19

Design and Construction of Gravity Dams

19.1. Definition, etc.

A gravity dam has been defined as a structure which is designed in such a way that its own weight resists the external forces. This type of a structure is most durable and solid, and requires very little maintenance. Such a dam may be constructed of masonry or concrete. However, concrete gravity dams are preferred these days and mostly constructed. They can be constructed with ease on any dam site, where there exists a natural foundation strong enough to bear the enormous weight of the dam. Such a dam is generally straight in plan, although sometimes, it may be slightly curve. The line of the upstream face of the dam, or the line of the crown of the dam if the upstream face in sloping, is taken as the reference line for layout purposes, etc. and is known as the *Base line of the dam* or the '*Axis of the Dam*'. When suitable conditions are available, such dams car be constructed up to great heights. The highest gravity dam in the world is Grand Dixence Dam in Switzerland (284 m), followed by Bhakra dam in India (226 m); both are of concrete gravity type. The ratio of base width to height of all these structures is less than 1 : 1.

19.2. Typical Cross-section

A typical cross-section of a concrete gravity dam is shown in Fig. 19.1. The upstream face may by kept throughout vertical or partly slanting for some of its length, as shown. A drainage gallery is provided in order to relieve the uplift pressure exerted by the seeping water.

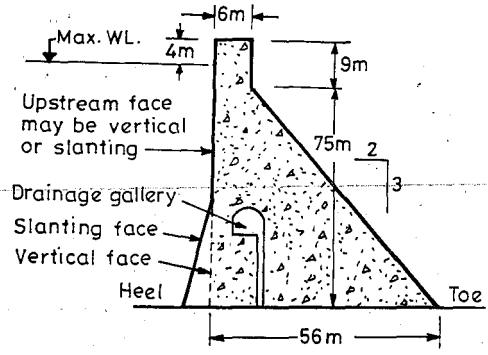


Fig. 19.1. A typical cross-section of a concrete gravity dam.

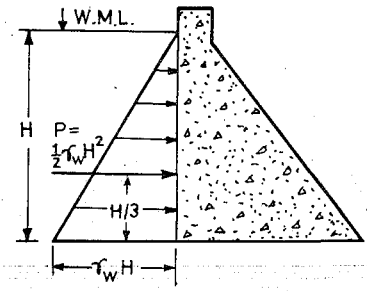
19.3. Forces Acting on Gravity Dam

The various external forces acting on a gravity dam may be :

(1) Water Pressure
(2) Uplift Pressure
(3) Pressure due to earthquake forces
(4) Silt Pressure
(5) Wave Pressure
(6) Ice Pressure
(7) The stabilising force is the weight of the dam itself.

An estimation and description of these forces is given below :

(1) **Water Pressure.** Water pressure (P) is the most major external force acting on such a dam. The horizontal water pressure, exerted by the weight of the water stored on the upstream side on the dam can be estimated from rule of hydrostatic pressure distribution ; which is triangular in shape, as shown in Fig. 19.2 (a) and (b). When the upstream face is vertical, the intensity is zero at the water surface and equal to $\gamma_w H$ at the base ; where γ_w is the unit weight of water and H is the depth of water : as shown in Fig. 19.2 (a). The resultant force due to this external water $= \frac{1}{2}\gamma_w H^2$, acting at $H/3$ from base.



Where γ_w = unit weight of water
9.81 kN/m^3 = 1000 kgf/m^3
Fig. 19.2. (a)

When the upstream face is partly vertical and partly inclined [Fig. 19.2 (b)], the resulting water force can be resolved into horizontal component (P_h) and vertical component (P_v). The horizontal component $P_h = \frac{1}{2}\gamma_w H^2$ acts at $\frac{H}{3}$ from the base ; and the vertical component (P_v) is equal to the weight of the water stored in column $ABCA$ and acts at the c.g. of the area.

Similarly, if there is tail water on the downstream side, it will have horizontal and vertical components, as shown in Fig. 19.2. (b).

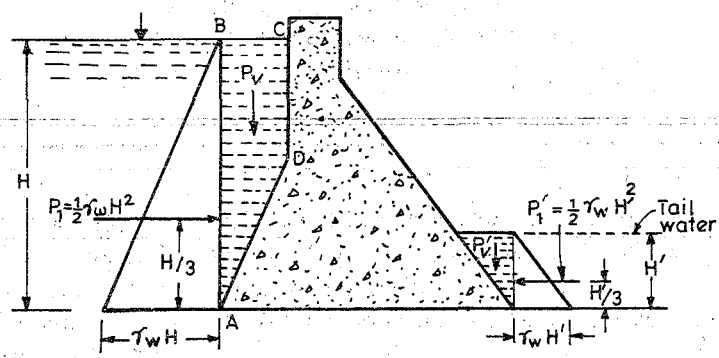


Fig. 19.2 (b)

(2) **Uplift Pressure.** Water seeping through the pores, cracks and fissures of the foundation material, and water seeping through dam body and then to the bottom through the joints between the body of the dam and its foundation at the base ; exert an uplift pressure on the base of the dam. It is the second major external force and must be accounted for in all calculations. Such an uplift force virtually reduces the downward weight of the body of the dam and hence, acts against the dam stability.

The amount of uplift is a matter of research and the present recommendations which are followed, are those suggested by United States Bureau of Reclamation (U.S.B.R.). According to these recommendations, the uplift pressure intensities at the heel and the toe should be taken equal to their respective hydrostatic pressures and joined by a

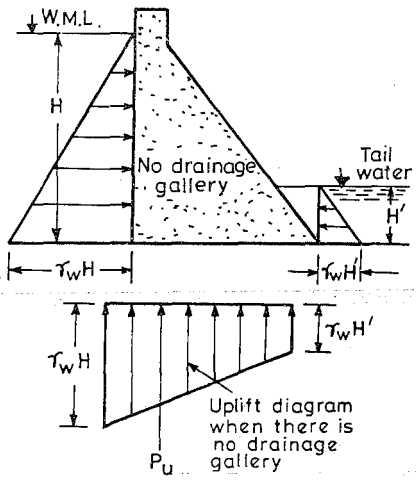


Fig. 19.3 (*a*) Uplift pressure (*U*) diagram, when no drainage gallery is provided.

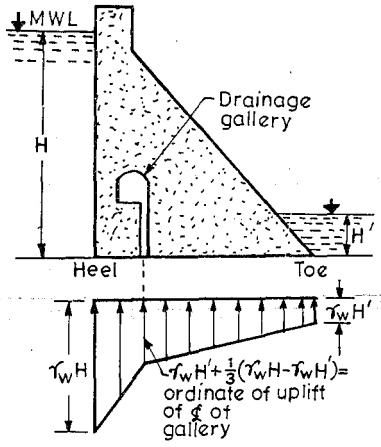


Fig. 19.3 (*b*) Uplift pressure (*U*) diagram, when drainage gallery is provided.

straight line in between, as shown in Fig. 19.3 (*a*). When drainage galleries are provided to relieve the uplift, the recommended uplift at the face of the gallery is equal to the hydrostatic pressure at toe ($\gamma_w \cdot H'$) plus $\frac{1}{3}$rd the difference of the hydrostatic pressures at the heel and the toe ; as shown in Fig. 19.3 (*b*) ; *i.e.* $\left[\gamma_w \cdot H' + \frac{1}{3}(\gamma_w \cdot H - \gamma_w \cdot H')\right]$. It is also assumed that the uplift pressures are not affected by the earthquake forces.

The uplift pressures can be controlled by constructing cut-off walls under the upstream face, by constructing drainage channels between the dam and its foundation, and by pressure grouting the foundation.

(3) **Earthquake Forces.** If the dam to be designed, is to be located in a region which is susceptible to earthquakes, allowance must be made for the stresses generated by the earthquakes.

An earthquake produces waves which are capable of shaking the Earth upon which the dam is resting, in every possible direction.

The effect of an earthquake is, therefore, equivalent to imparting an acceleration to the foundations of the dam in the direction in which the wave is travelling at the moment. Earthquake wave may move in any direction, and for design purposes, it has to be resolved in vertical and horizontal components. Hence, two accelerations, *i.e.* one horizontal acceleration (α_h) and one vertical acceleration (α_v) are induced by an earthquake. The values of these accelerations are generally expressed as percentage of the acceleration due to gravity (g), *i.e.* $\alpha = 0.1\ g$ or $0.2\ g$, etc.

In India, the entire country has been divided into five seismic zones depending upon the severity of the earthquakes. Zone V is the most serious zone and includes Himalayan regions of North India. A map and description of these zones is available in "Physical and Engineering Geology" (1999 edition) by the same author, and can be referred to, in order to obtain an idea of the value of the α which should be chosen for designs. On an average, a value of α equal to 0.1 to 0.15 g is generally sufficient for high dams in seismic zones. A value equal to 0.15 g has been used in Bhakra dam design, and 0.2 g in Ramganga dam design. However, for areas not subjected to extreme earthquakes,

$\alpha_h = 0.1\,g$ and $\alpha_v = 0.05\,g$ may by used. In areas of no earthquakes or very less earthquakes, these forces may be neglected. In extremely seismic regions and in conservative designs, even a value upto 0.3 g may sometimes be adopted.

Effect of vertical acceleration (α_v). A vertical acceleration may either act downward or upward. When it is acting in the upward direction, then the foundation of the dam will be lifted upward and becomes closer to the body of the dam, and thus the effective weight of the dam will increase and hence, the stress developed will increase.

When the vertical acceleration is acting downward, the foundation shall try to move downward away from the dam body ; thus reducing the effective weight and the stability of the dam, and hence is the worst case for designs.

Such acceleration will, therefore, exert an **inertia force** given by

$$\frac{W}{g}\alpha_v \text{ (}i.e.\text{ force} = \text{Mass} \times \text{Acceleration)}$$

where W is the total weight of the dam.

$\therefore$ The net effective weight of the dam $= W - \frac{W}{g} \cdot \alpha_v$.

If $\alpha_v = k_v \cdot g$

[where k_v is the fraction of gravity adopted for vertical acceleration, such as 0.1 or 0.2, etc.].

Then, the net effective weight of the dam

$$= W - \frac{W}{g} \cdot k_v \cdot g = W[1 - k_v].$$

In other words, vertical acceleration reduces the unit weight of the dam material and that of water to $(1 - k_v)$ times their original unit weights.

Effects of horizontal acceleration (α_h). Horizontal acceleration may cause the following two forces :

(*i*) Hydrodynamic pressure ; and

(*ii*) Horizontal inertia force.

Both these forces are discussed below :

(*i*) *Hydrodynamic pressures.* Horizontal acceleration acting towards the reservoir causes a momentary increase in the water pressure, as the foundation and dam accelerate towards the reservoir and the water resists the movement owing to its inertia. The extra pressure exerted by this process is known as *hydrodynamic pressure.*

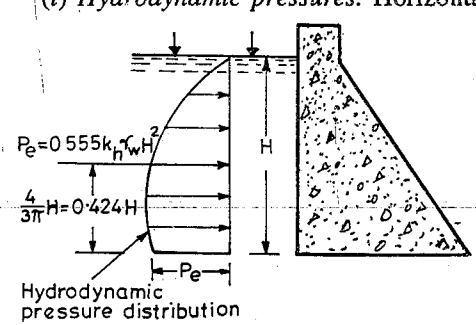


Fig. 19.4. Showing development of Hydrodynamic pressure by a horizontal earthquake moving towards the reservoir. A similar pressure will be developed on d/s tail water when the earthquake is reversed.

According to **Von-Karman**, the amount of this hydrodynamic force (P_e) is given by.

$$P_e = 0.555 \cdot k_h \gamma_w \cdot H^2 \qquad \text{...(19.1)}$$

and it acts at the height of $\frac{4H}{3\pi}$ above the base, as shown in Fig. 19.4.

(2) Rook-fill dams.

(3) Solid masonry gravity dams.

These types are discussed below :

(1) **Earth Dams**. Earth dams are made of soil that is pounded down solidly. They are built in areas where the foundation is not strong enough to bear the weight of a concrete dam, and where earth is more easily available as a building material compared to concrete or stone or rock.

Some important earth dams of the world are :

(*i*) Green mountain dam on Colorado river in U.S.A.

(*ii*) Swift dam in Washington in U.S.A.

(*iii*) Side flanks of Nagarjun Sagar dam in India.

(*iv*) Trinity Dam in California in U.S.A.

(*v*) Maithan Dam in India (which is partly Earthen and partly Rockfill).

(2) **Rockfill Dams.** Rockfill are formed of loose rocks and boulders piled in the river bed. A slab of reinforced concrete is often laid across the upstream face of a rockfill dam to make it water-tight.

Some important rock-fill dams of the world are ;

(*i*) The Salt Springs Dam in California (345′ high) in U.S.A.

(*ii*) The San Gabriel No. 1 Dam (321′ high) in U.S.A.

(*iii*) Cougar Dam on Mc-Knezie River in Oregon (445′ high) in U.S.A.

(3) **Solid-masonry Gravity dams.** These are familiar to us by now, after we have talked about Aswan, Roosevelt, Hoover, and above all Bhakra dam.

These big dams are expensive to be built but are more durable and solid than earth and rock dams. They can be constructed on any dam site, where there is a natural foundation strong enough to bear the great weight of the dam.

These three types of dams were all found in ancient days. In recent times, *four other types of dams have come into practice.* They are :

(4) Hollow masonry gravity dams ;

(5) Timber dams ;

(6) Steel dams ; and

(7) Arch dams.

(4) **The hollow masonry gravity dams**. These are essentially designed on the same lines on which the solid masonry gravity dams are designed. But they contain less concrete or masonry ; about 35 to 40% or so. Generally, the weight of water is carried by a deck of R.C.C. or by arches that share the weight burden. They are difficult to build and are adopted only if very skilled labour is easily available, otherwise the labour cost is too high to build its complex structure.

(5) **Steel dams.** These are not used for major works. Today, steel dams are used as temporary coffer dams needed for the construction of permanent dams. Steel coffer dams are usually reinforced with timber or earthfill.

(6) **Timber dams.** These are short lived, since in a few years time, rotting sets in. Their life is not more than 30 to 40 years and must have regular maintenance during that time. However they are valuable in agricultural areas, where a cattle raiser may need a pool for his live stock to drink from, and for meeting other such low-level needs.

(7) **Arch dams.** Arch dams are very complex and complicated. They make use of the horizontal arch action in place of weight to hold back the water. They are best suited at sites where the *dam must be extremely high and narrow*. Some examples are :

(*i*) *Sautet dam* on the Drac River in France, 414′ high, but only 230′ long at top and 85′ long at bottom of the gorge, 56′ thick at bottom and 8′ thick at top.

(*ii*) *The Tignes dam* in France (592′ high).

(*iii*) *Mauvoisin dam* on the Drause River in Switzerland, (780′ high).

(*iv*) *Idduki* dam in Kerala State, across the Periyar river, which is the only arch dam in India. It is 366 m (1200′) long double curvature arch dam, made in concrete, and has a height of about 170 m (560′).

17.3. Problems in Dam Construction

Dams are extremely useful things. Anyone who lives in Punjab or at Asansol in West Bengal, knows how valuable dams are. The farmers of Punjab and people getting electricity from the Bhakra sing praises for it. The people of areas benefitted by various dams and other ancillary works on Damodar river are really thankful to those human beings who have miraculousy harnessed the Damodar river for them. The prosperity and welfare of millions and billions of people depend directly on these towering handsome dams with which the nation's rivers have been harnessed.

But dams can cause problems too. Dams have drawbacks and disadvantages also. Let us here discuss some of the negative features of dams and let us see what can be done to overcome them. There are four major problems, in general, which are posed by such huge constructions. They are:

(1) Fish Problem ;

(2) Submergence Problem ;

(3) Failure Problem; and

(4) Bomb Problem

They are described below:

(1) **Fish Problem.** On large rivers, in late summer season, fish move from downstream to upstream to lay their eggs. These eggs are fertilised by male fish. The old fish may get exhausted and the new born fish again move downstream. They, after two to three years, return to their ancestral spawning place and may die after getting exhausted, while the newborns move downstream. The cycle goes on for years.

The fish which move to their ancestral spawning place (upstream) are called *anadromous fish. Salmon and Hilsa* are typical examples of such a fish. These are commercially valuable fish, and important industries are dependent on them.

When a dam barrier is constructed on a river, these fish can not move upstream to lay their eggs; because it is impossible for these fish to overtop such a barrier. But surprisingly, even when they find a barrier in their path of advancement towards their ancestral spawning ground, these fish do not return to their downstream dwelling place (*i.e.* sea). However, they go on fighting against the barrier, trying furiously to overtop it, till they get exhausted and die down. This results in a serious large scale killing of fish, causing great damage to fish industry and economy of the nations.

In the beginning, much attention was not paid to this problem; but a little later, it was realised, and serious attempts were made to find out solution to the problem.

Sometimes, fish were trapped on one side of the dam and passed on to the other side by giant steel and plastic nets. An external arrangement called *Fish Ladder* was also devised.

Fish Ladder. Just as river-going vessels can bypass a dam by using a navigation lock, so a series of 'locks' enable the fish to get over the dam. A separate channel is created, consisting of a series of little dams that form a row of pools, rising up over the big dam to reservoir level. The salmon, entering the lowest rung of the ladder at the base of the dam, could leap from pool to pool until they had crested the dam. Then, they could continue on through the reservoir to the spawning grounds. The new born fish called *finger lings* could later return to the sea (downstream) in the same fashion via the ladder. A section, plan and photographic view of a fish ladder has already been shown in the chapter on Weirs.

In the beginning, the fish ladders worked better in theory than in practice. The fish seemed to prefer to mill ground in splashing water under spillway, instead of entering the ladder. This difficulty was overcome by careful design that put the fish ladder in the place where it was most likely to attract the fish. Another problem was that the slow moving water was stranger to fish and they tended to collect in the lower pools without going onward.

Millions and billions were spent into fish-ladder research. Improvements in design made the fish ladder more attractive to fish, more like the rapids they were accustomed to.

Fish ladders are not always practicable from engineering stand point. In such cases, other steps have to be taken to protect the fish.

Meanwhile, other experiments are going forward to see if fish can be successfully induced to spawn in waters other than their own ancestral spawning grounds. In the long run, it may save millions of currency to construct *fish hatcheries* instead of *fish-ladders.* There are many possible solutions to the problem of anadromous fish, and research is being undertaken in different regions of the world to find out a better solution to the problem.

(2) **Submergence Problem.** Whenever a dam is constructed across a river to store water on the upstream side, a large area gets submerged due to the rise in the water levels. The entire area which gets submerged, forming a reservoir, has to be calculated and acquired before a dam can be constructed. The owners of the land have to be persuaded, adequately compensated, and well settled somewhere else, before, the work can be taken up in hand. Hence it is necessary to investigate the probable damage caused by this submergence.

(3) **Failure Problem.** We try our best to build dams to last as long as possible. Every person whosoever has worked on a dam hopes that the dam will live as long as the pyramids of Egypt. But many a times, the dam give way under the continued insistent pressure of the water penned up behind them. This failure of the dam may be caused either due to bad workmanship or due to faulty design or due to the occurrence of unanticipated floods.

Luckily, these disasters have been comparatively rare in this century. Dams used to give way easily in olden times, but due to engineering advancement in modern times, their failure has been considerably reduced.

These huge structures are now properly designed, keeping in view the various forces which they are going to face. Proper and rational design, good supervision and constant vigil and watch during maintenance period ensures their safety and makes us fairly confident of it. Bhakra Dam on Satluj River in India and Boulder Dam on Colorado River in U.S.A. *cannot fail* in one attempt, how furiously these rivers may try to move their foundations. We are fairly confident of this, but sometimes the confidence is *rudely and cruelly repaid with tragedies.*

Dams may sometimes fail due to excessive and unanticipated earthquakes. The *Koyna Dam* in India was at the verge of failure in 1968 earthquake. Thanks to the efforts of the Indian engineers who saved that dam by toiling hard day and night. A very confident dam called *Vega de Tera Dam* is Spain failed in January 1959. The people were tucked in the town of Rivaldelago. The disaster caused was tremendous. Rivaldelago was flattened. Telephone poles were snapped like matchsticks. Within moments, 123 villages were drowned. Several hundred luckier ones were saved, but were rendered homeless. This was a case where a dam had simply not been built strong enough to bear the full weight of its intended reservoir. Heavy rains wrecked it. *Faulty design and bad engineering must be blamed.*

Another important dam called *The Malpasset Dam,* a 200 feet high arch dam on the Reyran River, was completed in 1954. This dam gave way in December 1956, causing 421 persons to die in floods. Investigations revealed that the dam had failed because the foundation rock has shifted along a thin clay seam in the left abutment, making the dam unstable and vulnerable to any serious stress.

We learnt from our mistakes;and several other dams of the same type, then under construction in Europe, were quickly resurveyed to find the possibility of such a geological formation. This was very very small comfort to the relatives of those who died when Malpasset failed; but at least, we should learn from our mistakes and there should be no such repetitions.

(4) **The Bomb Problem.** The dams create dangers in wars, especially in modern atomic age. One single atom bomb may cause the failure of Hoover Dam (Boulder Dam) or Bhakra Dam. The resultant failure of such a dam will create catastrophes, but also, it will get contaminated by radioactivity from which there could be no escape.

This is an important point which is generally stressed by opponents of big dams. But the only answer to this argument is that it would not be advisable to deprive ourselves of the benefits of big dams simply because they are hazards in war time. After all, an atom bomb dropped in Calcutta, Delhi, or New York would also cause tremendous damage and catastrophe, but this *does not mean* that we should not *develop big cities.*

Atomic war is dangerous to every aspect of living and not only to the construction of dams. We don't refuse riding in automobiles or aeroplanes because of the fear of accidents. *Certain risk has to be accepted if there is to be progress.*

So, without denying the very great damage that could be caused by atomic explosions at our dams, we must go on building dams. We need them and we must devote our energies to the cause of continued peace, so that bombs will never be able to fall. We may also take more precautions, and anticraft guns and radars can be established at and in the vicinity of such important works. The use of atomic energy for peaceful purposes and a general feeling of brotherhood is the only possible way to reduce such threats.

17.4. Selection of the Type of Dam and Their Classifications

(1) *Classification.* Dams can be classified in various ways depending upon the purpose of the classification.

(1) *Classification According to the Material used for Dam Construction :*

The dams classified according *to the material used* for construction are: Solid masonry gravity dams, Earthen dams, Rockfill dams, Hollow masonry gravity dams, Timber dams, Steel dams, and R.C.C. Arch dams. They have already been explained in a previous article.

(2) *Classification According to Use*

(*i*) **Storage Dams.** They are constructed in order to store water during the periods of surplus water supply, to be used later during the periods of deficient supply. The stored water may be used in different seasons and for different uses. They may be further classified depending upon the specific use of this water, such as navigation, recreation, water supply, fish, electricity, etc.

(*ii*) **Diversion Dams.** These small dams are used to raise the river water level, in order to feed an off-taking canal and or some other conveyance systems. They are very useful as irrigation development works. A diversion dam is generally called a **weir** or a **barrage.**

(*iii*) **The Detention Dams.** They detain food-waters temporarily so as to retard flood runoff and thus minimise the bad effects of sudden flood.

Detention dams are sometimes constructed to trap sediment. They are often called debris dams.

(3) *Classification According to Hydraulic Designs*

(*i*) **Overflow Dams.** They are designed to pass the surplus water over their crest. They are often called Spillways. They should be made of materials which will not be eroded by such discharges.

(*ii*) **Non-overflow Dams.** They are those which are not designed to be overtopped. This type of design gives us wider choice of materials including earthfill and rockfill dams.

Many a times, the overflow dam and the non-overflow dam are combined together to form a composite single structure.

(*iii*) **Rigid Dams and Non-rigid Dams.** Rigid dams are those which are constructed of rigid materials like masonry, concrete, steel, timber, etc.; while non-rigid dams are constructed of earth and rock-fill. They have already been explained.

17.5. Factors Governing the Selection of a Particular Type of Dam

Whenever we decide to construct a dam at a particular place, the first baffling problem which faces us, is to choose the kind of the dam. *Which type will be the most suitable and most economical?* Two, three kinds of dams may be technically feasible, but only one of them will be the most economical. Various designs and their estimates have to be prepared before signalling one particular type. The various factors which must be thoroughly considered before selecting one particular type are described below:

(1) **Topography.** Topography dictates the first choice of the type of dam. For example:

(*i*) A narrow U-shaped valley, *i.e.* a narrow stream flowing between high rocky walls, would suggest a concrete overflow dam.

(*ii*) A low, rolling plain country, would naturally suggest an earth fill dam with a separate spillway.

(*iii*) The availability of a 'Spillway Site' is very important while selecting a particular kind of a dam.

(*iv*) A narrow V-shaped valley indicates the choice of an arch dam. It is preferable to have the top width of the valley less than one-fourth of its height. But a separate site for the spillway must also be available.

(2) **Geology and Foundation Conditions.** The foundations have to carry weight of the dam. The dam site must be thoroughly surveyed by geologists, so as to detect the thickness of the foundation strata, presence of faults, fissured materials, and their permeability, slope, and slip, etc.

The various kinds of foundations generally encountered are discussed below:

(*i*) *Solid Rock Foundations.* Solid rock foundations such as granite, gneiss, etc. have a strong bearing power. They offer high resistance to erosion and percolation. Almost every kind of dam can be built on such foundations. Sometimes, seams and fractures are present in these rocks. They must be grouted and sealed properly.

(*ii*) *Gravel Foundations.* Coarse sands and gravels are unable to bear the weight of high concrete gravity dams and are suitable for earthen and rock-fill dams. Low concrete gravity dams up to a height of 15 m may also be suggested on such foundations.

These foundations have high permeability and, therefore, subjected to water percolation at high rates. Suitable cut-offs must be provided to avoid danger of undermining.

(*iii*) *Silt and Fine Sand Foundations.* They suggest the adoption of earth dams or very low gravity dams (upto height of 8 m). A rockfill dam on such a foundation is not suitable. Seepage through such a foundation may be excessive. Settlement may also be a problem. They must be properly designed to avoid such dangers. The protection of foundations at the downstream toe from erosion must also be ensured.

(*iv*) *Clay Foundations.* Unconsolidated and high moisture clays are likely to cause enormous settlement of the dam. They are not fit for concrete gravity dams or for rock-fill dams. They may be accepted for earthen dams, but that too, after special treatment. (*v*) *Non-uniform Foundations.* At certain places, a uniform foundation of the types described above may not be available. In such a case, a non-uniform foundation of rock and soft material may have to be used if the dam is to be built. Such unsatisfactory conditions have to be dealt with by special designs. However, every problem is an individual problem and a solution has to be found by experienced engineers. For example—

The Jawahar Sagar Dam in Rajasthan offered such a problem. A bed of clay was encountered, between the base of the dam and solid rock foundation. It was not economically feasible to remove this clay bed. The solution adopted was to anchor the base of the dam to the foundations below, by means of prestressed cables.

(3) **Availability of Materials.** In order to achieve economy in the dam, the materials required for its construction must be available locally or at short distances from the construction site.

Sometimes, good soil is easily available, which naturally calls for an earthen dam. If sand, cement and stone, etc., are easily available, one should naturally think of a concrete gravity dam. If the material has to be transported from far off distances, then a hollow concrete dam (Buttress) is a better choice.

(4) **Spillway Size and Location.** Spillway, as defined earlier, diposes of the surplus river discharge. The capacity of the spillway will depend on the magnitudes of the floods to be by- passed. The spillway will, therefore, become much more important on streams with large flood potential. On such rivers, the spillway may become dominant structure, and the type of dam may become the secondary consideration.

The cost of constructing a separate spillway may be enormous or sometimes a suitable separate site for a spillway may not be available. In such cases, combining the spillway and the dam into one structure may be desirable, indicating the adoption of a concrete overflow dam.

At certain places, where excavated material from a separate spillway channel may be utilised in dam embankment, an earthfill dam may prove to be advantageous. Small spillway requirement often favours the selections of earth fill or rockfill dams even in narrow dam sites.

The practice of building a concrete spillway on earth and rock embankments is being discouraged these days, because of their conservative design assumptions and the vigil and watch that has to be kept during their operations.

(5) **Earthquake Zone.** If the dam is to be situated in an earthquake zone, its design must include the earthquake forces. Its safety should be ensured against the increased stress induced by an earthquake of worst intensity. The type of structures best suited to resist earthquake shocks without danger are earthen dams and concrete gravity dams.

(6) **Height of the Dam.** Earthen dams are usually not provided for heights more than 30 m or so. Hence, for greater heights, gravity dams are generally preferred.

(7) **Other Considerations.** Various other factors such as, the life of the dam, the width of the roadway to be provided over the dam, problem of skilled labour, legal and aesthetic point must also be considered before a final decision is taken. Overall cost of construction and maintenance and the funds available will finally decide the choice of a particular kind of a *dam at a particular place.*

17.6. Selection of Dam Site

The selection of a site for constructing a dam should be governed by the following factors:

(1) Suitable foundations (as determined in the previous article) must be available.

(2) For economy, the length of the dam should be as small as possible, and for a given height, it should store the maximum volume of water. It, therefore, follows, that the river valley at the dam site should be narrow but should open out upstream to provide a large basin for a reservoir. A general configuration of contours for a suitable site is shown in Fig. 17.1.

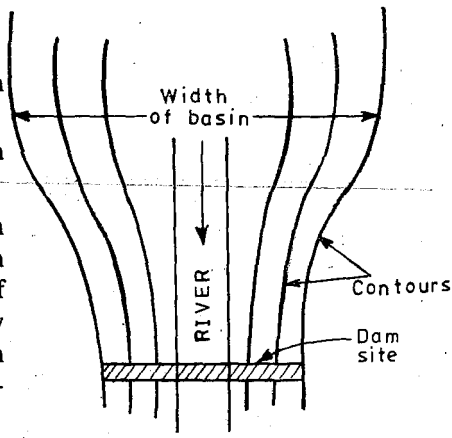


Fig. 17.1

(3) The general bed level at dam site should preferably be higher than that of the river basin. This will reduce the height of the dam and will facilitate the drainage problem.

(4) A suitable site for the spillway should be available in the near vicinity. If the spillway is to be combined with the dam, the width of the gorge should be such as to accommodate both.

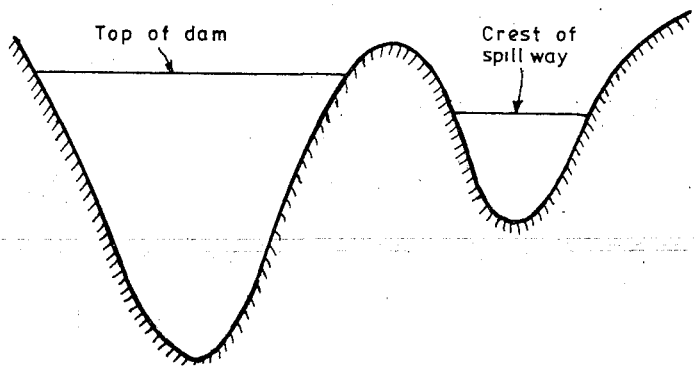


Fig. 17.2

The best dam site is one, in which a narrow deep gorge is separated from the flank by a hillock with its surface above the dam, as shown in Fig. 17.2.

If such a site is available, the spillway can be located separately in the flank, and the main valley spanned by an earthen or similar dam. Sometimes, the spillway and concrete masonry dam may be compositely spanned in the main gorge, while the flanks are in earth at low cost.

(5) Materials required for the construction should be easily available, either locally or in the near vicinity, so that the cost of transporting them is as low as possible.

(6) The reservoir basin should be reasonably water-tight. The stored water should not escape out through its side walls and bed.

(7) the value of land and property submerged by the proposed dam should be as low as possible.

(8) The dam site should be easily accessible, so that it can be economically connected to important towns and cities by rails, roads, etc.

(9) Site for establishing labour colonies and a healthy environment should be available in the near vicinity.

STORIES OF A FEW IMPORTANT DAMS

Before we take up the actual planning and design of concrete gravity dams and earthen dams in subsequent chapters, let us narrate the stories of certain such important dams. This will give us an idea as to what actually happens in the field and to what kind of difficulties are encountered and how they are overcome.

17.7. Hoover Dam

Hoover dam (Fig. 17.3) is a concrete gravity dam, constructed on the Colorado river in California (U.S.A.). The construction of this dam was taken off the drawing boards on January 26, 1892 when the Colorado River Commission presided by Mr. Harbert Hoover discussed its construction in their first meeting. But the actual construction of this dam could start only in late 1930. The construction took about 2 years of non-stop work, every minute of the hour, 24 hours a day, and 365 days a year. On the day of Christmas, the pouring of concrete was tremendous even under searing desert sun by day and under floodlights at night.

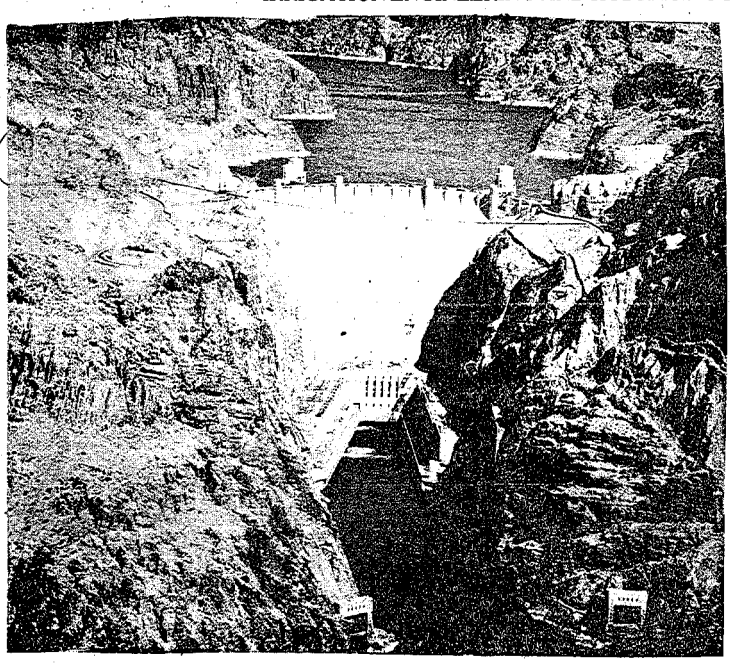


Fig. 17.3. Photo view of Hoover Dam.

