_2 = \frac{1}{2}\,[30 + (30 + 6\,h_1)]\,h_1 = [30 + 6\,(h_1/2)]\,h_1 = (30 + 3h_1)\,h_1$

[i.e. storage = (water spread area at $h_1/2$) $\times\ h_1$]

Applying the basic equation for the first interval :

Total inflow = Total outflow + Change in storage

$\therefore$ $2.916 = (1.62\,h_1^{3/2}) + (30 + 3\,h_1)\,h_1$

or $h_1^2 + 0.54\,h_1^{3/2} + 10\,h_1 - 0.972 = 0$

Solving this by hit and trial, we get $h_1 = 0.0954$ m

Note: In the first trial, neglecting h_1^2 and $h_1^{3/2}$ we get $h_1 = 0.972/10 = 0.0972$ m. Hence the value of h_1 for the next trial will be less than 0.0972 m.

2. For the second interval of 3 hours to 6 hours

Let the head over the crest be h_2 m at the end of the interval

Total inflow $= \frac{1}{2}(480 + 900)\,3 \times 3600 = 7.452$ M . m^3

Total outflow $= \frac{1}{2}[300\,(0.0954)^{3/2} + 300\,h_2^{3/2}]\,3 \times 3600 = 0.0477 + 1.62\,h_2^{3/2}$

$\Delta s = [(30 + 3\,h_2)\,h_2 - (30 + 3 \times 0.0954)\,0.0954] = 30\,h_2 + 3h_2^2 - 2.8893$

Hence $7.452 = (0.0477 + 1.62\,h_2^{3/2}) + (30\,h_2 + 3\,h_2^2 - 2.8893)$

or $h_2^2 + 0.54\,h_2^{3/2} + 10\,h_2 - 3.4312 = 0$

Solving this by hit and trial, we get $h_2 = 0.323$ m

3. For the third interval of 6 hours to 9 hours:

Let the head over the spillway crest be h_3 m at the end of the interval.

Total inflow $= \frac{1}{2}\,(900 + 470)\,3 \times 3600 = 7.398$ M . m^3

Total outflow $= \frac{1}{2}\,[300\,(0.323)^{3/2} + 300\,(h_3)^{3/2}] \times 3 \times 3600 = 0.2974 + 1.62\,h_3^{3/2}$ M . m^3

$\Delta s = \left[\left\{(30 + 3\,h_3)\,h_3\right\} - \left\{30 + 3\,(0.323)\right\}0.323\right] = 30\,h_3 + 3\,h_3^2 - 10.0030$ M . m^3

$\therefore$ $7.398 = (0.2974 + 1.62\,h_3^{3/2}) + (30\,h_3 + 3\,h_3^2 - 10.0030)$

or $h_3^2 + 0.54\,h_3^{3/2} + 10\,h_3 - 5.7012$

Solving this by hit and trial, we get $h_3 = 0.522$ m

4. For the fourth interval of 9 hours to 12 hours

Let the head over spill way crest be h_4 m at the end of the interval.

Total inflow $= \frac{1}{2}\,(470 + 270)\,3 \times 3600 = 3.996$ M . m^3

Total outflow $= \frac{1}{2}[300\,(0.522)^{3/2} + 300\,h_4^{3/2}] \times 3 \times 3600 = 0.611 + 1.62\,h_4^{3/2}$ M . m^3

$\Delta s = \left[\left\{(30 + 3\,h_4)\,h_4\right\} - \left\{30 + 3\,(0.522)\right\}0.522\right] = 30\,h_4 + 3\,h_4^2 - 16.4775$

$\therefore$ $3.996 = (0.611 + 1.62\,h_4^{3/2}) + (30\,h_4 + 3\,h_4^2 - 16.4775)$

or $h_4^2 + 0.54\,h_4^{3/2} + 10\,h_4 - 6.6208 = 0$

Solving this by hit and trial, we get $h_4 = 0.601$ m

5. For the fifth interval of 12 hours to 15 hours

Let the head over the spillway crest be h_5 m at the end of the interval

Total inflow $= \frac{1}{2}\,(270 + 160)\,3 \times 3600 = 2.322$ M . m^3

Total outflow $= \frac{1}{2}\,[300\,(0.601)^{3/2} + 300\,h_5^{3/2}]\,3 \times 3600 = 0.7548 + 1.62\,h_5^{3/2}$ M . m^3

$\Delta s = \left[\left\{(30 + 3\,h_5)\,h_5\right\} - \left\{30 + 3\,(0.601)\right\}0.601\right] = 30\,h_5 + 3\,h_5^2 - 19.1136$ M . m^3

$\therefore$ $2.322 = (0.7548 + 1.62\,h_5^{2/2}) + (30\,h_5 + 3\,h_5^2 - 19.1136)$

or $h_5^2 + 0.54\,h_5^{3/2} + 10\,h_5 - 6.8936 = 0$

Solving this by hit and trial, we get $h_5 = 0.624$ m

6. For the sixth interval of 15 hours to 18 hours

Let the head over the spillway crest be h_6 m at the end of the interval.

Total inflow $= \frac{1}{2}\,(160 + 110)\,3 \times 3600 = 1.458$ M . m^3

Total outflow $= \frac{1}{2}\,[300(0.624)^{3/2} + 300\,h_6^{3/2}]\,3 \times 3600 = 0.7985 + 1.62\,h_6^{3/2}$ M . m^3

$\Delta s = \left[\left\{(30 + 3\,h_6)\,h_6\right\} - \left\{30 + 3\,(0.624)\right\}0.624\right] = 30\,h_6 + 3\,h_6^2 - 19.8881$

$\therefore$ $1.458 = (0.7985 + 1.62\,h_6^{3/2}) + (30\,h_6 + 3\,h_6^2 - 19.8881)$

or $h_6^2 + 0.54\,h_6^{3/2} + 10\,h_6 - 6.8492 = 0$

Solving this by hit and trial, we get $h_6 = 0.620$ m

Since $h_6 < h_5$, the head has started reducing.

Hence $h_{max} = h_5 =$ **0.624 m**

Corresponding maximum outflow discharge $= 300\,(0.624)^{3/2} =$ **147.88 cumecs**

Example 6.6. *A multipurpose project has a total cost of 240 million rupees. For the data given below, calculate the allocations to each project purpose, by the following methods.*

(*a*) *Remaining benefits method* (*b*) *Alternate justifiable expenditure method.*

Item	*Flood control*	*Power generation*	*Irrigation*
1. Separable costs	*Rs. 32 million*	*Rs. 88 million.*	*Rs 72 million*
2. Estimated benefits	*Rs 40 million*	*Rs 138 million*	*Rs 112 million*
3. Alternate single purpose cost	*Rs 47 million*	*Rs 104 million*	*Rs 101 million*

Solution : Total cost of project = 240 million $= 240 \times 10^5$

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Total separable cost = (32 + 88 + 72) million = 192×10^6

∴ Total joint cost = $(240 - 192)\ 10^6 = 48 \times 10^6$

(a) *Allocation of cost by remaining benefits method*

The computation for allocation are done in the tabular form below.

(All cost in millions of rupees)

Line	*Item*	*Flood control*	*Power generation*	*Irrigation*	*Total*
1.	Benefits limited by alternate cost	40	104	101	245
2.	Remaining benefits	40 – 32 = 8	104 – 88 = 16	101 – 72 = 29	245 – 192 = 53
3.	Allocated joint cost	7.2453	14.4906	26.2641	48.0
4.	Total allocation	39.2453	102.4906	98.2641	240.0
5.	Total allocation (%)	16.352	42.705	40.943	100.0

In the table, *line 1* gives the estimated benefits, or alternate single purpose cost, whichever is less. Thus the figures for flood control, power generation and irrigation are 40, 104 and 101 million rupees respectively, making a total of Rs 245 millions.

Line 2 gives the remaining benefits, which are obtained by subtracting the separate costs from the data of line 1.

Line 3 gives the allocated joint costs for each purpose, obtained as under :

$$\textit{Allocated joint cost} = \frac{\textit{Total joint cost}}{\textit{Total remaining benefits}} \times \textit{remaining benefit of that purpose}$$

Thus, allocated joint cost for flood control $= \frac{48}{53} \times 8 = 7.2453$ million

Allocated joint cost for power $= \frac{48}{53} \times 16 = 14.4906$ mllion

Allocated joint cost for irrigation $= \frac{48}{53} \times 29 = 26.2641$ million

Line 4 gives the total allocation, which is equal to the sum of separable cost and the allocated joint cost. Finally, *line 5* gives the total allocation in percentage.

(b) *Allocation of cost by alternate justifiable expenditure method*

The computations for cost allocation are done in Tabular form below.

(All costs in millions of Rupees)

Line	*Item*	*Flood control*	*Power generation*	*Irrigation*	*Total*
1.	Alternate cost less separable cost	47 – 32 = 15	104 – 88 = 16	101 – 72 = 29	60
2.	Allocated joint cost	12	12.8	23.2	48.0
3.	Total allocation	32 + 12 = 44	88 + 12.8 = 100.8	72 + 23.2 = 95.2	240.0
4.	Total allocation (%)	18.333	42	39.667	100

Line 1 in the above table is obtained by subtracting separable cost from alternate single purpurpose cost, for each allocation.

Line 2 gives the allocated joint costs, obtained from the relation :

$$\textit{Allocated joint cost} = \left[\frac{\textit{Total joint cost}}{\textit{Total alternate cost less separable cost}}\right] \times \left[\textit{Alternate cost less separable cost of that purpose}\right]$$

Thus, allocated joint cost for flood control $= \frac{48}{60} \times 15 = 12$ millions

Allocated joint cost for power $= \frac{48}{60} \times 16 = 12.8$ millions

Allocated joint cost for irrigation $= \frac{48}{60} \times 29 = 23.2$ millions

Line 3 gives the *total allocation* which is equal to the sum of separable cost and allocated joint cost for each purpose.

Line 4 gives the total allocation in percentage.

6.14. EXAMPLES FROM COMPETITIVE EXAMINATIONS

Example 6.7. *The Muskingnum method by McCarthy assumes the reach storage in a stream to be given by $S = K[x\,I + (1-x)\,O]$ where K is the storage constant. Also basic routing equation written for discrete time is*

$$\frac{I_1 + I_2}{2} - \frac{O_1 + O_2}{2}\,t = S_2 - S_1$$

Derive from these Muskingnum equation and incidently determine the coefficients therein. What is the sum of coefficients ? **(*Engg. Services Exam. 1986*)**

Solution

Given the reach storage equation as under :

$$S = K\,[x\,I + (1-x)\,O] \qquad ...(1)$$

Also, the basic routing equation is

$$\frac{I_1 + I_2}{2}\,t - \frac{O_1 + O_2}{2}\,t = S_2 - S_1 \qquad ...(2)$$

Substituting the following values of S_1 and S_2 from (1) into (2), we have

$$S_1 = K\,[x\,I_1 + (1-x)\,O_1] \text{ and } S_2 = K\,[x\,I_2 + (1-x)\,O_2]$$

$$\therefore \quad \frac{I_1 + I_2}{2}\,t - \frac{O_1 + O_2}{2}\,t = K\,[x\,I_2 + (1-x)\,O_2] - K\,[x\,I_1 + (1-x)\,O_1]$$

or $$\frac{I_1 + I_2}{2}\,t + K\,[x\,I_1 + (1-x)\,O_1] = \frac{O_1 + O_2}{2}\,t + K\,[x\,I_2 + (1-x)\,O_2]$$

or $$(I_1 + I_2) + \frac{2K}{t}\,[x\,I_1 + (1-x)\,O_1] = O_1 + O_2 + \frac{2K}{t}\,[x\,I_2 + (1-x)\,O_2]$$

or $$I_1 + I_2 + \frac{K\,x\,I_1}{0.5\,t} + \frac{K}{0.5\,t}(1-x)\,O_1 = O_1 + O_2 + \frac{x\,K\,I_2}{0.5\,t} + \frac{K\,O_2\,(1-x)}{0.5\,t}$$

or $$\left[I_1 + \frac{K\,x\,I_1}{0.5\,t}\right] + \left[I_2 - \frac{x\,K\,I_2}{0.5\,t}\right] + \left[\frac{K\,(1-x)}{0.5\,t}\,O_1 - O_1\right] = \left[O_2 + \frac{K\,(1-x)}{0.5\,t}\,O_2\right]$$

or $$O_2\left[\frac{0.5\,t + K\,(1-x)}{0.5\,t}\right] = I_1\left[\frac{0.5\,t + K\,x}{0.5\,t}\right] + I_2\left[\frac{0.5\,t - K\,x}{0.5\,t}\right] + O_1\left[\frac{K\,(1-x) - 0.5\,t}{0.5\,t}\right]$$

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or $$O_2(K - Kx + 0.5t) = I_1(Kx + 0.5t) + I_2(-Kx + 0.5t) + O_1(K - Kx - 0.5t)$$

or $$O_2 = I_1\left[\frac{Kx + 0.5t}{K - Kx + 0.5t}\right] + I_2\left[\frac{-Kx + 0.5t}{K - Kx + 0.5t}\right] + O_1\left[\frac{K - Kx - 0.5t}{K - Kx + 0.5t}\right]$$

or $$O_2 = C_0 I_2 + C_1 I_1 + C_2 O_1$$

Which is the required Muskingnum equation, where

$$C_0 = \frac{-Kx + 0.5t}{K - Kx + 0.5t}, \quad C_1 = \frac{Kx + 0.5t}{K - Kx + 0.5} \quad \text{and} \quad C_2 = \frac{K - Kx - 0.5t}{K - Kx + 0.5t}$$

Sum of the coefficients $C_0 + C_1 + C_2$

$$= \frac{1}{K - Kx + 0.5t}[-Kx + 0.5t + Kx + 0.5t + K - Kx - 0.5t] = 1$$

Example 6.8. *The storage in a stream reach has been studied; x and K have been identified as 0.28 and 1.6 days. If the inflow hydrograph in the stream reach, as the flood starts coming in and passes, is given by the following table, compute, the outflow hydrograph (Plotting is not needed).*

Time (hours)	0	6	12	18	24	30
Inflow (m^3/sec)	35	55	92	130	160	140

Solution : **(Engg. Services Exam. 1986)**

The Muskingum equation has been derived in the previous Example, in the form

$$O_2 = C_0 I_2 + C_1 I_1 + C_2 O_1$$

Here $x = 0.28$ and $K = 1.6$ days $= 1.6 \times 24 = 38.4$ hours, $t = 6$ hours.