Name Problem. The name of the Hoover Dam became a political issue. It was first of all named as *Boulder Dam.* Later when Herbert Hoover became the President of America, its name was changed to *Hoover Dam.* In 1933, when the Democrats replaced the Republicans, President Roosevelt changed this name again to *Boulder Dam.* It was known as *Boulder Dam* for 14 years. When again in 1946, Republicans came to power, the name was once again changed to *Hoover Dam,* which still exists today.

Dimensions of the Dam. The original dimensions of the dam are given below in F.P.S. units:

Height	= 726 ft
Span	= 1300 ft from rim to rim of the Canyon
Thickness at the base	= 660 ft
Thickness at the top	= 45 ft
Name of the reservoir formed at the back of the dam	= Lake Mead
Length of the reservoir	= 115 miles.

Something about the river. This big dam straddles the Colorado river. The Colorado river rises in the State of Colorado, runs down through Utah and into Arizona, then California, emptying finally into the Gulf of California in Mexico.

The Colorado is a river with muscles. It is a very strong river - containing huge amounts of silt and mud in it. It cuts numerous deep narrow canyons while it flows. The mile deep walls of the Grand canyon proves the power of this river.

It is a young river and all such rivers have strong powers for forming deep narrow gorges (*i.e.* canyons). They are the most turbulent, and the deepest digging. They are the best sources of hydroelectric power. Irrigation or navigation is not possible on this kind of rivers, unless, they are tamed.

Attempts were made to tame this river by constructing dikes or levees, but failed.

Spring floods of 1904 brought havoc. Another catastrophe occurred in 1916. Gila River, a tributary of Colorado, flooded at a rate of about 5, 560 cubic metres/sec. The town of Yuma in Arizona, where the Gila empties into the Colorado, was submerged to a depth of 1.2 metres.

It was extremely desirable to tame this river. But how? A solution was dreamt of. The idea of constructing a big dam was visualised by Arthur Davis. It was an ambitious plan.

Planning for the Dam. A scheme was planned. A concrete gravity dam, of about 700 to750 feet height was thought. A reservoir at the back of the dam could hold every drop of water, the Colorado could send in any 2 years of steady flow. It was to be the biggest man-made lake in the world. It was going to be a multi- purpose dam, generating about 6 billion Kilo watt-hours of electrical energy each year for the growing cities of southern California. The reservoir would hold the flood waters, and the spillways would release the required amounts of water to the downstream farmers.

Flood control, irrigation and electricity were the three main purposes of this project. Supply of drinking water to 13 cities of California and creation of a recreational and navigable lake reservoir were the additional advantages.

Selection of a suitable site. A thorough search was made for the spectacular gorges of the Colorado, seeking the best possible site. They studied 70 such sites before choosing 'Black canyon' on the border between Arizona and Nevada, 48 km from the city of Las Vegas (Nevada).

The preliminary survey of *Black canyon* took about 3 years. Here, the river flows through cliffs 1000-2000 feet (300 m to 600 m) high. At the water line, the rock walls were 350 feet (105 m) apart.

Engineers roamed in heat and sun, testing the rocks, drilling into it, to make sure that it could stand the burden of enormous weight of the concrete that would be laid upon it. *Their conclusion was that it could. They recommended that the giant dam be built.*

After the technical green-signal was obtained, some political issues such as to who will be benefitted and up to what extent, were settled with a great difficulty. The rift between the different States always persists in such huge projects.

Construction of the Dam. The real work began in late 1930. Herbert Hoover was the President of America at that time. He himself was an engineer. The work started under his vigilance, Arthur Powell Davis, 70 years old and about to retire, saw his life long dream fulfilled as he stood high above the Colorado and watched thousands of workers working hard with picks and dynamite far below.

The dam site was very hot. It was not a congenial surroundings to work. But a *dam had to be built.*

First of all, a town was built to house the workers. A permanent city, now called Boulder-city, was settled at an expenditure of about $70 million. This was spread in an area of about 300 acres and could accommodate 5, 000 workers.

Whenever a dam is constructed, the water of the river is first of all to be diverted so that the construction could start. This is called by engineers as 'to turn the river off.' 4 tunnels, each 56 feet wide and 4000 feet long were dug into the solid rock of the canyon walls. These *bypass tunnels* received the flow of the river and carried it down

to the downstream, to a point beyond the construction site. About 1 $\frac{1}{2}$ million cubic feet of rock had to be removed for building these four tunnels.

Then, the Coffer dam was built. A coffer dam is a temporary retaining embankment upstream of the site. Huge amounts of rock and earth was heaped up, forcing the river into four bypass tunnels. The bed of the river was thus laid bare.

Workers then descended into this river bed to lay the foundations for the dam. Seven million tons of concrete had to be laid. It was 660 feet thick at the base.

The dam curves upstream, so that the water load is held back in part by the walls of the canyon. The completion of the dam took about 2 years of non-stop work.

After completion, the bypass tunnels were blocked up and the water started coming and collecting against the dam. The lake formed on the upstream side was called *Lake Mead.* Electrical power houses, which are of the size of 20-storey sky-scrapers were constructed. The lake, the power houses, the dam galleries etc. are open for visitors and for inspection. It was a great achievement indeed.

17.8. Bhakra Dam

Bhakra Dam is a concrete gravity dam. It is 740 feet **(226 m)** high, spanning the V-shaped gorge in the lower Shivalik hills. The dam is 1700 ft long at the top and only 325 ft at the bottom. The thickness of the dam at foundations is 1320 ft and it tapers to 30 ft at the top where a road runs. *Bhakra dam was the highest concrete gravity dam of the world when built, thus surpassing the existing* 726 ft (221 m) high Hoover dam. But the highest concrete gravity dam of the world, at present, is **Grand Dixence dam** in Switzerland **(284 m high).** Bhakra dam is situated in Himachal Pradesh State of India near a village used to be called Bhakra. It has been constructed on Satluj river. Satluj is a river coming from Himalayas. It is a perennial river but carries enormous water during floods and rains.

Downstream and upstream views of Bhakra dam are shown in Photo Fig. 17.4.

This dam has given tremendous prosperity to India and has given her a high name in the world. The various functions served by this dam are:

(*i*) Flood control (*ii*) Irrigation

(*iii*) Electricity (*iv*) Fish development.

Bhakra Project is not a single Bhakra Dam but consists of the following:

(*i*) 740 ft high Bhakra Dam (*ii*) 95 ft high Nangal Dam

(*iii*) Nangal Hydel Channel (*iv*) Ganguwal and Kotta Power houses

(*v*) Bhakra Canal System.

Planning and Construction. The survey works for the construction of this multi-purpose project started in 1919. From 1919 to 1930, the survey continued and various sites were considered for various purposes. From 1932 to 1946, the work was interrupted, and finally in 1946, a railway line was spread in this area, and then the actual construction started.

After selecting a suitable narrow canyon for the construction of Bhakra Dam, the dam site was dewatered after the river was 'turned off'. Two diversion tunnels, one in either abutment were constructed in order to carry the river water. Two coffer dams enclosing the foundation area, were also constructed. Both the tunnels are 50 ft, in finished diameter, half a mile long and are lined with 3.6 ft thick heavily reinforced concrete. The work on these two tunnels was started in 1948 and was completed in 1953. The total expenditure incurred on them was approximately Rs. 3.6 crores.

Fig. 17.4 (*a*) Upstream view of Bhakra Dam.

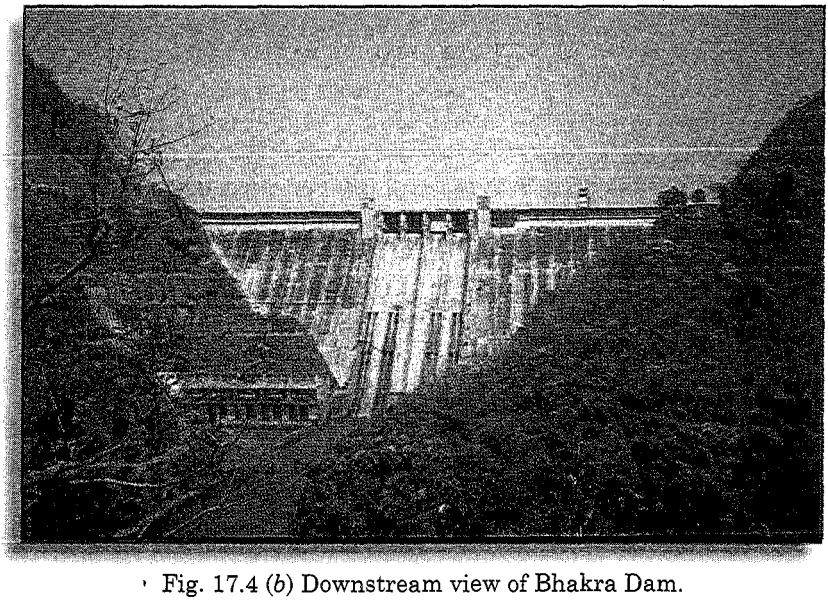


Fig. 17.4 (*b*) Downstream view of Bhakra Dam.

Fig. 17.4. Photoviews of Bhakra Dam.

After the construction of these tunnels, two earth (rolled) and rock coffer dams were constructed to enclose the operation area of the dam site. The construction of 215 ft high coffer dam began in 1956. The water was forced into the tunnels and taken downstream beyond the construction point. The 15 ft high downstream coffer dam did not let the water come back in the pit.

The foundations of these rolled fill earthen dams or dikes were carried down to 60ft and 70 ft below the river bed level. After the construction of coffer dams, the dewatering of the area was expeditiously completed and the excavation of the river bed started. The rock was blasted by an explosive. Desert-shovels were employed for loading the material in special trucks having capacity of 914 cubic yards. The material was carried and dumped at the dumping site. The total excavation was of the order of 700 million cubic yards. About one million cubic yard was excavated per day.

While the excavation was going on, a large constructional plant, capable of producing high quality concrete economically and efficiently was also installed. This required a $4\frac{1}{2}''$ long belt conveyor system, aggregate processing plant, cement handling plant, cooling plant, batching and mixing plants, high steel tristles, revolving cranes and other electricity operated cranes, etc. etc. It was an expensive, completely automised concrete plant. It was capable of handling about *600 tons* of concrete per hour.

Then, the actual concreting work of the dam started in November 1956. Huge electric cranes including 6 cantilever-cranes and 2 stiff-legged-cranes were employed for concreting of dam.

A very low heat cement was used in the construction of this dam. Because, when cement sets, it produces a large amount of heat which is liable to cause cracks in the structure. So in order to avoid this cracking, steel pipes were embedded in the structure and ice-cold water was circulated through them.

The first stage of the dam (390 ft.) was completed by 1959. Work in the 2nd stage was interrupted by a flood in the diversion tunnel that drowned ten workers and damaged the power house. The tunnel and plugging of the dam was completed in 1962.

Dimensions and other data about Bhakra Dam are given below:

Height of the dam	= 740′
Length at the bottom	= 325′
Length at the top	= 1200′
Breadth or thickness at the bottom	= 1320′
Breadth or thickness at the top	= 30′
Concrete required for the dam	= 55 lakh cubic yard
Electricity generated	= 12 lakh Kilowatt
Name of the reservoir formed	= Govind Sagar
Length of the reservoir	= 90 miles
Maximum depth	= 740′
Minimum depth	= 300′
Total irrigation	= 1 crore acre of area
Population of these villages	= 30, 000
Total storage capacity	= 8 million acre ft.
Catchment area	= 22, 000 sq. miles
Live storage	= 6.35 million acre ft.
No. of Inspection galleries	= 46

Power Houses. There are two power houses called (*i*) The Left Power House; and (ii) The Right Power House. The left power house is more important and was first constructed.

The left power house is a reinforced concrete structure. It is standing on a stone consisting partly of clay stone and partly of sand stone. The foundation chosen were adverse to the geological conditions required for a good dam site. The permeability of sand stone is high, and thus there was a possibility of leakage and danger of undermining of the foundations. It necessitated a heavily reinforced raft foundation. The left power house building is seven storeyed and its construction took about two to three years. It required about 635, 000 cubic yards of concrete, about one million sq.ft. of finishing work and 6, 000 tons of reinforced steel.

The machinery used in this power house is of cosmopolitan in nature. The turbines were supplied by Japan, the generators and transformers by U.K., and over-head cranes by Yogoslavia.

The local manufacture of 10 ft diameter penstocks, made out of 1.75″ thick steel plate was considered as a big engineering feat of the country in such a short period of independence. The left power house had costed nearly Rs. 10 crores. The work on right power house was also completed afterwards..

Problem of Wood Transport. Like all other perennial rivers of India, the River Satluj has been the cheapest means of transporting wood from the Himalayan forests down to the plains. But since the dam is very high above the river water level, it has become impossible to use this river for this purpose.

The problem is overcome by bringing logs of woods from Govind Sagar to Nangal railway station by means of an aerial rope-way. This is about $5\frac{1}{2}$ miles long. A wooden log loom has been put across Govind Sagar to obstruct timber. From there, the timber is taken by the inclined carriage way to the loading station on the upstream of the right side.

17.8.1. Nangal Dam. Nangal Dam is 95 ft high subsidiary dam, 8 miles downstream of Bhakra on Satluj. It falls within the jurisdiction of Punjab State in India. The length of the dam is 1, 000 ft. The object of this dam is to head up water of the river Satluj and then divert it into the canal off-taking from the left bank of the river. The canal is called Nangal Hydel Canal, and is a 40 miles long concrete lined canal.

This dam has 29 strong gates of span 30 ft each. An enormous tunnel called the Inspection gallery has been made in the river Satluj in the lower portion of this dam. This was the first tunnel constructed by the Government under Bhakra Project Scheme. In order to enter into this tunnel, one has to go 70 ft down. The tunnel goes across Satluj River.

Bhakra Nangal Project is something tremendous, stupendous, something which shakes up and thrills us when we see it. It marks the India's progress after her Independence. It is something which cannot be forgotten easily, if we see it once.

17.9. Nagarjuna Sagar Dam

The multipurpose Nagarjuna Sagar dam is located across Krishna River, near Nandikonda village in Nalgonda District (Andhra Pradesh). It is named after Buddhist Savant, Acharya Nagarjuna, who lived at the spot about 2,000 years ago to fulfil a mission.

This Dam irrigates in Guntur, Kurnool, Nellore, Nalgonda, Khamman and Krishna Districts. Its irrigation Potential is about 35 lakh acres of land and electrical potential is 1 lakh kilowatt hours of firm power (guaranteed-power generation) and 4.6 lakh kilowatt hours of seasonal power.

A photoview of this dam is shown in Fig. 17.5.

Salient Features of the Dam. The river gorge is blocked by a masonry dam 409 ft (124.6 m) high above the deepest foundation level and 4756 ft long. The full reservoir level is +590.0 ft above the mean sea level (M.S.L.). The most unique feature of this dam is the adoption of Stone masonry for its construction, deviating from the traditional concrete. The use of stone had resulted in a large saving and had created huge employment potential for a large labour force. Hence, Nagarjuna Sagar Project ranks first in the man-power utilisation among the modern gigantic projects of its own kind in the world. It was designed and executed entirely by Indian engineers. On either side of the masonry dam, earth dams have been constructed for a length of about two miles, the maximum height being 85 ft. The spillway crest has been installed with 26 Radial gates each of size 45′×44′. Other component-works of this mighty dam include 8 penstock pipes on the left side, three Power sluices and 9 irrigation sluices on the right side, two chute sluices and a diversion-cum-irrigation tunnel. Two canals-off-take on either side of the dam for irrigation. The expenditure on the project was of the order of Rs. 80 crores. The crest level of the dam is +605 ft and the crest level of spillway is +546 ft. Various details of the dam are given below:

(1) **Location.** Lat. = 16°34′ North, Long. = 71°19′ East, $1\frac{1}{2}$ mile downstream of Nandikonda village Miryalaguda Taluk, Nalgonda District, 90 miles from Hyderabad.

(2) **States Covered.** ANDHRA PRADESH

(3) **Hydrology**

(*a*) Water-shed area at dam site	... 83,087 sq. miles
(*b*) Maximuml flood discharge (observed)	... 11.7 lakh cumecs.

(4) **Reservoir**

(*a*) Full Reservoir level	... + 590′.0
(*b*) Maximum water level	... + 594′.0
(*c*) Gross storage capacity	... 9.37 M.a.ft
(*d*) Net (live) storage capacity	... 5.51 M.a.ft
(*e*) No. of villages submerged	... 57
(*f*) Population displaced	... 4,824 families

(5) **Masonry dam**

(*a*) Total length	... 4,756 ft
(*b*) Spillway length	... 1545 ft
(*c*) Non-overflow length including power dam	... 3,211 ft
(*d*) Height of dam (maximum)	... 409 ft
(*e*) Base width (maximum)	... 320 ft
(*f*) Top width	... 28 ft

(*g*)	Top level	... + 605′.0
(*h*)	Top of crest in spillway	... + 546′.0
(*i*)	Chute sluices 2 No. 10′ × 25′ provided in blocks 25 and 51 with sill level elevation	... + 450′.0
(*j*)	8 No. of 16′ diameter penstocks provided one in each in blocks 16 to 23 with central line in elevation	... + 405′.0
(*k*)	3 No. power sluices 15′ × 38′ provided (two in block 71 and one in block 72) with sill level at elevation	... + 479′.0
(*l*)	Radial Crest Gates 26 No. 45′ × 44′, with crest elevation	... + 546′.0
(*m*)	Thickness of spillway pipes	... + 15′
(*n*)	Nine No. sluices 10′ × 15′ are provided for right canal head regulator with sill at elevation	... + 489′.0

(6) **Quantities of Work**

(*a*)	Excavation for foundations	... + 42.07 M. cft
(*b*)	Volume of masonry and concrete	... 198 M. cft
(*c*)	Quantity of cement	... About 11 lakh tons
(*d*)	Quantity of steel	... About 60,000 tons

(7) **Earth Dams**

(*a*)	Length of Left Earth Dam	... 8,400 ft
(*b*)	Length of Right Earth Dam	... 2,800 ft
		11,200 ft
(*c*)	Maximum height above foundation level	... 85 ft
(*d*)	Top width	... 30 ft
(*e*)	Top level	... 610′
(*f*)	Excavation of foundation	... 9.2 M. cft
(*g*)	Earthwork for embankment	... 88 M. cft

(8) **Irrigation-cum-Diversion Tunnel (horse-shoe in Section)**

(*a*)	Length	... 2,590 ft
(*b*)	Diameter	... 27 ft

(9) **Power Plant**

(*a*)	Left side 8 No. of penstocks 16 ft. diameter to develop	... 3 lakh kW
(*b*)	Right side 3 power sluices of size 15′ × 38′ to develop	... 0.6 lakh kW

(10) **Man-Power** ... 50,000

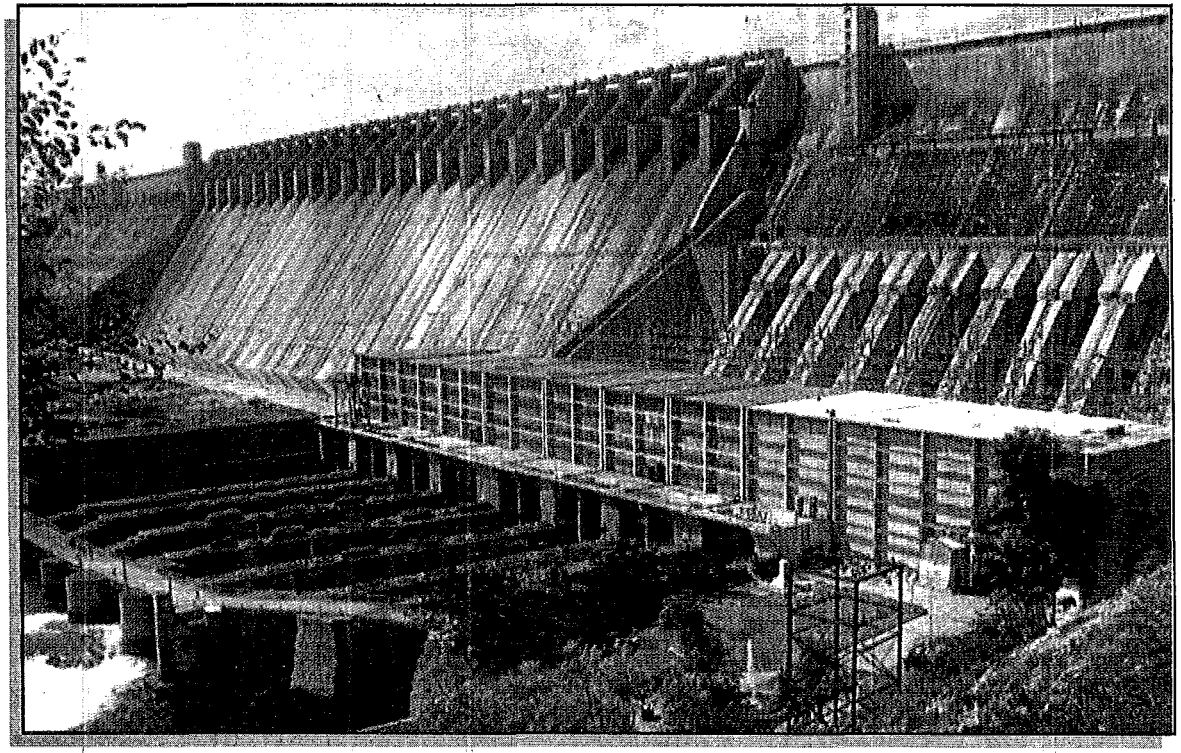


Fig. 17.5. Photoview of Nagarjuna Sagar Dam.

Organisation. In the earlier years of construction, a Control Board was the orverall incharge of the Nagarjun Sagar Project including technical, financial and administration aspects. The project was then located in the Andhra Pradesh and the Hyderabad States. After the reorganisation of States, the project came entirely in Andhra Pradesh State. The State Government assumed its full responsibility w.e.f. 1.8.1959. The Control Board became an advisory Body to the State Government, whose advice is accepted as a matter of convention.

The programme of construction of dam and canals was so adjusted that partial benefits started accruing even from 1967. The reservoir was able to deliver water to first crop of Krishna delta in time for raising substantial acreage in 2nd crop. Letting out waters in the two canals was inaugurated, by Smt. Indira Gandhi, the then Prime Minister of India, on 4.8.1967.

Rehabilitation. The man-made lake of Nagarjuna Sagar fully submerged lands of 52 villages and partially submerged that of 5 villages.

About 4,900 families were displaced and rehabilitated in 24 Rehabilitation centres. Good facilities, liberal compensation and other amenities were provided to those who had to be uprouted.

Nagarjunakonda Excavations. As the Nagarjuna Sagar reservoir was to completely submerge the famous relics of Nagarjuna Konda, which was the seat of Ikshwaku Kings and one of the principal centres of Mahayana System of Buddhism, the whole area was excavated by the Archeological Department of the Governmernt of India. The more important of the relics were located in a museum constructed for the purpose, on the top of an adjoining hill. Practically, all the relics had been unearthed and shifted to the museum. Expenditure to the extent of 12 lakh rupees was debited to the Project and the balance was met by the Archeological department.

PROBLEMS

1. (*a*) What is meant by a "dam and a reservoir" ? What are the different meterials that are commonly used for dam construction and what are their comparative advantages and disadvantages ?

(*b*) Discuss the geological and tropological features which affect the selection of the type of dam.

2. (*a*) What are 'arch' and 'buttress' dams ? Illustrate with sketches and mention site conditions favourable for construction of such dams.

(*b*) Discuss the factors which are considered in the selection of the site for a proposed dam. It is assumed that the type of the dam has already been selected for the project.

3. (*a*) What useful purpose is served by a dam ? What are the illeffects of dam construction ?

(*b*) How do you classify dams according to :

(*i*) their use ;

(*ii*) their hydraulic designs ;

(*iii*) their materials of construction.

(*c*) Discuss the various factors which govern the selection of a particular type of dam for a particular project.

4. "Dams are the sources of sorrow and grief". Debate the above statement giving points in favour as well as against it.

5. Narrate briefly the story of construction of any major dam of India.

18

Reservoirs and Planning for Dam Reservoirs

18.1. Definition and Types

When a barrier is constructed across some river in the form of a dam, water gets stored on the upstream side of the barrier, forming a pool of water, generally called a *dam reservoir* or an *impounding reservoir* or *a river reservoir.*

The quality of water stored in such a reservoir is not much different from that of a natural lake. The water so stored in a given reservoir during rainy season can be easily used almost throughout the year, till the time of arrival of the next rainy season, to refill the emptying reservoir again.

Depending upon the purpose served by a given reservoir, the reservoirs may be broadly divided into the following three types.

(1) *Storage or Conservation reservoirs* ;

(2) *Flood Control reservoirs* ; and

(3) *Multipurpose reservoirs.*

The fourth type of a reservoir is a simple storage tank constructed within a city water supply system, and is called a *Distribution reservoir* ; and such a reservoir is evidently not a river reservoir, but is a simple storage tank.

18.1.1. Storage or Conservation Reservoirs. A city water supply, irrigation water supply, or a hydroelectric project drawing water directly from a river or a stream may fail to satisfy the consumers demands during extremely low flows; while during high flows, it may become difficult to carry out their operations due to devastating floods. *A storage or a conservation reservoir can retain such excess supplies during periods of peak flows, and can release them gradually during low flows as and when the need arises.*

Incidentally, in addition to conserving water for later use, the storage of flood waters may also reduce flood damage below the reservoir. Hence, a reservoir can be used for controlling floods either solely or in addition to other purposes. In the former case, it is known as a 'Flood Control Reservoir' or a 'Single Purpose Flood Control Reservoir'; and in the latter case, it is called a 'Multipurpose Reservoir'.

18.1.2. Flood Control Reservoirs. A flood control reservoir, generally called a flood-mitigation reservoir, stores a portion of the flood flows in such a way as to minimise the flood peaks at the areas to be protected downstream. To accomplish this, the entire inflow entering the reservoir is discharged till the outflow reaches the safe capacity of the channel downstream. The inflow in excess of this rate is stored in the

reservoir, which is then gradually released, so as to recover the storage capacity for the next flood.

The flood peaks at the downstream of the reservoir are thus reduced by an amount *AB*, as shown in Fig. 18.1. A flood control reservoir differs from a conservation reservoir only in its need for a large sluiceway capacity to permit rapid drawdown before or after a flood.

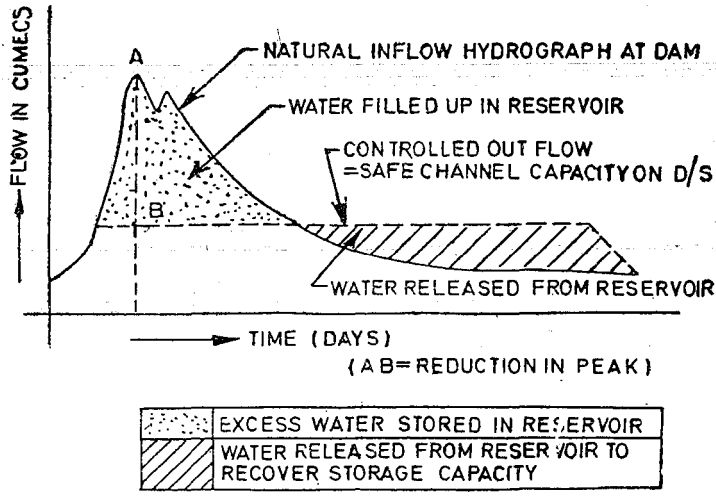


Fig. 18.1.

Types of flood control reservoirs. There are two basic types of flood-mitigation reservoirs; *i.e.*

(*i*) *Storage reservoirs or Detention basins* ; and

(*ii*) *Retarding basins or Retarding reservoirs.*

A reservoir having gates and valves installation at its spillway and at its sluice outlets is known as a storage reservoir; while on the other hand, a reservoir with uncontrolled and ungated outlets is known as a **retarding basin.**

Functioning and Advantages of a Retarding Basin

A retarding basin is usually provided with an uncontrolled spillway and an uncontrolled orifice type sluiceways. The automatic regulation of outflow, depending upon the availability of water, takes place from such a reservoir. The maximum discharging capacity of such a reservoir should be equal to the maximum safe carrying capacity of the channel downstream. As floods occur, the reservoir gets filled, and discharges through sluiceways. As the reservoir elevation increases, the outflow discharge increases. The water level goes on rising until the flood has subsided, and the inflow becomes equal to or less than the outflow. After this, the water gets automatically withdrawn from the reservoir until the stored water is completely discharged. The advantages of a retarding basin over a gate controlled detention basin are:

(*i*) Cost of the gate installation is saved.

(*ii*) There are no gates and hence, the possibility of human error and negligence in their operation is eliminated.

(*iii*) Since such a reservoir is not always filled, much of the land below the maximum reservoir level will be submerged only temporarily and occasionally, and can be successfully used for agriculture, although no permanent habitation can be allowed on this land.

Functioning and Advantages of a Storage Reservoir

A storage reservoir with gated spillway and gated sluiceways, provides more flexibility of operation, and thus gives us better control and increased usefulness of the reservoir. Storage reservoirs are, therefore, preferred on large rivers, which require better control; while retarding basins are preferred on small rivers. In storage reservoirs, the flood crest downstream, can be better controlled and regulated properly, so as not to cause their coincidence. This is the biggest advantage of such a reservoir and outweighs its disadvantages of being costly and involving risk of human error in installation and operation of gates.

18.1.3. Multipurpose Reservoirs. A reservoir planned and constructed to serve not only one purpose but various purposes together is called a multipurpose reservoir. Reservoir, designed for one purpose, incidentally serving other purposes, shall not be called a multipurpose reservoir, but will be called so, only if designed to serve those purposes also in addition to its main purpose. Hence, a reservoir designed to protect the downstream areas from floods and also to conserve water for water supply, irrigation, industrial needs, hydroelectric purposes, etc. shall be called a multipurpose reservoir. Bhakra dam and Nagarjun Sagar dam are the important multipupose projects of India.

18.1.4. Distribution Reservoirs. A distribution reservoir is a small storage reservoir constructed within a city water supply system. Such a reservoir can be filled by pumping water at a certain rate and can be used to supply water even at rates higher than the inflow rate during periods of maximum demands (called critical periods of demand). Such reservoirs are, therefore, helpful in permitting the pumps or the water treatment plants to work at a uniform rate, and they store water during the hours of no demand or less demand, and supply water from their 'storage' during the critical periods of maximum demand.

In this chapter, we shall however, confine ourselves to the river reservoirs only.

18.2. Capacity-Elevation and Area-Elevation Curves of a Reservoir Site

Whatever be the size or use of a reservoir, the main function of a reservoir is to store water and thus to stabilize the flow of water. Therefore, the most important physical characteristic of a reservoir is nothing but its *storage capacity*. The capacity of reservoirs on dam sites, is determined from the contour maps of the area. A topographic survey of the dam site is carried out, and a contour map such as shown in Fig. 18.2 is drawn. The area enclosed within each contour can be measured with a planimeter.

In fact, the general practice adopted for capacity computations is to actually survey the site contours only at vertical distances of 5 m or so. The areas of the intervening contours at smaller intervals of say 0.5 m or so, are then interpolated by taking the square root of the surveyed contours, and to assume that the square root of the interpolated ones, vary in exact proportion to their vertical distance apart. For example, suppose the area of the reservoir at 200 m contour is A_1 hectares, and that at 205 m contour is A_2 hectares; then

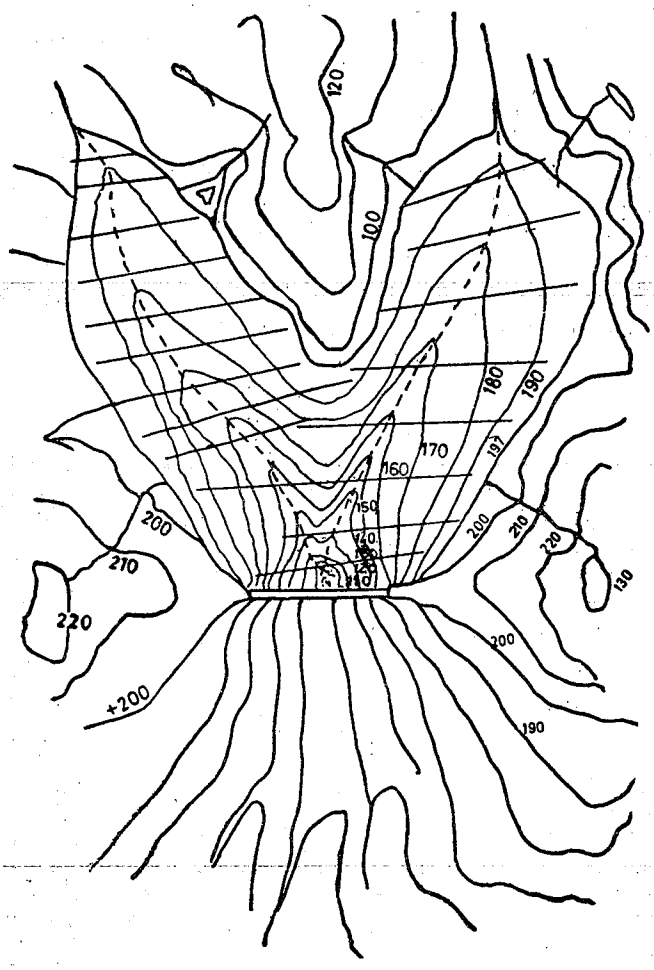


Fig. 18.2. A typical contour map of a dam site.

The area at 200.5 m contour is

$$= \left[\sqrt{A_1} + \frac{200.5 - 200}{205 - 200}(\sqrt{A_2} - \sqrt{A_1})\right]^2$$

$$= \left[\sqrt{A_1} + \frac{0.5}{5.0}(\sqrt{A_2} - \sqrt{A_1})\right]^2 \qquad \text{...[18.1 (a)]}$$

Similarly, the area at 201.0 m contour

$$= \left[\sqrt{A_1} + \frac{201.0 - 200}{205 - 200}(\sqrt{A_2} - \sqrt{A_1})\right]^2$$

$$= \left[\sqrt{A_1} + \frac{1.0}{5.0}(\sqrt{A_2} - \sqrt{A_1})\right]^2 \qquad \text{...[18.1 (b)]}$$

Similarly, the area at 204.5 m contour

$$= \left[\sqrt{A_1} + \frac{204.5 - 200}{205 - 200}(\sqrt{A_2} - \sqrt{A_1})\right]^2$$

$$= \left[\sqrt{A_1} + \frac{4.5}{5.0}(\sqrt{A_2} - \sqrt{A_1})\right]^2 \quad \text{...[18.1 (c)]}$$

In this way, the areas can be computed at sufficiently low contour intervals (0.5 m), which can be used to determine the incremental volumes (ΔS) stored between two successive contours, by using the simple average method, *i.e.* by multiplying the average of the two areas at the two elevations, by the elevation difference (Δh). The summation of these incremental volumes below any elevation, is the storage volume below that level.

Instead of using the simple average formula, *i.e.*

$$\Delta S = \frac{a_1 + a_2}{2}(\Delta h), \text{ sometimes the formula}$$

$$\Delta S = \frac{\Delta h}{3}\left[a_1 + a_2 + \sqrt{a_1 a_2}\right] \text{ can also be used, where}$$

$a_1, a_2, \ldots$ represent the areas at 0.5 m contour interval.

Prismoidal formula can also be used preferably where three consecutive sections at equal height are taken. According to this,

$$\Delta S = \text{Storage} = \frac{\Delta h}{6}\left[A_1 + 4A_2 + A_3\right] \quad \text{...(18.2)}$$

where A_1, A_2 and A_3 are the areas of succeeding contours, and Δh is the vertical distance between two alternate contours.

It thus becomes evident that the areas at different elevations (contours), as well as the storage at different elevations, can be mathematically worked out, and both plotted on a graph paper, to obtain **Area-Elevation curve, and Storage-Elevation curve** (*Capacity-Elevation curve*), respectively, as shown in Fig. 18.3.

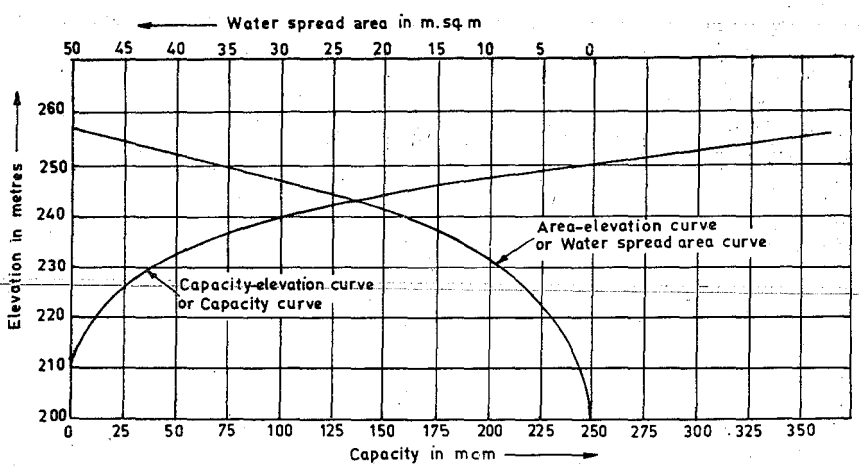


Fig. 18.3. Area elevation curve and Capacity elevation curve.

Another method for computing capacity may be the use of integration technique. Infact, the area-elevation curve, when integrated, will yield nothing but the capacity-elevation curve. Hence the surveyed areas at large-intervals, may be plotted on a simple graph paper, and a smooth curve, *i.e.* area-elevation curve, is first of all drawn. The equation of this curve is now obtained by statistical methods, which can be integrated to obtain the equation of the capacity-elevation curve, as follows:

The equation of the area-elevation curve, will generally be of the form:

$$A = \alpha + \beta \,.\, h + \gamma \,.\, h^2 + \ldots \eta \,.\, h^{n-1} \qquad \ldots(18.3)$$

where A represents the area at any elevation h; and $\alpha, \beta, \gamma \ldots \eta$ are all constants.

This equation can be determined and then integrated to obtain the storage (capacity), as explained below:

Let y be the height of the water surface in the reservoir above any assumed datum, over which the storage/capacity is to be worked out.

Let A_y represents symbolically the area of the contour at this height. Then, assume that the *equation of area-elevation curve* is given by :

$$A_y = \alpha + \beta \,.\, y + \gamma \,.\, y^2 + \ldots \eta \,.\, y^{n-1} \qquad \ldots(18.3\ a)$$

where $\alpha, \beta, \gamma, \ldots, \eta$ are all constants.

From the actual survey, or from the points falling on the area-elevation curve, the area of any required number of contours (n) are known.

Let them be A_0, A_1, A_2.....corresponding to the known level values, *i.e.*, heights $0, y_1, y_2, \ldots$ above the datum.

Substituting these values in equation (18.3 *a*), we get

$$A_0 = \alpha$$
$$A_1 = \alpha + \beta \,.\, y_1 + \gamma \,.\, y_1^2 + \ldots \eta \,.\, y_1^{n-1}$$
$$A_2 = \alpha + \beta \,.\, y_2 + \gamma \,.\, y_2^2 + \ldots \eta \,.\, y_2^{n-1}$$
$$A_3 = \alpha + \beta \,.\, y_3 + \gamma \,.\, y_3^2 + \ldots \eta \,.\, y_3^{n-1}$$

Thus, we get n simultaneous equations to determine n number of constants $(\alpha, \beta, \gamma, \ldots, \eta)$ Hence, the equation of the area-elevation curve, *i.e.* $A_y = \alpha + \beta.y + \gamma.y^2 + \ldots \eta.y^{n-1}$ becomes defined, with $\alpha, \beta, \gamma, \ldots \eta$ all known.

This equation can now be integrated between the limits 0 to y,

$$\int_{y=0}^{y=y} A_y \cdot dy = \int_{y=0}^{y=y} (\alpha + \beta.y + \gamma.y^2 + \ldots \eta.y^{n-1})\, dy$$

or S_y = storage (capacity) between 0 to y

$$= \int_{y=0}^{y=y} (\alpha + \beta.y + \gamma.y^2 + \ldots \eta.y^{n-1})\, dy$$

$$= \left[\alpha.y + \beta.\frac{y^2}{2} + \gamma.\frac{y^3}{3} + \ldots \eta \cdot \frac{y^n}{n} \right] + K$$

where K is a constant, which obviously is the reservoir capacity at datum.

The use of this method can be best understood by solving a numerical example.

Example 18.1. *A contour survey of a reservoir site gives the following data:*

Contour value	*Area*
At 200 m contour	*6.0 hectares*
At 210 m contour	*18.1 hectares*
At 220 m contour	*34.0 hectares*

The capacity of the reservoir upto 200 m elevation is found to be 14.1 ha. m. Determine the general equation for the area-elevation curve and capacity-elevation curve. Also determine the reservoir capacity at RL 225 m.

Solution. Use equation (18.3 *a*), wherein y is the height above *RL* 200 m, as

$$A_y = \alpha + \beta . y + \gamma . y^2$$

Now substituting the given values, we have

$$A_0 = 6.0 \text{ hectares}$$

$\therefore$ $6.0 = \alpha + \beta (0) + \gamma (0) = \alpha$

or $\alpha = \mathbf{6.0}$ ...(*i*)

Also $A_1 = 18.1$ hectares, at $y_1 = 10$ m (given)

$\therefore$ $18.1 = \alpha + \beta . (10) + \gamma . (10)^2$

or $18.1 = 6.0 + 10\beta + 100\gamma$

or $12.1 = 10\beta + 100\gamma$

or $\beta + 10\gamma = 1.21$ ...(*ii*)

Also $A_2 = 34.0$ hectares at $y_2 = 20$ m (given)

$\therefore$ $34.0 = \alpha + \beta (20) + \gamma (20)^2 = 6.0 + 20\beta + 400\gamma$

or $20\beta + 400\gamma = 28.0$

$\beta + 20\gamma = 1.4$ ...(*iii*)

Solving (*ii*) and (*iii*), we get

$\beta + 10\gamma = 1.21$

$\beta + 20\gamma = 1.40$

or $10\gamma = 0.19$ or $\gamma = \mathbf{0.019}$

$\therefore$ $\beta + 0.19 = 1.21$ or $\beta = \mathbf{1.02}$

Hence, the equation of area-elevation curve is given by

$$A_y = 6.0 + 1.02y + 0.019y^2 \quad \textbf{Ans.}$$

Integrating this equation, we get

$$S_y = 6.0y + 1.02\frac{y^2}{2} + 0.019\frac{y^3}{3} + K$$

The constant K is obviously the reservoir capacity (storage) upto *RL* 200.0 m, which is given to be 14.1 ha. m.

$\therefore$ $S_y = 6.0y + 0.51y^2 + 0.0063y^3 + 14.1$

Hence, the capacity-elevation curve is given by

$$S_y = 0.0063y^3 + 0.51y^2 + 6.0y + 14.1 \quad \textbf{Ans}$$

To determine the capacity at *RL* 225 m, we have to substitute $y = 225 - 200 = 25$ m in the above eqn. to obtain the requisite capacity in ha.m., *i.e.*

Required capacity at *RL* 225 m

$$= 0.0063\,(25)^3 + 0.51\,(25)^2 + 6.0 \times 25 + 14.1$$
$$= 98.96 + 318.75 + 150.0 + 14.1 = \mathbf{581.81\,ha.m.} \quad \textbf{Ans.}$$

Conclusions. It can thus be seen that the capacity of a reservoir by this method can be determined by surveying only a few contours. It is also found that the method does not give more than 3% error, when it is cross-checked with the capacity worked out by surveying large number of contours. This error is not considered much, in the light of the fact that the areas of contours are themselves not very precise figures.

Sometimes, storage capacity may be expressed as a single term function of y, such as

$$S_y = K.y^n \qquad \text{where } K \text{ and } n \text{ are constants.}$$

Infact, in practical life, no one bothers about the equations, and only curves, as shown in Fig. 18.3 are drawn. The required capacity at any elevation is read out, from such a curve.

It may also be pointed out here that the best capacity curve for a reservoir is the one in which the rise to the straight line is the quickest. It results from a cup-shaped catchment, with gentle longitudinal slope. Fig. 18.4 shows the good and bad capacity curves.

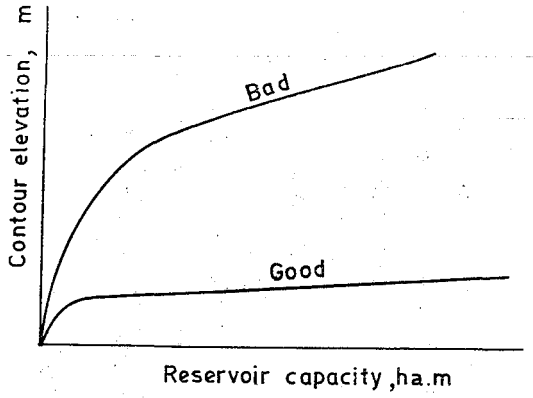


Fig. 18.4. Good and Bad capacity curves for a reservoir

18.3. Storage Zones of a Reservoir

These zones are defined w.r. to Fig. 18.5.

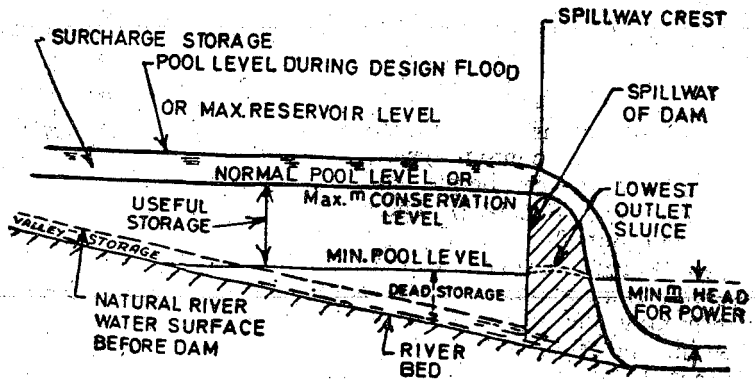


Fig. 18.5. Storage zones of a reservoir.

18.3.1. Normal Pool Level or Maximum Conservation Level. It is the maximum elevation to which the reservoir water surface will rise during normal operating conditions. (See Fig. 18.5). It is equivalent to the elevation of the spillway crest or the top of the spillway gates, for most of the cases.

18.3.2. Minimum Pool Level. The lowest water surface elevation, which has to be kept under normal operating conditions in a reservoir, is called the minimum pool level (See Fig. 18.5). This level may be fixed by the elevation of the lowest outlet in the dam or may be guided by the minimum head required for efficient functioning of turbines.

18.3.3. Useful and Dead Storage. *The volume of water stored in a reservoir between the minimum pool and normal pool levels is called the useful storage.* Water stored in the reservoir below the minimum pool level is known as the *Dead Storage,* and it is not of much use in the operation of the reservoirs. The useful storage may be subdivided into conservation storage and flood-mitigation storage, in a multipurpose reservoir.

18.3.4. Maximum Pool Level or Full Reservoir Level. During high floods, water is discharged over the spillway, but will cause the water level to rise in the reservoir above the normal pool level. The maximum level to which the water rises during the worst design flood is known as the *maximum pool level.*

18.3.5. Surcharge Storage. The volume of water stored between the normal pool level and the maximum pool level is called *surcharge storage.* Surcharge storage is an uncontrolled storage, in the sense that it exists only till the flood is in progress and cannot be retained for later use.

18.3.6. Bank Storage. When the reservoir is filled up, certain amount of water seeps into the permeable reservoir banks. This water comes out as soon as the reservoir gets depleted. This volume of water is known as the *bank storage,* and may amount to several percent of the reservoir volume depending upon the geological formations. The bank storage effectively increases the capacity of the reservoir above that indicated by the elevation capacity curve of the reservoir.

18.3.7. Valley Storage. Even before a dam is constructed, certain variable amount of water is stored in the stream channel, called *valley storage.* After the reservoir is formed, the storage increases, and the actual net increase in the storage is equal to the storage capacity of the reservoir minus the natural valley storage. The valley storage thus reduces the effective storage capacity of a reservoir. It is not of much importance in conservation reservoirs, but the available storage for flood mitigation is reduced, as given by the following relation:

Effective storage for flood mitigation

= Useful Storage + Surcharge Storage − Valley Storage corresponding to the rate of inflow in the reservoir.

DESIGNING RESERVOIR CAPACITY

18.4. Catchment Yield and Reservoir Yield

Long range runoff from a catchment is known as the yield of the catchment. Generally, a period of one year is considered for determining the yield value. *The total yearly runoff, expressed as the volume of water entering/passing the outlet point of the catchment, is thus known as the catchment yield, and is expressed in M m^3 or M.ha.m.*

The annual yield of the catchment upto the site of a reservoir, located at the given point along a river, will thus indicate the quantum of water that will annually enter the reservoir, and will thus help in designing the capacity of the reservoir. This will also help to fix the outflows from the reservoir, since the outflows are dependent upon the inflows and the reservoir losses.

The amount of water that can be drawn from a reservoir, in any specified time interval, called the *reservoir yield,* naturally depends upon the inflow into the reservoir and the reservoir losses, consisting of reservoir leakage and reservoir evaporation.

The annual inflow to the reservoir, i.e. the *catchment yield,* is represented by the *mass curve of inflow;* whereas, the outflow from the reservoir, called the *reservoir yield,* is represented by the *mass demand line* or *the mass curve of outflow.* Both these curves decide the reservoir capacity, provided the reservoir losses are ignored or separately accounted.

The inflows to the reservoir are however, quite susceptible of variation in different years, and may therefore vary throughout the prospective life of the reservoir. *The past available data of rainfall or runoff in the catchment is therefore used to work out the optimum value of the catchment yield.* Say for example, in the past available records of say 35 years, the minimum yield from the catchment in the worst rainfall year may be as low as say 100 M.ha.m; whereas, the maximum yield in the best rainfall year may be as high as say 200 M.ha.m. The question which then arises would be as to whether the reservoir capacity should correspond to 100 M. ha.m yield or 200 M. ha.m yield. If the reservoir capacity is provided corresponding to 100 M. ha.m yield, then eventually the reservoir will be filled up every year with a dependability of 100%; but if the capacity is provided corresponding to 200 M.ha.m yield, then eventually the reservoir will be filled up only in the best rainfall year (*i.e.* once in 35 years) with a dependability of about $\frac{1}{35} \times 100 \approx 3\%$.

In order to obtain a sweet agreement, a via media is generally adopted and an intermediate *dependability percentage value* (p), such as 50% to 75%,* may be used to compute the **dependable yield** or **the design yield.** The yield which corresponds to the worst or the most critical year on record is however, called the **firm yield** or **the safe yield.** Water available in excess of the firm yield during years of higher inflows, is designated as **the secondary yield.** Hydropower may be developed from such secondary water, and sold to the industries *'on and when available basis'.* The power commitments to domestic consumers must, however, be based on the firm basis, and should not exceed the power which can be produced with the firm yield, unless thermal power is also available to support the hydroelectric power.

The arithmetic average of the firm yield and the secondary yield is called the **average yield.**

18.4.1. Computing the Design or the Dependable Catchment Yield. The dependable yield, corresponding to a given dependability percentage p, is determined from the past available data of the last 35 years or so. The yearly rainfall data of the reservoir catchment is generally used for this purpose, since such long runoff data is rarely available . The rainfall data of the past years is therefore used to work out the dependable rainfall value corresponding to the given dependability percentage p. This dependable rainfall value is then converted into the dependable runoff value by using the available empirical formulas connecting the yearly rainfall with the yearly runoff.

It is, however, an adopted practice in Irrigation Departments to plan the reservoir project by computing the dependable yield from the rainfall data, but to start river gauging as soon as the site for the reservoir is decided, and then correlate the rainfall-runoff observations to verify the correctness of the assumed empirical relation between the rainfall and the runoff. Sometimes, on the basis of such observations, the initially assumed yield value may have to be revised.

* This percentage will depend upon the risk which can be absorbed for the proposed use of water. Say for example, city water supply projects can absorb lesser risk as compared to the irrigation projects, and hence should consider higher dependability percentage value.

The procedure which is adopted to compute the dependable rainfall value for a given dependability percentage p is explained below, and has been further used in solving example 18.2.

(*i*) The available rainfall data of the past N years is first of all arranged in the descending order of magnitude.

(*ii*) The order number m, given by the equation

$$m = N \cdot \frac{p}{100} \qquad ...(18.4)$$

is then computed, and rainfall value corresponding to this order number in the tabulated data will represent the required dependable rainfall value.

(*iii*) If the computed value of m is a fraction, then the arithmetic mean of the rainfall values corresponding to whole number m values above and below this fraction value is taken as the **dependable rainfall** value.

This method of computing the rainfall value of the given dependability (such as 50%, 75%, etc.) will become more clear when we solve example 18.2. The rainfall value of given dependability can finally be converted into the dependable yield, by using empirical methods, discussed in article 18.4.1.1.

The dependable yield of a given dependability (p per cent) can also be determined from the stream gauging, by computing the annual yields, from the observed discharges, and arranging the annual yield values in descending order to determine order numbers (instead of rainfall) as exhibited in solved example 18.3.

Example 18.2. *The yearly rainfall data for the catchment of a proposed reservoir site for 35 years is given in Table 18.1. Compute from this data, the values of dependable rainfalls for 60% and 75% dependability percentage.*

Table 18.1

Year	*Rainfall in cm*	*Year*	*Rainfall in cm*
1956	98	1974	88
1957	100	1975	94
1958	101	1976	107
1959	99	1977	110
1960	85	1978	208
1961	112	1979	114
1962	116	1980	104
1963	78	1981	120
1964	160	1982	108
1965	66	1983	102
1966	184	1984	80
1967	90	1985	109
1968	76	1986	122
1969	118	1987	115
1970	86	1988	140
1971	92	1989	138
1972	96	1990	60
1973	93		

Solution. The given data of table 18.1 is arranged in descending order, mentioning serial number (order number m) in front of each, as shown in Table 18.2.

Table 18.2

S.No. i.e order number (m)	*Rainfall in desecending order in cm*	*S. No. i.e. order number (m)*	*Rainfall in descending order in cm*
1	208	19	101
2	184	20	100
3	160	21	99
4	140	22	98
5	138	23	96
6	122	24	94
7	120	25	93
8	118	26	92
9	116	27	90
10	115	28	88
11	114	29	86
12	112	30	85
13	110	31	80
14	109	32	78
15	108	33	76
16	107	34	66
17	104	35	60
18	102		

Now, using equation (18.4), we compute the order number (m) for the given dependability percentage $p = 60\%$, as:

$$m = N \cdot \frac{p}{100} = 35 \times \frac{60}{100} = 21$$

The rainfall value tabulated above in table 18.2 at order no.21 is 99 cm; and hence the required dependable rainfall = 99cm. $\therefore$ $P_{60\%} = \mathbf{99\ cm}$ Ans.

Similarly, for dependability $p = 75\%$, we calculate the order no. (m)

$$= N \cdot \frac{p}{100} = 35 \times \frac{75}{100} = 26.25.$$

Since reqd. order no. is not an integer, the mean value of rainfall corresponding to order no. 26 and 27 will be taken as the value of p of 75% dependability.

$$\therefore \qquad P_{75\%} = \frac{92+90}{2} = 91 \text{ cm} \quad \textbf{Ans.}$$

Example 18.3. *The daily flows in a river for three consecutive years are given in the table by class interval alongwith the number of days the flow belonged to this class. What are the 50% and 75% dependable flows (annual and daily) for the river ?*

Table 18.3

		Year 1981	*Year 1982*	*Year 1983*
S. No.	*Daily mean discharge m^3/s (range)*	*No. of days the flow belonged to the given range (class interval)*	*No. of days the flow equalled the class range of col (2)*	*No. of days the flow equalled the range given in col (2)*
(1)	*(2)*	*(3)*	*(4)*	*(5)*
1	*100-90.1*	*0*	*6*	*10*
2	*90-80.1*	*16*	*19*	*16*
3	*80-70.1*	*27*	*25*	*38*
4	*70-60.1*	*21*	*60*	*67*
5	*60-50.1*	*43*	*51*	*58*
6	*50-40.1*	*59*	*38*	*38*
7	*40-30.1*	*64*	*29*	*70*
8	*30-20.1*	*22*	*48*	*29*
9	*20-10.1*	*59*	*63*	*26*
10	*10-negligible*	*54*	*26*	*13*

(Engg. Services, 2002)

Solution. With the given daily flows, we will calculate the annual yields for the given 3 years of records, as computed in Table 18.4. Since daily discharge figs. are given in range (interval), mid value of range (interval) can be assumed to be the flow discharge, which when multiplied by no. of days, will give yield during those days. Summation of yield values over 365 days will give annual yield, as shown in Table 18.4, which otherwise is self explanatory.

Table 18.4

			Year 1981		*Year 1982*		*Year 1983*	
S. No.	*Daily mean discharge (range) (m^3/s) in descending order*	*Mean value of daily discharge range*	*No. of days for which the flow equalled to that given in col (3)*	*Yield (Mm^3) = (3) × (4) × $\frac{86400}{10^6}$ [(3) × (4) × 0.0864]*	*No. of days for which flow equalled to that of col (3)*	*Yield (Mm^3) = 0.0864 × col (3) × col (6)*	*No. of days for which flow equaled to that of col (3)*	*Yield ($M\ m^3$) = 0.0864 × col (3) × col (8)*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*	*(8)*	*(9)*
1	100-90.1	95.05	0	0	6	49.25	10	82.08
2	90-80.1	85.05	16	117.50	19	139.54	16	117.50
3	80-70.1	75.05	27	174.96	25	162.00	38	246.24
4	70-60.1	65.05	21	117.94	60	336.96	67	376.27
5	60-50.1	55.05	43	204.34	51	242.35	58	275.62
6	50-40.1	45.05	59	229.39	38	147.77	38	147.74
7	40-30.1	35.05	64	193.54	29	87.70	70	211.68
8	30-20.1	25.05	22	47.52	48	103.68	29	62.64
9	20-10.1	15.05	59	74.34	63	81.65	26	33.70
10	10-Negligible	5.05	54	23.33	26	11.23	13	5.62
	Σ		365	1182.86	365	1361.98	365	1559.09

The river yield for 3 year is thus, calculated by summation of col (5), (7) & (9). Let these yields be arranged in descending order.

Year	*Annual yield ($M\ m^3$)*	*Order No. (m)*
1983	1559.09	1
1982	1361.98	2
1981	1182.86	3
		$N = 3$

Order No. (m) for p% dependability

$$= \frac{p}{100} \times N \qquad \text{...(Eqn. 18.4)}$$

(1) Order No for 75% dependability $= \frac{75}{100} \times 3 = 2.25$.

Since order No. of 2.25 is not an integer, the mean value of yield corresponding to S. NO. 2 and 3 will be taken as yield for 75% dependability, given as

$$= \left[\frac{1361.98 + 1182.86}{2}\right] = 1272.42\ M\ m^3$$

Hence, Annual flow with 75% dependability = **1272.42 M m³** **Ans.**

∴ Daily flow (discharge) with 75% dependability

$$= \frac{1272.42 \times 10^6}{365 \times 86400}\ m^3/s = \mathbf{40.34^3\ m^3/s} \quad \textbf{Ans.}$$

(*ii*) Order No. for 50% dependability $= \frac{p}{100} \times N = \frac{50}{100} \times 3 = 1.5$

Since order No. of 1.5 is not an integer, the mean value of yield corresponding to order No. 1 and 2 shall be taken as the yield of 50%. Dependability, given as :

$$= \frac{1559.09 + 1361.98}{2} = 1460.54\, M\, m^3$$

Hence, Annual flow with 50% dependability = **1460.54 M m^3** **Ans.**

∴ Daily flow (discharge) with 50% dependability

$$= \frac{1460.54 \times 10^6}{365 \times 86400}\, m^3/s = \mathbf{46.31\ m^3/s} \quad \textbf{Ans.}$$

18.4.1.1. Converting the dependable rainfall value into the dependable yield value. Certain empirical relations are available for converting the yearly rainfall value for the given catchment into the yearly runoff value expected from that catchment. Some of these formulas are:

1. *Binnie's percentages;*
2. *Strange's tables;*
3. *Barlow's tables;*
4. *Lacey's formula;*
5. *Inglis formula; and*
6. *Khosla's formula.*

These empirical relations are briefly discussed below:

(1) **Binnie's percentages.** The first effort ever made in India to connect the long range rainfall and the runoff (yield), was from Sir Alexander Binnie. He made observations *on two rivers in the central provinces,* and worked out certain percentages to connect the monthly rainfall with the monthly yield for the entire monsoon period from June to October. These percentages have been further adjusted by Mr. Garret, and are given in table 18.5. From the values given in table 18.5, *the total monsoon rainfall Vs percentage of rainfall that becomes runoff* are plotted in Fig. 18.6(*a*).

Table 18.5. Binnie's Monsoon Yield Percentages (as adjusted by Garret)

Monsoon Rainfall (P) in cm	*Monsoon Yield percentage*	*Monsoon Rainfall (P) in cm*	*Monsoon Yield percentage*
25	7.0	107.5	39.5
30	9.0	110.0	40.3
35	11.0	112.5	41.0
40	13.0	115.0	41.7
45	15.0	117.5	42.4
50	17.0	120.0	43.1
55	19.0	122.5	43.7
60	21.0	125.0	44.4
65	23.0	127.5	45.0
70	25.0	130.0	45.6
75	27.0	132.5	46.2
80	29.0	135.0	46.8
85	31.0	137.5	47.3
90	33.0	140.0	47.9
95	35.0	142.5	48.4
100	37.0	145.0	49.0
102.5	37.9	147.5	49.5
105.0	38.7	150.0	50.0

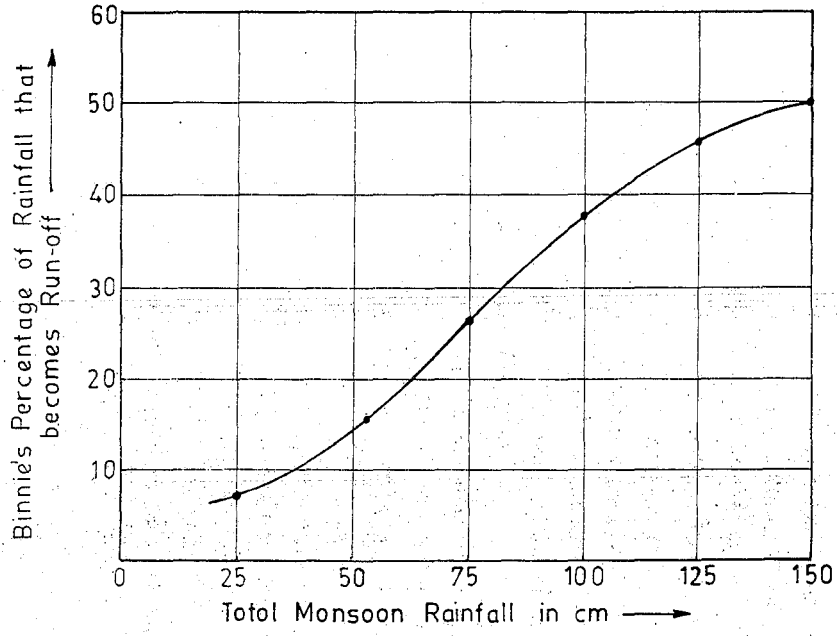


Fig. 18.6(*a*). Binnie's Monsoon Rainfall-runoff Curve.

(2) **Strange's percentages and tables.** Mr. W.L. Strange carried out investigations on catchments in Bombay Presidency, and worked out percentages for converting monsoon rainfall into monsoon yield. He even worked out such percentages for converting daily rainfalls into daily runoffs. It was an improvement over Binnie's tables, since he divided the catchments into three categories to account for the general characteristics of the catchments. The catchments prone to producing higher yields, such as those with more paved areas, etc. were categorised as *good catchments;* and those prone to producing low yields were termed as *bad catchments.* The intermediate types were called *average catchments.* Different runoff percentages were given for different types of catchments for different values of rainfalls, as shown in table 18.6(*a*). These values have also been drawn in the shape of curves, as shown in Fig. 18.6(*b*).

Table 18.6. Values of Strange's Run off Percentages

Monsoon Rainfall in cm	*Runoff Percentages for catchments designated as*			*Monsoon Rainfall in cm*	*Runoff Percentages for catchments designated as*		
	Good	*Average*	*Bad*		*Good*	*Average*	*Bad*
25.0	4.3	3.2	2.1	107.5	40.9	30.6	20.4
30.0	6.2	4.6	3.1	110.0	42.0	31.5	21.0
35.0	8.3	6.2	4.1	112.5	43.1	32.3	21.5
40.0	10.5	7.8	5.2	115.0	44.3	33.2	22.1
45.0	12.8	9.6	6.4	117.5	45.4	34.0	23.2
50.0	15.0	11.3	7.5	120.0	46.5	34.8	23.8
55.0	17.3	12.9	8.6	122.5	47.6	35.7	23.8
60.0	18.5	14.6	9.7	125.0	48.8	36.6	24.4
65.0	21.8	16.3	10.9	127.5	49.9	37.4	24.9
70.0	24.0	18.0	12.0	130.0	51.0	38.2	25.5
75.0	26.3	19.7	13.1	132.5	52.0	39.0	26.0
80.0	28.5	21.3	14.2	135.0	53.3	39.9	26.6
85.0	30.8	23.1	15.4	137.5	54.4	40.8	27.2
90.0	33.0	24.7	16.5	140.0	55.5	41.6	27.7
95.0	35.3	26.4	17.6	142.5	56.6	42.4	28.3
100.0	37.5	28.1	18.7	145.0	57.8	43.3	28.9
102.5	38.5	28.9	19.3	147.5	58.9	44.1	29.4
105.0	39.5	29.8	19.9	150.0	60.0	45.0	30.0

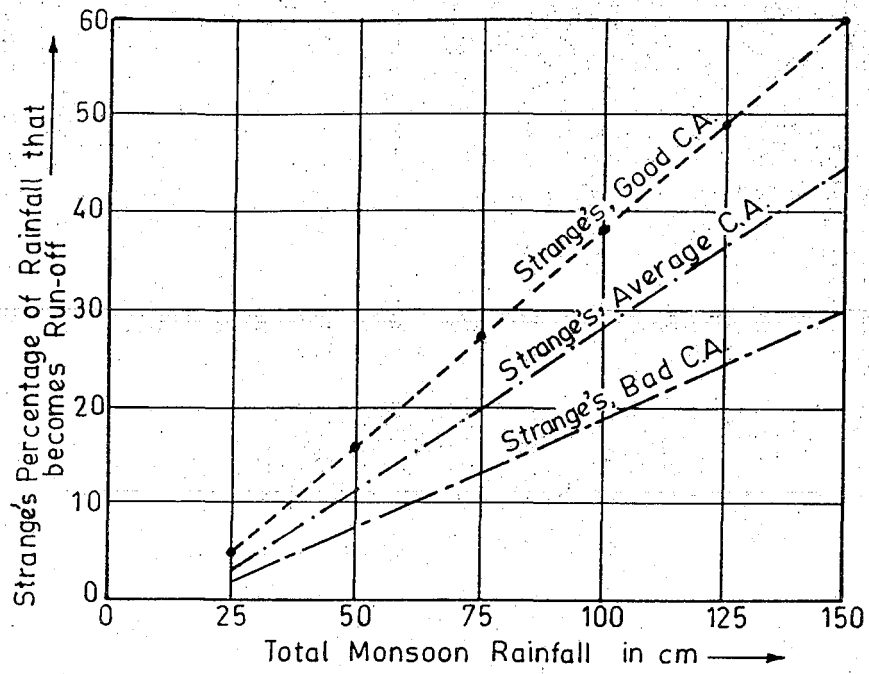


Fig. 18.6(*b*). Strange's monsoon Rainfall-Runoff curves.