Hence $$C_0 = \frac{-Kx + 0.5t}{K - Kx + 0.5t} = \frac{-38.4 \times 0.28 + 0.5 \times 6}{38.4 - 38.4 \times 0.28 + 0.5 \times 6} = -0.253$$

$$C_1 = \frac{Kx + 0.5t}{K - Kx + 0.5t} = \frac{38.4 \times 0.28 + 0.5 \times 6}{38.4 - 38.4 \times 0.28 + 0.5 \times 6} = 0.449$$

$$C_2 = \frac{K - Kx - 0.5t}{K - Kx + 0.5t} = \frac{38.4 - 38.4 \times 0.28 - 0.5 \times 6}{38.4 - 38.4 \times 0.28 + 0.5 \times 6} = 0.804$$

Sum : $C_0 + C_1 + C_2 = -0.253 + 0.449 + 0.804 = 1$ (OK)

The computations for outflow rate (i.e. O_2) at the end of each interval by the following Muskingum equation are carried out in the Tabular form below.

$$O_2 = C_0 I_2 + C_1 I_1 + C_2 O_1$$

Time (hours)	Inflow (cumecs)	$C_0 I_2 = -0.253 I_2$	$C_1 I_1 = 0.449 I_1$	$C_2 O_1 = 0.804 O_1$	Outflow (cumecs)
(1)	(2)	(3)	(4)	(5)	(6) = (3) + (4) + (5)
00	35	–	–	–	35.00
06	55	$-0.253 \times 55 = -13.915$	$0.449 \times 35 = 15.715$	$0.804 \times 35 = 28.14$	29.94*
12	92	$-0.253 \times 92 = -23.276$	$0.449 \times 55 = 24.695$	$0.804 \times 29.94 = 24.072$	25.49*
18	130	$-0.253 \times 130 = -32.89$	$0.449 \times 92 = 41.308$	$0.804 \times 25.49 = 20.494$	28.91
24	160	$-0.253 \times 160 = -40.48$	$0.449 \times 130 = 58.37$	$0.804 \times 28.91 = 23.244$	41.13
30	140	$-0.253 \times 140 = -35.42$	$0.449 \times 160 = 71.84$	$0.804 \times 41.13 = 33.068$	69.49

Column (6) of the above table gives the values of the outflow hydrograph. The initial computed outflow drops because of sharp increase in inflow.

Example 6.9. *The following Table gives the monthly inflow into and contemplated demand from a reservoir*

Month	Monthly demand ($M.m^3$)	Evaporation (cm)	Rainfall (cm)	Monthly inflow ($M.m^3$)
Jan.	50	6	1	50
Feb.	75	8	0	40
March	80	13	0	30
April	85	17	0	25
May	130	22	0	20
June	120	22	19	30
July	25	14	43	200
Aug.	25	11	39	225
Sept.	40	13	22	150
Oct.	45	12	6	90
Nov.	50	7	2	70
Dec.	60	5	1	60

Assume the reservoir area as 30 km^2. The runoff coefficient of area flooded by the reservoir is 0.4. Estimate the minimum storage to meet the demand.

(Civil Services Exam. 1989)

Solution Note that 1 cm-$km^2 = \frac{1}{100} \times 10^6\ m^3 = 10^4\ m^3 = 0.01\ M.m^3$.

Table below gives the data about evaporation, rainfall, effective rainfall and net inflow/outflow.

(1)	Month	Jan.	Feb.	March	April	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
(2)	Evap. (E) (cm)	6	8	13	17	22	22	14	11	13	12	7	5
(3)	Rainfall (R) (cm)	1	0	0	0	0	19	43	39	22	6	2	1
(4)	Eff. Rainfall $R_e = 0.4R$ (cm)	0.4	0	0	0	0	7.6	17.2	15.6	8.8	2.4	0.8	0.4
(5)	Net inflow/outflow $= R_e - E$ (cm)	– 5.6	– 8.0	– 13.0	– 17.0	– 22.0	– 14.4	+3.2	+4.6	– 4.2	– 9.6	– 6.2	– 4.6
(6)	Net inflow/ outflow volume ($M.m^3$) $= (5) \times 0.01 \times A$	– 1.68	– 2.4	– 3.9	– 5.1	– 6.6	– 4.32	+0.96	+1.38	– 1.26	– 2.88	– 1.86	– 1.38

Note that – ve = outflow and + ve = inflow into reservoir

The computations for reservoir capacity are done in a tabular form below.

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Month	Monthly demand (Mm^3)	Monthly inflow (Mm^3)	Net inflow/outflow (from above Table) Mm^3	Net demand ($M.m^3$)	Cumulative demand ($M.m^3$)	Cumulative inflow (Mm^3)	Excess demand (Mm^3)	Excess supply (Mm^3)
(1)	(2)	(3)	(4)	(5)= (2) + (4)	(6)	(7)	(8)= (6) – (7)	(9)
Jan.	– 50	50	– 1.68	– 51.68	51.68	50	1.68	
Feb.	– 75	40	– 2.40	– 77.40	129.08	90	39.08	
March	– 80	30	– 3.90	– 83.90	212.98	120	92.96	
April	– 85	25	– 5.10	– 90.10	303.08	145	158.08	
May	– 130	20	– 6.60	– 136.60	439.68	165	274.68	
June	– 120	30	– 4.32	– 124.32	564.00	195	369.00	
July	– 25	200	+0.96	– 24.04	588.04	395	193.04	
Aug.	– 25	225	+1.38	– 23.62	611.66	620		8.34
Sept.	– 40	150	– 1.26	– 41.26	652.92	770		117.08
Oct.	– 45	90	– 2.88	– 47.88	700.80	860		159.20
Nov.	– 50	70	– 1.86	– 51.86	752.66	930		177.34
Dec.	– 60	60	– 1.38	– 61.38	814.04	990		175.96

Minimum storage required = Max. of excess demand = 369.00 Mm^3

Example 6.10. *The amounts of water flowing from a certain catchment area at the proposed dam site are tabulated blow. Determine.*

(i) The minimum capacity of the reservoir if water is to be used to feed the turbines of hydropower plant at an uniform rate and no water is to be spilled over.

(ii) The initial storage required to maintain the uniform demand as above. (***Engg. Services Exam. 1999***)

Monthly	Inflow ($\times 10^5 m^3$)
January	2.83
February	4.25
March	5.66
April	18.40
May	22.64
June	22.64
July	19.81
August	8.49
September	7.10
October	7.10
November	5.66
December	5.66

Solution

The computations are done in a tabular form below.

Month	Inflow ($\times 10^5 m^3$)	Av. Demand ($\times 10^5 m^3$)	Deficit ($\times 10^5 m^3$)	Surplus ($\times 10^5 m^3$))
(1)	(2)	(3)	(4)= (3) – (2)	(5)= (2) – (3)
January	2.83	10.8533	8.0233	
February	4.25	10.8533	6.6033	
March	5.66	10.8353	5.1933	
April	18.40	10.8533		7.5467
May	22.64	10.8533		11.7867
June	22.64	10.8533		11.7867
July	19.81	10.8533		8.9567
August	8.49	10.8533	2.3633	
September	7.10	10.8533	3.7533	
October	7.10	10.8533	3.7533	
November	5.66	10.8533	5.1933	
December	5.66	10.8533	5.1933	
Sum	130.24×10^5	130.24×10^5		40.0768×10^5

Average demand = (Σ Inflow)/12 = $130.24 \times 10^5/12 = 10.853 \times 10^5$ m^3

This value has been entered in column (3) of the above Table.

Since no water is to be spilled, minimum capacity will be equal to the sum of the surplus water

∴ Minimum capacity = Σ Surplus = $\mathbf{40.0768 \times 10^5\ m^3}$

Initial storage = Σ Surplus – Σ Deficit before January

$= [40.0768 - (2.3633 + 3.7533 + 3.7533 + 5.1933 + 5.1933)] \times 10^5$

$= \mathbf{19.8203 \times 10^5\ m^3}$

PROBLEMS

1. Explain various types of reservoirs. What do you understand by multipurpose reservoir?
2. Describe in brief various investigations required for reservoir planning.
3. What are the factors on which the selection of the site of a reservoir depend?
4. Define the following : surcharge storage, valley storage, safe yield and secondary yield.
5. What do you understand by mass inflow curve and how is it prepared?
6. What do you understand by demand curve ? Explain the method of calculating reservoir capacity for a specified yield, from the mass inflow curve.
7. Explain how would you determine safe yield from a reservoir of a given capacity.
8. Write a note on reservoir sedimentation. How do you estimate the probable life of a reservoir ?
9. Discuss various methods of reservoir sediment control.
10. What is flood routing ? Explain the basic flood routing equation and outline its method of solution.
11. Explain the graphical method of flood routing.
12. Describe the trial and error method of flood routing.

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7

Dams-I : General

7.1. INTRODUCTION

A dam is a hydraulic structure constructed across a river to store water on its up-stream side. It is an impervious or fairly impervious barrier put across a natural stream so that a reservoir is formed. This water is then utilised as and when it is needed. Due to the construction of the dam, water level in the river at its up-stream side is very much increased, and a large area may be submerged depending upon the water spread of the reservoir so formed.

Classification of Dams. Dams may be classified into different categories, depending upon the purpose or basis of the classification. Table 8.1 gives a summary of various types of dams.

TABLE 7.1. TYPES OF DAMS

Basis of Classification	*Types*	*Common Examples*
(a) Classification according to use	(*i*) Storage dam	Gravity dam, earth dam, rockfill dam, Arch dam etc.,
	(*ii*) Diversion dam	Weir, barrage
	(*iii*) Detention dam	Dike, water spreading dam, debris dam
(b) Classification by hydraulic design	(*i*) Overflow dam	Spillway
	(*ii*) Non-overflow dam	Gravity dam, earth dams, rockfill dam
(c) Classification by materials	(*i*) Rigid dams	Gravity dam, arch dam, buttress dam, steel dam, timer dam
	(*ii*) Non-rigid dams	Earth dam, rockfill dam

7.2. CLASSIFICATION ACCORDING TO USE

Based on use, dams are classified as follows :

(*i*) Storage dam,
(*ii*) Diversion dam,
(*iii*) Detention dam.

Storage dam. This is the most common type of dam normally constructed. Storge dam is constructed to impound water to its upstream side during the periods of excess

supply in the river (*i.e.* during rainy season) and is used in periods of deficient supply. Behind such a dam, a reservoir or lake is formed. The storage dams may be constructed for various purposes, such as for irrigation, water power generation or for water supply for public health-purposes, or it may be for a mult-purpose project. A storge dam may be constructed of wide variety of materials, such as stone, concrete, earth and rockfill etc.

Diversion dam. The purpose of a diversion dam is essentially different. While a storage dam stores water at its upstream for future use, a diversion dam simply raises water level slightly in the river and thus provides head for carrying or *diverting* water into ditches, canals, or other conveyance systems to the place of use. A diversion dam is, therefore, of smaller height and no reservoir is formed to store water. The common examples of diversion dams are weirs and barrages. During the floods, water passes over or through these diversion dams while during periods of normal flow, the river water, partly or wholly, is diverted to irrigation channel etc. A diversion dam may be constructed for irrigation or municipal or industrial uses.

Detention dam. A detention dam is constructed to store water duriong floods and release it gradually at a safe rate, when the flood recedes. By the provision of artificial storage during the floods. flood damage downstream is reduced. See Fig. 6.1 for the flood hydrograph which is moderated by the flood control reservoir formed by the construction of a detention dam. There are usually two types of detention dams. In the first type discussed above, water is temporarily stored and released through a suitable outlet structure. In the other type of detention dam, water is not released and no outlet structure is provided. Instead, water is held in the reservoir as long as is possible. This held water seeps into pervious banks and foundation strata. Due to this seepage of water, water-level in wells in the adjoining area is increased and lift irrigation may be possible. Also, irrigation may be done in the river bed itself at the downstream side. The seepage water may be sufficient for the growth of the crop and no additional surface watering may be necessary. Such a detention dam is sometimes called *water-spreading dam* or *dike*, and is similar to a weir in constructional features. Some times, detention dams are constructed across tributaries carrying large silt and sediment. In such a case, it is known as a *debris dam* used to trap the sediment and thus to exclude the sediment to flow to the main reservoir formed on the main river to which the tributary meets.

In a multipurpose river valley project, the dam may serve the purposes of storage, flood protection and recreation. The stored water may also be used for irrigation, water power generation, municipal and industrial supply and other purposes.

7.3. CLASSIFICATION ACCORDING TO HYDRAULIC DESIGN

According to hydraulic design dams may be classified as follows (Fig.7.1) :

1. Non-overflow dam. 2. Overflow dam.

Non-overflow dam. A non-overflow dam is the one in which the top of the dam is kept at a higher elevation than the maximum expected high flood level. Water is not permitted to overtop the dam. Hence a non-overflow dam may be constructed of wide variety of materials, such as a earth, rockfill, masonry, concrete etc. Fig. 7.1(*a*) shows a typical non-overflow type concrete gravity dam.

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Overflow dam. An overflow dam is the one which is designed to carry surplus discharge (including floods) over its crest. Its crest level is kept lower than the top of the other portion of the dam (*i.e.* non-overflow dam). Since water glides over its downstream face it should be made of such a material which is not easily eroded by flowing water. Such dams are generally made of concrete or masonry. An overflow dam is commonly known as a *spillway* [Fig. 7.1(*b*)].