(3) **Barlow's tables.** Mr. T.G. Barlow, the first Chief engineer of Hydro-Electric Survey of India, carried out extensive investigations on catchments mostly under 130 square km in U.P. State of India. On the basis of his investigations, he divided catchments into five classes and assigned different percentages of rainfall that become runoff (over long periods) for each class of catchment, as indicated in table 18.7.

Table 18.7. Values of Barlow's Runoff Percentages

S.No.	*Class of catchment*	*Description of the catchment*	*Runoff percentages (K)*
1.	A	Flat, cultivated, adsorbent soils	10
2.	B	Flat, partly cultivated stiff soils	15
3.	C	Average catchment	20
4.	D	Hills and plains with little cultivation	25
5.	E	Very hilly and steep catchment with little or no cultivation	33

The above values of runoff percentages are for the average type of monsoons, and are to be modified by the application of the following coefficients (Table 18.8) according to the nature of the season.

Table 18.8. Values of Barlow's coefficient to be multiplied with K values of table 18.7 to obtain True Runoff Percentages

S.No.	*Nature of season*	*Class of catchment*				
		A	*B*	*C*	*D*	*E*
1.	Light rain, no heavy down pour	0.7	0.8	0.8	0.8	0.8
2.	Average or varying rainfall, no continuous down pour	1.0	1.0	1.0	1.0	1.0
3.	Continuous down pour	1.5	1.5	1.6	1.7	1.8

(4) **Lacey's formula.** This formula connects the monsoon *rainfall (P) with the yield (Q)* by the equation.

$$\text{Yield } (Q) = \left[\frac{P}{1 + \dfrac{304.8\,m}{p.n}}\right] \qquad ...(18.5)$$

where m = a constant, called *monsoon duration factor,* the values of which are given in table 18.9.

n = a constant, called *catchment factor,* the values of which are given for different classes of catchments (as defined by Barlow) in table 18.10.

Table 18.9. Values of m to be used in Eq. (18.5)

S.No.	*Duration of Monsoon*	*Monsoon Duration factor m*
1.	Bad year	0.5
2.	Normal year	1.2
3.	Good year	1.5

Table 18.10. Values of *n* to be used in Eq. (18.5)

S.No.	*Class of catchment*	*Description of the catchment*	*Values of n*
1.	A	Flat cultivated, absorbent soils	0.25
2.	B	Flat, partly cultivated stiff soils	0.60
3.	C	Average catchments	1.00
4.	D	Hills and plains with little cultivation	1.70
5.	E	Very hilly and steep catchments with little or no cultivation	3.45

(5) **Inglis formula.** Inglis derived his formula for catchments of West Maharashtra State of India. He divided the areas as ghat areas (*Sahyadri ranges*) where rainfall is 200 cm or more; and non-ghat areas where rainfall is less than 200 cm. His formulas are:

(*a*) *For ghat areas, with rainfall (P) equalling or exceeding 200 cm:*

$$\text{Yield} = (0.85P - 30.48) \text{ cm} \qquad ...(18.6)$$

where P is the rainfall in cm.

(*b*) *For non-ghat areas with rainfall P less than 200 cm.*

$$\text{Yield} = \frac{P\,(P - 17.78)}{254} \text{ cm} \qquad ...(18.7)$$

where P is the rainfall in cm.

(6) **Khosla's formula.** This formula is based upon the recent research work conducted in this field, and is a very simple and useful formula. It can be easily applied to the entire country, without bothering for the region of its origin. This formula states that

$$\text{Yield } (Q) = P - 0.48\,T_m \qquad ...(18.8)$$

where Q = the yield in cm

P = the rainfall in cm

T_m = mean annual temperature of the area.

Example 18.4. *The design annual rainfall for the catchment of a proposed reservoir has been computed to be 99 cm. The catchment area has been estimated to have the mean annual temperature of 20°C. The catchment area contributing to the proposed reservoir is 1000 sq.km. Calculate the annual design catchment yield for this reservoir. Make use of Khosla's formula.*

Solution. Use Khosla's formula connecting the design rainfall (P) with the design yield (Q) by Eq. (18.8) as:

$$Q = P - 0.48\, T_m$$

where $P = 99$ cm (given)

$T_m = 20°C$ (given)

$$\therefore \quad Q = 99 - 0.48 \times 20 = 89.4 \text{ cm} = \mathbf{0.894\ m}$$

The total yield produced in m^3 from the given catchment of 1000 sq.km (*i.e.* $1000 \times 10^6\ m^2$)

$$= 0.894 \times 1000 \times 10^6\ m^3 = 894\ M.m^3 \quad \textbf{Ans.}$$

18.4.2. Use of Flow Duration Curves for Computing Dependable Flow

As we know, the stream flow varies widely over a **water year***. This variability of stream flow can be studied by plotting *flow duration curves* for the given stream. A **flow duration curve**, also called as **discharge-frequency curve**, is a curve plotted between *stream flow (Q)* and *percent of time the flow is equalled or exceeded (P_p)*, and is of the type shown in Fig. 18.7 (*a*), (*b*) and (*c*).

Such a curve can be plotted by first arranging the stream flow values (Q) in descending order using class intervals, if the number of individual values is very large. The data to be used can be of daily values, weekly values, or monthly values. If N data values are used, the plotting position of any discharge (or class value) Q is given, as :

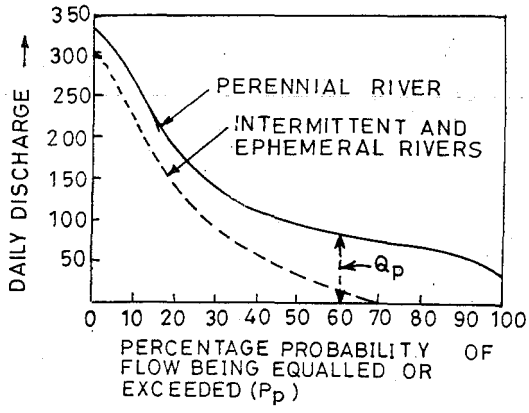


Fig. 18.7(*a*). Typical flow-duration curves on an ordinary paper.

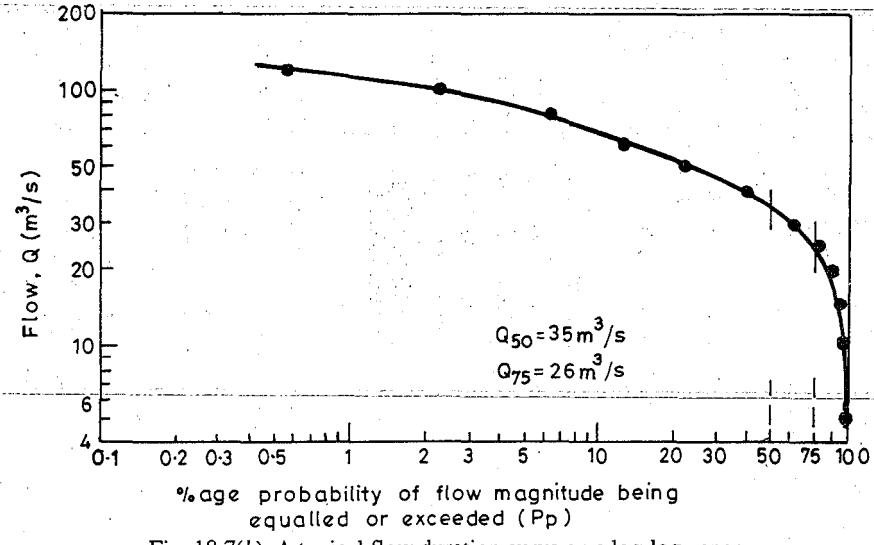


Fig. 18.7(*b*). A typical flow duration curve on a log-log paper.

* 1st June to 30st May.

$$P_p = \frac{m}{N+1} \times 100\%$$

where m = is the order No. of that discharge (or class value)

P_p = percentage probability of the flow magnitude being equalled or exceeded.

The ordinate Q at any percentage probability p (such as 60%), *i.e.* Q_p, will represent the flow magnitude of the river that will be available for 60% of the year, and is hence termed as *60% dependable flow* (Q_{60}). Q_{100} (*i.e.* 100% dependable flow) for a perennial river can, thus, be read out easily from such a curve. Q_{100} for an *ephmeral* or for an *intermittent river* shall evidently be zero.

A flow-duration curve represents the cumulative frequency distribution, and can be considered to represent the stream flow variation of an average year. Such a curve can be plotted on an ordinary arithmetic scale paper, or an semi-log or log-log paper. The following *characteristics of flow-duration curves* have been noticed.

(1) The slope of the flow-duration curve depends upon the interval of data used. Say for example, a daily stream flow data gives a steeper curve than a curve based on monthly data for the same river. This happens due to smoothening of small peaks in monthly data.

(2) The presence of a reservoir on a stream upstream of the gauging point will modify the flow-duration curve for the stream, depending upon the reservoir-regulation effect on the released discharges. A typical reservoir regulation effect on flow duration curve is shown in Fig. 18.7 (*c*).

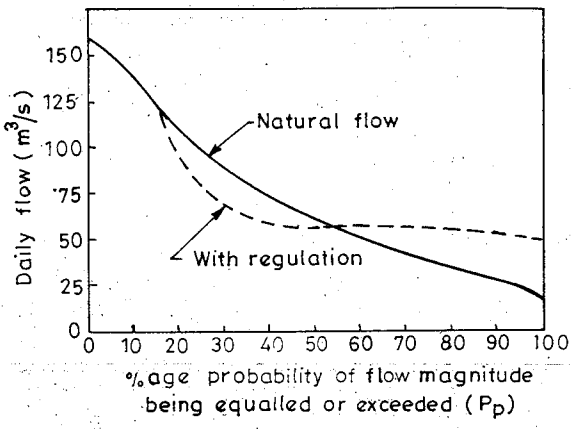


Fig. 18.7. (*c*) Reservoir regulation effect on the flow-duration curve of a stream.

(3) The flow-duration curve, when plotted on a log probability paper, is found to be a straight line at least over the central region, as shown in Fig. 18.7 (*c*). From this property, various coefficients expressing the variability of the flow in a stream can be developed for the description and comparison of different streams.

(4) The flow-duration curve plotted on a log-log paper (Fig. 18.7 (*b*)) is useful in comparing the flow characteristics of different streams. Say for example, a steep slope of the curve indicates a stream with a highly variable discharge ; while a flat slope of the curve indicates a small variability of flow and also a slow response of the catchment to the rainfall. A flat portion on the lower end of the curve indicates considerable base flow. A flat portion on the upper end of the curve is typical of river basins having large flood plains, and also of rivers having large snowfall during a wet season.

(5) The chronological sequence of occurrence of the flow gets hidden in a flow duration curve. A discharge of say 500 m^3/s in a stream will, thus, have the same percentage probability (P_p), irrespective of whether it occurred in January or June. This aspect, a *serious handicap of such curves*, must be kept in mind while interpreting a flow-duration curve.

Flow-duration curves find a considerable use in water-resources planning and development activities. Some of their important uses are indicated below :

(*i*) For evaluating dependable flows of various percentages, such as 100%, 75%, 60%, etc. in the planning of water-resources engineering projects.

(*ii*) In evaluating the characteristics of the hydropower potential of a river.

(*iii*) In comparing the adjacent catchments with a view to extend the stream flow data.

(*iv*) In computing sediment load and dissolved solids load of a stream.

(*v*) In the design of drainage systems, and

(*vi*) in flood control studies.

18.5. Fixing the Reservoir Capacity for the Computed Value of the Dependable Yield of the Reservoir Catchment

After deciding the dependable yield for the proposed reservoir (tank), the reservoir capacity is decided as follows:

The water demand (annual of course) is computed by estimating the *crop water requirement* (including transit losses) and any other water demand required to meet the water supply needs, or the downstream commitments of water release, if any. Reservoir losses @ about 15% of the water demand is then added to obtain the *live* or the *net storage* required to meet the given demand. *Dead storage* is now added to this live storage to obtain the *gross storage* required to meet the demand. The *reservoir capacity*, however, cannot exceed the *catchment yield* (inflow into the reservoir), and hence the reservoir capacity is fixed at a value which is lesser of the value of the assessed gross storage required to meet the demand; and ii) the assessed dependable yield for the reservoir site. The full tank level (FTL) or the full reservoir level (FRL) is finally computed from the elevation-capacity curve.

The *dead storage* level or dead storage required in the above computation is usually fixed at higher of the following values:

(*a*) dead storage = rate of silting x Life of the reservoir

(*b*) dead storage = 10% of gross storage or net water demand.

(*c*) dead storage level being equal to the full supply level of the off taking canal at the tank site

Example 18.5. *The lowest portion of the capacity-elevation curve of a proposed irrigation reservoir, draining 20 km^2 of catchment, is represented by the following data:*

Elevation in m	*Capacity in ha.m*
RL 600	*24.2*
602	*26.2*
604	*30.3*
606	*36.8*

The rate of silting for the catchment has been assessed to be 300 m^3/km^2/year. Assuming the life of the reservoir to be 50 years, (a) compute the dead storage, and the lowest sill level (LSL), if the main canal is 6km long with a bed slope of 1 in 1000, and the canal bed level at the tail end is at RL 594.5 m. The FSD of the canal at the head is 80 cm. The crop water requirement is assessed as 250 ham.

(b) If the dependable yield of the catchment is estimated to be 0.29m, what will be the gross capacity of the reservoir ?

Solution. The *dead storage* is first of all computed as maximum of the following three values :

(*a*) dead storage = rate of silting × life of the reservoir

$$= 300\ \text{m}^3/\text{km}^2/\text{year} \times \underset{\text{(C.A.)}}{20\ \text{km}^2} \times \underset{\text{(Life)}}{50\ \text{year}} = 300000\ \text{m}^3$$

$$= 30\ \text{ha.m.} \qquad ...(i)$$

(*b*) dead storage

$$= 10\% \times \text{net water demand or crop water requirement} = 10\% \times 250\ \text{ha.m}$$

$$= 25\ \text{ha.m} \qquad ...(ii)$$

(*c*) Dead Storage Level = FSL of canal at headworks

$$= 594.5 + (6 \times 1000)\frac{1}{1000} + 0.8 = 601.4 \text{ m}$$

Dead Storage Capacity at RL 601.4 m is interpolated

$$= 24.2 + \frac{(26.2 - 24.2) \text{ ham}}{RL\,(602 - 600)} \times RL\,(601.4 - 600) = 24.2 + \frac{2 \times 1.4}{2}$$

$$= 24.2 + 1.4 \text{ ham} = 25.6 \text{ ham} \qquad ...(iii)$$

The dead storage is fixed at maximum of the three values obtained at (*i*), (*ii*) and (*iii*) above, *i.e.* 30 ham, 25 ham, and 25.6 ham. Hence, choose the dead storage at **30 ham. Ans.**

The *LSL* (x) corresponds to 30 ham capacity, which is computed as :

$$30 = 26.2 \text{ ham} + \frac{(30.3 - 26.2) \text{ ham}}{RL\,(604 - 602)} \times RL\,(x - 602)$$

or $$3.8 = \frac{4.1}{2}(x - 602)$$

$$(x - 602) = \frac{3.8 \times 2}{4.1} = 1.85$$

$$x = \mathbf{RL\ 603.85\ m.}$$

The lowest sill level *i.e.* dead storage level is thus fixed at **RL 603.85 m. Ans.**

(*b*) Net water demand = Crop water requirement including transit losses = 250 ham

Reservoir losses = 15% × 250 ham = 37.5 ham

Live storage required

to meet the given demand = (250 + 37.5) ham = **287.5 ham**

Dead storage = 30 ham (computed above)

∴ Gross reservoir storage required

= Live storage + Dead storage = 287.5 + 30 = **317.5 ham**

Dependable yield = 0.29 m (depth) = 0.29 m × (20×10^6 m^2) = **580 ham**

The gross capacity of the reservoir is fixed at the lesser of the gross storage required to meet the demand (*i.e.* 317.5 ham) and the dependable yield (*i.e.* 580 ha.m.). Hence, the reservoir capacity = **317.5 ham. Ans.**

18.6. Relation between Inflow, Outflow, and storage Data for a Reservoir

The *inflow* to the reservoir and the *outflow* from the reservoir are the only two factors which govern the storage capacity of a reservoir. Since the inflow to the reservoir is variable, water is stored in the reservoir to cater to the required outflow from the reservoir, particularly during the critical periods in non-monsoon season. Naturally, if more outflow is required, more capacity has to be provided.

As a matter of fact, after assessing the monthly or annual inflows into the reservoir and representing it by the mass inflow curve, the demand pattern is specified. The reservoir is then usually designed to meet this specified demand, represented by the mass outflow curve.

The reservoir capacity, the reservoir inflow, and the outflow from the reservoir are governed by the storage equation, given by:

Inflow – Outflow = Increase in storage

∴ Increase in Reservoir Storage = Inflow – Outflow ...(18.9)

18.7. Fixing the Reservoir Capacity from the Annual Inflow and Outflow Data

The capacity of reservoir may be determined by determining the storage needed to accommodate the given inflow minus the given outflow, as governed by the above equation. However, this study involves numerous factors, as discussed below:

Streamflow data at the reservoir site must be known. Monthly inflow rates are sufficient for large reservoirs, but daily data may be required for small reservoirs. When the inflow data at the dam site are not known, the data at a station elsewhere

on the stream or on a nearby stream may be collected and adjusted to the dam site. The available data may sometimes be extended so as to include a really drought period.

Besides determining the streamflow data at the dam site, an adjustment has to be made for the water required to be passed from the reservoir to satisfy the prior water rights and to obey the agreements between various sharing States through which the river is passing.

Moreover, the construction of a reservoir increases the exposed area of the water surface above that of the natural stream, and thus, increases the evaporation losses. There is, thus, a net loss of water occurring due to reservoir construction, Sometimes, these losses may be so huge that the entire purpose of the reservoir may be defeated. Seepage from the reservoir may also add to the loss resulting from the reservoir.

All these factors make this study very very complex. An approximate easy solution for determining the reservoir capacity may be obtained graphically with the help of *mass curves* as explained in the next article. On the other hand, a tabular solution is necessary in order to account for all important factors. For more precise results and for all complex systems, computers may be used for programming the analysis (called operation study).

Example 18.6. *Monthly inflow rates during a low-water period at the site of a proposed dam are tabulated in Col. (2) of table 18.9. The corresponding monthly pan evaporation and precipitation at a nearby station are also tabulated in Col. (3) and Col. (4) of the same table. Prior water rights make it obligatory to release the full natural flow or 15 hectare-metres per month, whichever is minimum. If the estimated monthly demands are as given in col. (5) of table 18.9 and the net increased pool area is 400 hectares, find the required storage capacity for the reservoir. Assume pan evaporation coefficient=0.7 and also assume that only 28% of the rainfall on the land area to be flooded by reservoir has reached the stream in the past.*

Table 18.11

Month	*Inflow at dam site in hectare metres*	*Pan evaporation in cm*	*Precipitation in cm*	*Demand in hectare-metres*
(1)	*(2)*	*(3)*	*(4)*	*(5)*
January	1.2	1.8	1.3	15.8
February	0.0	1.8	1.7	14.3
March	0.0	2.6	0.6	9.6
April	0.0	10.2	0.0	4.8
May	0.0	15.4	0.0	3.5
June	0.0	1.6	1.1	3.4
July	240.0	10.8	16.1	5.0
August	480.0	11.7	16.4	5.0
September	1.0	10.8	2.2	10.0
October	0.6	9.6	0.8	15.6
November	0.5	7.8	0.0	16.8
December	0.2	2.0	0.0	16.8
Σ	723.5	101.1	40.2	120.6

Solution. Table 18.11 is extended as shown in Table 18.12. The 28% precipitation is already reaching and is included in the given inflow [Col. (2)], and hence, only 72% of the precipitation is to be included, as worked out in Col. (8). The table is otherwise self- explanatory and the monthly water drawn from the reservoir is worked out in Col. (10).

Table 18.12. Solution Table for Example 18.6

Month	*Inflow at dam site in hectare-metres*	*Pan evapo-ration in cm*	*Precipi-tation in cm*	*Demand in hectare-metres*	*Require-ment due to prior rights equal to Col. (2) or 15 ha.m. whichever is minimum*	*Evaporation in hectare-metres* $\frac{400 \times Col.(3)}{100} \times 0.7 = 2.8 \times Col.(3)$	*Precipitation in hectare metres* $\frac{400 \times Col.(4)}{100} \times 0.72 = 2.88 \times Col.(4)$	*Adjusted inflow. Col. (2) + Col. (8) – Col. (6) – Col. (7) in hectare metres*	*Water required from storage in ha. m. Col. (5) – Col. (9) (only +ve values)*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*	*(8)*	*(9)*	*(10)*
January	1.2	1.8	1.3	15.8	1.2	5.03	3.74	(–) 1.29	17.09
February	0.0	1.8	1.7	14.3	0.0	5.03	4.9	(–) 0.13	14.43
March	0.0	2.6	0.6	9.6	0.0	7.28	1.73	(–) 5.55	15.15
April	0.0	10.2	0.0	4.8	0.0	28.56	0.0	(–) 28.56	33.36
May	0.0	15.4	0.0	3.5	0.0	43.1	0.0	(–) 43.1	46.60
June	0.0	16.6	1.1	3.4	0.0	46.4	3.17	(–) 43.23	46.63
July	240.0	10.8	16.1	5.0	15.0	30.3	46.4	(+) 241.1	Nil
August	480.0	11.7	16.4	5.0	15.0	32.8	47.3	(+) 479.5	Nil
September	1.0	10.8	2.2	10.0	1.0	30.3	6.34	(–) 23.96	33.96
October	0.6	9.6	0.8	15.6	0.6	26.9	2.31	(–) 24.39	39.99
November	0.5	7.8	0.0	16.8	0.5	21.8	0.0	(–) 21.8	38.60
December	0.2	2.0	0.0	16.8	0.2	5.6	0.0	(–) 5.6	22.40
Σ	723.5	101.1	40.2	120.6	33.5	283.1	116.49		308.21

Finally, their summation (*i.e.* **308.21 hectare-metres**) works out to be the required storage capacity of the reservoir. **Ans.**

18.8. Fixation of Reservoir Capacity with the Help of Mass Curves of Inflow and Outflow

After the flow hydrographs for the stream at the dam site have been plotted for a large number of years (say 25 to 30 years), the required storage capacity for a reservoir with a given outflow pattern can be approximately calculated with the help of mass curves. A hydrograph is a plot of discharge vs. time, while a *mass curve* is a plot of accumulated flow vs. time. The area under the hydrograph between times $t = 0$ and $t = t$ will represent nothing but the accumulated flow up to the time t, and hence, the ordinate of the mass curve at time t.

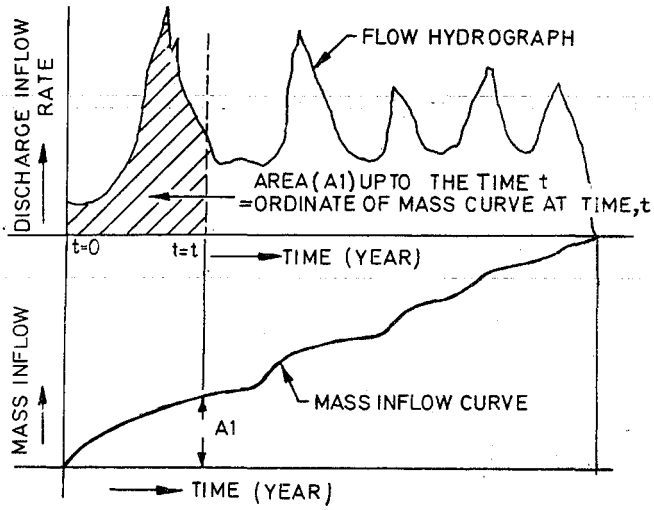


Fig. 18.8 (*a*) and (*b*).

A typical annual inflow hydrograph is shown in Fig. 18.8 (*a*), and the mass curve for this inflow hydrograph is plotted in Fig. 18.8 (*b*). The area under the first curve upto a time (t) is equal to the ordinate of the second curve at the same time (t). Adjustments for the scales and units of the two curves must be made while plotting.

It is evident that a mass curve will continuously rise, as it is the plot of the accumulated inflow. Periods of no inflow would be represented by the horizontal lines on the mass inflow curve. To differentiate such a mass curve of runoff from the mass curve of rainfall, this mass curve is usually called as the **flow mass curve** and is an integral of the flow hydrogrph.

The mass curve may also be called the *ripple diagram.* The slope of the mass curve at any time is a measure of the inflow rate at that time.

After the inflow mass curve has been plotted, the mass curve of demand may also be plotted by accumulating the required outflow. *If a constant rate of withdrawal is required from the reservoir, the mass curve of demand will be a straight line having a slope equal to the demand rate.* Demand curves or demand lines are generally straight lines (representing uniform withdrawal) although, in practice, they may be curved also [See Fig. 18.9 (*a*) and (*b*)].

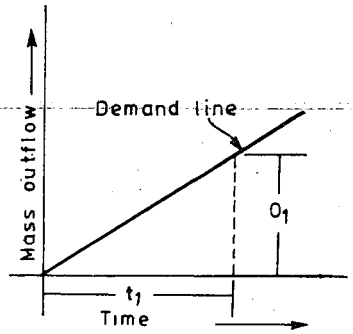


Fig. 18.9 (*a*)

O_1/t_1 = Slope of the line = Demand rate

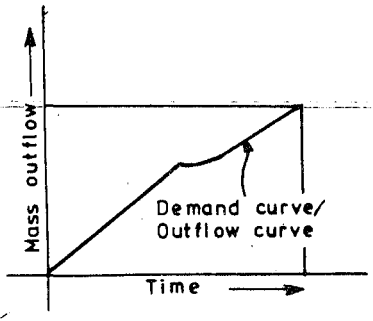


Fig. 18.9 (*b*).

Determining Reservoir Capacity for a Given Demand

The mass curve of inflow and the demand line can be used to determine the required storage capacity. In Fig. 18.10, it is evident that the demand lines drawn tangent to the high points A_1, A_2, A_3... of the mass curve, represent the rate of withdrawal from the reservoir. Assuming the reservoir to be full whenever the demand line intersects the mass curve (points F_1, F_2 ...), the maximum departure (B_1C_1, B_2C_2...) between the two curves represents the reservoir capacity just required to satisfy the demand.

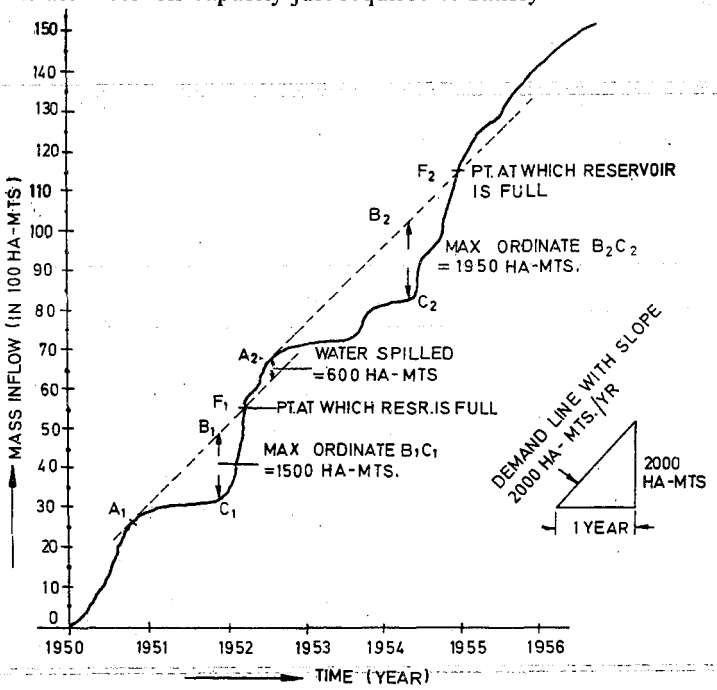


Fig. 18.10.

Fig. 18.10 has been drawn for a demand of 2,000 hectare- metres/year. The biggest departure ordinate (*i.e.* the maximum of B_1C_1, B_2C_2...) works out to be 1,950 hectare-metres/year, which represents the required storage capacity for the reservoir.

The vertical distance between the successive tangents A_1B_1, and A_2B_2, etc., represent the water wasted over the spillway. The spillway must have sufficient capacity to discharge this flood volume.

For Fig. 18.10, the spillway capacity works out to be 600 hectare-metres, and the reservoir capacity as 1,950 hectare-metres. It can also be observed that:

(*i*) Assuming the reservoir to be full at A_1, it is depleted to 1,950 – 1,500 = 450 ha-m at C_1 and is again full at F_1.

(*ii*) The reservoir is full between F_1 and A_2, and the quantity of water spilled over the spillway is equal to 600 ha-m.

(*iii*) From A_2, the water starts reducing in the reservoir till it becomes fully empty at C_2.

(*iv*) The water again starts collecting in the reservoir and it is again full at F_2.

Note 1. It may also be noted that a demand line, when extended, must intersect the mass curve. If it does not, the reservoir will not refill.

Note 2. When the demand curve is not a straight line, then the two mass curves are superimposed over each other in such a way that their origins and axis coincide. The larger ordinate between the two, gives the required storage capacity, as shown in Fig. 18.11.

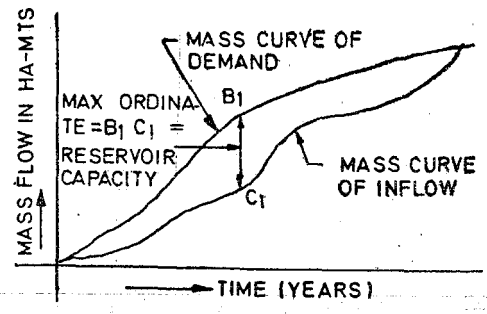


Fig. 18.11.

Fixing the Demand for a Reservoir of a Given Capacity

In the previous article, we have explained as to how the reservoir capacity can be determined for a given demand. The reverse, *i.e.* fixing the demand for a given reservoir capacity, may also be done with the help of mass curve of inflow.

In this case, the tangents are drawn to the high points (A_1, A_2, ...) of the mass inflow curve in such a way that the maximum departure from the mass curve is equal to the reservoir capacity. The slopes of the lines so drawn represent the demand rate which can be obtained with this capacity during different periods. The minimum value of these slopes will represent the withdrawal rate, which can certainly be obtained from the given reservoir, and will, thus, represent its *firm yield.*

For example in Fig. 18.12, *a mass inflow curve is given. It is further required to find out the possible yields and the safe yield for a reservoir capacity of 750 ha-m.* To determine these values, we shall proceed as follows :

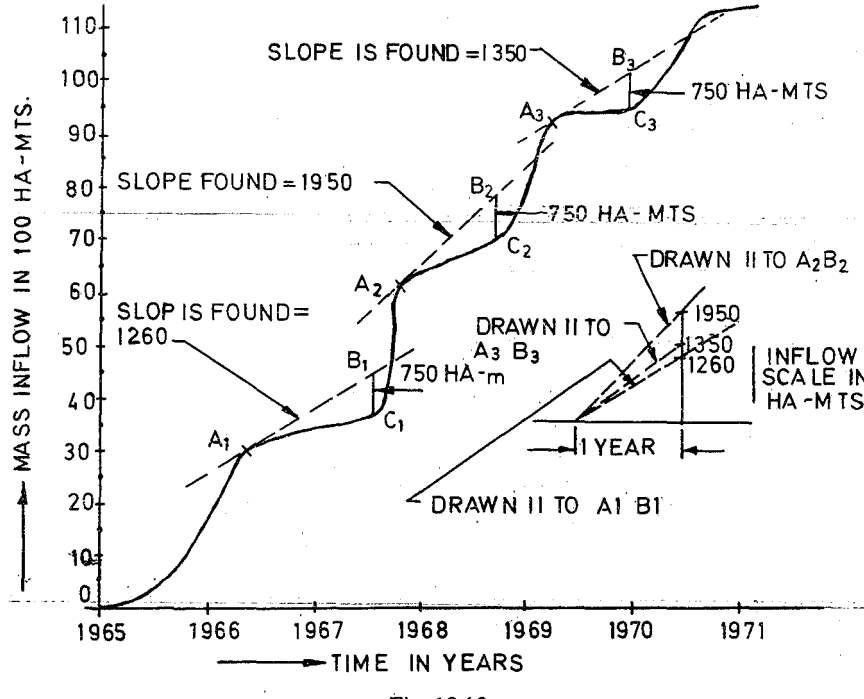


Fig. 18.12.