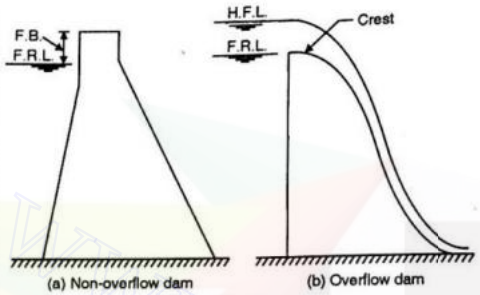

FIG. 7.1

Very often, in a river valley project, the two types of dams are combined. The main dam is kept as non-overflow dam made of either rigid materials such as masonry or concrete or non-rigid material such as earth and rockfill and some portion of dam is kept as overflow dam (spillway) at some suitable location along the main dam.

7.4. CLASSIFICATION ACCORDING TO MATERIAL

According to this most common classification, the dam may be classifed as follows:

1. Rigid dams. 2. Non-rigid dams

Rigid dams. Rigid dams are those which are constructed of rigid materials such as masonry, concrete, steel or timber. Rigid dams may be further classified as follows:

1. Solid masonry or concrete gravity dam.
2. Arched masonry or concrete dam. 3. Concrete buttress dam.
4. Steel dam. 5. Timber dam.

Non-rigid dams. Non-rigid dams are those which are constructed of non-rigid materials such as earth and/or rockfill. The most common types of non-rigid dams are :

1. Earth dam. 2. Rockfill dam. 3. Combined earth and rockfill dam.

7.5. GRAVITY DAMS

A *gravity dam* is the one in which the external forces (such as water pressure, wave pressure, silt presure, uplift pressure etc.) are resisted by the weight of the dam itself. Thus the forces disturbing the stability of the dam are resisted by the gravity forces of the mass of the dam. A *gravity dam* may be constructed either of masonry or of concrete. *Masonry gravity dams* are now-a-days constructed of only small heights. All major and important gravity dams are now constructed of concrete only [Fig.7.1(*a*)]. A gravity dam may be either straight or curved in plan.

Advantages of gravity dams. There is no type of dam more permanent than one of the solid concrete, nor does any other type require less for maintenance. As compared to earth and rockfill dams gravity dams have the following advantages :

1. Gravity dams are relatively more strong and stable than earth dams. They are particularly suited across gorges having very steep side slopes where earth dam, if constructed, might slip.

2. Gravity dams are well adapted for use as an overflow spillway crest. Earth dams cannot be used as overflow dams. Due to this, a gravity overflow dam is often used for the spillway feature of earth and rockfill dams.

3. Gravity dams can be constructed of any height, provided suitable foundations are available to bear the stresses. The height of an earth dam is usually limited by the stability of its slopes requring a very wide base width. Highest dams in the world are made of gravity dams only.

4. Gravity dam is specially suited to such areas where there is likelihood of very heavy downpour. The slopes of earth dam might get washed away in such a situation.

5. A gravity dam requires the least maintenance.

6. The failure of a gravity dam, if any, is not sudden. It gives enough warning time before the area to downstream side is flooded due to the damage to the gravity dams. On the contrary, an earth dam generally fails suddenly.

7. Deep-set sluices can be used in the gravity dams, to retard the sedimentation or silt deposit in the reservoir. The trap efficiency of a reservoir of an earth dam is more than that of a reservoir of gravity dam.

8. A gravity dam is cheaper in the long run since it is more permanent than any other type. Thus the *benefit-cost ratio* of such a dam is always higher.

Disadvantages. The disadvantages of gravity dam, as compared to an earth dam are as follows :

1. Gravity dams can be constructed only on sound rock foundations. They are unsuitable on weak foundations or on permeable foundations on which earth dams can be constructed with suitable foundation treatment.

2. The initial cost of a gravity dam is always higher than an earth dam. Hence, where funds are limited and where suitable materials are availble for the construction of an earth dam, the earth dam may be preferred.

3. If mechanised plants, such as manufacturing and transporting mass concrete, curing of concrete etc. are not available, a gravity dam may take more time to construct.

4. Gravity dams require skilled labour or mechanised plants for its construction.

5. It is very difficult to allow subsequent rise in the height of a gravity dam, unless specific provisions have been made in the intial design.

7.6. ARCH DAMS

An arch dam (Fig. 7.2) is a dam curved in plan and carries at major part of its water load horizontally to the abutments by arch action. This part of water load depends primarily upon the amount of curvature. The balance of the water load is transferred to the foundation by cantilever action. The thrust developed by the water load carried by arch action essentially requires strong side walls of the canyon to resist the arch forces. The weight of arch dams is not counted on to assist materially in

The solid dam made of concrete that is curved upstream in plan is called arch dam

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the resistance of external loads. For this reason, uplift on the base is not an important design factor.

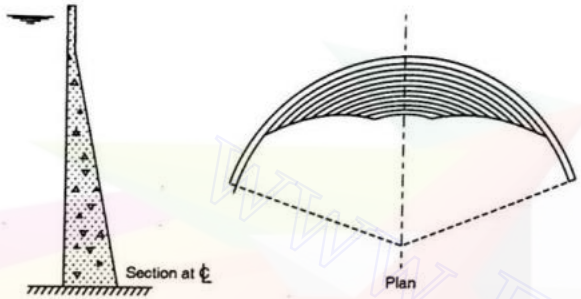

FIG. 7.2. ARCH DAM.

Advantages of Arch Dams

1. Arch dams are particularly adapted to the gorges where the length is small in proportion to the height.
2. For a given height, the section of an arch dam is much lesser than a corresponding gravity dam. Hence, an arch dam requires less material and is, therefore, cheaper.
3. Because of much less base width, the problems of uplift pressure are minor.
4. Since only a small part of water load is transferred to the foundation by cantilever action, an arch dam can be constructed in moderate foundations where gravity dam requiring sound foundation rock may be unsuitable.

Disadvantages

1. It requires skilled labour and sophisticated form work. The design of an arch dam is also quite specialized.
2. The speed of construction is normally slow.
3. It requires very strong abutments of soild rock capable of resisting arch thrust. Hence, it is not suitable in the locations where strong abutments are not available. Unfortunately, only few sites are suitable for this type of dam.

7.7. BUTTRESS DAMS

A buttress dam (Fig.7.3) consists of a number of buttresses or piers dividing the space to be dammed into a number of spans. To hold up water and retain the water between these buttresses, panels are constructed of horizontal arches or flat slabs. When the panels consist of arches, it is known as *multipe arches type buttress dam.* If the panels consist of flat slab, it is known as *deck type buttress dam.* Other types of buttress dam are described in chapter 9.

Advantages of Buttress Dams

1. A buttress dam is less massive than a gravity dam. Hence, the foundation pressures are less in the case of a buttress dam, and it can be constructed even on weak foundations on which the gravity dam cannot be supported.

2. The water load acts normal to the inclined deck. Hence the vertical component of the water load stablises the dam against both overturning and sliding and the butteress dam, possesses a factor of safety much greater than that obtained in a gravity dam.
3. The ice pressure is relatively unimportant since the ice tends to slide over the inclined u/s deck.
4. In the case of gravity dam, the height of the dam can raised only by the provision of crest shutter at overflow section. However in the case of a buttress dam, further raising of the height is possible and convenient by extending buttress and slab as shown in Fig. 7.4. Consequently, buttress dams are used where a future increase in reservoir capacity is contemplated.

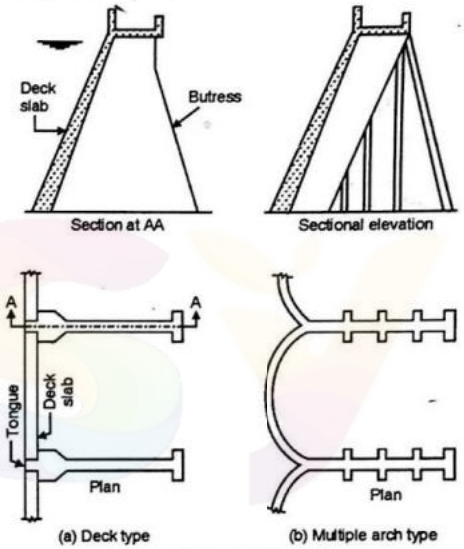

FIG. 7.3. BUTTRESS DAMS.

5. Power houses and water treatment plants can be housed in between buttresses, thus saving some cost of construction.
6. The amount of concrete used in buttress dam is about 1/2 to 1/3 of the concrete used in gravity dam of the same height. However, the cost of construction of a buttress dam is not low in that ratio because of the increased cost of reinforcement and of form work.
7. Access is possible to the back of upstream face and to foundations between buttresses for periodic inspection and for subsequent grouting and drilling of pressure relief holes if required.
8. Depending on the degree of articulation or structural isolation provided, buttress dams may be designed to accommodate moderate amounts of foundation movement without serious damages.
9. The most efficient use of strength of concrete lead to the economy in quantity required, though the financial economy will not necessarily be directly proportional to the quantity of concrete saved.

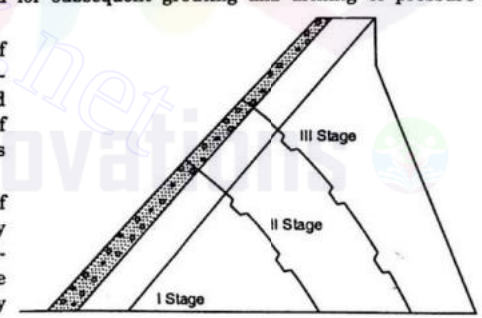

FIG. 7.4. RAISING HEIGHT OF BUTTRESS DAM.

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10. The reduction in concrete volume and increase in the surface area to volume ratio provide for better heat dissipation during construction and possibly increased speed of construction because of the larger exposed area and the thinner section do not produce a problem for cooling.

Disadvantages of Buttress Dams

1. Skilled labour requirements and the shuttering concrete ratio are greater than for solid dams. This may lead to higher limit rates and so offset some of the saving due to reduction in the quantity of concrete.

2. Deterioration of upstream concrete surface has serious effects on buttress dams with very thin concrete face.

3. Buttress dam is more susceptible to willful damage. The degree of vulnerability would probably depend upon the thickness of the upstream face and the facility for access from the downstream side.

7.8. STEEL DAMS

Steel dams are constructed with a framework of steel with a thin skin plate as deck slab on the upstream side. In India, no such dams has been constructed. However, in United States three such dams have been constructed : Ash Fork Dam in Arizona (1898), Redridge Dam in Michigan (1905) and Hauser Lake Dam in Montana (1901). Out of these the first two dams gave satisfactory results while the third dam failed only after one year of service. The failure was mainly due to undermining of the foundation by leakage through or under the steel sheet pile.

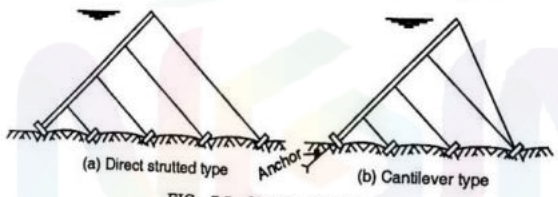

FIG. 7.5. STEEL DAMS.

Steel dams (Fig. 7.5) are generally of two types : *direct strutted type and (ii) cantilever type*. In the direct strutted type, the load on the deck plate is carried directly to the foundations through inclined struts. In the cantilever type, the section of the bent supporting the upper part of the deck is formed into a cantilever truss. This arrangement introduces a tensile force in the deck girders which is resisted either by anchoring the deck girder into the foundation at the upstream toe or by framing the entire bent rigidly together so that the moment of the weight of the water on the lower part of the deck may be utilised to offset the moment of the cantilever.

Advantages of Steel Dams

1. With the modern methods of steel fabrication, greater speed can be achieved in construction.

2. They have been found to be cheaper than other rigid dams.

3. Stresses in steel dams are more determinate. Hence, section can be designed economically and with confidence.

4. They have greater flexibility to resist unequal settlement without excessive leakage.

5. They are not affected by frost action.

6. Leaky joints, if any, can be repaired more easily by modern welding processes, than in hollow concrete dams.

Disadvantages

1. Steel dams are lighter, hence are not as adaptable to absorb the shock from vibrations of spilling water.

2. The life of steel dam is known to be shorter than a concrete dam.

3. Steel dam requires greater and more constant maintenance than concrete.

4. They have to be anchored at foundation which is difficult and precarious.

5. There is considerable concentration of bearing stresses.

7.9. TIMBER DAMS

A timber dam is constructed of framework of timber struts and beams, with timber plank facing to resist water pressure. A timber dam is an ideal temporary dam , though a well designed, constructed and maintained timber dam may last 30-40 years. They are suitable to places where timber can be available in plenty.

Timber dams are normally found to be of three types :

1. A-frame type (Fig. 7.6).
2. Rock-filled crib type (Fig. 7.7).
3. Beaver type. (Fig. 7.8.)

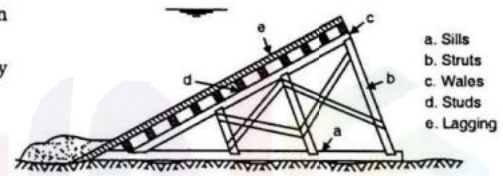

FIG. 7.6 A-FRAME TYPE TIMBER DAM.

1. A-frame Type Timber Dam

Fig. 7.6 Shows typcial A-frame type timber dam. It consists of five component parts : (*a*) sills, (*b*) struts, (*c*) wales, (*d*) studs and (*e*) lagging. The sills should be fastened to the ledge rock by wedge bolts or anchor bolts. The lagging should not be less than 5 cm thickness.