First of all, the high points A_1, A_2 and A_3 are marked on the mass inflow curve. The points B_1, B_2, and B_3 are determined in such a way that their maximum departure from the curve is equal to 750 ha-m. The tangent lines A_1B_1, A_2B_2 and A_3B_3 are then drawn. The slopes of these lines are determined and are found to be 1,260, 1,950, 1,350

ha-m/year, respectively. These values represent the possible yields in different periods. *The minimum of them, i.e.* 1,260 ha-m/year represents the safe yield or the firm yield of the reservoir. **Ans.**

Example 18.7. *Annual runoff in terms of depth over the catchment area of 1675 sq.km. of a reservoir is given below :*

Year	*1962*	*63*	*64*	*65*	*66*	*67*	*68*	*69*
Runoff (cm)	*98*	*143.5*	*168.3*	*94*	*95.3*	*152.4*	*110*	*131.3*

Draw the flow mass diagram. What is the average yield from the catchment? What should be the live storage capacity of the reservoir to use the source fully? If the dead storage is 20% of the live storage, what is the gross storage? Mark the filling and emptying periods on the mass curve. *(Bhopal University, 1980)*

Solution. The cumulative runoff values are worked out in Col. 4 of table 18.13, and they are plotted against the values of corresponding years (col. 1) of the same table, so as to obtain the desired mass diagram [Fig. 18.13].

Table 18.13

Year	*Yearly runoff (cm)*	*Runoff as volume in* $\text{M.m}^3 = \frac{Col.\ (2)}{100} \times 1675$	*Cumulative runoff as volume in* M.m^3
(1)	*(2)*	*(3)*	*(4)*
1962	98	1642	1642
63	143.5	2404	4046
64	168.3	2819	6865
65	94	1575	8440
66	95.3	1596	10,036
67	152.4	2553	12,589
68	110	1842	14,431
69	131.3	2199	16,630
	$\Sigma = 992.8$	16630	

The **average annual yield** of the catchment is the arithmetic mean of the given annual yields, and is equal to

$$= \frac{992.8}{8} = 124.1 \text{ cm of runoff}$$

$$= 1.241 \text{ m} \times CA \text{ in m}^2 \qquad \text{(volume of runoff)}$$

$$= 1.241 \text{ m} \times (1675 \times 10^6) \text{ m}^2$$

$$= 2078.68 \text{ Mm}^3; \text{ say } \mathbf{2079\ Mm^3} \quad \textbf{Ans.}$$

Now, to utilise the source fully, there should not be any spilling over of the water, and the yearly demand should be equal to the average yield *i.e.* 2079 Mm^3.

To determine the required reservoir capacity to meet this demand rate, a line is drawn from the high point A_1, parallel to this demand rate, as shown in Fig. 18.13; and the maximum departure of this line from the mass curve is read out as $B_1C_1 = 1065\ \text{Mm}^3$.

Hence, the required reservoir capacity is 1065 Mm^3. **Ans.**

Physically speaking, the reservoir will be empty at trough pts C_1 and C_2, and full at ridge point A_1.

Alternatively, the mass demand line can be plotted, and both curves extended back up to the origin O, as shown in Fig. 18.14.

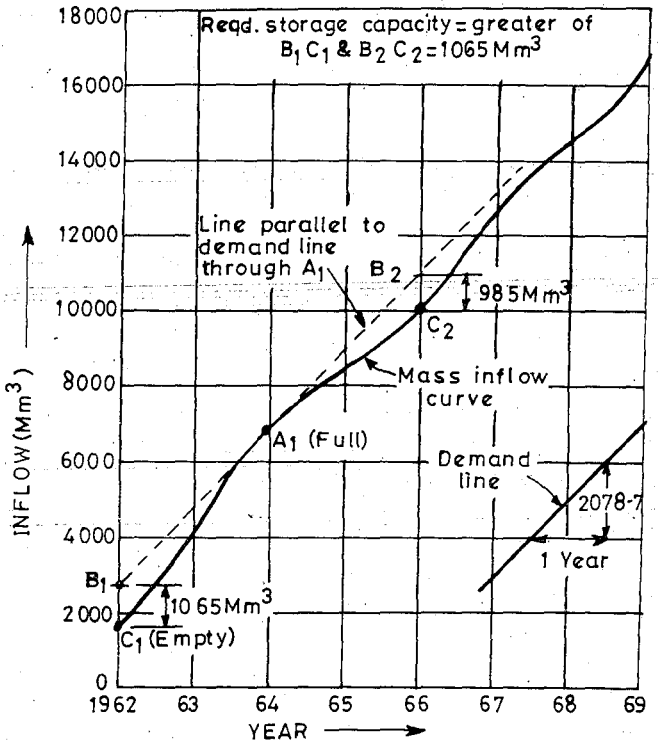


Fig. 18.13.

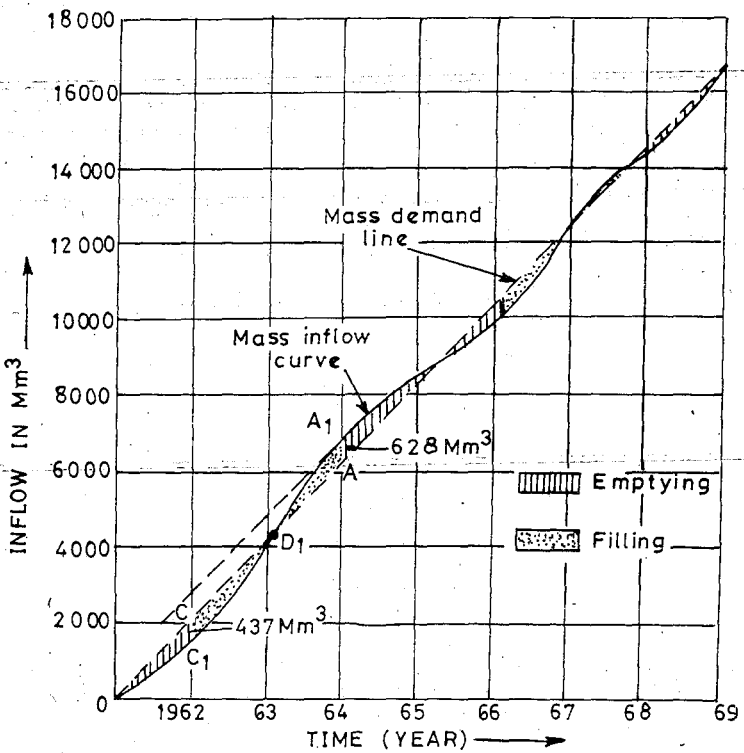


Fig. 18.14.

It can be seen from this curve, that from O to C_1, the slope of the inflow curve is less than that of the demand curve, indicating that the inflow is less than the outflow, and the reservoir is *emptying*. From C_1 to D_1, the slope of the inflow curve is in excess of the demand, and reservoir is *filling*. The vertical intercept C_1 C, from the inflow curve at C_1 to the demand curve at C, represents the initial storage to meet the demand from O to C_1. At D_1, the reservoir level is the same as at O. After D_1, and upto A_1, the slope of the inflow curve is steeper than that of the demand curve, and as such, the reservoir is still rising. Point A_1 represents the full reservoir level. Vertical intercept A_1A represents the storage between the initial water level corresponding to the point O and the full reservoir level. The total minimum storage capacity of the reservoir to meet the demand is thus given by the vertical intercepts C_1C and A_1A.

Max. withdrawl from storage + Max. stored in storage

$= 437 + 628 = 1065$ mcm. **Ans.**

The emptying and filling processes are also shown in Fig. 18.14 ; filling will occur when the slope of the inflow curve is more than that of the demand curve, and *vice versa.*

18.9. Fixation of Reservoir Capacity Analytically using Sequent Peak Algorithm

The *sequent peak algorithm* is a simple and straight forward analytical procedure, for computing reservoir capacity, and is used as an excellent alternative to the mass curve method of determining reservoir capacity.

In the mass curve analysis, the reservoir is assumed to be full at the beginning of the dry period, and storage required to pass the dry period is estimated. If the mass curve contains only one ridge point, and if there is no well defined subsequent trough point, it may become necessary to repeat the given data for one more cycle to arrive at the desired storage determination. Also, the demand rate is usually not a straight line (as assumed in mass curve analysis), since the demand (out flow) generally becomes non-uniform due to seasonal variations in the demand. *The sequent peak algorithm technique helps us to device simple mathematical solution to the problem of computing the reservoir capacity.*

Sequent peak algorithm (Fig. 18.15) is a plot between time (say, in months) on X-axis, and cumulative inflow minus cumulatives outflow on Y-axis. The quantity taken on Y-axis, corresponding to each month equals to [Σ Inflow $-$ Σ outflow] = [Σ (Inflow-Outflow]. This value is also called *cumulative net inflow.*

The + *ve* values of cumulative net inflow, representing cumulative surplus of inflow will be plotted above X-axis, while its negative values, representing cumulative deficit of inflow, will be plotted below x-axis.

The obtained plot will consist of peaks and troughs, as shown in Fig. 18.15.

The first ridge point A_1 in this plotting (*i.e.* first peak of cumulative net surplus) is called the *first peak* ; while subsequent ridge points A_2, A_3, A_4, etc., are called *sequent peaks.*

Similarly, the first trough point B_1 is called the first trough, while subsequent trough points B_2, B_3, etc. may be called the *sequent troughs.*

The difference between the first peak and first trough (height A_1 B_1) will, in this plot, evidently, represent the reservoir storage required under normal inflows, and

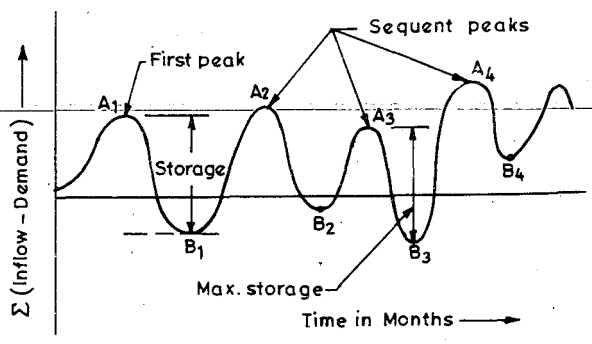


Fig. 18.15. Sequent Peak Algorithm.

is called the *normal storage* ; whereas, the maximum difference between any sequent peak and the just following trough will represent the *maximum storage* required for the reservoir.

The normal and maximum storage through sequent peak algorithm is calculated as follows :

(1) Convert the monthly inflows into the volume units for the period of the available data.
(2) Estimate the monthly volumes of all the outflows from the reservoir. This should include losses from evaporation, seepage, and other losses.
(3) Compute the cumulative values of Inflows.
(4) Compute the cumulative values of outflows.
(5) Compute the values of cumulative inflow minus cumulative outflow ; *i.e.* [Σ Inflow – Σ Outflow].
(6) Plot a graph by taking months (time) on X-axis, and $\Sigma(I-0)$ of step (5) on Y-axis, on an ordinary graph paper.
(7) The data will plot peaks and troughs. The second and subsequent peaks are called sequent peaks.
(8) The maximum difference between any sequent peak and the just following trough is the maximum storage required for the reservoir. The difference between the first peak and the trough following it, is the storage required under normal inflows.

The method will become more clear on solving example 18.8.

Example 18.8. *Monthly inflows at a proposed reservoir site for a drought period of 15 months are given along with targetted demands (found from a working table) in the table below. Compute the storage required by plotting sequent peak algorithm.*

Table 18.14

Month	*June*	*July*	*Aug*	*Sept*	*Oct*	*Nov*	*Dec*	*Jan*	*Feb*	*March*	*April*	*May*	*June*	*July*	*Aug*
River Inflows ($M\,m^3$)	250	350	400	200	150	150	100	50	150	300	400	450	150	200	450
Targetted demand $M\,m^3$	150	150	200	250	350	400	250	200	150	150	100	250	350	300	100

Solution. Calculations are carried out in Table 18.15 to compute $\Sigma(I-O)$ in col (6).

Table 18.15

Month (1)	*Inflow* (I) $M\,m^3$ (2)	*Outflow* (O) $M\,m^3$ (3)	*Cumulative Inflow* ΣI ($M\,m^3$) (4)	*Cumulative Outflow* ΣO ($M\,m^3$) (5)	($\Sigma I - \Sigma O$) (6)
June	250	150	250	150	+ 100
July	350	150	600	300	+ 300
Aug	400	200	1000	500	+ 500
Sept	200	250	1200	750	+ 450
Oct	150	350	1350	1100	+ 250
Nov	150	400	1500	1500	0
Dec	100	250	1600	1750	(–) 150
Jan	50	200	1650	1950	(–) 300
Feb	150	150	1800	2100	(–) 300
March	300	150	2100	2250	(–) 150
April	400	100	2500	2350	+ 150
May	450	250	2950	2600	+ 350
June	150	350	3100	2950	+ 150
july	200	300	3300	3250	+ 50
Aug	450	100	3750	3350	+ 400

To calculate values of $\Sigma(I-O)$, values of ΣI (cumulative inflow) are written in col (4), and values of ΣO (cumulative outflow) are written in col (5).

Values of $(\Sigma I - \Sigma O) = \Sigma (I - O)$ are new calculated by subtracting values of col (5) from values of col (4). These values are written in col (6) with their + *ve* or – *ve* sign, representing excess & cumulative deficit, respectively. When values of $(\Sigma I - \Sigma O)$ from col (6) are plotted on *y*-axis along with corresponding values of month (col 1) taken on *x*-axis, a plot containing peaks and troughs will be obtained, as shown in Fig. 18.16, called sequent peak algorithm. The difference between first peak and the trough following it, is found to be **800 M m³**, which represents the reqd. storage capacity. **Ans.**

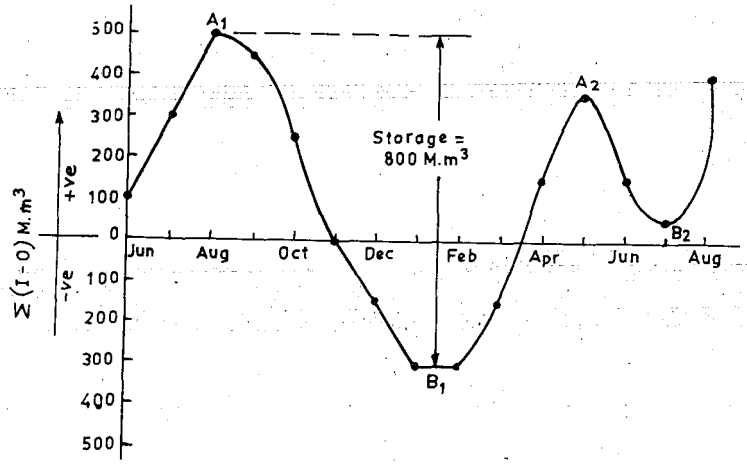


Fig. 18.16. Sequent peak algorithm for Example 18.8.

Alternatively, the values of $\Sigma (I - O)$ can be more easily computed by first computing + *ve* and – *ve* values of $(I - O)$, as in col (4) & (5) of Table 18.16 ; and then computing their summation values, *i.e.* values of $\Sigma (I - O)$ as in col (6) and (7) of Table 18.6. + *ve* values represent cumulative excess inflow ; while – *ve* values represent cumulative deficit. The maximum value out of all the values of Col (6) and (7) will represent the minimum storage required to accommodate the surplus (or to supply the deficit). This method thus, avoids the necessity of plotting peaks & troughs (curve) and the value of reqd storage capacity can be computed easily and mathematically, which procedure can even be programmed in a digital computer to help in estimation of required storage capacity. In present calculations, this peak value in col (6) & (7) is found to be **800 M.m³**. **Ans.**

Table 18.16 *

Month	*Inflow I*	*Outflow (O)*	*(I – O)*		*Σ (I – O)*	
			+ ve values	*– ve values*	*Cumulative*	*Cumulative*
	M m³	*M m³*	*Excess (M m³)*	*Deficit (M m³)*	*excess (M m³)*	*deficit (M m³)*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*
June	250	150	100		100	
July	350	150	200		300	
Aug	400	200	200		500	
Sept	200	250		50		50
Oct	150	350		200		250
Nov	150	400		250		500
Dec	100	250		150		650
Jan	50	200		150		**800***
Feb	150	150	0		0	(Peak value
March	300	150	150		150	in col 6 & 7)
April	400	100	300		450	
May	450	250	200		650	
June	150	350		200		200
July	200	300		100		300
Aug	450	100	350		350	

* Max. of all the values in col (6) & (7)

Example 18.9. *Solve Example 18.7 analytically without using mass curve.*

Solution. The data given in example 18.7 is used in Table 18.17 to compute $(I - O)$ values in col (4) and (5) [– *ve i.e.* deficit values in col (4), and + *ve i.e.* Excess values in col (5)]. The values of $\Sigma (I - O)$ are finally computed in col (6) & (7). as shown :

Table 18.17

Year	*Inflow (I)* Mm^3	*Outflow (O)* Mm^3	$I - O$		$\Sigma (I - O)$	
			– ve *Deficit* Mm^3	*+ ve* *Surplus* Mm^3	*Cumulative Deficit* Mm^3	*Cumulative Surplus* Mm^3
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*
1962	1642	2079	437		437	
63	2404	2079		325		325
64	2819	2079		740		1065*
65	1575	2079	504		504	
66	1596	2079	483		987	
67	2553	2079		474		474
68	1842	2079	237		237	
69	2199	2079		120		120

Out of all the values of col. (6) and (7), the max. value is 1065 Mm^3, which represents the min. storage required to accommodate this 1065 Mm^3 surplus water entering during the years 1963 and 1964. **Ans.**

Gross storage reqd. = Dead storage + Live storage

= 20% of Live storage + Live storage

= 1.2 Live storage = 1.2×1065 Mm^3 = 1278 Mm^3 **Ans.**

Example 18.10. *The yield of water in Mm^3 from a catchment area during each successive month is given in the table below :*

1.4	*2.1*	*2.8*	*8.4*	*11.9*	*11.9*
7.7	*2.8*	*2.52*	*2.24*	*1.96*	*1.68*

Determine the minimum capacity of a reservoir required to allow the above volume of water to be drawn off at a uniform rate assuming that there is no loss of water over the spillway.

Solution. The total inflow of water in 12 months

= Summation of inflow values = 57.4 Mm^3.

∴ Average monthly rate at which water is withdrawn to use the inflow fully = Av. demand rate = 57.4/12 = 4.78 Mm^3.

Now, to draw the mass curve of inflow, the cumulative inflow values are worked out in table 18.18.

Table 18.18

Month	*Yield* (Mm^3)	*Cumulative yield* (Mm^3)
1	1.4	1.4
2	2.1	3.5
3	2.8	6.3
4	8.4	14.7
5	11.9	26.6
6	11.9	38.5
7	7.7	46.2
8	2.8	49.0
9	2.52	51.52
10	2.24	53.76
11	1.96	55.72
12	1.68	57.40

The mass inflow curve is now plotted, as shown in Fig. 18.17. A line parallel to the demand rate line is now drawn through the high point A_1 on the inflow mass curve, as shown. The maximum departure between the inflow mass curve and this line, *i.e.* B_1C_1, gives the min. storage capacity reqd. This value is read out as 20.78 Mm^3. **Ans.**

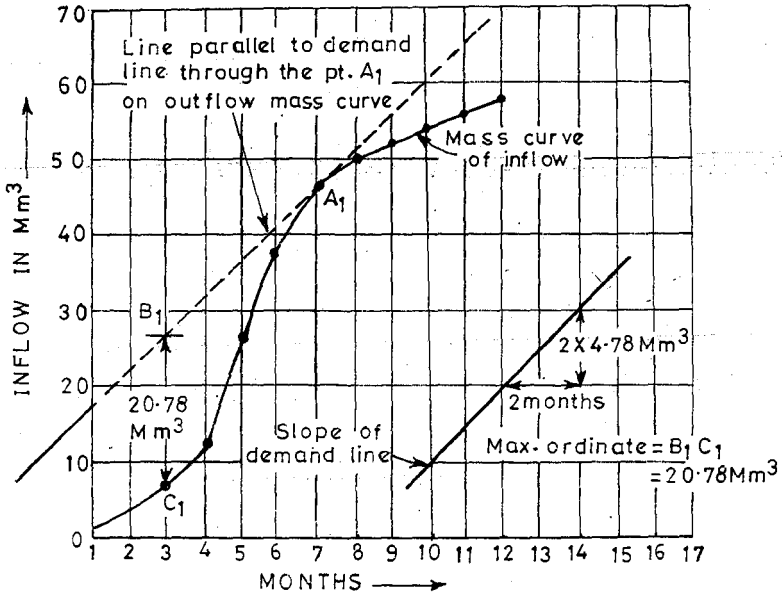


Fig. 18.17.

Analytically, the solution to the problem can be worked out, as shown in table 18.19.

Table 18.19

Month	*Inflow* Mm^3	*Outflow (Demand)* Mm^3	*Deficit* Mm^3	*Surplus* Mm^3	*Cumulative Deficit* Mm^3	*Cumulative Surplus* Mm^3
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*
1	1.4	4.78	3.38			
2	2.1	4.78	2.68			
3	2.8	4.78	1.98		8.04	
4	8.4	4.78		3.62		
5	11.9	4.78		7.12		
6	11.9	4.78		7.12		
7	7.7	4.78		2.92		20.78
8	2.8	4.78	1.98			
9	2.52	4.78	2.26			
10	2.24	4.78	2.54			
11	1.96	4.78	2.82			
12	1.68	4.78	3.10		12.70	

The highest value in col. (6) and 7 (as no spilling is allowed), is 20.78 Mm^3, which gives the min. required reservoir storage. **Ans.**

Example 18.11. *The runoff data for a river during a lean year are given below :*

Month	*Jan*	*Feb*	*Mar*	*Apr*	*May*	*Jun*	*Jul*	*Aug*	*Sept*	*Oct*	*Nov*	*Dec*
River flow in $10^6 m^3$	*140*	*27*	*35*	*26*	*16*	*48*	*212*	*180*	*116*	*92*	*67*	*37*

What is the maximum uniform demand that can be met? What is the storage capacity required to meet this demand? What minimum initial storage is necessary? When does the reservoir become empty?

Solution. Total inflow in the year = Summation of the given monthly discharges

$= 996$ M. m^3.

Average monthly rate at which water can be withdrawn to avoid any wastage *i.e. max average monthly rate*

$$= \frac{996}{12} = \mathbf{83\ Mm^3} \quad \textbf{Ans.}$$

Now, to determine storage capacity etc. we carry out the computations in table 18.20.

Table 18.20

Month end	*Monthly Inflow in reservoir in Mm^3.*	*Monthly outflow from reservoir in Mm^3.*	*Monthly deficit in reservoir (to be supplied from storage) in Mm^3*	*Monthly surplus in reservoir in Mm^3*	*Consecutive cumulative deficit in Mm^3*	*Consecutive cumulative surplus in Mm^3*	*Net water available in the reservoir in Mm^3*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*	*(8)*
Jan	140	83		57		57	(+) 57
Feb	27	83	56				
March	35	83	48				
April	26	83	57				
May	16	83	67				
June	48	83	35		263		(–) 206
July	212	83		129			
Aug	180	83		97			
Sept	116	83		33			
Oct	92	83		9		268	(+) 62
Nov	67	83	16				
Dec	37	83	46		62		0

The highest value in col. (6) and (7) is **268 Mm^3**, which represents the minimum storage capacity required to meet the demand without any spilling. **Ans.**

To compute the min. initial storage, we compute in col. (8), the net storage left in the reservoir with the above inflows and outflows. The maximum negative storage here works out to be 206 Mm^3. In order that the reservoir fully meets the demand with the above inflows & outflows, there should be no negative storage in it, and in the limiting case the max. negative storage should be equal to zero. Hence, the min. initial storage in the reservoir should be **206 Mm^3**, which will just meet the shortage created in the reservoir by *June end,* when the reservoir will become empty. **Ans.**

Example 18.12. *The storage capacity of a reservoir for a flood control project is to be determined. The estimated cost of damage if the emergency spillway is topped is Rs. 10 lacs for each event. The interest rate is 6% and the reservoir life is 50 years. Six reservoir designs of different storage capacities ; the probabilities of exceeding those capacities in any given year, and the estimated initial cost for construction of the reservoir are as follows :*

Reservior design number	*1*	*2*	*3*	*4*	*5*	*6*
Flood storage capacity in lac cubic metres	*30*	*35*	*40*	*45*	*50*	*55*
Probability of exceeding storage capacity in any given year	*0.15*	*0.10*	*0.06*	*0.04*	*0.02*	*0.01*
Estimated initial cost, in lac rupees	*25*	*30*	*35*	*40*	*45*	*50*

Determine the optimal storage capacity of the reservoir.

(A.M.I.E. 1989)

Solution. The required computations are carried out in table 18.21 below.

Table 18.21

Reservior Design No.	*Reservior Capacity in lakh cum*	*Probability of exceeding storage capacity*	*No. of times the flood is likely to exceed the capacity in 50 yrs.*	*Damages caused due to flood exceeding capacity in Rs. Lakh = Rs. 10 lakh × Col (4)*	*Extra benefit over proposal No. 1 of lowest storage as datum; Rs. 75 lakh – col (5)*	*Imtial cost of each proposal in Rs. Lakh*	*Extra costs involved over proposal 1 as datum, in Rs. Lakh Col. 7 – Rs. 25 lakh*	*Net additional benefit in Rs. lakh; col. (6) – col. (8)*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*	*(8)*	*(9)*
1	30	15%	7.5	75	—	25	—	—
2	35	10%	5	50	25	30	5	20
3	40	6%	3	30	45	35	10	35
4	45	4%	2	20	55	40	15	40
5	50	2%	1	10	65	45	20	45
6	55	1%	0.5	5	70	50	25	45

The above table computes the damages caused by overflooding for each proposal, the max. damage of Rs. 75 lakhs being for proposal 1 - having min. reservoir capacity; and the min. damage of Rs. 5 lakh being for proposal 6 – having max. storage capacity.

The additional benefits thus provided by proposals 2 to 6 over proposal 1 are computed in col (6) as 75 lakhs – col (5). Extra costs are also worked out in col (8).

These calculations reflect that in proposal 2, we spend extra Rs. 5 lakh (over proposal 1) and get a benefit of 25 lakhs (over proposal 1 of course) ; *i.e.* a net benefit of Rs. 20 lakh. Similarly, in proposal 3, we spend extra Rs. 10 lakhs & get a benefit of Rs. 45 lakhs over proposal 1; *i.e.* a net benefit of Rs. 35 lakhs. Similarly, we find that our net benefit increases up to proposal 5, and afterwards, as in proposal 6, it becomes equal to that in proposal 5. Hence, the increase in reservoir capacity up to proposal 5 gives us increasing net returns, but thereafter we don't get any extra net returns. Investing more, beyond proposal 5, would, therefore, not be optimal, and thus, proposal 5 can be considered as the most optimum design.

Hence, the optimum storage capacity of the reservoir will correspond to that of proposal 5, *i.e.* **50 lakh m^3. Ans.**

18.10. Estimation of Demands and Optimal Reservoir Operations

In the previous articles, it was explained as to how the reservoir capacity can be determined for satisfying a certain given downstream demand or *vice versa.* This demand may be constant or may vary throughout the year.

The demand pattern mainly depends upon the purpose for which the reservoir has been constructed. Hence, it has to be different for different types of reservoirs. Moreover, the demands are man made, and should be adjusted in such a way that the maximum benefits can be obtained with the minimum cost. If by reducing the demands slightly, a lot of expenditure can be saved, then we must go in for that. This is known as *optimal planning* for dam reservoirs, and involves economic considerations, discussed afterwards.

Demand patterns for various types of reservoirs are explained below :

(*i*) *Single Purpose Conservation Reservoir.* A single purpose conservation reservoir shall store water only to meet one particular demand, such as water supply or irrigation or hydroelectric power generation, etc. In all such cases, the demand for water can be predicted with a fair degree of accuracy for different parts of the year, and the storage capacity can be worked out, as explained earlier. The storage capacity so worked out should be otherwise feasible and should be justified on cost benefit considerations.

(*ii*) *Single Purpose Flood Control Reservoir.* In case of a single purpose flood control reservoir, there is no specific downstream demand at all, except to satisfy that the downstream release should not exceed the safe carrying capacity of the channel. Since the effect of constructing a flood control reservoir is to moderate and reduce the flood peaks by absorbing certain volume of flood, and then gradually releasing it when the flood subsides (Fig. 18.18), the capacity of such a reservoir should be sufficient to absorb this excess volume of flood.

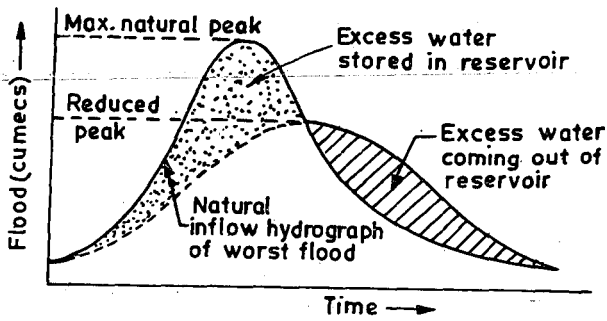


Fig. 18.18.

It, therefore, becomes evident that the capacity of such a

reservoir does not depend upon the pattern of demand but mainly depends upon the hydrograph of the worst flood that is likely to enter this reservoir and also upon the downstream permissible H.F.L. and the safe carrying capacity of the channel.

The hydrograph of the worst inflow flood can be found from the hydrological investigations, and the safe peak rate of outflow can be determined from the downstream channel conditions. The capacity of the reservoir required to moderate the inflow peak to a value equal to or less than the downstream safe peak can then be found by hit and trial method with the help of *flood routing*. Flood routing is the process by which the hydrograph of the moderated flood can be determined, and is explained a little later.

(*iii*) *Multipurpose Reservoirs.* Single purpose reservoirs are seldom constructed these days. Reservoirs are therefore, generally designed to serve more than one purpose. For example, a reservoir constructed for conserving water for irrigation may be combined with its flood control purpose. The head available may simultaneously be utilised for generating hydro-electric power. This combination of irrigation, flood control, and power is generally adopted in designing multipurpose reservoirs in India.

For proper optimal planning of such a multipurpose reservoir, a schedule of operations must be finalised from a number of tentative schedules drawn on the basis of available data and past similar experiences. The schedule which gives maximum total benefits for the various design purposes, without encroaching upon the lower or upper limits of storage, is accepted for estimation and for initial operations. This schedule may be tried and further modified on the basis of the past experience and future needs.

A multipurpose reservoir constructed on a perennial snow fed river in North India (such as Bhakra reservoir) can be operated on the following lines :

Fig. 18.19 shows the typical schedule of operations for such a reservoir. The reservoir water will normally fluctuate between minimum pool level and normal pool level (*i.e.* maximum conservation level) for satisfying irrigation needs. The minimum pool level will ensure the generation of firm power, as the water level shall not normally

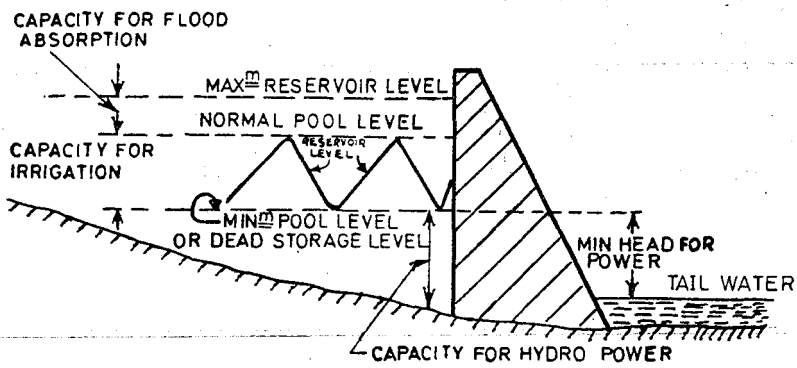


Fig. 18.19.

be allowed to go below the minimum pool level. Any serious flood which may occur after the reservoir is full, shall be absorbed between the normal pool level and the miximum pool level, at all times. As soon as the water level reaches the normal pool level, the spillway will start functioning. But the rate of water discharged over the spillway shall be in accordance with the downstream safe carrying capacity of the

channel, and hence, may be less than the rate of inflow. The water shall, then, be stored temporarily between the normal pool level and the maximum reservoir level, and discharged at a safe rate. As the flood subsides, and the excess water goes out slowly, the water level will again fall up to the normal pool level, and the reservoir will be ready to absorb another flood.

For irrigation, we generally require water from April to June for Kharif crops, and from middle of November to middle of February for Rabi crops. There are heavy rains during monsoon period of say June to middle of October. At the start of the monsoon season, the reservoir is quite depleted because of heavy outflow for Kharif crops and constant release for power generation. Thus, just before the monsoon, a large reservoir capacity is available, which not only conserves the water but also serves to control floods that may occur at this time. During the monsoon, the irrigation demand will almost be nil and water shall be released only for power generation. The reservoir level will, therefore, go on increasing steadily and will be allowed to go up to the normal pool level by the end of normal monsoon season, say middle of October. Any untimely flood occurring after the reservoir is full up to normal pool level, shall be absorbed between normal pool level and the maximum reservoir level, as explained earlier.

After the monsoon is over, the inflow is considerably reduced (almost nil) and water is constantly drawn for Rabi crops and power generation. Due to this, the reservoir level goes on falling up to about middle of February, when it is usually the lowest, somewhere near the minimum pool level (*i.e.* dead storage level). At this time, snow starts melting in the head reaches of the river, which augments the river discharge. Some winter cyclonic rains during December and January may also be helpful in augmenting the reservoir storage. From February-March onwards, there are large supplies due to melting of snow, and an equal burden is to be absorbed by the reservoir for satisfying the Kharif irrigation withdrawals. Thus, during April, May and June, the reservoir level may rise or fall slightly at slow rate; and at the start of monsoon, *i.e.* at the end of June or beginning of July, the reservoir will be quite depleted and prepared for full replenishment of its supplies.