2. Rock Filled Crib Type Timber Dam

Fig. 7.7 shows such a type of dam in which cribs of square or round timber are drift bolted together. The timbers are spaced about

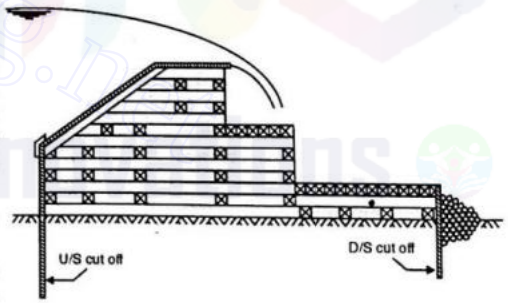

FIG. 7.7 ROCK-FILLED TIMBER CRIB DAM.

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2–2.5 m centre to center both ways. The space between them is filled with rock fragments or boulders. In the case of rock foundation the bottom cribs are pinned to the rock foundation. If, however, the dam is constructed on earth foundation, sheet pilling is provided both at the u/s as well as d/s side as shown in Fig. 7.7.

3. Beaver Type Timber Dam

Fig. 7.8 shows Beaver type timber dam which is the lowest in cost if plenty of timber is available. This is used only for low height and u/s slope of the dam is not kept steeper than 1 on 2. It consists of a number of timbers the butts of which point downstream, and between the butts are placed spacer logs which are drift pinned. Fill of earth or sand is placed over the bottom of deck for its stability.

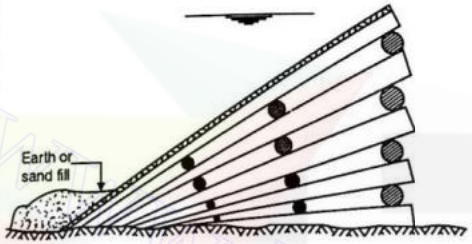

FIG. 7.8. BEAVER TYPE TIMBER DAM.

Advantages

1. Low initial cost.
2. Suitable for any type of foundation.
3. Where only temporary dams are to be constructed timber dams are more suitable.
4. Greater speed in construction can be achieved

Disadvantages

1. High maintenance cost.
2. Life is short.
3. Suitable only for small heights.
4. Greater seepage loss through the body of the dam.

7.10. EARTH DAMS AND ROCKFILL DAMS

Earth dams are made of locally available soils and gravels and therefore, are most common types of dams used upto moderate heights. Their construction invloves utilization of materials in the natural state requiring a minimum of processing. With the advancing knowledge of soil mechanics and with the advent of more sophisticated earth moving equipment, earth dams are now becoming more common, even for higher heights. The foundation requirements of earth dams are less stringent than for other types. Fig 7.9(*a*) shows a typical section of composite earth dam.

A rockfill dam is an embankment which uses variable sizes of rock to provide stability and an impervious membrane to provide water tightness. In modern practice, the rockfill dam has four fundamental parts : (1) dumped rockfill at the downstream, (2) upstream rubble cushion of laid-up stone bounding into the dumped rock, (3) upstream impervious facing resting on rubble cushion, and (4) upstream cutoff to check under seepage. Fig. 7.9(*b*) shows a typical section. Fig. 7.9(*c*) shows a combined earth and rockfill dam in which the usual derrick laid rockfill is replaced by the hydraulic earthfill and no upstream impervious membrane is provided.

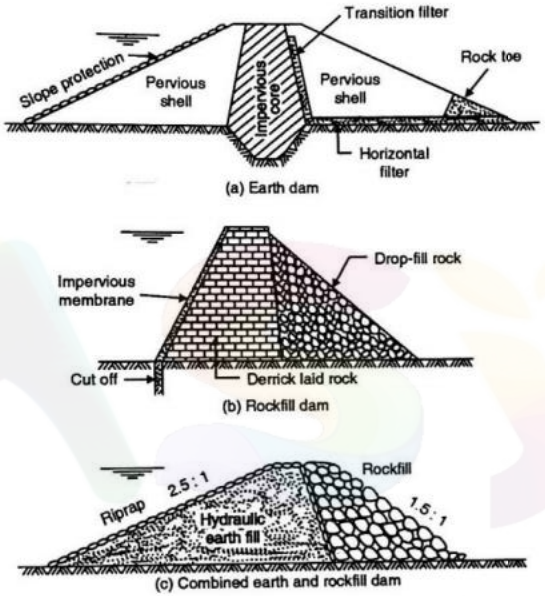

FIG. 7.9 EARTH AND ROCKFILL DAMS.

Advantages of Earth and Rockfill Dams

1. Earth dams can be constructed almost on any type of available foundations. However, rockfill dams impose some restriction on the quality of foundation but they are much less rigid than those required for any other type.
2. They can be constructed rapidly with relatively unskilled labour and with materials available on the spot. Hence, it can be constructed even under famine relief work.
3. They are generally cheaper than other types.
4. They can be subsequently raised in height without much difficulty if such a need arises in future.

Disadvantages

1. They are more vulnerable to damage by floods and fail suddenly without sufficient warning.
2. They cannot be used as overflow dams. Separate spillways are to be provided at some site away from the dam, to pass the surplus discharge.
3. They are not suitable at the locations where heavy downpour is more common.
4. They require heavy maintenance cost and constant supervision.

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7.11. PHYSICAL FACTORS GOVERNING SELECTION OF TYPE OF A DAM

The selection of a type of a dam at a given site depends upon many physical factors such as topography, geological and foundation conditions, available materials, suitable site for spillway, data about earthquake etc. It is only in exceptional circumstances that an experienced designer can say that only one type of dam is suitable or most economical. Preliminary designs and estimates are required for several types of dams before one can be shown to be the most economical. Before selecting the best type of dam at a particular site, one must consider the characteristics of each type of dam, as related to the physical features of the site and the adaption to the purposes the dam is supposed to serve, as well as economy, safety and other pertinent limitations. The choice of a dam may also be guided by many local problems such as availability of labour and equipment, accessibility of site, limitations imposed by outlet works and cost of protection needed from spillway discharge as well as time required for its constuction. Some of the physical factors governing the selection of type of dam are discussed below.

1. **Topography**. The first choice of dam is usually governed by the topography for the site. A low rolling plains country suggests an earth dam with a separate spillway. A low narrow V-shaped valley suggests an arch dam, provided the top width of valley is less than one-fourth its height and separate site for spillway is available. A narrow stream flowing between high rocky walls (giving rise to a U-shaped valley) would suggest a concrete overflow dam. For intermediate conditions, other factors such as foundation condition, location of suitable site for spillway and availability of materials of construction play an important role in the selection of the type.

2. **Geology and Foundation Conditions**. The next important factor is the geology and foundation conditions. If the foundation consists of sound rock, with no fault or fissures, any type of dam can be constructed on it. Rocks like granite, gniss and schist make very satisfactory foundation for gravity dam. However, these rocks may have seams or fractures. The removal of disintegrated rock together with the sealing of seams and fractures by grouting will frequently be neccesary. *Poor rock or gravel foundations* are suitable for earth dam, rockfill dam or low concrete gravity dam. Since there will be considerable under-seepage in this case, effective water cutoffs or seals have to be provided. *Silt or fine sand foundations* have the problems of settlement, seepage and toe-erosion. Hence, such foundations are suitable only for either earth dam or low concrete gravity dam but not rockfill dams. *Clay foundations* have often the problems of long range consolidation under the weight of the dam, resulting in cracks. Hence, only earth dams are suitable with proper foundation treatment. Gravity dams or rockfill dams are not suitable on clay foundations.

3. **Materials of Construction**. The next important factor is the availability of materials of construction for dam. The cost of construction of a particular type of dam will depend upon the availability of the materials in nearby area so that transportation charges are reduced. If sand, gravel and crushed stone is available, a concrete gravity dam may be more suitable. If, however, coarse and fine grained soils are available an earth dam may be suitable. The preliminary selection of a particular type, based on the first two physical factors, must correspond with the easy availability of the materials required for its construction otherwise that type of dam should be dropped.

4. **Spillway Size and Location**. The safe discharge of flood water through dam is very essential, and for that suitable site for spillway should be available. If the area is such that a large spillway capacity is required, an overflow concrete gravity dam should be preferred. Where small spillway capacity is required and where seperate site for spillway location is available, an earth dam may be preferred. Where large discharges are to be derived during the construction of dam, a concrete gravity dam is preferred to an earth dam. If no other site is available for spillway, it has to be accommodated with the main dam across the main river section. In that case, a concrete gravity dam having overflow section in its middle would be a better choice, though an earth dam can also be constructed with central overflow section of concrete.

5. **Roadway.** If a roadway is to be passed over the top of the dam, an earth dam or gravity dam would be preferred.

6. **Length and Height of Dam**. If the length of the dam is very long and its height is low, an earth dam would be a better choice. If the length is small but height is more, gravity dam is preferred.

7. **Life of Dam**. Concrete or masonry gravity dams have very long life. Earth and rockfill dams have intermediate life. However, timber dams are adopted only for temporary storages.

7.12. SELECTION OF SITE FOR A DAM

The following are the requisites of good sites for various types of dams :

1. ***Foundations.*** Suitable foundations should be available at the site selected for a particular type of dam. For gravity dams, sound rock is essential. For earth dams, any type of foundation is suitable with proper treatment. In general, however, the foundation should be free from seams, open pockets or fault planes. Formation in which hard and soft layers alternate are not generally good, because the penetration of water may weaken the soft layers and lead to movement along them. Alternations of sand, stones and shales may also lead to slipping during excavation of the trench. Beds which dip gently upstream and strike across the valley provide probably the best foundation conditions among sedimentary rocks, as the resultant of the weight of the dam and water pressure will then act nearly at right angles to the bedding planes of the strata. No dam should be built across a fault known to have been active in recent times. The best conditions are when a dam can be built on one uniform formation; if more than one kind of rock is present in the foundation, different bearing strengths may lead to differential settlement of the structure.

2. ***Topography.*** (*i*) The river cross-section at the dam site should preferably have a narrow gorge to reduce the length of the dam. However, the gorge should open out upstream to provide large basin for a reservoir.

(*ii*) A major portion of the dam should preferably be on high ground, as this would reduce the cost and facilitate drainage.

3. ***Site for Spillway.*** Good site for the location of a separate spillway is essential especially in the case of earth or rockfill dam. However, in the case of gravity dam, spillway may be located at its middle. The best site for a dam may be considered to be one where a deep gorge and a flank at its sides are separated by a hillock

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higher than the height of the dam. In that case the main dam may be located in the deep gorge while the spillway can be located in the flank.

4. ***Materials.*** Materials required for a particular type of dam should be available nearby, without requiring much of transportation. This would very much reduce the cost of construction.

5. ***Reservoir and Catchment Area.*** (*i*) The side should ensure adequate storage capacity of reservoir basin at a minimum cost.

(*ii*) The cost of land and property submerged in the water spread area should be minimum.

(*iii*) The reservoir site should be such that quantity of leakage through its side and bed is minimum. Reservoir site having the presence of higher permeable rocks reduce the water-tightness of the reservoir.

(*iv*) The geological conditions of the catchment area should be such that percolation losses are minimum and maximum run-off is obtained.

(*v*) The reservoir site should be such that it avoids or excludes water from those tributaries which carry a high percentage of silt in water.

(*vi*) The site should be such that a deep reservoir is formed. A deep reservoir is preferable to shallow one because of (*a*) lower cost of land submerged per unit of capacity, (*b*) less evaporation losses because of reduction in the water spread area and (*c*) less likelihood of weed growth.

6. ***Communication.*** It would be preferable to select a site which is connected by a road or rail link or can be conveniently connected to the site for transportation of cement, labour, machinery food and other equipmemt.

7. ***Locality.*** The surroudings near the site should perferably be healthy and free of mosquitoes etc., as labour and staff colonies have to be constructed near the site.

PROBLEMS

1. Classify various types of dams. Distinguish clearly between rigid and non-rigid dams.
2. Discusss in brief merits and demerits of various types of dam.
3. Discuss, with illustrations the phyical factors that govern the selection of type of dam.
4. What are the factors on which selection of site for a dam depends?
5. Write a note on foundation problems for dams and their remedies.

8

Dams-II : Gravity Dams

8.1. INTRODUCTION

A gravity dam is a structure so proportioned that its own weight resists the forces exerted upon it. This type of dam is the most permanent one, requires little maintenance and is most commonly used. The most ancient gravity dam on record was built in Egypt more than 400 years B.C. of uncemented masonry. Archeological experts believe that this dam was kept in perfect condition for many centuries.

A gravity dam may be constructed either of mosonry or of conrete. However, now-a-days with improved methods of construction, quality control and curing, concrete is most commonly used for the construction of gravity dams. A gravity dam is mostly straight in plan and is known as '*straight gravity dam*'. However, it may also be slightly curved in plan. A curved gravity dam resists the external forces by its weight and not by arch action. Most of the gravity dams are *solid*, so that no bending stress is introduced at any point and hence, they are sometimes known as '*solid gravity dams*'. A gravity dam, however, can be hollow and is known 'hollow gravity dam'.

Gravity dams are particulary suited across gorges with very steep side slopes where earth dams might slip. Where good foundations are available, gravity dams can be built upto any height. The highest dams in the World are of gravity type.

8.2. FORCES ACTING ON A GRAVITY DAM

Following are the forces acting on a gravity dam. Some of these forces have been shown in Fig. 8.1.

1. Water pressure.
2. Weight of dam.
3. Uplift pressure.
4. Pressure due to earthquake.
5. Ice pressure.
6. Wave pressure.
7. Silt pressure.
8. Wind pressure

8.3. WATER PRESSURE

This is the major external force acting on dam. When the upstream face of the dam is vertical, the water pressure acts horizontally. The intensity of pressure varies triangulary, with a zero intensity at the water surface, to a value wh at any depth h below water surface. When the upstream face is partly vertical and partly inclined, as shown in Fig. 8.1, the resultant water pressure can be resolved into two components:

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(*i*) Horizontal component P_{wH} and (*ii*) Vertical component P_{wv} due to weight of water supported by the inclined face.

Thus, horizontal force,

$$P_{wH} = \frac{w\,H^2}{2} \qquad ...(8.1)$$

This acts at a height $H/3$ from the base of the dam.

Vertical component, P_{wv} = Weight of water contained by column $AA'C'B$ and acting at the C.G. of the area.

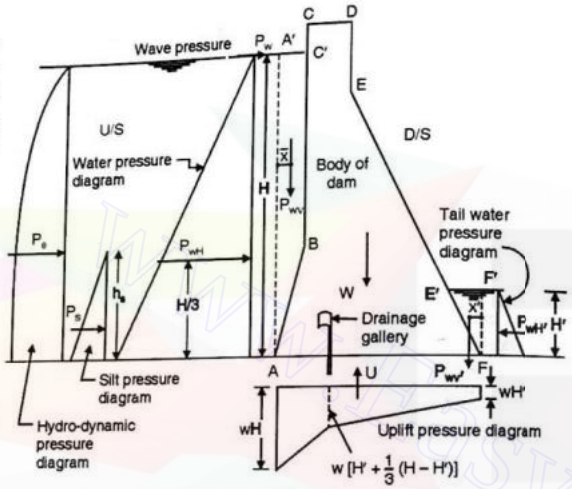

FIG. 8.1 FORCES ACTING ON A GRAVITY DAM.

Similarly, if there is tail water of height H' on the downsteam side, it exerts both horizontal pressure ($P_{wH'}$) as well as vertical pressure ($P_{wv'}$).

Thus, horizontal pressure $P_{wH'} = \dfrac{w\,H'^2}{2}$

and vertical pressure $P_{wv'}$ = Weight of water contained by column $FF'E'$

8.4. WEIGHT OF THE DAM

The weight of the dam is major resisting force. For analysis purposes, generally, unit length of the dam is considered. The cross-section of the dam may be divided into several triangles and rectangles, and the weights W_1, W_2, W_3, etc. of each of these may be computed conveniently, along with determination of their lines of action. The total weight W of the dam acts at the C.G. of its section.

8.5. UPLIFT PRESSURE

Water has a tendency to seep through the pores and fissures of the foundation meterial. It also seeps through the joints between the body of the dam and its foundation at the base, and through the pores of the material in the body of the dam. The seeping water exerts pressure and must be accounted for in the stability calculations.

The *uplift pressure* is defined as the upward pressure of water as it flows or seeps through the body of the dam or its foundation. A portion of the weight of the dam will be supported on the upward pressure of water ; hence net foundation reaction due to vertical force will reduce.

Area over which uplift acts. The area over which the uplift pressure acts has been a question of investigation from the early part of this century. There are two schools of thought existing in the old literature for this. One school of thought recommends that a value one-third to two-thirds of the area should be considered as effective over which the uplift acts.

Terzaghi and Leliavaski, the propagators of the second school of thought, recommend that the effective area may be taken approximately equal to the total area. The present practice in the design is to follow U.S.B.R. recommendation that the total area should be considered as effective to account for uplift.

U.S.B.R. Recommendations. U.S.B.R. suggests the adoption of uplift pressure intensities equal to the hydrostatic pressure of water at the toe and heel joined by a straight line in between. Sometimes drainage galleries are also provided in the body of the dam which releases the uplift pressure built up under it. The magnitude of the uplift pressure recommended at the face of the gallery is equal to the hydrostatic pressure at the toe plus 1/3rd the difference of the hydrostatic pressure at the heel and toe. From Fig. 8.1

The uplift pressure at heel $A = wH$

Uplift pressure at toe $F \quad = wH'$

Uplift pressure at gallery $\quad = w\,[H' + \frac{1}{3}(H - H')]$. ...(8.2)

Indian Standard Recommendations (IS : 7894-1975) : According to the Indian Standard, there are two constituent elements in uplift pressure : the *area factor* or the percentage of area on which uplift acts and the *intensity factor* or the ratio which the actual intensity of uplift pressure bears to the intensity gradient extending from head water to tail water at various points.

Effective downstream drainage, whether natural or artificial, will generally limit the uplift at the toe of the dam to tail water pressure. Formed drains in the body of the dam and drainage holes drilled subsequent to grouting in the foundation, where maintained in good repair, are effective in giving a partial relief to the uplift pressure intensities under and in the body of the dam. The degree of effectiveness of the system will depend upon the character of the foundation and the dependability of the effective maintenance of the drainage system. In any case, observation of the behaviour of the dam will indicate the uplift pressures actually acting on the structure and when the uplift pressures are seen to approach or exceed design pressures, prompt remedial measures should necessarily be taken to reduce the uplift pressures to values below the design pressures.

Criteria for design. This following criteria are recommended for the calculating uplift forces :

(*a*) Uplift pressure distribution in the body of the dam shall be assumed, in case of both preliminary and final designs, to have an intensity which at the line at the formed drains exceeds the tailwater pressure by one-third the differential between reservoir level and tailwater level. The pressure gradient shall then be extending linearly to heads corresponding to reservoir level and tailwater level. The uplift shall be assumed to act over 100 percent of the area.

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(*b*) Uplift pressure distribution at the contact plane between the dam and its foundations and within the foundation shall be assumed for preliminary designs to have an intensity which at the line of drains exceeds the tailwater pressure by one-third the differential between the reservoir and tailwater heads. The pressure gradient shall then be extended linearly to heads corresponding to reservoir level and tailwater level. The uplift shall be assumed to act over 100 % area. For final designs, the uplift criteria in case of dams founded on compact and unfissured rock shall be as specified above. In case of highly jointed and broken foundation, however, the pressure distribution may be required to be based on electrical analogy or other methods of analysis taking into consideration the foundation condition after the treatment proposed. The uplift shall be assumed to act over 100 % of the area.

(*c*) For the extreme loading conditions *F* and *G* (see § 8.11), the uplift shall be taken as varying linearly from the appropriate reservoir water pressure at the u/s face to the appropriate tailwater pressure at the d/s face. If the reservoir pressure at the section under consideration exceeds the vertical normal stress (computed without uplift) at the u/s face, a horizontal crack is assumed to exist and to extend from the u/s face towards the d/s face of the dam to the point where the vertical normal stress (computed on the basis of linear pressure distribution without uplift) is equal to the reservoir pressure at the elevation. The uplift is assumed to be the reservoir pressure from the u/s face to the end of the crack and from there to vary linearly to the tailwater pressure at the d/s face. The uplift is assumed to act over 100 % of the area.

(*d*) No reduction in uplift is assumed at the d/s toe of spillways on account of the reduced water surface elevation (relative to normal tailwater elevation) that may be expected immediately downstream of the structure.

(*e*) It is assumed that uplift pressure are not affected by earthquakes.

8.6. PRESSURE DUE TO EARTHQUAKE

The earthquake sets up primary, secondary, Raleigh and Love waves in the earth's crust. The waves impart accelerations to the foundations under the dam and causes its movement. In order to avoid rupture, the dam must also move along with it. This acceleration introduces an inertia force in the body of dam and sets up stresses initially in lower layers and gradually in the whole body of the dam. Earthquake wave may travel in any direction. However, it is customary to resolve the general direction of acceleration into horizontal and vertical directions.

Intensity of Earthquake

The intensity of an earthquake at a place is a measure of the strength of shaking during the earthquake, and is indicated by a *number* according to the Modified Mercalli Scale or M.S.K. Scale of seismic intensities. The comprehensive intensity scale (MSK 64), recommended by IS 1893 (Part 1) 2002, has rating numbers from 1 to 12. The detailed description about MSK 64, along with the damages caused corresponding to each number on the scale is available in IS 1893 (Part 1) : 2002.

Earthquake Zones of India : There are World maps which are divided into different zones according to the severity of earthquake. According to IS 1893-1984, India

was divided in five zones : zone I, zone II, zone III, zone IV and zone V. However, according to its revised version (IS 1893-2002), the seismic zone map is revised with only *four zones*, instead of five. Erstwhile zone I has been merged to zone II. Hence zone I does not appear in the new zoning map; only zone II, III, IV and V appear. The Killari area has been included in zone III and necessary modifications made, keeping in view the probabilistic hazard evaluation. The Bellary isolated zone has been removed. The part of eastern coast areas have shown similar hazard to that of Killari area; the level of zone II has been enhanced to zone III and connected with zone III of Godavari Graben area.

Spectra of Earthquake : Acceleration Spectra : Spectrum of an earthquake is the representation of the maximum dynamic response of idealised structure during an earth quake. The *idealized structure* is a 'single degree of freedom system' having a certain period of vibration and damping. The maximum response is plotted against the natural period of vibration (*T*) and can be expressed in terms of the following :

(*i*) Maximum absolute acceleration

(*ii*) Maximum relative velocity

or (*iii*) Maximum relative displacement.

For the purpose of design, *acceleration spectra* are very useful as they give the *seismic force* on the structure directly by multiplying it with the generalized or modal *mass* of the structure, i.e. Force = mass × earthquake acceleration. The *earthquake acceleration* is usually designated as fraction of the acceleration due to gravity and it is expressed as αg, where α is known as the *seismic coefficient*.

Design seismic coefficients for different zones (IS : 1893-1984)

According to IS : 1893-1984, the design value of *horizontal seismic coefficient* (α_h) may be determined by one of the two methods.

(*a*) Seismic coefficient method (*b*) Response spectrum method

As per IS : 1893-1984, the seismic coefficient method has been suggested for dams upto 100 m in height and the response spectrum method has to be used for dams above this height. The idea of choosing a value of 100 m is that most of the dams upto this height will have periods (*T*) shorter than 0.3 seconds and the spectral acceleration may not be influenced by change of period in this range. Further, dams beyond 100 m in height may be thought to be more important requiring more sophisticated analysis.

(a) Seismic coefficient method

As per IS : 1893-1984, the design value of horizontal seismic coefficient (α_h) shall be computed as given by the following expression :

$$\alpha_h = \beta I \alpha_0 \qquad \text{....(3.8 } a\text{)}$$

where β = *Soil-foundation system factor*, the value of which may be taken as 1.0 for dams

I = *Importance factor*, the value of which is taken as 2.0 for dams

α_0 = *Basic seismic coefficient*, the value of which for each of the four zones is given in Table 8.1 below.

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TABLE 8.1. BASIC SEISMIC COEFFICIENT α_0 (IS : 1893-1984)

Seismic zone	II	III	IV	V
Basic seismic coefficient α_0	0.02	0.04	0.05	0.08

Thus, substituting the values of β and *I for dams*, Eq. 8.3 (*a*) reduces as under:

$$\alpha_h = 2\,\alpha_0 \qquad ...(8.3)$$

As α_0 varies from 0.02 to 0.08, the value of α_h will vary from 0.04 to 0.16 for zones II to zone V.

(*b*) **Response spectrum method** : As per IS : 1893-1984, the design value of horizontal seismic coefficient is computed from the following expression :

$$\alpha_h = \beta\, I\, F_0 \frac{S_a}{g} \qquad ...(8.4a)$$

where β = *Soil foundation system factor*, the value of which may be taken as 1.0 for dams

I = *Importance factor*, the value of which is taken as 2.0 for dams

F_0 = *Seismic zone factor* for average acceleration spectra, the value of which for each of four zones is given in Table 8.2 below.

$\frac{S_a}{g}$ = average acceleration spectra as read from Fig 8.2 for appropriate natural period (T) and damping of the structure.

TABLE 8.2. SEISMIC ZONE FACTOR (IS : 1893-1984)

Seismic zone	II	III	IV	V
Seismic zone factor F_0	0.10	0.20	0.25	0.40

Substituting the value of β = 1.0 and I = 2.0, for dams, Eq. 8.4 (*a*) reduces as under :

$$\alpha_h = 2\,F_0 \frac{S_a}{g} \qquad ...(8.4)$$

For determining the value of S_a/g from Fig. 8.2, IS 1893-1984 recommends a damping of 5 percent for dams, and the natural period of vibration (T) is determined from the following expression :

$$T = 5.55\,\frac{H^2}{B}\sqrt{\frac{w_m}{g\,E_s}} \qquad ..(8.5)$$

where

H = height of the dam (m)

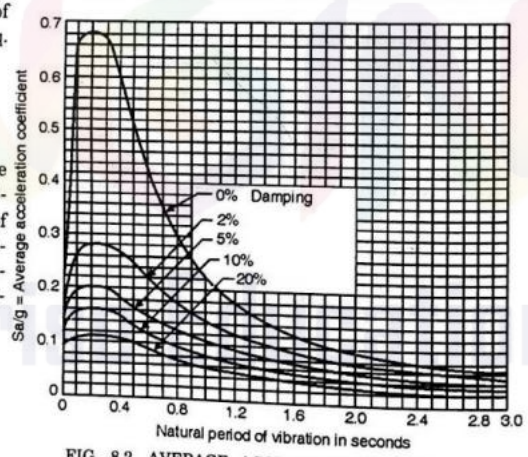

FIG. 8.2. AVERAGE ACCELERATION SPECTRA

B = base width of the dam (m)

w_m = unit weight of the material of dam (kN/m^3)

g = acceleration due to gravity (m/s^2)

and E_s = modulus of elasticity of the dam material (kN/m^2)

Taking $w_m = 24$ kN/m^3, $E_s = 2.1 \times 10^7$ kN/m^2, $g = 9.81$ m/sec^2 and $B = 0.7\,H$, Eq. 8.5 reduces to

$$T = 5.55\left(\frac{H^2}{0.7\,H}\right)\sqrt{\frac{24}{9.81 \times 2.1 \times 10^7}} = 2.71 \times 10^{-3}\,H \qquad ...(8.5\ a)$$

Thus, when H = 100 m, T = 0.271 seconds for which S_a/g for 5% damping comes out to be 0.19 from Fig. 8.2.