FLOOD ROUTING OR FLOOD ABSORPTION

The hydrograph of a flood entering a reservoir, will change in shape as it emerges out of the reservoir, because certain volume of its water is stored in the reservoir temporarily and is let off as the flood subsides. The base of the hydrograph, therefore, gets broadened, its peak gets reduced, and, of course, the time of peak is delayed. The extent by which the inflow hydrograph gets modified due to the reservoir storage can be computed by a process known as *flood routing,* and more particularly as *reservoir routing* (to differentiate it from routing though 'river channels').

Since the flood protection reservoirs are generally located many km upstream of the cities which are to be saved against floods, it is sometimes necessary to route the outflow hydrograph of the reservoirs up to these downstream localities. The reservoir outflow hydrograph may then be routed through this much length of river channel, so as to obtain the final shape of the hydrograph at the affected cities. This routing, in which the stream itself acts like an elongated reservoir, is known as *channel routing* or *river routing.*

A variety of routing methods are available, and they can be broadly classified into two categories, *viz*

(*i*) *hydrologic routing ; and*

(*ii*) *hydraulic routing*

Hydrologic routing methods employ essentially the *equation of continuity ; whereas the hydraulic routing methods employ the continuity equation together with the equation of motion of unsteady flow.* The basic differential equations used in hydraulic routing are popularly known as *St. Venant equations*, and afford a better description of the unsteady flow than the hydrologic methods. We shall, however, cofine ourselves in the book to the hydrologic methods only.

18.11. Hydrologic Reservoir Routing Methods

The passage of a flood wave through a reservoir or a river reach is an *unsteady flow* phenomenon. In hydraulics, we classify it as a *gradually varied flow.* The *equation of continuity* used in all the hydrologic routing methods, as the primary equation, states that the difference between the inflow and the outflow rate is equal to the rate of change of storage ; *i.e.*

$$I - O = \frac{dS}{dt} \qquad ...(18.9)$$

where I = Inflow rate

O = Outflow rate

S = Storage

Alternatively, in a small time interval Δt, the difference between the total inflow volume and the total outflow volume is equal to the change in storage volume ; *viz.*

$$\underset{\substack{\text{(Inflow volume)}\\ I(v)}}{\bar{I}\,\Delta t} - \underset{\substack{\text{(Ouflow vol.)}\\ O(v)}}{\bar{O}\,\Delta t} = \Delta S \qquad ...(18.10)$$

where $\bar{I}$ = Average inflow (rate) in time Δt

$\bar{O}$ = Average outflow (rate) in time Δt

ΔS = change in storage during the time Δt

Since $\bar{I} = \dfrac{I_1 + I_2}{2}$;

$\bar{O} = \dfrac{O_1 + O_2}{2}$;

$\Delta S = S_2 - S_1$;

where suffixes 1 and 2 denote the beginning and the end of the time interval Δt.

Eq. (18.10) can then be written as :

$$\left(\frac{I_1 + I_2}{2}\right)\Delta t - \left(\frac{O_1 + O_2}{2}\right)\Delta t = S_2 - S_1 \qquad ...(18.11)$$

The time interval Δt should be sufficiently short, so that the inflow and outflow hydrographs can be assumed to be in straight line, in that time interval. Moreover, Δt must be shorter than the time of transit of the flood wave through the reservoir or the given river reach.

The above relationship seems to be very simple, but its evaluation is not easily possible without drastic simplifying assumptions. This is because of the fact that the relations between time and rate of inflow, elevation and storage of reservoir, and

elevation and rate of outflow, cannot be expressed by simple algebraic equations. They are, respectively represented by the *inflow flood hydrograph, the elevation-storage curve, and the outflow-elevation curve.* The first two curves obviously cannot be represented by any simple equations, and the third may be represented by the spillway-discharge equation ($Q = 1.71\ LH^{3/2}$) only if, the discharge through the outlets is neglected. If the discharge though the outlets is also not neglected, then all the three curves will be unamenable to simple mathematical treatment, without drastic assumptions.

Several procedures have, however, been suggested by different investigators to solve the above basic equation (Eq. 18.11) by rearranging the components in different manners. Depending upon the different procedures adopted for solving the above basic equation, the following hydrologic methods may be used for reservoir routing :

(1) *Trial and Error method*

(2) *Modified Pul's method or Storage indication method ; and*

(3) *Goodrich method.*

The first method is discussed here, while the detailed description of other methods is available in authors another book titled "*Hydrology and Water Resources Engineering*" and may be referred to in specific needs.

18.11.1. Trial and Error Method of Reservoir Routing. Trial and Error method is widely adopted with the assistance of computers to reduce the time taken in long calculations involved in this method. This method arranges the basic routing equation (Eq. 18.11), as follows :

$$\frac{I_1 + I_2}{2} \cdot \Delta t = \frac{O_1 + O_2}{2} \cdot \Delta t + (S_2 - S_1) \qquad ...(18.11\ a)$$

The procedure involves assuming of a particular level in the reservoir at the end of the interval Δt, and computing the values on the right side of the above Eq. (18.11 *a*). The summation of $\frac{O_1 + O_2}{2} \cdot \Delta t$ and $(S_2 - S_1)$ is then compared with the known value of $\frac{I_1 + I_2}{2} \cdot \Delta t$. If the two values tally, then the assumed reservoir elevation at the end of the interval is supposed to be O.K. ; otherwise this is changed, and the process is repeated till the required matching is obtained.

This method gives quite reliable results, provided the chosen time interval (Δt) is sufficiently small, so that the mean of the outflow rates at the start and the end of the given interval may be taken as the average throughout the interval.

Procedure : The following detailed procedure may be adopted in this method, to complete the involved computations :

Data to be given : (*i*) the inflow hydrograph

(*ii*) Elevation capacity curve or Elevation area curve

(*iii*) Elevation outflow curve.

Steps involved in computations

(*i*) Divide the inflow flood hydrograph into a number of small intervals. The time interval should be so chosen, as not to miss the peak values.

(*ii*) Fix the normal pool level at which the spillway crest is provided, and the level at which the flood enters the reservoir ; the two are generally taken to be the same, as

it is assumed that this worst design flood enters the reservoir only after the reservoir is full up to the normal pool level.

(*iii*) Work out the spillway and the outlet discharge rating curves, if not given.

(*iv*) Work out the elevation-capacity curve for the reservoir from the elevation-area curve, if the former is not given, using cone formula, *i.e.* $V = \Sigma \frac{h}{3}\left[A_1 + A_2 + \sqrt{A_1 A_2}\right]$, where h is the contour interval. (18.12)

(*v*) Start with the first interval and compute the total inflow during the interval by multiplying the average inflow rate at the beginning and the end of the interval, with the period of the interval.

$$\therefore \qquad I_{(V)} = \frac{I_1 + I_2}{2}(\Delta t)$$

where I_1 = Inflow rate at the start of the interval

I_2 = Inflow rate at the end of the interval

Δt = Duration of the interval

$I_{(V)}$ = Total inflow volume during the interval.

(*vi*) The reservoir level at the start of the flood (*i.e.* start of first interval) is known. Assume a trial value for the reservoir level at the end of the interval.

(*vii*) compute the total outflow during the interval

$$O_{(V)} = \frac{O_1 + O_2}{2} \cdot \Delta t$$

where O_1 = Outflow rate at the start of the interval, corresponding to the given reservoir level.

O_2 = Outflow rate at the end of the interval, corresponding to the assumed reservoir level.

Δt = Duration of the interval

$O_{(V)}$ = Total outflow volume during the interval.

(*viii*) Using the elevation-storage curve for the reservoir, determine the storage S_1 and S_2 at the beginning and the end of the interval, corresponding to the known and the assumed reservoir levels, respectively. Their difference $S_2 - S_1 = \Delta S$, represent the amount of flood stored in the reservoir during the interval.

(*ix*) Add the volume of outflow $O_{(V)}$ obtained in step (*vii*) to the values of ΔS obtained in step (*viii*), and compare it with the inflow volume $I_{(V)}$, calculated in step (*v*). The two values must be equal (*i.e.* $I_{(V)} = O_{(V)} + \Delta S$). If this is so, the assumed reservoir level is correct, otherwise, change it and repeat the procedure till this coincidence is obtained.

(*x*) All the above steps should be repeated for other time intervals, till the entire flood is routed or still further, till the reservoir level returns to pre-flood pool level.

(*xi*) Outflow ordinates are plotted so as to obtain the outflow hydrograph. The point at which it crosses the inflow hydrograph gives the peak outflow rate. From this time, the rate of outflow begins to fall due to decrease in the inflow rate.

(*xii*) The time lag between the two peaks is evaluated as to give the time lag.

An example has been solved to make the procedure very clear.

Example 18.13. *The inflow flood discharges for a possible worst flood are tabulated in Table 18.22 at suitable intervals starting from 0.00 hours on august 20, 1975.*

Table 18.22

Time from start in hr	0	6	12	18	24	30	36	42	48	51	60	66	78	90	102	114
Dis-charge in cumecs	0	50	280	610	1290	1900	2130	1900	1600	1440	1060	780	500	370	220	130

This flood approaches a reservoir with an uncontrolled spillway, the crest of which is kept at RL 140.0 m. Determine the maximum reservoir level and the hydrograph of the routed flood. Values of reservoir capacity (above spillway crest) and outflow discharge at various elevations are tabulated in Tables 18.23 and 18.24 respectively.

Table 18.23

Elevation in metres	140.0	141.0	142.0	143.0	144.0	145.0	146.0
Reservoir storage with spillway crest as datum in million cubic metres (m.c.m.)	0.0	15.0	35.0	60.0	95.0	140.0	240.0

Table 18.24

Elevation	*Outflow discharge in cumecs*
140.0	0
141.0	170
142.0	482
143.0	883
144.0	1,360
145.0	1,905
146.0	2,500

Solution. The elevation storage curve and the elevation outflow curve are plotted with the help of Tables 18.23 and 18.24, as shown in Fig. 18.20 and 18.21 respectively. The hydrograph of the given flood is plotted in Fig. 18.22. Flood routing is carried out by hit and trial method as shown in Table 18.25 and as explained earlier. This table is otherwise self-explanatory.

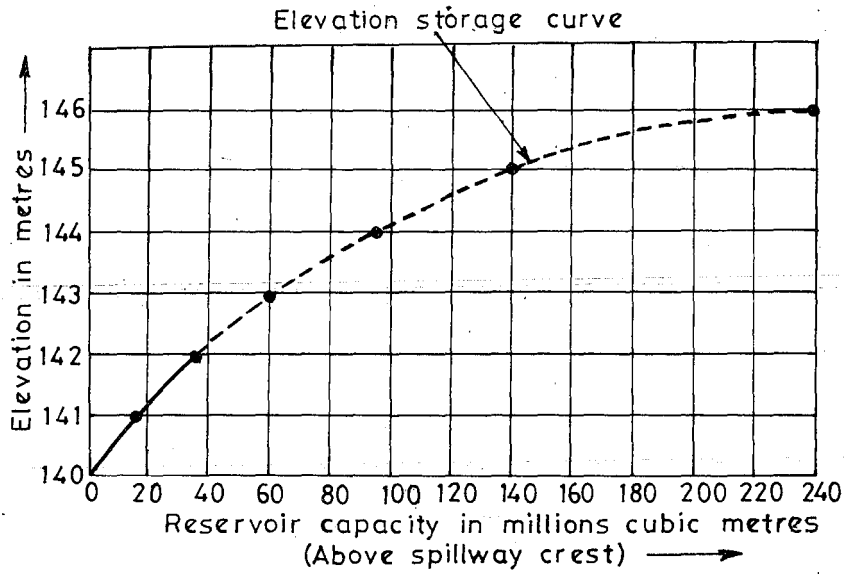


Fig. 18.20. Elevation Storage curve for example 18.13.

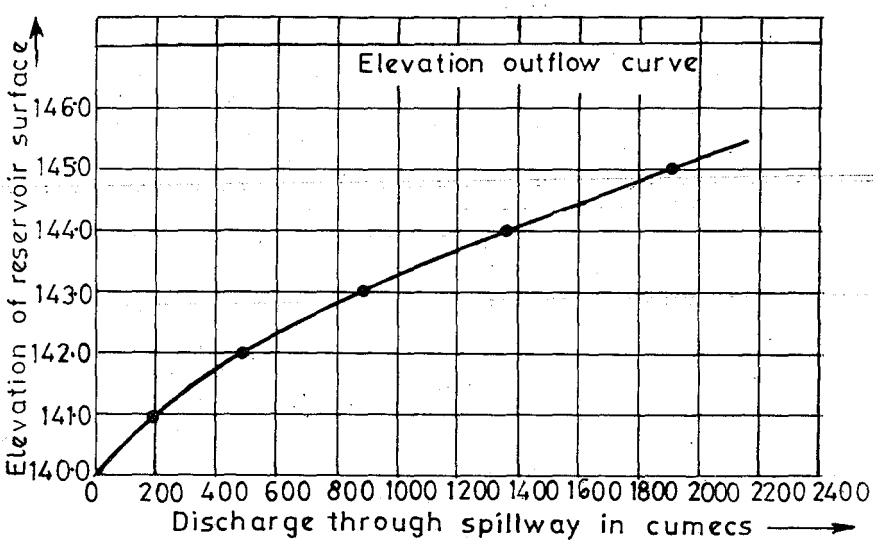


Fig. 18.21. Elevation-Outflow curve for example 18.13.

The outflow hydrograph is plotted from col. (7) of table 18.25, as shown in Fig. 18.22 by the dotted curve. The point at which it intersects the inflow hydrograph, represents the peak of the routed flood.

The peak of outflow flood works out to be **1458 cumecs,** while the peak of the inflow flood was 2,130 cumecs. The time lag is found to be **15 hours** (from Fig. 18.22). The maximum reservoir level is found from Table 18.25, to be **144.18 metres. Ans.**

Table 18.25

Time from start	*Time interval (Δt)*	*Reservoir elevation at the beginning of interval*	*Inflow at the start of interval (I_1)*	*Inflow at the end of interval (I_2)*	*Volume of inflow during the interval $I_{(V)} = (I_1 + I_2)/2$*	*Outflow at the start of interval (O_1)*	*Trial reservoir elevation at the end of interval*	*Outflow at the end of interval (O_2)*	*Mean outflow volume $O_{(V)} = [(O_1 + O_2)/2]\,\Delta t$*	*Storage capacity at start of interval (S_1)*	*Storage capacity at the end of interval (S_2)*	*Change in storage during the interval (ΔS) = ($S_2 - S_1$)*	*Outflow volume + change in storage; i.e. $O_{(V)} + \Delta S$*	*Compare $O_{(V)} + \Delta S$ with $I_{(V)}$, i.e. col. (14) with col. (6) and Remarks*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*	*(8)*	*(9)*	*(10)*	*(11)*	*(12)*	*(13)*	*(14)*	*(15)*
			I_1	I_2	I	O_1		O_2	$O_{(V)}$	S_1	S_2	$(S_2 - S_1)$	$O_{(V)} + \Delta S$	Whether
Hrs.	Hrs.	m	m^3/s	m^3/s	$M.m^3$	m^3/s	m	m^3/s	$M.m^3$	reckoned over spilway crest $M.m^3$	M.m3	*i.e.* (ΔS) $M.m^3$	$M.m^3$	$(O_{(V)} + \Delta S)$ is large or small as compared to $I_{(V)}$
0—6	6	140.0	0.0	50	0.54	0.0	140.1	5.67	0.06	0.0	1.2	1.2	1.26	Large and hence change the trial value
							140.03	0.88	0.01		0.525	0.525	0.535	O.K.

...contd

Table 18.25 (Contd.)

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
6-12	6	140.03	50	280	3.56	0.88	140.3	28.7	0.31	0.525	4.0	3.475	3.785	large
							140.25	21.2	0.23	"	3.85	3.325	3.555	O.K.
12-18	6	140.25	280	610	9.62	21.2	140.5	60.0	0.88	3.85	8.0	4.15	5.03	small
							141.0	170.0	2.06	"	15.0	11.15	13.21	large
							140.8	121.6	1.54	"	13.0	9.15	10.69	large
							140.75	110.2	1.42	"	12.05	8.20	9.62	O.K.
18-24	6	140.75	610	1290	20.5	110.2	141.5	312.2	4.57	12.05	26.0	13.95	18.52	small
							141.6	343.0	4.89	"	27.7	15.65	20.54	O.K.
24-30	6	141.06	1290	1900	34.4	343.0	142.5	672	10.95	27.7	47.0	19.3	30.25	small
							142.6	713	11.4	"	50.0	22.3	33.7	small
							142.63	725	11.52	"	50.5	22.8	34.32	O.K.
30-36	6	142.63	1900	2130	43.6	725	143.5	1113	19.85	50.5	76.0	25.5	45.35	large
							143.4	1065	19.23	"	73.0	22.5	41.73	small
							143.45	1087	19.55	"	74.5	24.0	43.55	O.K.
36-42	6	143.45	2130	1900	43.6	1087	144.0	1360	26.44	74.5	95.0	20.5	46.94	large
							143.95	1335	26.16	"	92.0	17.5	43.65	O.K.
42-48	6	143.95	1900	1600	37.8	1335	144.5	1620	13.9	92.0	115.0	23.0	64.9	large
							144.2	1461	30.2	"	102.0	10.0	40.2	large
							144.15	1435	29.9	"	100.0	8.0	37.9	O.K.

...contd

Table 18.25 (Contd.)

(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*	*(8)*	*(9)*	*(10)*	*(11)*	*(12)*	*(13)*	*(14)*	(15)
48-51	3	144.15	1600	1440	16.4	1435	114.18	1458*	15.5	100.0	101.0	1.0	16.5	O.K.
51-60	9	144.18*	1440	1060	40.5	1458*	144.10	1410	46.5	101.0	98.0	–3.0	43.5	large
							144.05	1385	46.1	"	95.5	–5.0	41.1	O.K.
60-66	6	144.05	1060	780	19.9	1385	143.8	1258	28.5	95.5	87.0	–8.5	20.0	O.K.
66-78	12	143.80	780	500	27.8	1258	142.2	974	48.2	87.0	67.0	–20.0	28.2	O.K.
78-90	12	143.20	500	370	18.8	974	142.60	713	35.4	67.0	50.0	–17.0	18.4	O.K.
90-102	12	142.60	370	220	12.7	713	142.0	482	25.8	50.0	35.0	–15.0	10.8	small
							142.08	510	26.5		36.2	–13.8	12.7	O.K.

*Results : Peak rate of outflow = 1,458 cumecs
Maximum reservoir level = 144.18 metres
Time lag = 15 hours

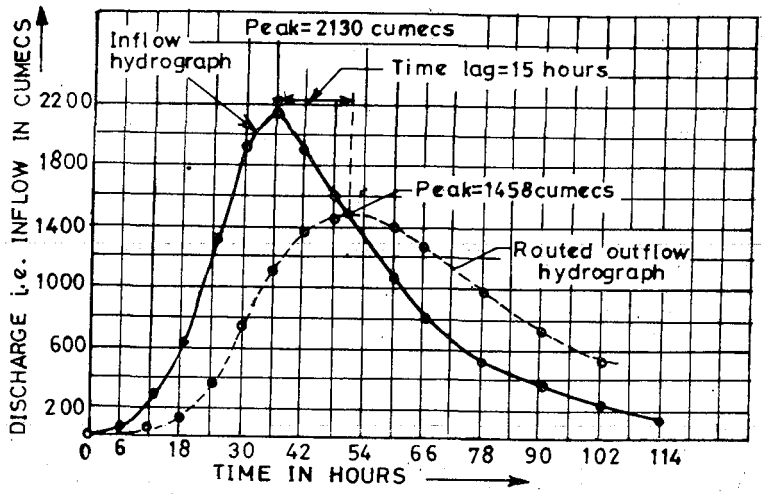


Fig. 18.22. Hydrograph of the given flood and that of the routed flood for example 18.10.

Example 18.14. *The hydrograph of inflow to a reservoir is given in the table below.*

Time(days)	*0*	*2*	*4*	*6*	*8*	*10*	*12*	*14*	*16*	*18*	*20*
Flow (m^3/s)	*60*	*115*	*425*	*550*	*440*	*320*	*260*	*200*	*150*	*110*	*60*

The reservoir is full at the start of the flood inflow. The storage S of reservoir above the spillway crest, is given in million cubic metres by : $S = 8.64\,h$, where h is the head in metres above the crest. The discharge over the spillway is given in cumecs by $Q = 60\,h$. Find the head over the spillway crest at the end of 8th day of the flood.

Solution. Let h_1 be the head over the spillway crest at the end of 2 days ; h_2 at the end of 4 days ; h_3 at the end of 6 days ; and h_4 at the end of 8 days. Now we will consider the position, interval by interval.

(*i*) **0—2 day interval**

Inflow $I_1 = 60$ cumecs.

$I_2 = 115$ cumecs.

$$\therefore \quad I_{(V)} = \left(\frac{I_1 + I_2}{2}\right) t = \left(\frac{60 + 115}{2}\right) 2 \times (24 \times 60 \times 60)\ \text{m}^3 = 15.12\ \text{M.m}^3.$$

Outflow $O_1 = 0$

$O_2 = 60\,h_1$

$$O_{(V)} = \frac{O_1 + O_2}{2} \cdot t = \left(\frac{60h_1}{2}\right) \times 2 \times (24 \times 60 \times 60)\ \text{m}^3 = 5.184\,h_1\ \text{M.m}^3.$$

Storage $S_1 = 0$

$S_2 = 8.64\,h_1$ M.m^3

$\therefore \quad \Delta S = 8.64\,h_1$

Owner- Tepeyan
08114404476
Tilahun w.

Now using $I_{(V)} = O_{(V)} + \Delta S$, we get

$$15.12 = 5.184\, h_1 + 8.64\, h_1 \quad \text{or} \quad 15.12 = 13.824\, h_1$$

$$\therefore \quad h_1 = \frac{15.12}{13.824} = \mathbf{1.093\ m.}$$

(*ii*) 2—4 day interval

Inflow $I_1 = 115$ cumecs ; $I_2 = 425$ cumecs

$$I_{(V)} = \left(\frac{115 + 425}{2}\right) 2 \times 86400\ \text{m}^3 = 46.656\ \text{Mm}^3.$$

Outflow $O_1 = 60\, h_1$; $O_2 = 60\, h_2$

$$O_{(V)} = \left(\frac{60h_1 + 60h_2}{2}\right) 2 \times (86400)\ \text{m}^3 = (h_1 + h_2) \times 5.184\ \text{Mm}^3.$$

Storage $S_1 = 8.64\, h_1$; $S_2 = 8.64\, h_2$

$$\Delta S = 8.64\,(h_2 - h_1)$$

Using $I_{(V)} = O_{(V)} + \Delta S$, we get

$$46.656 = (h_1 + h_2)\, 5.184 + 8.64\,(h_2 - h_1)$$

Using $h_1 = 1.093$ m, we get

$$46.656 = 5.666 + \mathbf{5.184\, h_2 + 8.64\, h_2} - 9.443$$

or $$50.433 = 13.824\, h_2 \quad \text{or} \quad \mathbf{h_2 = 3.648\ m.}$$

(*iii*) 4—6 day interval

Inflow $I_1 = 425$ cumecs ; $I_2 = 550$ cumecs

$$\therefore \quad I_{(V)} = \frac{425 + 550}{2} \times 2 \times 86400\ \text{m}^3 = 84.24\ \text{M.m}^3.$$

Outflow $O_1 = 60h_2$; $O_2 = 60h_3$

$$O_{(V)} = \frac{60h_2 + 60h_3}{2} \times 2 \times 86400\ \text{m}^3 = (h_2 + h_3)\, 5.184\ \text{M.m}^3.$$

Storage $S_1 = 8.64h_2$; $S_2 = 8.64h_3$

$$\Delta S = 8.64\,(h_3 - h_2)$$

Using $I_{(V)} = O_{(V)} + \Delta S$, we get

$$84.24 = (h_2 + h_3)\, 5.184 + 8.64\,(h_3 - h_2)$$

Using $h_2 = 3.648$ m in this eqn., we get

$$84.24 = 18.911 + \mathbf{5.184\ h_3 + 8.64\, h_3} - 31.518$$

$$\therefore \quad 96.848 = 13.824\, h_3 \qquad \therefore \quad \mathbf{h_3 = 7.00\ m.}$$

(*iv*) 6–8 day interval.

Inflow $I_1 = 550$ cumecs ; $I_2 = 440$ cumecs

$$I_{(V)} = \left(\frac{550 + 440}{2}\right) 2\,(86400)\ \text{M.m}^3 = 85.536\ \text{M.m}^3.$$

Outflow $O_1 = 60\, h_3$; $O_2 = 60\, h_4$

$$O_{(V)} = \frac{60h_3 + 60h_4}{2} \times 2 \times 86400\ \text{m}^3 = (h_3 + h_4)\, 5.184\ \text{M.m}^3.$$

Storage $S_1 = 8.64\, h_3$; $S_2 = 8.64\, h_4$

$$\therefore \quad \Delta S = S_2 - S_1 = 8.64\,(h_4 - h_3)$$

Using $h_3 = 7.00$ m in this eqn., we get

$$85.536 = 36.288 + \mathbf{5.184\, h_4} + \mathbf{8.64\, h_4} - 60.48$$

or $109.728 = 13.824\, h_4$ or $\boldsymbol{h_4 = 7.94}$ **m.**

Hence, the head over the spillway crest at the end of the 8th day = 7.94 m. **Ans.**

Example 18.15. *A small reservoir has an area of 5000 hectares* ($50 \times 10^6\ m^2$) *at spillway crest level. Banks are essentially vertical above the spillway crest level. The spillway is 50 m long and has a coefficient C of 2.2 in equation* $q = CH^{3/2}$. *The inflow to the reservoir is given in the table below :*

Time from start (hrs.)	*0*	*6*	*12*	*24*	*30*	*36*	*42*	*48*	*54*
Inflow (m^3/s)	*40*	*340*	*722*	*320*	*192*	*118*	*80*	*56*	*40*

Compute the maximum outflow discharge over the spillway; and the reservoir level to be expected if the reservoir level was at the spillway crest at the start.

(Bhopal University, 1980)

Solution. As done in example 18.11, let us assume that the head over the spillway crest at the end of the various intervals be h_1, h_2, h_3, h_4 metres. Then

For interval 0—6 hr.

Inflow $I_1 = 40$ cumecs ; $I_2 = 340$ cumecs

$$I_{(V)} = \left(\frac{40+340}{2}\right)(6 \times 60 \times 60)\ \text{m}^3 = 4.104\ \text{M.m}^3.$$

Outflow $O_1 = 0$

$$O_2 = 2.2 \times 50 \times h_1^{3/2} = 110\, h_1^{3/2} \qquad [\because Q \text{ over spillway} = \text{C.L.H.}^{3/2}]$$

$$O_{(V)} = \left(\frac{O_1 + O_2}{2}\right) \cdot t = \left(\frac{0 + 110 h_1^{3/2}}{2}\right)(6 \times 60 \times 60)\ \text{m}^3 = 1.19\, h_1^{3/2}\ \text{M.m}^3.$$

Storage $S_1 = 0$

$$S_2 = (50 \times 10^6)\, h_1\ \text{m}^3. \qquad [\because \text{Storage above the spillway is rectangular} = \text{Area of the reservoir} \times \text{ht. of water}]$$

$$\therefore \qquad \Delta S = 50\, h_1\ \text{M.m}^3.$$

Now, using $I_{(V)} = O_{(V)} + \Delta S$, we have

$$4.104 = 1.19\, h_1^{3/2} + 50 h_1 \qquad \text{or} \qquad h_1^{3/2} + 42 h_1 - 3.45 = 0$$

Solving by hit and trial, we get

$$\boldsymbol{h_1 = 0.082}\ \textbf{m.}$$

For interval 6–12 hr.

$$I_{(V)} = \left(\frac{340+722}{2}\right)(6 \times 60 \times 60)\ \text{m}^3 = 11.47\ \text{M.m}^3$$

$$O_{(V)} = \frac{110\, h_1^{3/2} + 110\, h_2^{3/2}}{2} \cdot (6 \times 60 \times 60)\ \text{m}^3 = (h_1^{3/2} + h_2^{3/2})\ 1.19\ \text{M.m}^3$$

$$\Delta S = 50\,(h_2 - h_1)\ \text{M.m}^3$$

$$\therefore \qquad 11.47 = 1.19\,(h_1^{3/2} + h_2^{3/2}) + 50(h_2 - h_1)$$

Using $h_1 = 0.082$, we have

$$9.65 = 0.02 + h_2^{3/2} + 42h_2 - 3.44 \quad \text{or} \quad 13.07 = h_2^{3/2} + 42h_2,$$

or $$h_2^{3/2} + 42h_2 - 13.07 = 0 \quad \therefore \quad \mathbf{h_2 = 0.31\ m.}$$

For interval 12—24 hr.

Inflow $$I_{(V)} = \left(\frac{722 + 320}{2}\right)(12 \times 60 \times 60)\ \text{m}^3 = 22.51\ \text{M.m}^3$$

Outflow $O_1 = 110\ h_2^{3/2}$; $O_2 = 110\ h_3^{3/2}$

$$O_{(V)} = \frac{110\ (h_2^{3/2} + h_3^{3/2})}{2} \times (12 \times 60 \times 60)\ \text{m}^3 = 2.38\ (h_2^{3/2} + h_3^{3/2})\ \text{M.m}^3$$

Storage $S_1 = 50\ h_2$ $\quad S_2 = 50\ h_3$

$$\Delta S = 50\ (h_3 - h_2)$$

$$I_{(V)} = O_{(V)} + \Delta S.$$

$$\therefore \quad 22.51 = 2.38\ (h_2^{3/2} + h_3^{3/2}) + 50\ (h_3 - h_2)$$

Using $h_2 = 0.31$ m, we have

$$9.46 = 0.17 + h_3^{3/2} + 12.2\ h_3 - 3.78$$

$$13.07 = h_3^{3/2} + 12.2\ h_3.$$

or $$h_3^{3/2} + 12.2\ h_3 - 13.07 = 0 \quad \therefore \quad \mathbf{h_3 = 0.99\ m.}$$

For interval 24—30 hr.

Inflow $$I_{(V)} = \left(\frac{320 + 192}{2}\right)(6 \times 60 \times 60)\ \text{m}^3 = 5.53\ \text{M.m}^3$$

Outflow $$O_{(V)} = \frac{110\ (h_3^{3/2} + h_4^{3/2})}{2}\ (6 \times 60 \times 60) = 1.19\ (h_3^{3/2} + h_4^{3/2})$$

$$\Delta S = 50\ (h_4 - h_3)$$

$$I_{(V)} = O_{(V)} + \Delta S.$$

$$\therefore \quad 5.53 = 1.19\ (h_3^{3/2} + h_4^{3/2}) + 50\ (h_4 - h_3)$$

or $$4.65 = (h_3^{3/2} + h_4^{3/2}) + 9.04\ (h_4 - h_3)$$

Using $h_3 = 0.99$. we have

$$4.65 = 0.99 + h_4^{3/2} + 9.04\ h_4 - 8.95$$

or $$h_4^{3/2} + 9.04 h_4 - 12.61 = 0 \quad \therefore \quad \mathbf{h_4 = 1.24\ m.}$$

For interval 30—36 hr.

$$I_{(V)} = \left(\frac{192 + 118}{2}\right)(6 \times 60 \times 60)\ \text{m}^3 = 3.35\ \text{M.m}^3.$$

$$O_{(V)} = 1.19\ (h_4^{3/2} + h_5^{3/2})$$

$$\Delta S = 50\ (h_5 - h_4)$$

$$\therefore \quad 3.35 = 1.19\ (h_4^{3/2} + h_5^{3/2}) + 50\ (h_5 - h_4)$$

or $$2.81 = (h_4^{3/2} + h_5^{3/2}) + 42\,(h_5 - h_4)$$

Using $h_4 = 1.24$ m, we get

$$2.81 = 1.38 + h_5^{3/2} + 42\,h_5 - 52.08$$

or $$h_5^{3/2} + 42h_5 - 50.65 = 0 \qquad \therefore \qquad h_5 = 1.17 \text{ m}$$

Since $h_5 < h_4$, it means that the head has started reducing, reaching its max. value of **1.24 m.**

$\therefore$ max. outflow discharge over the spillway

$$= 110\,(1.24)^{3/2} \text{ cumecs.} = 151.8 \text{ cumecs.} \quad \textbf{Ans.}$$

and, the max. reservoir level will be 1.24 m higher than the spillway crest level, which cannot be worked out as the spillway crest level is not given. **Ans.**

RESERVOIR REGULATION

18.12. Rule Curves and Operating Tables for Reservoirs

Multipurpose reservoirs need to be operated and regulated efficiently with a high degree of intelligence, intuition, and experience, in order to ensure that *they are neither left partially empty by the end of the rainy season, nor they are found full at the time of arrival of a series of peak floods, leading to heavy releases, causing floods in the downstream valley.*

Reservoir regulation committees, consisting of experts are, therefore, generally constituted to ensure issue of proper and timely directives to the staff operating the gated openings of the reservoir, to avoid any bad and inefficient operations.

Guiding tables and curves, called **rule curves** or **guide curves,** are drawn in advance, and kept ready for use for the efficient regulation of the reservoir waters, with time. Such guiding curves are normally required only for the flood season, because for the rest of the year, the reservoir will only discharge water for irrigation and hydel needs. Sometimes, **reservoir regulation manuals** are also prepared and made available to the officers, dealing with the operation of the gates of the reservoir. Such manuals provide guidelines for gate operations, and for the overall maintenance and upkeep of the dam and the reservoir.

A rule curve, either in tabular or graphical form, as framed by the experts on the basis of the past data, broadly reflects the maximum reservoir levels to be achieved by the different dates of the rainy season. *Such a curve, therefore, reflects the vacant space to be left in the reservoir on different dates or weeks of the rainy season.* Such guides may have to be revised from time to time, based on their performance during their actual use.

A typical set of such rule curves (tabular form) being followed for the operation of *Hirakud dam reservoir** are shown in table 18.26 (*a*) and (*b*).