Vertical seismic coefficient (α_v) : The vertical seismic coefficient may be taken as *half* of the horizontal seismic coefficient. Hence

$$\alpha_v = 0.5\,\alpha_h \qquad ...(8.6)$$

Recommendations of IS : 1893-2002 : Design spectrum

According to IS 1893-2002, (Part 1) the *design horizontal seismic coefficient* (re-designated as A_h) shall be determined from the following expression :

$$A_h = \frac{Z}{2}\cdot\frac{I}{R}\cdot\frac{S_a}{g} \qquad ...(8.7)$$

where Z = *Zone factor*, given in Table 8.3. These zone factors are for the *maximum considered Earthquake* (MCE) and service life of the structure in a zone. The factor 2 in the denominator of Z is used so as to reduce the maximum considered earthquake (MCE) zone factor to the factor for design basis earthquake (DBE).

I = *Importance factor*,

R = *Response reduction factor*, depending on the perceived seismic damage performance of the structure, characterised by ductile or brittle deformations.

$\frac{S_a}{g}$ = average response acceleration coefficient for rock or soil sites, the values of which are given in Fig. 8.3 for 5% damping. For rocky or hard soil sites, the value of S_a/g may be found from the following expression :

$$\frac{S_a}{g} = \begin{cases} 1 + 15\,T & (\text{for } 0.00 \le T \le 0.10 \\ 2.50 & (\text{for } 0.10 \le T \le 0.40 \\ 1.00/T & (\text{for } 0.40 \le T \le 4.00 \end{cases} \qquad ...(8.8)$$

Thus, for gravity damn having T between 0.1 and 0.4 seconds, the value of S_a/g = 2.5

TABLE 8.3. ZONE FACTOR Z (IS : 1893 : 2002)

Seismic zone	II	III	IV	V
Seismic intensity	Low	Moderate	Severe	Very severe
Zone factor Z	0.10	0.16	0.24	0.36

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Design acceleration spectrum for vertical motion : The design vertical acceleration may be taken as two-thirds of the design horizontal acceleration spectrum

Note Though the Code, IS 1893 (Part 1) 2002 has given the values of I and R for buildings, it has yet not given the values of I and R for other structures, such as liquid retaining structures, bridges and retaining walls, stack like structures and dams and embankments. *Till parts 2, 3, 4 and 5 of the above Code are finalised, we will use the recommendations of IS 1893-1984 for dams.* However, we shall consider zone I to be merged with zone II, as recommended in IS 1893-2002.

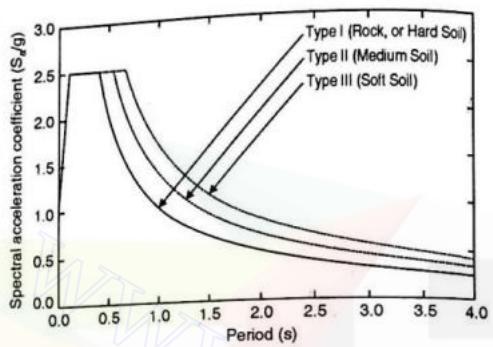

FIG. 8.3. RESPONSE SPECTRA FOR 5% DAMPING (IS : 1893-2002)

(a) EFFECT OF HORIZONTAL EARTHQUAKE ACCELERATION

Horizontal acceleration due to earthquake causes two effects :

1. Inertia force in the body of the dam

and 2. Hydrodynamic pressure of water

1. Horizontal Inertia force : The inertia force is the product of mass and acceleration and this force acts in the direction opposite to that of the ground motion. The horizontal inertia force acts from upstream to downstream as well as from downstream to upstream. For *reservoir full condition*, the worst case occurs when the earthquake acceleration acts towards the upstream direction and the corresponding inertia force acts in the *downstream direction*. However, for the *reservoir empty condition*, the worst case occurs when the acceleration due to earthquake acts towards the downstream direction and the corresponding inertia force acts in the upstream direction :

The inertia force dF on any elemental lamina of the dam is equal to the product of the mass of the lamina and the acceleration.

Let dW = weight of the elemental lamina at any height of the dam

g = acceleration due to gravity

α_h = horizontal acceleration coefficient $= \dfrac{\text{Earth quake acceleration}}{\text{Acceleration due to gravity}}$

Horizontal earthquake acceleration $= \alpha_h g$

$\therefore$ Inertia force = mass × acceleration

$$= \frac{dW}{g} \cdot \alpha_h g = dW \cdot \alpha_h \qquad ...(8.9)$$

The computation of horizontal inertia force on the dam can be done by *seismic coefficient method* for dams upto 100 m height and by *response spectrum method* for dams of height greater than 100 m.

(i) Seismic Coefficient Method

In the seismic coefficient method, the horizontal seismic coefficient is assumed to vary triangularly with a maximum value of $1.5\,\alpha_h$ at the top of the dam to a value of zero at the base the dam. Thus, the horizontal acceleration is assumed to have the shape of an inverted triangle. If one takes the shape of the dam as triagular (i.e. elementary profile), with zero width at the top and a width B at the bottom, with a height H, and the distribution of horizontal acceleration as an inverted triangle (Fig. 8.4), the value of base shear (F_H) will work out to be equal to $0.5\,W\,\alpha_h$, as shown under.

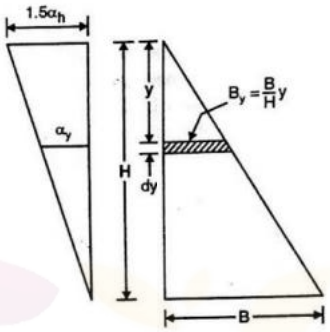

FIG. 8.4 VARIATION OF HORIZONTAL ACCELERATION ALONG THE DEPTH OF DAM

Consider an elementary strip of thickness dy at a depth y from the top. The width of the strip is

$$B_y = \frac{B}{H} \cdot y$$

The horizontal acceleration coefficient at this level is

$$\alpha_y = 1.5\,\alpha_h \left(1 - \frac{y}{H}\right)$$

If w_m is the unit weight of the material,

$$dW = B_y \cdot dy \cdot w_m$$

$\therefore$ Elementary horizontal inertia force on the strip $= dF_y = dW \cdot \alpha_y$, from Eq. 8.9

$\therefore$ Base shear $$F_H = \int_0^H dF_y = \int_0^H B_y \cdot dy \cdot w_m \cdot \alpha_y$$

$$= \int_0^H \left(\frac{B}{H} y\right)\left(1 - \frac{y}{H}\right) 1.5\,\alpha_h \cdot w_m\, dy$$

$$= 1.5\,\alpha_h \frac{B}{H} w_m \left(\frac{y^2}{2} - \frac{y^3}{3H}\right)_0^H$$

or $$F_H = 0.25\,\alpha_h BH\, w_m = 0.5\,\alpha_h W \qquad ...(8.10)$$

where $W = \frac{1}{2} BH\, w_m$ = total weight of the dam

However, in general, the dam has some top width from practical considerations.

Hence $$F_H \triangleq 0.6\,\alpha_h W \qquad ...(8.10\ a)$$

This compares well with the base shear coefficient given in the response spectrum method by the I.S. Code :

$$F_B = 0.6\,W\,\alpha_h \quad \text{(See Eq. 8.11)}$$

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Computation of total horizontal force on the dam and its moment

Let us now take the case of a general profile of a gravity dam with top with b_1 upto depth H_1 with b_2 at depth H_2 and width $b_3 = B$ at total depth H below the top, as shown in Fig. 8.5. The distribution of seismic coefficient is in the form of inverted triangle, with a maximum value of $1.5\,\alpha_h$ at the top and zero value at the base of the dam.

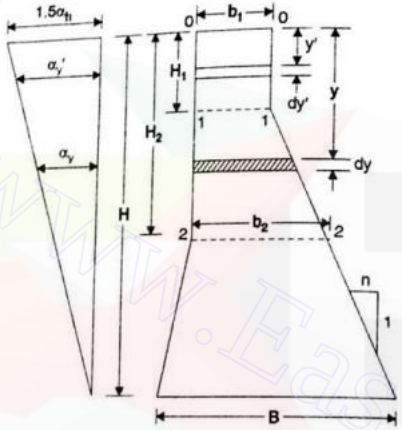

FIG. 8.5. GENERAL PROFILE OF DAM

For portion of dam between sections 0-0 to 1-1

At any depth y' below the top, consider a strip of thickness dy'. The width of the strip $b'_y = b_1$ and the acceleration coefficient $\alpha'_y = 1.5\,\alpha_h\,(1 - y'/H)$.

Hence elementary horizontal force $dF_1 = (1 \times b_1\,dy')\,w_m\,.\,\alpha_y'$

Hence horizontal inertial force for the portion of the dam between 0-0 and 1-1 is

$$F_1 = \int_0^{H_1} (b_1\,dy'\,.\,w_m)\,1.5\,\alpha_h\,(1 - y'/H)$$

or

$$F_1 = 1.5\,\alpha_h\,w_m\,b_1\,H_1\left(1 - \frac{H_1}{2H}\right) \qquad ...(1)$$

The moment of the inertia force F_1 about section 1-1 is

$$M_1 = \int_0^{H_1} (b_1\,dy'\,w_m)\,1.5\,\alpha_h\,(1 - y'/H)\,(H_1 - y')$$

or

$$M_1 = 1.5\,\alpha_h\,w_m\,b_1\,\frac{H_1^2}{2}\left(1 - \frac{H_1}{3H}\right) \qquad ...(2)$$

For portion of dam between sections 1-1 and 2-2

Consider an elementary strip of thickness dy at any depth y below the top of the dam. The width $b_y = b_1 + n\,(y - H_1)$, where n is the slope of the d/s face (i.e. n horizontal to 1 vertical)

Hence total horizontal inertia force on the portion of the dam above section 2-2 is given by

$$F_2 = F_1 + \int_{H_1}^{H_2} \left\{b_1 + n\,(y - H_1)\right\} dy\,w_m\,(1 - y/H)\,1.5\,\alpha_h$$

or

$$F_2 = F_1 + 1.5\,\alpha_h\,w_m\,(H_2 - H_1)\left[b_1\left(\frac{2H - H_2 - H_1}{2H}\right) + n\left(\frac{H_2 - H_1}{2} - \frac{2H_2^2 - H_1 H_2 - H_1^2}{6H}\right)\right] \qquad ...(3)$$

The moment of the inertia force F_2 about section 2-2 is given by

$$M_2 = M_1 + F_1\,(H_2 - H_1) + \int_{H_1}^{H_2} (b_y\,.\,dy\,.\,w_m)\,(1 - y/H)\,1.5\,\alpha_h\,(H_2 - y) \qquad ...(4)$$

Thus, adopting the procedure outlined above, the total inertia force F and its moment about the base, for the whole dam can be computed.

(*ii*) Response spectrum method

As per IS 1893-1984, the response spectrum method is adopted for dams of height more than 100 m. According to the response spectrum method, the base shear F_B and the base moment are obtained by the following formulae :

$$F_B = 0.6\,W\,\alpha_h \qquad ...(8.11)$$

and

$$M_B = 0.9\,W\,\bar{h}\,\alpha_h \qquad ...(8.12)$$

where W = total weight of the masonry or concrete in the dam

$\bar{h}$ = height of the centre of gravity of the dam above the base, and

α_h = design horizontal seismic coefficient.

For any horizontal section at a depth y below the top of the dam, shear force F_y and bending moment M_y may be obtained as follows

$$F_y = C_v'\,.\,F_B \qquad ...(8.13)$$

$$M_y = C_m'\,.\,M_B \qquad ...(8.14)$$

where coefficients C_v' and C_m' are obtained from Fig. 8.6.

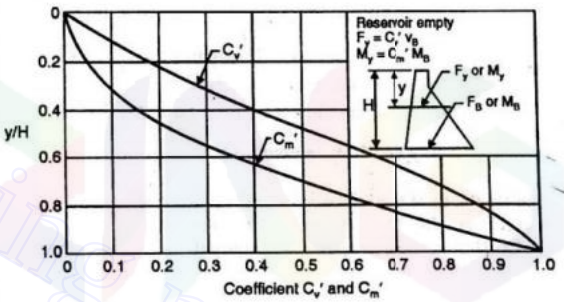

FIG. 8.6. VALUES OF C_v' and C_m'

2. Hydrodynamic pressure of water

The horizontal acceleration of the dam and foundation towards the reservoir causes a momentary increase in water pressure since the water resists the movement owing to its inertia. This *additional water pressure*, over and above the static water pressure, is known as *hydrodynamic pressure*. The following *simplified* methods are used to determine the hydrodynamic pressure which acts in the downstream direction.

(*i*) ***Von Karman's method*** : According to Von Karman, the hydrodynamic water pressure is assumed to be parabolic, and its magnitude (P_e) is given by

$$P_e = 0.555\,\alpha_h\,wh^2 \qquad ...(8.15)$$

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and it acts at a height of $4h/3\pi$ above the base of the dam, where h is the depth of water in the reservoir.

(iii) Zanger's method : In 1952, Zanger presented formulae for computing the hydrodyamic pressure exerted on vertical and sloping face of the dam. The formulae were derived by electrical analogy, based on the assumption that water is incompressible. IS 1893-1984 also recommends the same formulae. The pressure variation is elliptical-cum-parabolic. The hydrodynamic pressure intensity at a depth y below the maximum water level is given by

$$p_{ey} = C_y \, \alpha_h \, wh \qquad ...(8.16)$$

where C_y = a dimensionless pressure cofficient, at a depth y below water level. It is given by

$$C_y = \frac{C_m}{2}\left[\frac{y}{h}\left(2-\frac{y}{h}\right)+\sqrt{\frac{y}{h}\left(2-\frac{y}{h}\right)}\right] \qquad ...(8.17)$$

C_m = maximum value of the pressure coefficient for a given constant slope, and its value may be obtained from Fig. 8.7 for different values of θ, where θ is the angle in degrees which the u/s face makes with the vertical. An approximate value of C_m may be obtained from the following relation :

$$C_m = 0.735\left(1-\frac{\theta}{90°}\right) \qquad ...(8.18)$$

α_h = design horizontal seismic coefficient to be determined as follows :

In the seismic coefficient method, equivalent α_h at any depth y is given by $\alpha_y = \frac{F_y}{W_y}$ where F_y is the inertia force and W_y is the weight of the part of the dam above level y. In the response spectrum method, α_h is obtained from Eq. 8.4.