* Hirakud dam is constructed on Mahanadi river is Orissa State of India. It is 3 mile long composite dam, with max. height of 200 ft. Spillway capacity at FRL of 630′ is 14.83 lakh cusecs. It intercepts a catchment of 32,200 sq. miles, and the catchment below the dam and up to the head of delta, i.e. Naraj, is about 18,000 sq. miles. There is no flood reserve or conservation reserve as such, and it is a general use reservoir. The river capacity at Naraj is about 9 lakh cusecs.

Table 18.26 (*a*) Rule curve for filling the Hirakud reservoir for conservation

Date of the year	*Permissible reservoir water level (Dead storage RL=590ft)*
11th August (on and upto)	600′
21st August	605′
1st September	617′
11th September	623′
21st September	627′
1st October (on and after)	630′ (F.R.L.)

Table 18.26 (*b*) Rule curve for flood releases (under ordinary circumstances) for Hirakud dam reservoir

Range of levels RL in ft	*Permissible outflow in lakh cusecs*	*Safe depletion rate in lakh cusecs*
600 — 610	2.5	4.0
610 — 615	3.0	
615 — 620	4.0	
620 — 625	4.5	
625 — 628	5.5	
628 — 630	6.5	

Prior to the above rules, Hirakud dam reservoir was following another set of rules, as given in table 18.27, which proved to be inadequate, leading to insufficient storage at the end of flood seasons for some of the years, and heavy discharges from the reservoir synchronising with floods in the lower catchment in some other years.

Table 18.27. Old Rules, specified in the Manual for Operation of Hirakud reservoir, which have now been superceded by rules of tables 18.21 (*a*) and (*b*)

S. No. of Rule	*Rule*
Rule 1	There should be no full impounding till 1st of September in normal years. The entire inflow should be let out ; care being exercised that the combined discharge of reservoir outflow and the runoff from the catchment below the dam does not exceed 10 lakh cusecs at Naraj. Generally, the inflow up to 5 lakh cusecs can be passed without hesitation. When the inflow is more than 5 lakh cusecs, then the hydrometeorological conditions in the lower catchment should be carefully studied and outflow determined.
Rule 2	Safety of dam should be the prime consideration on and at every occasion when floods are absorbed in the reservoir for purposes of moderating and regulating outflow in the reservoir. A level of + 625.0' should be deemed to be the safe level normally for this purpose.

Another typical rule curve (graphical form), as drawn and used for Maithon dam reservoir, under *D.V.C. System**, is shown in Fig. 18.23. This operation schedule shows that the reservoir, which will be quite depleted by the start of June, will be filled up fully up to *Monsoon storage level* (RL 480′), latest by 3rd week of September, and will be maintained full, till the end of November. The reservoir will be allowed to be depleted only from December onward.

Any flood, which occurs during the last week of September, October and November, will thus, find the reservoir full, and will have to be accommodated within the *'flood reserve', i.e.* between monsoon storage level (RL 480′) and the max. permissible operational level of gates (RL 495′).

Depending upon the intensity of the coming and likely floods after the reservoir is full, the outflows will be regulated and so manipulated that the discharge from the reservoir together with the discharge contributed by the downstream area, does not exceed the safe carrying capacity of the river farther downstream.

As and when a late flood enters the reservoir, the reservoir-gate-operators may adjust the outflow to a lower value in the beginning, but may have to increase the outflow, as the flood waves continue and the weather forecasts for continued rains are received. In this operation, a stage may reach when the outflow may become quite high, causing floods in the down valley.

While manipulating these outflows, chances of synchronisation of the discharge of the downstream uncontrolled catchment will also have to be considered properly.

Say for example, if we release 1 lakh cusecs discharge from the reservoir, and another 1 lakh cusecs gets added to it from the catchment of the downstream 1 km length of the channel, then eventually 2 lakh cusecs will be flowing in the channel farther down the 1 km length. This 2 lakh cusecs may prove quite harmful, as it may exceed the safe carrying capacity of the downstream river (below 1 km).

Another possibility for gate operators could then be to release lesser discharge, say 50,000 cusecs, which with 1 km downstream contribution of 1 lakh cusecs, will make up 1,50,000 cusecs, which may not prove dangerous to farther down. But releasing lesser quantity would cause the reservoir water level to rise rapidly, encroaching the 'flood reserve' quickly, finally leading to a stage, wherein you may have to open the entire gates, passing say 4 lakh cusecs, to avoid over-topping and failure of the dam. This 4 lakh cusecs will eventually destroy the entire downstream area, and may prove worser than what would have happened, if 1 lakh cusecs would have been allowed in the beginning itself.

You can, thus, understand and appreciate as to how important it is, to properly man the outflows from a reservoir. The intelligent and timely manipulations, accompanied by proper rain forecasts, may help us to avoid flooding of the downstream area.

---

* D.V.C. (Damodar Valley Corporation) was constituted in India in July, 1948, on the line of T.V.A. (Tennesse Valley Authority) of USA. to plan and execute flood control works on river Damodar and its tributaries, which had created havoc in the year 1943, causing serious flood losses of the order of Rs. 8 crores, and cutting off the most important Calcutta city with the rest of the country for about ten weeks, due to breaches in G.T. Road and Rly link.

Consequently, 4 dams were constructed, including the Maithon on river Damodar, Konar on river Konar, and Panchat and Tallaiya on its tributary Barakar, along with a barrage at Durgapur (situated a few km downstream of the confluence of Damodar and Barakar. Konar and *Talliya* are smaller dams located in the upper reaches, whereas, Maithon and Panchat are bigger dams in lower reaches, bearing most of the flood burden.

Maithon is located about 13 km above the confluence, and 21 km northwest of Asansol and 40 km east of Dhanbad. Like Konar, it is partly earthen and partly of concrete. The primary purpose of the dam is flood control; whereas, *irrigation* and *hydropower* are secondary purposes.

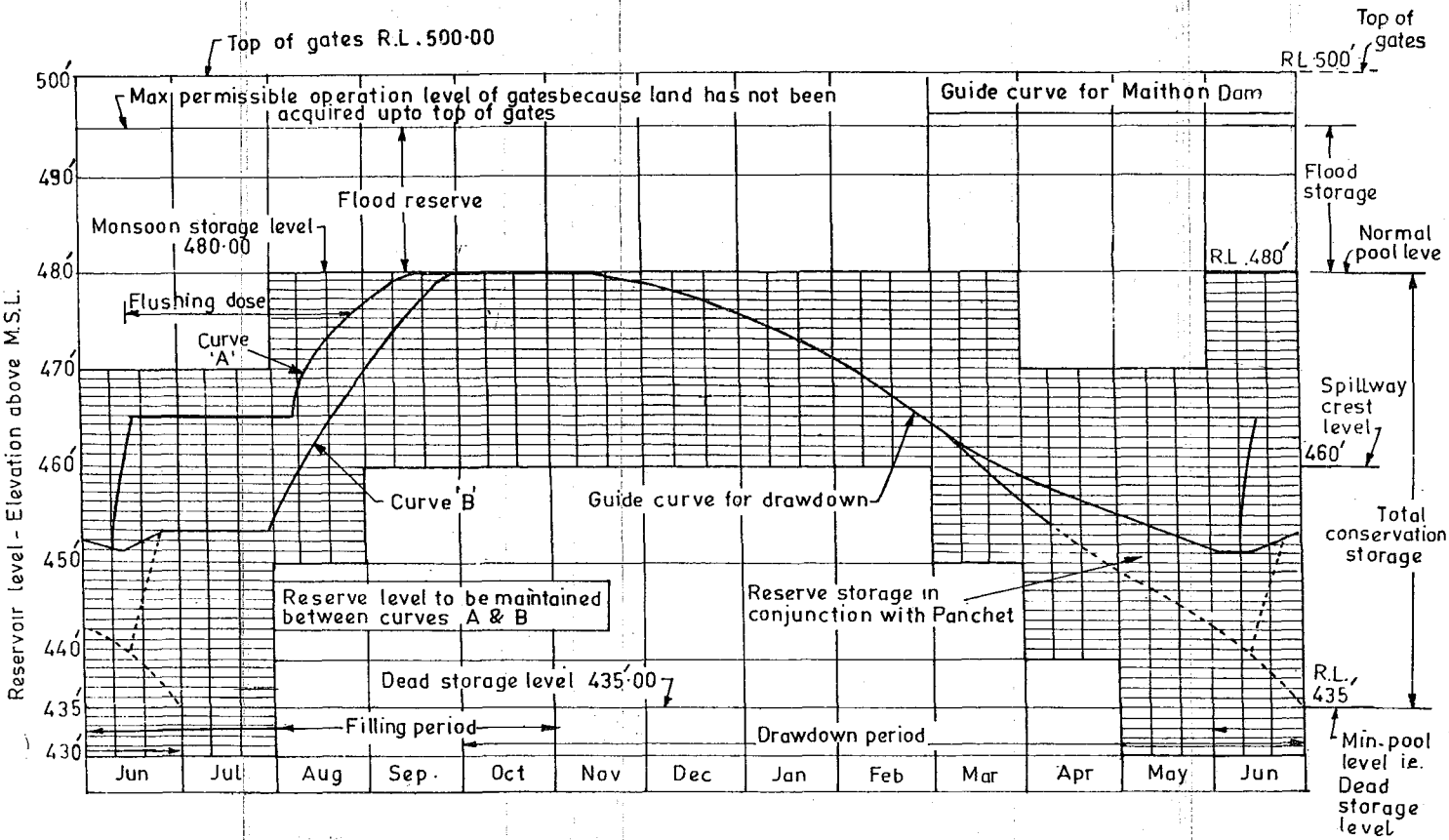


Fig. 18.23. A typical operating guide for Maithon Dam Reservoir.

18.13. Reservoir Sedimentation

Every river carries certain amount of sediment load. The sediment particles try to settle down to the river bottom due to the gravitational force, but may be kept in suspension due to the upward currents in the turbulent flow which may overcome the gravity force. Due to these reasons, the river carries fine sediment in suspension as suspended load, and larger solids along the river bed as bed load. When the silt laiden water reaches a reservoir in the vicinity of a dam, the velocity and the turbulence are considerably reduced. The bigger suspended particles and most of the bed load, therefore, gets deposited in the head reaches of the reservoir. Fine particles may travel some more distance and may finally deposit farther down in the reservoir, as shown in Fig. 18.24. Some very fine particles may remain in suspension for much longer period, and may finally escape from the dam along with the water discharged through the sluiceways, turbines, spillway, etc.

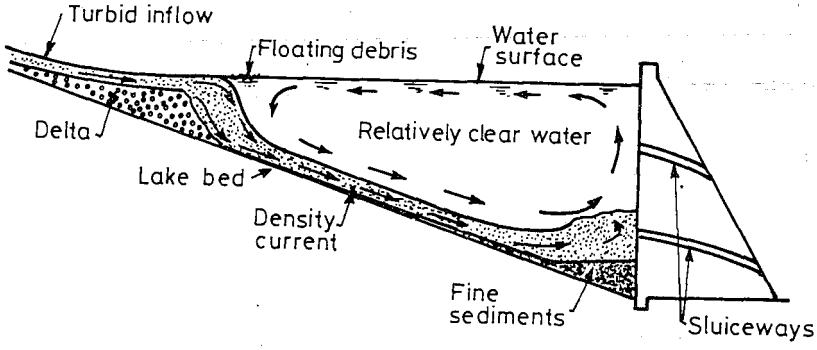


Fig. 18.24. Sediment accumulation in a typical reservoir.

The deposition of sediment in the reservoir is known as 'Reservoir Silting' or 'Reservoir Sedimentation'.

The deposition of the sediment will automatically reduce the water storing capacity of the reservoir, and if this process of deposition continues longer, a stage is likely to reach when the whole reservoir may get silted up and become useless.

Moreover, with the passage of time, the reservoir capacity will go on reducing. Thus, if today at the time of construction, a reservoir can store 10,000 cubic metres of water, tomorrow say after five years, it may be able to store only say 8,000 cubic metres of water. Therefore, in order to see that the capacity does not fall short of requirement ever during the design period, we must take this silting into account. The total volume of silt likely to be deposited during the designed life period of the dam is, therefore, estimated ; and approximately that much of volume is left unused to allow for silting, and is known as **dead storage.** The remainder is known as the *effective storage* or the **live storage.** The dead storage generally varies between 15 to 25% of the total capacity. For example in Bhakra dam, the gross capacity of the dam is 9,344 million cubic metres and the dead storage provided is 2,054 million cubic metres. All the outlets fetching water from the reservoir are provided above the dead storage level.

The importance of this silting can be understood by considering the following example :

Let the total capacity of a reservoir be 30 million cubic metres and the provision of dead storage be 6 million cubic metres. Let the average volume of sediment deposition be 0.15 million cubic metres per year. Then it is evident, that the dead storage will be filled up in $\frac{6}{0.15} = 40$ years, and the total storage in about $\frac{30}{0.15} = 200$ years.

Hence, the usefulness of this reservoir would start reducing after 40 years, and after 200 years it would be nothing but a collection of sand and sediment with no water in it, provided the siltation rate remains constant at 0.15 $M.m^3/yr$.

18.13.1. Density Currents. In a reservoir, the coarser sediment settles down along the bottom of the reservoir, as the muddy flow approaches the reservoir; while the finer sediment usually remains in suspension, and moves in a separate layer than the clear reservoir water, as shown in Fig. 18.24. This layer of water, containing the fine sediment, moves below the upper clearer reservoir water, as a *density current*, since its density is slightly more than the density of the main body of the reservoir water. *Because of their density difference, the water of the density current does not mix easily with the reservoir water, and maintains its identity for a considerable time.* The density current can thus be removed through the dam sluiceways, if they are located properly and at the levels of the density current. A lot of sediment load can, thus, be passed out of the reservoir, if it is possible to locate the dam outlets and sluiceways in such a fashion, as to vent out the density currents. Trap efficiency of reservoirs may thus be decreased by about 2 to 10%, if it is possible to vent such density currents through the outlets and sluiceways of the dams.

18.13.2. Trap Efficiency. Now, we introduce another very important term called Trap efficiency. Trap efficiency is defined as the percentage of the sediment deposited in the reservoir even inspite of taking precautions and measures to control its deposition.

Therefore, Trap Efficiency (η)

$$= \frac{\text{Total sediment deposited in the reservoir}}{\text{Total sediment flowing in the river}} \qquad ...(18.13)$$

Most of the reservoirs trap 95 to 100% of the sediment load flowing into them. Even if various feasible silt control measures are adopted, it has not been possible to reduce this trap efficiency below 90% or so.

18.13.3. Capacity Inflow Ratio. The ratio of the reservoir capacity to the total inflow of water in it, is known as the capacity-inflow ratio. It is a very important factor, because the trap efficiency (η) has been found to be a function of capacity-inflow ratio *i.e.*

$$\eta = f\left(\frac{\text{Capacity}}{\text{Inflow}}\right) \qquad ...(18.14)$$

The graph obtained for the existing reservoirs between trap efficiency and log of $\left(\frac{\text{Capacity}}{\text{Inflow}}\right)$ has been found to be of the type shown in Fig. 18.25.

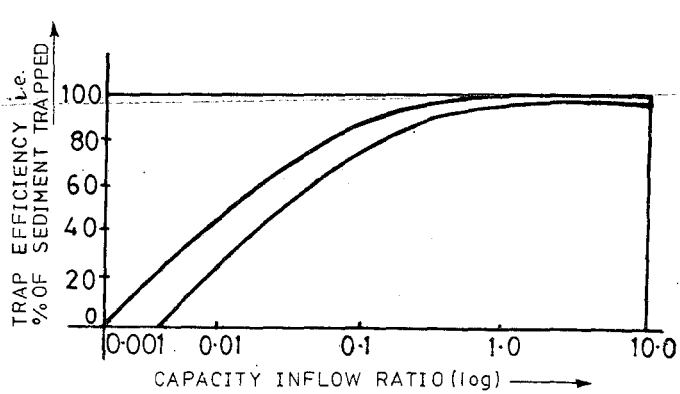


Fig. 18.25.

It is evident from the above curve, that if capacity reduces (with constant inflow), trap efficiency reduces, and hence, lesser sediment is trapped. Therefore, the silting rate in the reservoir shall be more in the beginning, and as its capacity reduces due to silting, the silting rate will reduce. Hence, the complete reservoir-silting may take longer period.

It can also be concluded that for small reservoirs (having small capacity) on large rivers (having large inflow rates), the trap efficiency is extremely low, because the capacity inflow ratio is very small. Such reservoirs silt very little and most of their sediment is passed downstream. On the other hand, large reservoirs on smaller rivers shall silt tremendously and almost complete deposition of sediment may take place.

18.13.4. Silting of Power Reservoirs. In case of reservoirs constructed solely for the purpose of power generation, the silting is comparatively less important. This is because of the fact, that for the proper and efficient functioning of a power reservoir, only a certain minimum head is necessary. This head remains available even after some silt gets deposited. So, only if sufficient water required for power generation remains available, the reservoir's efficiency remains unaffected by silting. But due to silted water, the abrasion of the blades of the turbines may occur very soon, and power production may be stopped over a considerable length of time.

18.13.5. Silting Control in Reservoirs. In order to increase the life of a reservoir, it is necessary to control the deposition of sediment. Various measures are undertaken in order to achieve this aim. The various methods which are adopted can be divided into two parts :

(1) Pre-constructing measures ; and (2) Post-constructing measures.

These measures are discussed below :

(1) **Pre-constructing measure.** They are those measures which are adopted before and during the execution of the project. They are innumerated below :

(*a*) *Selection of Dam Site.* The silting depends upon the amount of erosion from the catchment. If the catchment is less erodible, the silting will be less. Hence, the silting can be reduced by choosing the reservoir site in such a way as to exclude the run off from the easily erodible catchment.

(*b*) *Construction of the Dam in Stages.* The design capacity plays an important role in the silting of a reservoir. When the storage capacity is much less than the average annual runoff entering the reservoir, a large amount of water will get out of the reservoir, thereby, reducing the silting rate compared to what it would have been if the entire water would have been stored. Therefore, the life of a reservoir can be prolonged by constructing the dam in stages. In other words, first of all, the dam should be built lower, and raised subsequently when some of its capacity gets silted up.

(*c*) *Construction of Check Dams.* The sediment inflow can be controlled by building check dams across the river streams contributing major sediment load. These are smaller dams and trap large amounts of coarser sediments. They are quite expensive.

(*d*) *Vegetation Screens.* This is based on the principle that vegetations trap large amounts of sediment. The vegetation growth is, therefore, promoted at the entrance of the reservoir as well as in the catchment. These vegetative covers, through which flood waters have to pass before entering the reservoirs, are known as vegetation screens, and provide a cheap and a good method of silt control.

(*e*) *Construction of Under-sluices in the Dam.* The dam is provided with openings in its base, so as to remove the more silted water on the downstream side.

The sediment concentration will be more at some levels than at others. Therefore, sluices are located at the levels of higher sediment concentration. The method in itself, is not sufficient because the water digs out a channel behind the sluice for movement and leaves most of the sediment undisturbed. Therefore, this is simultaneously supplemented with mechanical loosening and scouring of the neighbouring sediment in order

to increase its effectiveness. But to provide large sluices near the bottom of the dam, is again a structural problem. The use of this method is, therefore, limited.

(2) **Post-constructing Measures.** These measures are undertaken during the operation of the project. They are given below:

(1) *Removal of Post Flood Water.* The sediment content increases just after the floods ; therefore, attempts are generally made not to collect this water. Hence, the efforts should be made to remove the water entering the reservoir at this time.

(2) *Mechanical Stirring of the Sediment.* The deposited sediment is scoured and disturbed by mechanical means, so as to keep it in a moving state, and thus, help in pushing it towards the sluices.

(3) *Erosion Control and Soil Conservation.* This includes all those general methods which are adopted to reduce erosion of soil and to make it more and more stable. This method is the most effective method for controlling siltation, because when the soil erosion is reduced, the sedimentation problem is reduced automatically. But the methods of treating the catchment in order to minimise erosion are very costly. It has been estimated that the investment required for treating 16% of the Indian catchment area is Rs. 1,000 crores. In India, only 1.5% of the catchment area has been treated to minimise silting.

Example 18.16. *The following information is available regarding the relationship between trap efficiency and capacity inflow ratio.*

Capacity inflow ratio	*0.1*	*0.2*	*0.3*	*0.4*	*0.5*	*0.6*	*0.7*	*0.8*	*0.9*	*1.0*
Trap efficiency percent	*87*	*93*	*95*	*95.5*	*96*	*96.5*	*97*	*97*	*97*	*97.5*

Find the probable life of the reservoir with an initial reservoir capacity of 30 million cubic metres, if the average annual flood inflow is 60 million cubic metres and the average annual sediment inflow is 2,00,000 tonnes. Assume a specific weight of the sediment equal to 1.2 gm per c.c. The usual life of the reservoir will terminate when 80% of its initial capacity is filled with sediment.

Solution.

Average annual sediment inflow = 2,00,000 tonnes

$$= 2 \times 10^5 \text{ tonnes} = 2 \times 10^{11} \text{ gm}$$

Volume of average annual sediment inflow

$$= \frac{2 \times 10^{11}}{1.2} \text{ c.c.} = \frac{2 \times 10^{11}}{1.2 \times 10^6} \text{ m}^3$$

$$= \frac{0.2}{1.2} \times 10^6 \text{ cubic metres} = \frac{1}{6} \text{ million cubic metre} = \frac{1}{6} \text{ M.m}^3$$

Initial Reservoir Capacity $= 30 \text{ M.m}^3$

Annual flood inflow $= 60 \text{ M.m}^3$

Let us assume that 20% of the capacity, *i.e.* 6 M.m^3 is filled up in the first interval.

Capacity inflow ratio at the start of the interval $= \frac{30}{60} = 0.5$

Trap efficiency at the start of the interval = 0.96.

Capacity inflow ratio at the end of the interval $= \frac{24}{60} = 0.4$

Trap efficiency at the end of interval = 0.955

Average trap efficiency during the interval $= \frac{0.96 + 0.955}{2} = \mathbf{0.9575.}$

Volume of sediment deposited annually till the 20% capacity is filled

$$= \frac{1}{6} \times 0.9575 \text{ M.m}^3$$

∴ No. of years during which 20% of the capacity, *i.e.* 6 M.m^3 shall be filled up

$$= \frac{6}{\frac{1}{6} \times 0.9575} \text{ years} = \frac{36}{0.9575} = \mathbf{37.6 \text{ years}}$$

Similarly, in the 2nd interval

Capacity inflow ratio at the start $= \frac{24}{60} = 0.4$

Capacity inflow ratio at the end $= \frac{18}{60} = 0.3$

Trap efficiency at the start $= 0.955$
Trap efficiency at the end $= 0.95$
Average trap efficiency $= \mathbf{0.9525.}$

∴ No. of years during which the next 20% of capacity shall be filled up

$$= \frac{6}{\frac{1}{6} \times 0.9525} = \frac{36}{0.9525} = \mathbf{37.8 \text{ years.}}$$

Similarly, in the 3rd interval

Capacity inflow ratio at start $= \frac{18}{60} = 0.3$

Capacity inflow ratio at the end $= \frac{12}{60} = 0.2$

Trap efficiency at the start $= 0.95$
Trap efficiency at the end $= 0.93$
Average trap efficiency during the interval $= 0.94$

No. of years during which the next 20% of the capacity shall be filled up

$$= \frac{6}{\frac{1}{6} \times 0.94} = \frac{36}{0.94} = \mathbf{38.3 \text{ years.}}$$

Similarly, in the 4th interval

Capacity inflow ratio at the start $= 12/60 = 0.2$
Capacity inflow ratio at the end $= 6/60 = 0.1$
Trap efficiency at the start $= 0.93$
Trap efficiency at the end $= 0.87$

Average trap efficiency during the interval $= 0.90$

No. of years during which the next 20% of the capacity shall be filled up

$$= \frac{6}{\frac{1}{6} \times 0.9} = \frac{36}{0.9} = \mathbf{40 \text{ years.}}$$

Total probable life till 80% capacity gets filled up

$$= 37.6 + 37.8 + 38.3 + 40.0 = \mathbf{153.7 \text{ years.}} \quad \textbf{Ans.}$$

The above calculations of dividing the entire capacity into intervals (20% each in the above case) can also be carried out in a tabular form, as shown below in Table 18.28.

Table 18.28

Capacity % (1)	Capacity Vol. in M. cum. (2)	Capacity/inflow $= \frac{\text{Col. (2)}}{60 \text{ Mcum}}$ (3)	Trap efficiency η (4)	Av. Trap eff. η_{av} during the interval (5)	Sediment trapped per year; Col. (5) × Av. annual sediment inflow = Col. 5 × $\frac{1}{6}$ M.cum. (6)	Years reqd. to fill up 20% capacity (6 Mcum.) $= \frac{6}{\text{Col. (6)}}$ in years (7)
100	30	0.5	0.96			
				0.9575	0.1596	36.6
80	24	0.4	0.955			
				0.9525	0.1588	37.8
60	18	0.3	0.95			
				0.94	0.1566	38.3
40	12	0.2	0.93			
				0.90	0.15	40.0
20		0.1	0.8			
						Σ = 153.7 yrs

Example 18.17. *A proposed reservoir has a capacity of 500 ha-m. The catchment area is 125 km², and the annual streamflow averages 12 cm of runoff. If the annual sediment production is 0.03 ha.m/km², what is the probable life of the reservoir before its capacity is reduced by 10% of its initial capacity by sedimentation? The relationship between trap efficiency η (%) and capacity inflow ratio C/I, is as under :*

C/I	0.01	0.02	0.04	0.06	0.08	0.1	0.2	0.3	0.5	0.7
η%	43	60	74	80	84	87	93	95	96	97

(U.P.S.C., Civil Services, 1987)

Solution. Av. annual streamflow = 12 cm of runoff

Area of catchment $= 125 \text{ km}^2 = 125 \times 10^6 \text{ m}^2$

∴ Annual flood inflow $= (125 \times 10^6) \cdot \frac{12}{100} \text{ m}^3$

$= 15 \times 10^6 \text{ m}^3 = \mathbf{15 \text{ M.m}^3 \text{ (Mcum)}}$.

Annual sediment inflow = 0.03 ha-m/km² of the catchment

$= 0.03 \times 125 \text{ ha–m} = 0.03 \times 125 \times 10^4 \text{ m}^3 = 3.75 \times 10^4 \text{ m}^3$

$= \frac{3.75}{100} \times 10^6 \text{ m}^3 = \mathbf{0.0375 \text{ M–m}^3 \text{ (Mcum)}}$.

It means that 0.0375 Mcum of sediment flows every year into the dam/reservoir site, but the quantum of this, which is trapped in the reservoir, depends upon the average trap efficiency (η) during that year, and this trap efficiency, in turn, depends upon the capacity/inflow ratio.

In the question, the total capacity to be filled up by sediment is 10% of the initial reservoir capacity,

i.e. 10% × 5 Mcum = 0.5 Mcum.

Now, we have to calculate the time during which this 0.5 Mcum of sediment will get deposited in the reservoir, as follows :

Capacity of the reservoir at the start = 5 Mcum

Capacity of the reservoir at the end

(*i.e.*, when 0.5 Mcum of sediment is filled up) = 4.5 Mcum

$\therefore$ Capacity/inflow at the start $= \dfrac{5\text{ Mcum}}{15\text{ Mcum}} = 0.333$

η at start = 95%. Capacity/inflow at the end $= \dfrac{4.5}{15} = 0.30$

η at the end of the interval = 95%. Average η = 95%

$\therefore$ Sediment load trapped/yr.

$= 0.0375 \times 95\% = 0.035625$

$\therefore$ No. of years during which 0.5 Mcum of sediment will get trapped

$= \dfrac{0.5}{0.035625}$ years = 14.04 years ; say **14 years.**

Hence, after 14 years, 10% reservoir capacity will get filled up. **Ans.**

18.14. Estimating Sediment Load likely to Enter a Proposed Reservoir

The quantum of sediment flowing into a dam reservoir along with river runoff primarily depends upon the erosion characteristics of its catchment and the characteristics of the rainfalls that produce the run off entering the reservoir.

The sediment is basically produced by rains by the process of sheet erosion. The flowing water is the most active agent for erosion of soil from the land. Other agents like *wind, gravity, ice* and *human activities* do help in the erosion process. The rain drops in itself, loosen the soil perticles and break the soil lumps. The action of flowing sheet of water on the land surface helps in eroding the top soil from the ground surface and transport it down to the channels. The erosion caused by rainfall and runoff, thus, constitutes of the following two parts :

(1) **Sheet erosion.** It includes the detachment of geological material from the land surface by the impact of raindrops, and its subsequent removal by overland flow; and

(2) **Channel erosion.** It includes river bank erosion and transportation of the materials by concentrated flow.

Human activities, like *overgrazing of grass land, cutting of forests, forest fires, ploughing of land,* and various *mining & other excavations* etc. have magnified the problem of water erosion in river channels.

18.14.1. Factors Affecting the Erodibility of a Soil. The factors which effect the erodibility of a soil are given below :

(*i*) *Particle size of soil.* Larger the size of soil particles, the lesser would be its chances for erosion.

(*ii*) *Land slope.* The greater is the land slope, the greater is the action of erosive agents, the optimum being at 40° slope.

(*iii*) *Vegetation.* The thicker is the vegetation cover over a soil, the lesser will be the scope of erosion of the soil from the area.

(*iv*) *Presence of Salt and Colloidal Matter in the Soil.* The binding materials like kaolinite, montmorillonite, biotite, etc., do help in increasing the force of cohesion between the soil particles, thereby reducing the erodibility of the soil.

(*v*) *Moisture Content of Soil.* The greater is the moisture content of the soil, the lesser is the scope of its erosion.

(*vi*) *Soil compaction.* The higher is the compaction of soil, the lesser is the chance of its erosion.

(*vii*) *Soil Properties.* The characteristics of the soil, such as soil texture, structure, stratification, permeability, composition etc. do affect the soil binding, which in turn will neutralise the force of weathering agents.

(*viii*) *Human Activities.* Human activities on the land like mining, agricultural operations, construction of projects, land use, etc. do increase the erosion from the given land.

(*ix*) *Rainfall characteristics.* The intensity, duration, quantity and distribution of rainfall over space and time are some of the important factors that affect sediment yield.

18.14.2. Estimation of Sheet Erosion. The sediment yield of a reservoir basically depends on sheet erosion which can be estimated by the following empirical equations :

(1) *Musgrave Equation.* Musgrave (1947) suggested the following equation to compute the annual gross sheet erosion from a catchment, on the basis of 19 widely scattered research stations in USA :

$$E = CR \cdot (S_0/10)^{1.35} (L/72)^{0.35} (P_{30}/1.25)^{1.75} \quad ...(18.15)$$

where E = Erosion of soil lost from the catchment in inches/year

C = the *soil erosion rate,* which varies from 0.43 to 0.53 inch/year depending on the soil type (which depends on texture and permeability of soil)

R = the *cover factor,* which varies from 0.95 for poorly covered land to 0.10 for row crops

S_0 = Land slope in percentage, the default being 10%

L = Length of the land slope in feet

P_{30} = The max. rainfall in inches having 30 min. duration and of 2 year frequency.

It was also stipulated that the erosion value of E computed above, in inch/year, can be multiplied by 150 to obtain the erosion value in ton/year/acre.

(2) *Universal Equation.* Agriculture Research Service of U. S. Deptt. of Agriculture developed a universal equation (1961) to predict erosion from **small catchments.** This universal equation is given as :

$$E_a = R_f \cdot K \cdot (LS)\, C_m \cdot P \quad ...(18.16)$$

where E_a = Average soil loss in ton/acre/year.

(*i*) R_f is **Rainfall-run off factor.** Its value takes into account the effect of rain drop impact as well as the resulting amount and rate of run off. It should also include the cumulative effects of many moderate sized storms as well as the effects of occasional severe storms.

The value of R_f is generally taken as equal to EI (**rainfall erosion index**), which is computed as :

$$E \cdot I = E \times I_{30} \quad ...(18.17)$$

where E = storm energy in 100 ft-ton/acre/inch

I_{30} = Max. 30-minute intensity in inch/hr.

The value of E is further computed as :

$$E = \frac{1}{100}\left[916 + 331 \log I\right] \quad ...(18.18)$$

where E = Storm energy in 100 ft-ton per acre per inch.

I = Storm intensity of the given storm with a limit of 3 inch/hr, since median drop size does not continue to grow beyond this limit.

The *EI* value can, thus, be computed by eqn (18.17) for a given storm. For a specified period, the individual storm *EI* values can be summed up, which provide a numerical measure of the erosive potential of the rainfall within that period (one year). *In this manner, the average annual total of the storm EI values in a particular area is obtained, which is called the rainfall-erosion index for that area,* which equals R_f.

(*ii*) K is **Soil erodibility factor.** Its value is determined experimentally for the given soil type. Representative values for different types of soils have been worked out and listed for different regions of USA by Soil Conservation Service of USA, as shown in Table 18.29.

Table 18.29. Computed *K* Values for Soils on Erosion Research Stations (After Wischmeier and Smith, 1978).