θ = angle, in degrees, the u/s face of the dam makes with vertical

w = unit weight of water

h = depth of water in reservoir

y = the vertical distance from the reservoir surface to the elevation in question.

Fig. 8.8 shows the variation of earthquake pressure coefficient C_y with y/h ratio and the angle of inclination.

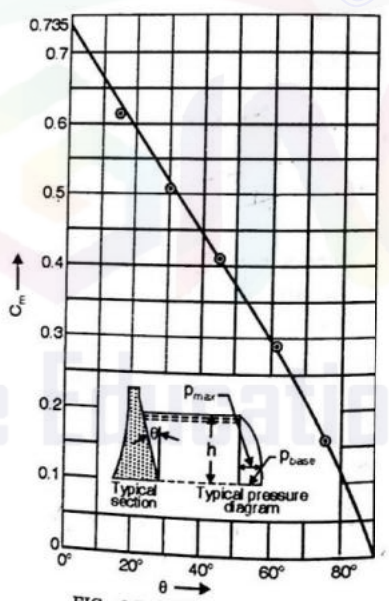

FIG. 8.7. VALUE OF C_m

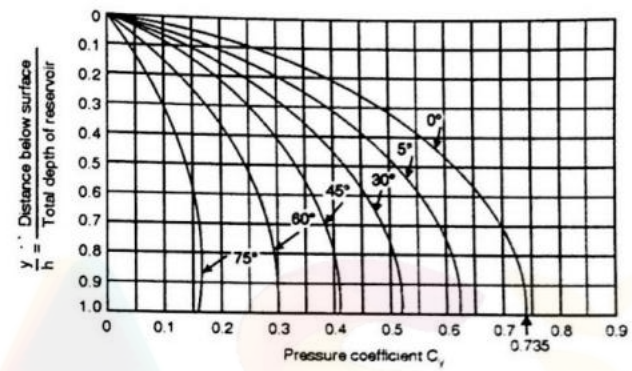

FIG. 8.8. EARTHQUAKE PRESSURE COEFFICIENT.

The pressure variation along the depth is shown in Fig. 8.9.

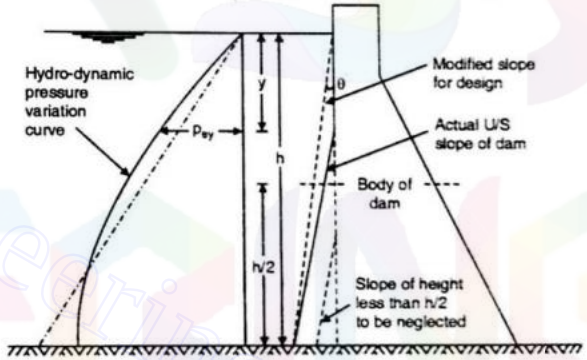

FIG 8.9 PRESSURE VARIATION DUE TO HYDRODYNAMIC LOAD.

If the u/s face is inclined, and does not extend to more than half the depth of reservoir, it may be considered as vertical for computation of C_m. However, if the slope extends to more than half the depth, overall slope θ upto the whole height may be considered for finding the value of C_m as illustrated in Fig. 8.9.

The *total horizontal pressure* or hydrodynamic horizontal shear at depth y may be found by integrating the pressure curve above that plane. Taking the pressure variation to be elliptical-cum-parabolic, the total pressure at depth y will be equal to the average of the areas of the quarter ellipse and semiparabola.

Hence $$P_{ey} = \frac{1}{2}\left(\frac{\pi}{4}\, p_{ey}\, y + \frac{2}{3}\, p_{ey}\, y\right) = \frac{1}{2}\, p_{ey}\, y\,(0.7854 + 0.6667) = 0.726\; p_{ey} \cdot y \qquad ...(8.19)$$

Similarly, the moment of pressure about the joint upto which the pressure is taken is given by half the sum of the moments of the quarter ellipse and semi-parabola.

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Hence $M_{cy} = \frac{1}{2}\left(\frac{4}{3\pi} y \times \frac{\pi}{4} p_{ey} \cdot y + \frac{2}{5} y \times \frac{2}{3} p_{cy} \cdot y\right) = \frac{1}{2} p_{ey} \cdot y^2 \left(\frac{1}{3} + \frac{4}{15}\right) = 0.299\ p_{ey} \cdot y^2$ ...(8.20)

The corresponding values at the base of the dam are obtained by subtituting the value of h for y in the above equation. Thus :

$$p_{eh} = C_m \cdot \alpha_h \cdot wh \quad ...(8.16\ a)$$

$$P_{eh} = 0.726\, p_{eh} \cdot h = 0.726\, C_m\, \alpha_h \cdot wh^2 \quad ...(8.19\ a)$$

and

$$M_{eh} = 0.299\, p_{eh} \cdot h^2 = 0.299\, C_m\, \alpha_h\, wh^3 \quad ...(8.20\ a)$$

If there is tail water on the down stream side, hydrodynamic pressure (or suction) will also be exerted on the d/s face of the dam, the value of which may be found from the above formulae, where h is the tail water depth.

Effect of horizontal earthquake acceleration on the vertical component of reservoir and tail water load

Since the hydrodynamic pressure (or suction) acts normal to the face of the dam, there shall, therefore, be a vertical component of this force if the face of the dam against which it is acting is sloping and its magnitude at any section is given by

$$W_{hv} = (P_{e2} - P_{e1}) \tan \varphi \quad ...(8.21)$$

where W_{hv} = increase (or decrease) in *vertical component* of load due to hydro-dynamic force

P_{e2} = total shear due to horizontal component by hydrodynamic force at the elevation of the section being considered.

P_{e1} = total shear due to horizontal component of hydrodynamic force at the elevation at which the slope of the dam face commence and

φ = Actual angle between the face of the dam and the vertical.

The moment due to the vertical component of reservoir and tail water load may be obtained by determining the lever arm from the centroid of the pressure diagram.

(b) EFFECT OF VERTICAL EARTHQUAKE ACCELERATION

Due to vertical earthquake acceleration, the dam as well as reservoir water are accelerated vertically upwards or downwards. This is resisted by the dam and water owing to their *inertia. Hence the effect of vertical earthquake acceleration is to change the unit weight of water* (w) and the dam material (w_m). An acceleration upwards increases the weights and an acceleration downwards decreases the weights. Thus the *altered* unit weights of the dam material and water, due to vertical acceleration will be $w_m\,(1 \pm \alpha_v)$ and $w\,(1 \pm \alpha_v)$ respectively.

To consider the effect of vertical earthquake acceleration, the vertical seismic coefficient would be as follows :

(a) For seismic coefficient method :

At the top of the dam, it would be equal to 0.75 times α_h value given by Eq. 8.3 and reducing linearly to zero, value at the base.

(b) For response spectrum method

At the top of the dam, it would be equal to 0.75 times α_h value given by Eq. 8.4 and reducing linearly to zero value at the base.

Knowing the value of α_{vy} at any depth y, by each of the above two methods, the increase or decrease in the unit weights (w_m and w) may be obtained by adopting the same procedure as followed for the computation of inertia forces due to horizontal earthquake acceleration.

Note : While considering the inertia forces due to horizontal acceleration, the effect of vertical acceleration on the unit weights are not considered because it is unlikely that both horizontal and vertical accelerations of full magnitude will occur simultaneously.

8.7. ICE PRESSURE

The ice pressure is more important for dams constructed in cold countries, or at higher elevations. The ice formed on the water surface of the reservoir is subjected to expansion and contraction due to temperature variations. The coefficient of thermal expansion of ice being five times more than that of concrete, the dam face has to resist the force due to expansion of ice. This force acts linearly along the length of the dam, at the reservoir level.

IS 6512-1984 recommends that till specific reliable procedures become available for the assessment of ice pressure, it may be provided for at the rate of 250 kN/m^2 applied to the face of the dam over the anticipated area of contact of ice with the face of the dam.

8.8. WAVE PRESSURE

Waves are generated on the reservoir surface because of the wind blowing over it. Wave pressure depends on the height of the waves developed. Wave height may be calculated from the following formula given by D.A. Molitor (IS : 6512-1972).

$$h_w = 0.0322\sqrt{V.F.} + 0.763 - 0.271\,(F)^{1/4} \quad \text{for } F < 32 \text{ km} \quad ...(8.22)$$

and

$$h_w = 0.0322\sqrt{V.F.} \quad \text{for } F > 32 \text{ km} \quad ...(8.22\ a)$$

where

h = height of waves in metres, between trough and crest

V = wind velocity in km per hour

F = fetch or straight length of water expanse in km

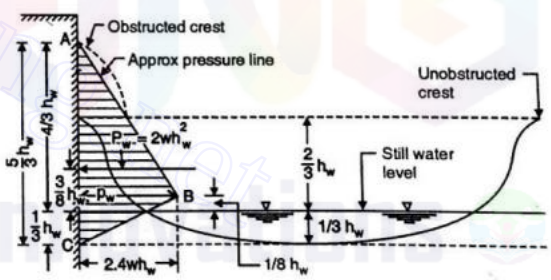

FIG. 8.10 WAVE PRESSURE.

Fig. 8.10 shows the configuration of the wave and the approximated wave pressure distribution diagram.

The pressure intensity due to waves is given by

$$p_w = 2.4\, wh_w \quad ...(8.23\ a)$$

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where p_w is the maximum unit pressure which occurs at $\frac{1}{8}h_w$ metres above still water surface. The pressure distribution is curvilinear. However, for design purpose, the pressure distribution may be assumed to be represented by a triangle of height equal to $\frac{5}{3}h_w$. Hence, the total pressure P_w is given by

$$P_w = (2.4\, w h_w) \times \frac{1}{2}\left(\frac{5}{3}h_w\right) = 2\, w\, h_w^2 \qquad ...(8.23\, b)$$

and this acts at a distance of $\frac{3}{8}h_w\ (= 0.375\, h_w)$ above the reservoir surface, as shown in Fig. 8.10.

8.9. SILT PRESSURE

The river brings silt and debris along with it. The silt load gets deposited to an appreciable extent when dam is constructed. The dam is, therefore, subjected to silt pressure in addition to the water pressure. If γ_s' is the submerged unit weight of silt and ϕ is the angle of internal friction and h_s is the height to which the silt is deposited, the silt pressure is given by

$$P_s = \frac{1}{2}\gamma_s'\, h_s^2\, \frac{1-\sin\phi}{1+\sin\phi} \qquad ...(8.24)$$

If the upstream face is inclined, the vertical weight of silt supported on the slope also acts as vertical force.

The *U.S.B.R. recommendation* presently followed in the design is to consider silt pressure as a fluid pressure which creates a horizontal pressure with its density equal to 360 kg/m^3 (3.53 kN/m^3), and a vertical pressure with its density equal to 920 kg/m^3 (9.03 kN/m^3). Thus, the total horizontal pressure will be equal to $360 \times \frac{h_s^2}{2} = 180\, h_s^2$ kg, and the vertical pressure will be equal to $920 \times \frac{h_s^2}{2} = 460\, h_s^2$ kg. These pressures are in addition to the usual pressures.

According to Indian Standard (IS : 6512-1984), the silt pressure and water pressure exist together in submerged silt. The following criteria is recommended for calculating forces due to silt :

(*a*) Horizontal *'silt and water pressure'* is assumed to be equivalent to that of a fluid weighing 1360 kg/m^3, (13.34 kN/m^3) and

(*b*) Vertical *'silt and water pressure'* is determined as if silt and water together have a density of 1925 kg/m^3 (18.88 kN/m^3).

Since the horizontal and vertical components due to water have already been taken with $w = 1000$ kg/m^3 (9.81 kN/m^3), the *additional forces* due *to silt alone* are computed by taking $\gamma_s' = 1360 - 1000 = 360$ kg/m^3 (3.53 kN/m^3) for horizontal pressure and $\gamma_s' = 1925 - 1000 = 925$ kg/m^3 (9.07 kN/m^3) for vertical pressure. These values compare favourably with the values recommended by USBR.

8.10. WIND PRESSURE

It is a minor force and need hardly be taken into account for the design of dams. Wind pressure is required to be considered only on that portion of the super structure which is exposed to the action of wind. Normally wind pressure is taken as 1 to 1.5 kN/m^2 for the area exposed to the wind pressure.

8.11. COMBINATION OF LOADING FOR DESIGN

(*a*) **U.S.B.R. Recommendations.** All the forces acting on a dam do not act simultaneoulsy. There are various combinations of loads which may act simultaneously. Design of the dam should be based on the most adverse combination of the 'probable load condition'. The U.S.B.R. specifies the 'normal' and 'extreme' load combinations as below:

1. *Normal Load Combination.* (*i*) Normal water surface elevation, ice pressure, silt pressure and normal uplift. This particular class of loading is to be considered where ice force may develop.

(*ii*) Normal water surface elevation, earthquake force, silt pressure and normal uplift.

(*iii*) Maximum water surface elevation, silt pressure and normal uplift.

2. *Extreme Load Combination.* Maximum flood water elevation, silt pressure and extreme uplift with no drain in operation to release the uplift.

Reservoir-Empty Condition. The design is also to be checked for the reservoir empty condition. The condition is primarily important for testing stresses inside the dam and around opening provided in the body of the dam.

(*b*) **Indian Standard Recommendations (IS 6512-1984).**

Gravity dam design shall be based on the most adverse load conditions *A, B, C, D, E, F* or *G*, given below using the safety factors prescribed. Depending on the scope and details of the various project components, site conditions and construction programme, one or more of the loading combinations *A, B, C, D* and *E* may not be applicable *ipso-facto* and may need suitable modifications.

(i) Load combination A (construction condition). Dam completed but no water in reservoir and no tailwater.

(ii) Load combination B (Normal operating condition). Full reservoir elevation (or top of gates at crest), normal dry weather tailwater, normal uplift, ice and silt (if applicable).

(iii) Load combination C (Flood discharge condition). Reservoir of maximum flood pool elevation, all gates open, tailwater at flood elevation, normal uplift and silt (if applicable).

(iv) Load combination D. Combination *A* with earthquake.

(v) Load combination E. Combination *B* with earthquake but no ice.

(vi) Load combination F. Combination *C*, but with extreme uplift (drainage inoperative).

(vii) Load combination G. Combination *E*, but with extreme uplift (drainage inoperative).

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Example 8.1. ***Seismic coefficient method***

A 100 m high concrete gravity dam, shown in Fig. 8.11 is located in seismic zone IV. The unit weight of dam material is 24 kN/m³ and the modulus of elasticity of the material is 2.05 × 10⁷ kN/m². Compute the forces induced due to earthquake, per unit length of the dam, using seismic coefficient method. Use recommendations of IS : 1893-1984.

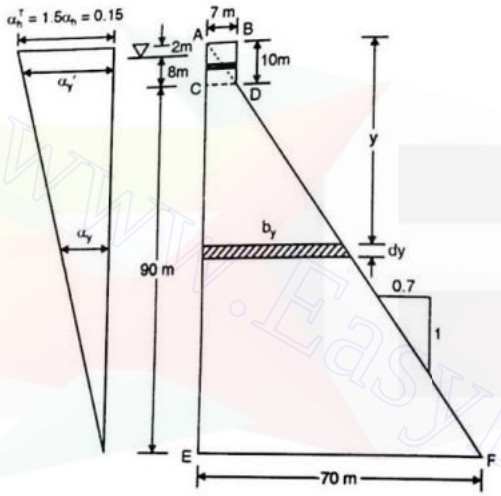

FIG. 8.11

Solution

(a) Inertia forces and moments

For zone IV, we get from Table 8.1,

$\alpha_0 = 0.05$

Hence from Eq. 8.3

$\alpha_h = 2\,\alpha_0 = 2 \times 0.05 = 0.1$

The distribution of horizontal seismic coefficient along the depth of the dam will be an inverted triangle, with $\alpha_h^T = 1.5\,\alpha_h = 0.15$ at the top and zero value at the bottom.

At section CD : 10 m below top

Consider an elementary strip of thickness dy' at y' below the top.

Weight of strip $W_y' = (1 \times 7\,dy')\,w_m = 7 \times 24\,dy' = 168\,dy'$

$$\alpha_y' = 1.5\,\alpha_h\,(1 - y'/H) = 1.5 \times 0.1\,(1 - y'/100) = 0.15\,(1 - 0.01\,y')$$

$\therefore$ Shear force, $$F_{10} = \int_0^{10} \alpha_y'\,W_y' = \int_0^{10} 0.15\,(1 - 0.01y')\,168\,dy' = 25.2\left[y' - \frac{0.01y'^2}{2}\right]_0^{10}$$

$$= 239.4 \text{ kN}$$

Moment, $$M_{10} = \int_0^{10} \alpha_y'\,W_y'\,(10 - y') = \int_0^{10} 0.15\,(1 - 0.01\,y')\,168\,(10 - y')\,dy'$$

$$= 25.2\left[10\,y' - \frac{0.1\,y'^2}{2} - \frac{y'^2}{2} + \frac{0.01y'^3}{3}\right]_0^{10} = 1218 \text{ kN-m}$$

At the base : 100 m below top

Consider a strip of thickness dy at y below the top.

Width of strip $b_y = 7 + 0.7\,(y - 10) = 0.7\,y$

Weight of strip $W_y = 1 \times 0.7\,y \times dy \times 24 = 16.8\,y\,dy$ kN

$\alpha_y = 0.15\,(1 - 0.01\,y)$

$\therefore$ Shear force, $$F_{100} = F_{10} + \int_{10}^{100} \alpha_y\,.\,W_y = 239.4 + \int_{10}^{100} 0.15\,(1 - 0.01\,y)\,16.8\,y\,dy$$

$$= 239.4 + 2.52\left[\frac{y^2}{2} - \frac{0.01\,y^3}{3}\right]_0^{100} = 4321.8 \text{ kN}$$

Moment $$M_{100} = M_{10} + F_{10}\,(100 - 10) + \int_{10}^{100} \alpha_y\,.\,W_y\,(100 - y)$$

$$= 1218 + 239.4 \times 90 + \int_{10}^{100} 0.15\,(1 - 0.01\,y)\,16.8\,y\,(100 - y)\,dy$$

$$= 22764 + 2.52\left[\frac{100\,y^2}{2} - \frac{y^3}{3} - \frac{y^3}{3} + \frac{0.01\,y^4}{4}\right]_{10}^{100} = 221781 \text{ kN-m}$$

(b) Hydrodynamic pressure and moment :

Depth of water in reservoir $= h = 98$ m. The hydrodynamic pressure at any depth y below the reservoir surface is given by

$$p_{ey} = C_y\,\alpha_h\,wh = 9.81 \times 98\,C_y\,\alpha_h = 961.38\,C_y\,\alpha_h$$

where $$C_y = \frac{C_m}{2}\left[\frac{y}{h}\left(2 - \frac{y}{h}\right)\right] + \sqrt{\frac{y}{h}\left(2 - \frac{y}{h}\right)}$$

Taking $C_m = 0.735$ for vertical face $(\theta = 0)$

$$C_y = \frac{0.735}{2}\left[\frac{y}{98}\left(2 - \frac{y}{98}\right) + \sqrt{\frac{y}{98}\left(2 - \frac{y}{98}\right)}\right]$$

Hence $$p_{ey} = \frac{961.38 \times 0.735}{2}\left[\frac{y}{98}\left(2 - \frac{y}{98}\right) + \sqrt{\frac{y}{98}\left(2 - \frac{y}{98}\right)}\right]\alpha_h$$

or $$p_{ey} = 353.307\left[\frac{y}{98}\left(2 - \frac{y}{98}\right) + \sqrt{\frac{y}{98}\left(2 - \frac{y}{98}\right)}\right]\alpha_h$$

At 10 m from the top of the dam : $y = 8$ m

$\therefore$ $$p_{ey} = 353.307\left[\frac{8}{98}\left(2 - \frac{8}{98}\right) + \sqrt{\frac{8}{98}\left(2 - \frac{8}{98}\right)}\right]\alpha_h = 195.142\,\alpha_h$$

where $$\alpha_h = \frac{F_{10}}{W_{10}} = \frac{239.4}{1 \times 7 \times 10 \times 24} = 0.1425$$

$\therefore$ $p_{e10} = 195.142 \times 0.1425 = 27.808$ kN/m²

Force (or shear) $P_{e10} = 0.726\,p_{ey}\,.\,y = 0.726 \times 27.808 \times 8 =$ **161.51 kN**

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$$M_{e10} = 0.299\, p_{ey}\, y^2 = 0.299 \times 27.808\,(8)^2 = \mathbf{532.13}\ \text{kN-m}$$

At the base of the dam

$$y = 98 \text{ m.}$$

$$\therefore \quad p_{ey} = 353.307\left[\frac{98}{98}\left(2-\frac{98}{98}\right)+\sqrt{\frac{98}{98}\left(2-\frac{98}{98}\right)}\right]\alpha_h = 706.614\,\alpha_h$$

where
$$\alpha_h = \frac{F_{100}}{W_{100}} = \frac{4321.8}{1\,[7\times 10+\frac{1}{2}(7+70)\,90]\,24} = 0.05094$$

$$\therefore \quad p_{e100} = 706.614 \times 0.05094 = 35.9953\ \text{kN/m}^2$$

$$\therefore \quad P_{e100} = 0.726\, p_{ey}\,.\,y = 0.726 \times 35.9953 \times 98 \simeq \mathbf{2561\ kN}$$

$$M_{e100} = 0.299\, p_{ey}\,.\,y^2 = 0.299 \times 35.9953\,(98)^2 = \mathbf{103364}\ \text{kN-m}$$

Example 8.2. ***Response spectrum method :***

Solve example 8.1 using response spectrum method

Solution : Design horizontal seismic coefficient is given by Eq. 8.4 :

$$\alpha_h = \beta\, I\, F_0 \frac{S_a}{g} = 2\, F_0 \frac{S_a}{g} \qquad \text{...(8.4)}$$

where $\beta = 1$ and $I = 2$ for gravity dams.

For zone IV, we get $F_0 = 0.25$ from Table 8.2, as per IS 1893-1984.

Hence
$$\alpha_h = 2 \times 0.25\,\frac{S_a}{g} = 0.5\,\frac{S_a}{g}$$

The fundamental period of vibration of the dam is given by Eq. 8.5.

$$T = 5.55\,\frac{H^2}{B}\sqrt{\frac{w_m}{g\,E_s}} = 5.55\,\frac{(100)^2}{70}\sqrt{\frac{24}{9.81 \times 2.05 \times 10^7}}$$

$$= 0.274 \text{ seconds}$$

Hence from Fig. 8.4, we get $S_a/g = 0.19$ for $T = 0.274\,s$ and 5% damping

$$\therefore \quad \alpha_h = 0.5\,\frac{S_a}{g} = 0.5 \times 0.19 = 0.095$$

(*a*) ***Base shear***. The inertia force or base shear per unit length is given by

$$F_B = 0.6\, W\, \alpha_h$$

where
$$W = \text{weight of the dam} = 1\left(7 \times 100 + \frac{63 \times 90}{2}\right)24 = 84840 \text{ kN}$$

$$\therefore \quad F_B = 0.6 \times 84840 \times 0.095 = \mathbf{4835.88\ kN}$$

(*b*) ***Base moment***. $M_B = 0.9\, W\, \bar{h}\, \alpha_h$

where $W = 84840$ kN, as determined above

$$\bar{h} = \frac{7 \times 100 \times \frac{100}{2} + \frac{1}{2} \times 63 \times 90 \times \frac{90}{3}}{7 \times 100 + \frac{1}{2} \times 63 \times 90} = 33.96 \text{ m}$$

$$\therefore \quad M_B = 0.9 \times 84840 \times 33.96 \times 0.095 = \mathbf{246342.6\ kN\text{-}m}$$

(*c*) ***Shear at y (= 10 m) from the top***

$$F_y = C_v'\,.\,F_B$$

For $y/h = 10/100 = 0.1$, we get $C_v' \simeq 0.08$ From Fig. 8.8.

$$\therefore \quad F_{10} = 0.08 \times 4835.88 \simeq \mathbf{387\ kN}$$

(*d*) ***Moment at y (= 10 m) from top***

$$M_y = C_m'\, M_B$$

For $y/h = 10/100 = 0.1$, we get $C_m' \simeq 0.02$

$$\therefore \quad M_{10} = 0.02 \times 246342.6 \simeq \mathbf{4927\ kN\text{-}m}$$

Note : Comparing seismic coefficient method and the response spectrum method, we find that the seismic coefficient method gives some what lower values at the base and considerably lower values near the top.

(*e*) ***Hydrodynamic pressure***

$$p_{ey} = C_y\,.\,\alpha_h\, wh$$

Here $\alpha_h = 0.095$ and is uniform for the entire height

$$h = 98 \text{ m}$$

$$w = 9.81\ \text{kN/m}^2$$

$$C_y = \frac{C_m}{2}\left\{\frac{y}{h}\left(2-\frac{y}{h}\right)+\sqrt{\frac{y}{h}\left(2-\frac{y}{h}\right)}\right\} \quad \text{where } C_m = 0.735 \text{ for } \theta = 0$$

$$= \frac{0.735}{2}\left\{\frac{y}{98}\left(2-\frac{y}{98}\right)+\sqrt{\frac{y}{98}\left(2-\frac{y}{98}\right)}\right\}$$

$$\therefore \quad p_{ey} = 0.095 \times 9.81 \times 98 \times \frac{0.735}{2}\left\{\frac{y}{98}\left(2-\frac{y}{98}\right)+\sqrt{\frac{y}{98}\left(2-\frac{y}{98}\right)}\right\}$$

or
$$p_{ey} = 33.564\left\{\frac{y}{98}\left(2-\frac{y}{98}\right)+\sqrt{\frac{y}{98}\left(2-\frac{y}{98}\right)}\right\}$$

At a depth of 8 m from reservoir level or 10 m from the top.

$$p_{e10} = 33.564\left\{\frac{8}{98}\left(2-\frac{8}{98}\right)+\sqrt{\frac{8}{98}\left(2-\frac{8}{98}\right)}\right\} = 18.538\ \text{kN/m}^2$$

$$\therefore \quad P_{e10} = 0.726\, p_{ey}\,.\,y = 0.726 \times 18.538 \times 8 = \mathbf{107.67\ kN/m}$$

and
$$M_{e10} = 0.299\, p_e\, y^2 = 0.299 \times 18.538\,(8)^2 = \mathbf{354.74\ kN\text{-}m/m}$$

At the base (y = 98 m)

$$p_{e100} = 33.564\left\{\frac{98}{98}\left(2-\frac{98}{98}\right)+\sqrt{\frac{98}{98}\left(2-\frac{98}{98}\right)}\right\} = 67.128\ \text{kN/m}^2$$

$$\therefore \quad P_{e100} = 0.726\, p_{ey}\,.\,y = 0.726 \times 67.128 \times 98 = \mathbf{4776\ kN/m}$$

and
$$M_{e100} = 0.299\, p_{ey}\,.\,y^2 = 0.299 \times 67.128\,(98)^2 = \mathbf{192764\ kN\text{-}m/m}$$

Example 8.3. ***Effect of vertical earthquake acceleration***

For the profile of the gravity dam of Fig. 8.11 (Example 8.1), determine the effect of vertical earthquake acceleration on the weight of the dam at 10 m below the top

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