Soil	*Source of Data*	*Computed K*
Dunkirm silt loam	Geneva, NY	0.69[a]
Keene silt loam	Zanesville, OH	0.48
Shelby loam	Bethany, MO	0.41
Lodi loam	Blacksburg, VA	0.39
Fayette silt loam	LaCrosse, WI	0.38[a]
Cecil sandy clay loam	Watkinsville, GA	0.36
Marshall silt loam	Clarinda, IA	0.33
Ida silt loam	Castana, IA	0.33
Mansic clay loam	Hays, KS	0.32
Hagerstown silty clay loam	State College, PA	0.31[a]
Austin clay	Temple, TX	0.29
Mexico silt loam	McCredie, MO	0.28
Honeoye silt loam	Marcellus, NY	0.28[a]
Cecil sandy loam	Clemson, SC	0.28[a]
Ontario loam	Geneva, NY	0.27[a]
Cecil clay loam	Watkinsville, GA	0.26
Boswell fine sandy loam	Tyler, TX	0.25
Cecil sandy loam	Watkinsville, GA	0.23
Zaneis fine sandy loam	Guthrie, OK	0.22
Tifton loamy sand	Tifton, GA	0.10
Freehold loamy sand	Marlboro, NJ	0.08
Blath flaggy silt loam with surface stones > 2 inches removed	Arnot, NY	0.05[a]
Albia gravelly loam	Beemerville, NJ	0.03

a Evaluated from continuous fallow. All others were computed from rowcrop data.

For soils containing less than 70% of silt and very fine sand, K can be computed as :

$$K = \frac{1}{100}\left[2.1\, M^{1.14}\,(10^{-4})\,(12-a) + 3.25\,(b-2) + 2.5\,(c-3)\right] \quad ...(18.19)$$

where M = particle size parameter, defined as percent silt & very find sand (size 0.1 mm-0.002 mm) times the quantity (100-percent clay)

a = percent organic matter

b = soil texture code used in USDA soil classification

c = profile permeability class

[Note : When the silt fraction does not exceed 70%, erodibility varies approximately as the 1.14 power of M, but addition of organic matter content, soil structure, and profile-permeability class as done in Eqn. (18.19) improves the prediction accuracy.]

(*iii*) (*LS*) = **Topographic Factor.** It is the ratio of soil loss per unit area from a field slope to that from a 72.6 ft length of uniform 9% slope under otherwise identical conditions. For a specified slope and its length, *LS* can be computed as :

$$(LS) = \left(\frac{\lambda}{72.6}\right)^m (65.41 \sin^2\theta + 4.56 \sin\theta + 0.065) \qquad ...(18.20)$$

where λ = slope length in ft

θ = angle of slope

M = 0.5 if the per cent slope is 5 or more

m = 0.4 on slopes of 3.5 to 4.5%

m = 0.3 for slopes of 1 to 3%

m = 0.2 for uniform slopes of less than 1%.

(*iv*) C_m = **Soil Cover and Management Factor.** It measures the combined effect of all inter-related cover and management variables, including the type of vegetation, plant spacing, the stand, the quality of growth, crop sequence, tillage practices, crop residues, incorporated residues, land use residues, fertility treatment, etc.

Values of C_m for pasture, range, idle land, and woodland for a combination of cover conditions are given in Table 18.30.

Table 18.30. Factor C_m for permanent Pasture, Range, and Idle Land (After Wischmeier and Smith 1978).

Vegetative canopy		*Cover that contracts the soil surface*						
			Percent ground cover					
Type and height[b]	*Percent cover*[c]	*Type*[d]	*0*	*20*	*40*	*50*	*80*	*95+*
No appreciable canopy		G	0.45	0.20	0.10	0.042	0.013	0.003
		W	0.45	0.24	0.15	0.091	0.043	0.011
Tall weeds or short brush with the average drop fall height of 20 in.	25	G	0.36	0.17	0.09	0.038	0.013	0.003
		W	0.36	0.20	0.13	0.083	0.041	0.011
	50	G	0.26	0.13	0.07	0.035	0.120	0.003
		W	0.26	0.16	0.11	0.076	0.039	0.001
	75	G	0.17	0.10	0.06	0.032	0.011	0.003
		W	0.17	0.12	0.09	0.068	0.038	0.011
Appreciable brush or bushes, with the average drop fall height of 6 ½ ft	25	G	0.40	0.18	0.09	0.040	0.013	0.003
		W	0.40	0.22	0.14	0.087	0.042	0.011
	50	G	0.34	0.16	0.08	0.038	0.012	0.003
		W	0.34	0.19	0.13	0.082	0.042	0.011
	75	G	0.28	0.14	0.08	0.036	0.012	0.003
		W	0.28	0.17	0.12	0.078	0.040	0.011
Trees, but no appreciable low brush Average drop fall height of 13 ft	25	G	0.42	0.19	0.10	0.041	0.013	0.003
		W	0.42	0.23	0.14	0.089	0.042	0.011
	50	G	0.39	0.18	0.09	0.040	0.013	0.003
		W	0.39	0.21	0.14	0.087	0.042	0.011
	75	G	0.36	0.17	0.09	0.039	0.012	0.003
		W	0.36	0.20	0.13	0.084	0.041	0.011

[a] The listed *C* values assume that the vegetation and mulch are randomly distributed over the entire area.

[b] Canopy height is measured as the average fall height of water drops falling from the canopy to the ground. Canopy effect is inversely proportional to the drop fall height and is negligible if fall height exceeds 33 ft.

[c] Portion of total area surface that would be hidden from view by canopy in a vertical projection (a bird's-eye view).

[d] G: cover at the surface is grass, grasslike plants, decaying compacted duff, or litter at least 2 in. deep.
W: cover at the surface is mostly broadleaf herbaceous plants (as weeds with little lateral-root network near the surface) or undecayed residues or both.

(v) P = **Support Practice Factor.** Its value depends upon crop land practice such as contour tillage, *strip cropping on the contour*, and *terrace systems*. Values of P have been given for each of these practices by Wischmeier and Smith (19 7 8). In general, tillage and planting on the contours reduce erosion. Table 18.31 gives P values for contouring.

Table 18.31. P values (Eq. 18.16) and Slope-Length Limits for contouring (Wischmeier & Smith)

Land Slope %	*P Value*	*Maximum Length* (feet)*
1 – 2	0.60	400
3 – 5	0.50	300
6 – 8	0.50	200
9 – 12	0.60	120
13 – 16	0.70	80
17 – 20	0.80	60
21 – 25	0.90	50

* Limit may be increased by 25% for residue cover after crop seedlings will regularly exceed 50%.

Since it is difficult to get proper data for the use of Eq. (18.16) for a developing country like India, this equation is not popular in developing countries.

18.14.3. Sediment Measurement by Sample Recorder. The sediment produced by sheet erosion from a catchment may not always reach at the point of measurement; *i.e.* the site of dam reservoir. Some part of sediment may be deposited en-route. *The ratio between the yield of sediment at the measuring site and the gross erosion in the catchment is called the* **sediment delivery ratio.**

Thus, the **sediment yield** is the gross sediment yield minus the quantity of sediment deposited en route. The sediment yield, infact, is important, since it is this sediment which will get deposited in the reservoir, affecting its useful life.

The *sediment yield of a reservoir* can be calculated either by using an appropriate empirical equation, out of the ones developed by various investigators; or by developing a rational appropriate relation between inflow and sediment, on the basis of actual measurements of sediment load at the site of the reservoir. The continuous measurements of suspended load and bed load at the reservoir site for a number of years will help in developing a rational relation (either a mathematical equation or a graphical curve) between sediment and the inflow into the reservoir. We will first of all discuss the method of the actual measurement of sediment and developing an appropriate relation between sediment yield (q_s) and the river discharge (Q) at the given site.

18.14.3.1. Measuring suspended sediment load. Sediment samplers are used to measure suspended sediment at a given site on the river to obtain the most reliable results of sediment yield. The bed load should also be calculated either by using the available empirical equations*, or on an adhoc basic of 2.5-25% of the suspended load, as to calculate the total sediment load (sum of suspended load and bed load).

A typical sediment sampler used in India is shown in Fig. 18.26. A depth integrating hand sampler used for small streams is also shown in Fig. 18.27.

* Please see article 4.9.

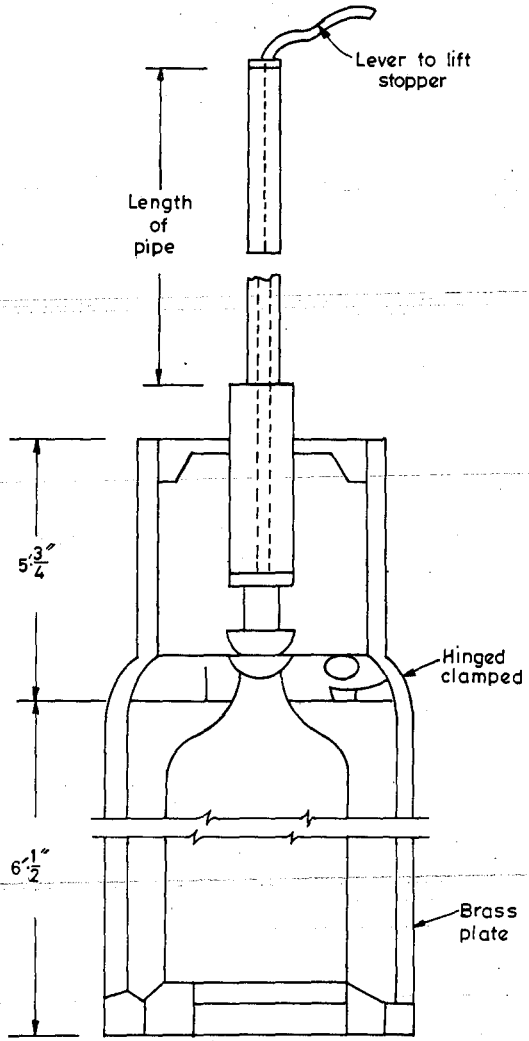


Fig. 18.26. Line diagram of a typical sediment sampler (Punjab bottle sampler).

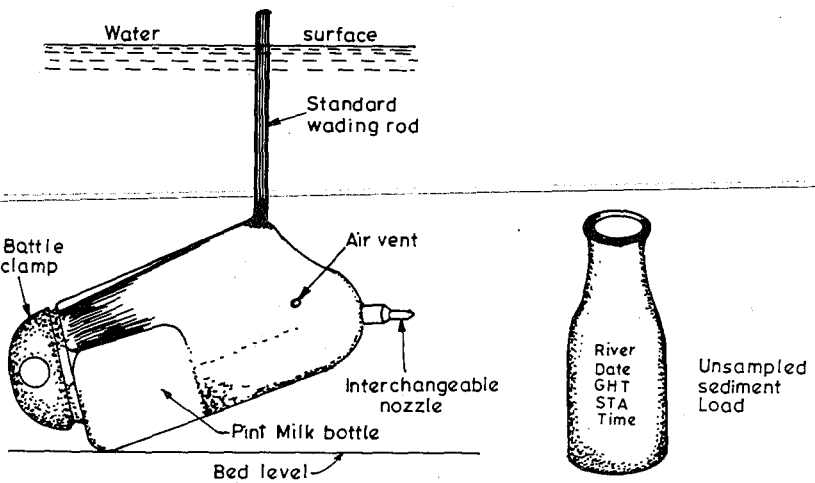


Fig. 18.27. AUS.DH-48 depth-integrating hand sampler for small streams.

The sampler is taken into the stream to a depth of 0.6y below the water surface, or at two depths at 0.2y and 0.8y depths to collect samples of sedimented water in the sampler bottle. The collected sample of sedimented water is then analysed in the laboratory either by a *gravimetric method*, or by *hydrometric method* to determine the quantum of coarse sediment (particle size higher than 0.2 mm), *medium size sediment* (particle size between 0.075 mm to 0.2 mm) and of *fine sediment* (particle size less than 0.075 mm), separately*. Their sum will indicate the total sediment load present in the given volume of water sample. Sediment load present in the water sample is then expressed in ppm (parts for million) as :

$$\text{Sediment load in ppm} = \frac{\text{Dry mass of sediment}}{\text{Total mass of original sample including the mass of sediment \& of water}} \times 10^6 \quad \text{...(18.21)}$$

This value can be converted into t/day by multiplying the average unit wt. of sediment (say 1.2 t/m^3) with the total volume of daily inflow in m^3.

When a large number of such sample records become available for the given site, then a curve can be plotted between the sediment load (q_s) in t/day on x-axis and daily discharge in m^3/s on y-axis, as to obtain a curve known as *sediment rating curve,* as shown in Fig 18.28. Such a plotting is usually done on a log-log paper.

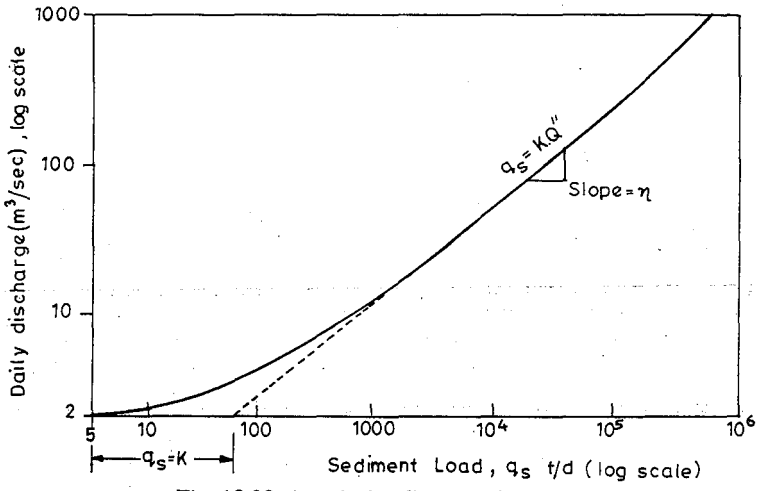


Fig. 18.28. A typical sediment rating curve.

When a standard sediment rating curve is established for a given site, the sediment yield (q_s) can be read out by simply knowing the discharge rate only. However, care must be taken to see that for different seasons of the year, different curves will have to be developed, since the sediment yield of a basin may vary with the season. The curve, for say March (non-monsoon period) may be entirely different from the curve for the month of say August (monsoon season).

* The sample collected by the sampler is first passed through a BSS-100 sieve and the coarse particles retained are taken out and oven dried. Thus, the quantity of coarse sediment (higher than 0.2 mm size) is obtained. Sedimented water passing 200 μ sieve is allowed to stand for 20 minutes, so that the finer particles settle down. The settled mass is removed by the process of decantation (pouring out water from the settled tank). The settled residue is dried and weighed, as to get the mass of fine sediment (0.075 mm – 0.2 mm). Poured water contains sediment of still finer particles. To isolate this, the sample is filtered through a filter paper, and the quantity retained therein is dried and weighed. This gives the mass of fine sediment (particle size < 0.075 mm). The summation of all the three masses will give the total mass of sediment.

The sediment rating curves can be used to compute daily sediment load for the given daily discharge values, and their summation can give us the monthly or the annual sediment load.

18.14.3.2. Mathematical equation for a sediment rating curve. The sediment rating curve (straight line portion) giving sediment load in tonnes/day (q_s) w.r. to daily discharge (Q) in m^3/sec can be expressed by a mathematical equation of the form :

$$q_s = K \cdot Q^n \qquad ...(18.22)$$

Taking log on both sides,

$$\log q_s = \log K + n \cdot \log Q$$

This equation is similar to the form

$$y = a + bx,$$

where $\log q_s$ is plotted on y-axis and $\log Q$ on x-axis.

$$n = b \qquad ...(18.23)$$

$$\log K = a \qquad ...(18.24)$$

A mathematical solution for such an equation can be obtained by the statistical **method of least squares,** where in the various known values of x (*i.e.* $\log Q$) and y (*i.e.* $\log q_s$) are analysed to estimate a and b values as :

$$a = \frac{\Sigma y \cdot \Sigma x^2 - \Sigma x \cdot \Sigma xy}{N \Sigma x^2 - (\Sigma x)^2} \qquad ...(18.25)$$

$$b = \frac{N \Sigma xy - \Sigma x \cdot \Sigma y}{N \cdot \Sigma x^2 - (\Sigma x)^2} \qquad ...(18.26)$$

By computing values of a and b, values of n and K become known to finally compute the relation between q_s and Q by Eq. (18.22).

The use of this method will become clear when we solve example 18.18.

Example 18.18. *A reservoir has the following sediment and discharge data :*

Year	*81*	*82*	*83*	*84*	*85*	*86*	*87*	*88*
Discharge (M m^3)	*1430*	*3850*	*2050*	*6510*	*2880*	*1120*	*6050*	*2220*
Sediment load (M.t) as measured by silt sampler	*2.65*	*5.82*	*3.60*	*7.15*	*5.22*	*1.95*	*6.88*	*3.94*

Calculate the average total sediment load/year/100 sq. km of the catchment at the site. Develop a regression relation and predict the total and observed sediment yield for the inflow of 3450 M.m^3 for the year 1978. Take the catchment area at the site as 3050 sq. km. What is the total sediment yield for 100 years ? Assume bed load as 10% of suspended load.

Solution. Average sediment load measured by silt sampler

$$= \frac{2.65 + 5.82 + 3.6 + 7.15 + 5.22 + 1.95 + 6.88 + 3.94}{8}$$

$$= \frac{37.21}{8} \text{ Mt/year} = 4.651 \text{ Mt/yr.}$$

Since sediment load is measured by a silt sampler, the given values of sediment are of suspended load.

Now, assume Bed load = 10% of suspended load

$\therefore$ Total load = Suspended load + Bed load

$\therefore$ Total sediment load $= 1.1 \times 4.651 = 5.116$

or $(q_s) = 5.116$ Mt/yr.

Sediment deposited in a reservoir consolidates gradually due to the increasing silt load on it every year and the wt. of water above it. Assuming the average sediment unit weight (sediment in a reservoir consists of sand, silt and clay in water) as 1.2 t/m^3, we have

$\therefore$ Total silt load/year $= 5.116$ Mt

$$q_s = \frac{5.116}{1.2} \text{ M·m}^3 = \mathbf{4.263 \text{ M m}^3}$$

Catchment area $= 3050$ sq. km

$$\therefore \text{ Sediment load/100 sq. km/year} = \frac{4.263 \text{ M m}^3}{3050} \times 100$$

$$= \mathbf{0.1397 \text{ M m}^3/100 \text{ sq. km/year}} \quad \textbf{Ans.}$$

Total sediment yield in 100 years $= 4.263$ M m^3/yr. $\times$ 100 yr. $= \mathbf{426.3 \text{ M m}^3}$ **Ans.**

In order to develop a regression type non-linear equation between total sediment load (q_s) and discharge (Q), represented as $q_s = K \cdot Q^n$, we carry out the required calculations in table 18.32 to evaluate the values of a and b by Eqn. (18.25) and (18.26), as :

Table 18.32. Computations to develop Equation $y = a + bx$ by the method of least squares.

year	*Discharge* Q ($M \cdot m^3$)	*Suspended Sediment measured in M.t*	*Total sediment i.e.* q_s *in M m*^{3} $col\,(3) \times \frac{1.1^*}{1.2}$	$\log Q = x$ $\log col\,(2)$	x^2 $(col\,5)^2$	$\log q_s = y$ $\log col\,(4)$	$x \cdot y$ $col\,(5) \times col\,(7)$
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*	*(8)*
81	1436	2.65	2.429	3.1553	9.9561	0.3855	1.2164
82	3850	5.82	5.335	3.5855	12.8555	0.7271	2.6070
83	2050	3.60	3.300	3.3118	10.9677	0.5185	1.7172
84	6510	7.15	6.554	3.8136	14.5433	0.8165	3.1138
85	2880	5.22	4.785	3.4594	11.9674	0.6799	2.3520
86	1120	1.95	1.788	3.0492	9.2977	0.2522	0.7690
87	6050	6.88	6.307	3.7818	14.3017	0.7998	3.0247
88	2220	3.94	3.612	3.3464	11.1981	0.5577	1.8663
			Σ	27.503	95.0875	4.7372	16.6664

* q_s (Total sediment in Mm^3) $= \dfrac{q_s \text{ in M} \cdot \text{t}}{1.2 \text{ t/m}^3} \times (1.1)$** $= \left(col\,(3) \times \dfrac{1.1}{1.2}\right)$

** Total sediment = 1.1 (Observed sediment by soil sampler)

Using $$a = \frac{\Sigma y \cdot \Sigma x^2 - \Sigma x \cdot \Sigma xy}{N \Sigma x^2 - (\Sigma x)^2}$$ ...(*i.e.* 18.25)

we get $$a = \frac{4.7372 \times 95.0875 - 27.503 \times 16.6664}{8 \times 95.0875 - (27.503)^2}$$

or $$a = \frac{-7.9275}{4.2850} = -1.8500$$

But $\log K = a = -1.8500$ $\therefore$ $K = 0.014$

Also $$b = \frac{N \cdot \Sigma xy - \Sigma x \cdot \Sigma y}{N \cdot \Sigma x^2 - (\Sigma x)^2}$$ ...(*i.e.* 18.26)

$$= \frac{8 \times 16.6664 - 27.503 \times 4.7372}{8 \times 95.0875 - (27.503)^2} = \frac{3.0440}{4.2850} = 0.7104$$

But $b = n = 0.7104$

Regression equation is hence given as :

$$q_s = K \cdot Q^n = 0.014\,(Q)^{0.7104} \quad \textbf{Ans.}$$

Total sediment yield for the year 1978 having inflow of 3450 M m^3

$$= 0.014\,(3450)^{0.7104} = \mathbf{4.565\ M\ m^3}$$

$$= 4.565\ \text{M m}^3 \times 1.2\ \text{t/m}^3 = \mathbf{5.48\ Mt} \quad \textbf{Ans.}$$

Observed or suspended sediment load for the year 1978

$$= \frac{5.48}{1.1}\ \text{Mt} = \mathbf{4.98\ Mt} \quad \textbf{Ans.}$$

18.14.3.3. Empirical equations for total sediment yield. The following empirical equations have been developed by several investigators, for estimating the annual sediment yield of a reservoir.

(1) *Swami's Regression Equation.* Swamy and Garde (1977) have proposed a relation correlating the cumulative volume of sediment deposited in a reservoir with the cumulative volume of water inflow, and initial bed slope of the river, as

$$V_s = C \cdot B \cdot (V_{ci})^{0.94}\,(S_0)^{0.84} \qquad ...(18.27)$$

where V_s = cumulative vol. of sediment deposited in the reservoir in M · m^3

C = Regression constant with safe value of the order of 1.16. However a value less than 1.16 may be adopted depending on the reservoir

B = Width of the reservoir at full reservoir level in m

V_{ci} = Cumulative vol. of inflow per unit width B of the reservoir

S_o = Bed slope of the river.

(2) *Jogelkar's Equation.* An equation proposed by Jogelkar (1960) is given as :

$$Q_s = 0.59\,(A)^{-0.24} \qquad ...(18.28)$$

where Q_s = Annual silting rate in M · m^3 per 100 sq. km of catchment area

A = Catchment area in sq km

(3) *Khosla's Equation.* Khosla (1953) has proposed the following empirical equation.

$$Q_s = 0.323\,(A)^{-0.28} \qquad ...(18.29)$$

where Q_s = Annual siltation rate in $M \cdot m^3/100$ s.q km/year

A = Catchment area in sq. km.

This equation always gives lower estimate of sediment yield at a site.

(4) *Varshneys Equations.* Varshney and Raichur have proposed the following equations for calculating sediment yield for an ungauged basin.

(*i*) *Up to 130 sq. km catchment for mountainous rivers*

$$Q_s = 0.395\,(A)^{-0.311} \qquad ...(18.30)$$

where Q_s = Annual sediment yield rate in $M \cdot m^3$ per 100 sq. km of catchment

A = catchment area in km^2.

(*ii*) *Rivers draining plain area up to 130 sq. km*

$$Q_s = 0.392\,(A)^{-0.302} \qquad ...(18.31)$$

Q_s & A have the same meaning as in Eqn. (18.30).

(*iii*) *For area greater than 130 sq. km for North Indian catchment*

$$Q_s = 1.534\,(A)^{-0.264} \qquad ...(18.32)$$

Q_s & A have the same meaning as in Eq. (18.30).

(*iv*) *For South Indian Rivers up to 130 sq. km*

$$Q_s = 0.46\,(A)^{-0.468} \qquad ...(18.33)$$

Q_s & A have the same meaning as in Eqn. (18.30).

(*v*) *For areas greater than 130 sq. km for South Indian catchments*

$$Q_s = 0.277\,(A)^{-0.194} \qquad ...(18.34)$$

Q_s & A have the same meaning as in Eqn. (18.30).

(5) *Using Known Data of Similar Catchment.* Sediment yield of an unmeasured watershed Q_{s_2} can be computed from sediment yield of measured catchment Q_{s_1} of similar topography, land cover and land use, on area proportion basis, as

$$Qs_2 = Qs_1 \left(\frac{A_2}{A_1}\right) \qquad ...(18.35)$$

Example 18.19. *Estimate the sediment load in tonne at the proposed dam site in North India with the following data using various empirical equations :*

Catchment area = 1830 sq. km

Width of reservoir at FRL = 560.0 m

River slope at the dam site = 0.006

Average inflows at the site are as follows :

Year	*1982*	*1983*	*1984*	*1985*	*1986*	*1987*	*1988*	*1989*	*1990*
Inflow $M \cdot m^3$	*2210*	*1290*	*1640*	*1780*	*2150*	*1980*	*2540*	*1285*	*1620*

Assume annual siltation rate per 100 sq. km from a similar catchment of 3050 sq. km to be 10.35 M · m³/100 sq. km.

Solution. The sedimentation rate is worked out by using various equations as described in the previous article.

(1) *By Swamy's Regression Method :* The computation is carried out in Table 18.28 by using Eqn. (18.27) as :

$$V_s = C \cdot B\,(V_{ci})^{0.94}\,(S_0)^{0.84}$$

where $C = 1.16$

$B = 560$ m

$S_0 = 0.006$

$\therefore$ V_s = cumulative vol. of sediment deposited in M · m³

$$= 1.16 \times 560\,(V_{ci})^{0.94} \times (0.006)^{0.84}$$

$$= 8.837\,(V_{ci})^{0.94}$$

where V_{ci} is the cumulative vol. of annual inflows per unit width B of reservoir, over the given years.

The computations are carried out in Table 18.33, which is self explanatory.

Table 18.33. Computation of sediment Load by Swamy's Regression Method

S. No. (1)	*Year* (2)	*Inflow in M · m³* (3)	*Cumulative inflow in M m³* (4)	V_{ci} = *cumulative inflow per unit width B of reservoir* = $\frac{col\,(4)}{560}$ (5)	$V_s = 8.837\,(V_{ci})^{0.94} = 8.837\,(col\ 5)^{0.94}$ *M · m³* (6)
1	1982	2210	2210	3.946	32.11
2	1983	1290	3500	6.250	49.48
3	1984	1640	5140	9.179	71.01
4	1985	1780	6920	12.357	93.91
5	1986	2150	9070	16.196	121.10
6	1987	1980	11050	19.732	145.80
7	1988	2540	13590	24.268	177.11
8	1989	1285	14875	26.563	192.81
9	1990	1620	16495	29.455	**212.48**

Q_s = sedimentation rate per 100 sq. km

$$= \frac{V_s \text{ (i.e. cumulative vol. of silt)}}{\text{No. of years (i.e. 9)}} \times \frac{100 \text{ sq. km}}{C \cdot A \text{ of } 1830 \text{ km}^2}$$

$$= \frac{212.48}{9} \times \frac{100}{1830}$$

or Q_s = **1.29 M m³/100 sq. km Ans.**

(2) *By Jogelkar's Equation*

$$Q_s = 0.59\,(A)^{-0.24}$$

$$= 0.59\,(1830)^{-0.24}$$

$$= \mathbf{0.10\ M\ m^3/100\ sq.\ km} \quad \textbf{Ans.}$$

(3) *By Khosla's Equation*

$$Q_s = 0.323\,(A)^{-0.28}$$

$$= 0.323\,(1830)^{-0.28}$$

$$= \mathbf{0.04\ M\ m^3/100\ sq.\ km} \quad \textbf{Ans.}$$

(4) *By Varshneys Equation for Norths Indian catchment exceeding 130 sq. km.*

$$Q_s = 1.534\,(A)^{-0.264}$$

$$= 1.534\,(1830)^{-0.264}$$

$$= \mathbf{0.211\ M\ m^3/100\ sq.\ km} \quad \textbf{Ans.}$$

(5) *From similar catchment.*

$Q_{s_1} = 4.56$ M m³/100 sq. km

$A_1 = 3050$ sq. km

$A_2 = 1830$ sq. km

$$Q_{s_2} = Q_{s_1} \times \left(\frac{A_2}{A_1}\right)$$

$$= 0.35 \times \left(\frac{1830}{3050}\right) \text{M m}^3/100 \text{ sq. km}$$

$$= \mathbf{0.21\ M\ m^3/100\ sq.\ km} \quad \textbf{Ans.}$$

18.15. Reservoir Sedimentation Studies on Existing Reservoirs

Sedimentation of storage reservoirs is a natural process, since large part of the silt eroded from the catchment and transported by the river, gets deposited on the bed of the reservoir. This causes reduction in the live as well as dead storage capacities of the reservoir. Progressive loss of capacity due to sediment accumulation results in reduced benefits and may even cause operational problems. *It, therefore, becomes necessary to monitor the sedimentation rates in the existing reservoirs at regular intervals, to help in planning and executing suitable remedial measures for controlling sedimentation in order to prolong the life of the reservoir and its benefits.*

Regular monitoring and updating of elevation-capacity curve of the reservoir immensely helps in better water management. With this aim, *conventional hydrographic surveys* are conducted at regular intervals at the existing reservoirs to determine the available capacities at different elevations, and to help compute the sedimentation volume at such regular intervals. Such conventional surveys will require computation

of water spread area at different water levels, and is quite a tedious and a costly process. **Remote sensing techniques** do offer a modern answer to the costly conventional surveys, as it offers a great potential for application in capacity evaluation of medium to large reservoirs. From the data provided by the remote sensing satellites, it has now been possible to compute loss of reservoir capacity due to sedimentation, and its distribution. The results obtained from this technique have been found to be quite comparable with those obtained from the costly and cumbersome conventional methods. One of the greatest advantage of this technique is that the capacity evaluation could be easily computed on yearly basis.

The methodology involved in this technique requires the use and analysis of the *satellite imageries* provided by the Remote Sensing satellites, which collect the data of the Earth surface features in different bands at regular intervals. In the case of Indian Remote Sensing Satellites IRS-1A and IRS-1B* (both identical satellites), this periodicity is 22 days for Indian sub-continent. These two satellites together are thus capable to provide us data of all our reservoirs at 11 days interval. The third satellite of this series IRS-1C has also been launched recently on 28.01.1996 and helps in providing better pictures even of cloud bound areas. IRS-1D has further been launched on 29.09.1997, through our first Indian rocket launcher.

Due to the water withdrawals from an existing reservoir, its water spread area goes on changing throughout the year. The reservoirs are generally full just after the monsoon period in October, and get depleted to almost dead storage/minimum drawdown level just before start of monsoon season in May or June, every year. The satellite data of various dates during the period from October to May, provide us an array of water spread areas between maximum water level (i.e. around the FRL) and the minimum reservoir level (i.e. around the dead storage level/minimum drawdown level). From the whole set of the satellite data, a few of them which are cloud free and of good quality and representative of the whole range of reservoir levels at close intervals, are selected for analysis.

The method of analysis depends upon the data products. The selected CCT's/FCC's of various dates are analysed for determining the water spread areas. The corresponding water levels are obtained from the daily gauge record of the reservoir. From these, the water volumes between two consecutive water levels are computed using Prismoidal or any appropriate formula. Volume of water below the minimum water level (as recorded by the satellite) and the "new zero"** elevation, are estimated based upon the previous hydrographic surveys. In case, these informations are not available in the hydrographic survey data, then the elevation-area relationship obtained from the hydrographic surveys as well as the one obtained from the satellite data interpretation, should be extended to get the new zero elevation, and then the volume between the minimum mapped water level and this new zero level is estimated. After this, the cumulative water volume at each reservoir level is computed, and then the revised elevation-area-capacity curve is drawn. By comparing the original area-capacity curve or any other such curve, the total sediment volume and its distribution can be computed.

18.16. Observed Sedimentation Rates for Various Important Indian Reservoirs

The observed sedimentation rates for various important dam reservoirs in India are indicated in col. (6) & (7) of Table 18.34.

* IRS-1A was launched on 19.03.1988, and IRS-1B was launched on 23.08.1991.

** The minimum reservoir bed level is raised due to sedimentation, which is termed as the new zero.

Table 18.34. Annual Sedimentation Rates of Various Indian Reservoirs

S. No. (1)	*Name and Location of reservoir* (2)	*Catchment area in 100 sq. km* (3)	*Capacity of reservoir at FRL in* $M \cdot m^3$ (4)	*Surface area at MRL* $M \cdot m^2$ (5)	*Annual sediment rate in ham/100 sq. m* (6)	*Annual volume of sediment deposit* $M \cdot m^3$ $\frac{col\ (6)}{100} \times col\ (3)$ (7)	*Dead storage capacity provided in* $M \cdot m^3$ (8)
1	Bhakra *i.e.* Govind Sagar) (HP)	568	9351	169	6.00	**34.08**	95
2	Gandhi Sagar (MP)	226	7413	660	10.08	22.71	586
3	Hirakund (Orissa)	834	8146	725	3.89	32.44	2318
4	Lower Bhawani (TN)	61.5	929	—	4.10	2.52	0
5	Maithan (DVC)	63	1275	—	13.02	8.20	165
6	Matatila (UP)	207.5	883	—	3.50	7.26	50
7	Mayurkshi (WB)	18.6	617	—	**20.09**	3.74	68
8	Nizam Sagar (AP)	216.94	715	130	6.57	14.25	—
8	Panchat (DVC)	111	1475	153	9.92	11.01	170
10	Ramganga (UP)	30.76	2448	—	17.30	5.32	395
11	Sriram Sagar or Shivaji Sagar (AP)	8.19	3454	—	15.20	1.24	849
12	Tawa (MP)	—	2312	—	8.10	—	263

18.17. Reservoir Losses

Huge quantity of water is generally lost from an impounding reservoir due to *evaporation, absorption,* and *percolation.* Depending upon which, the following losses may occur from such a reservoir :

1. *Evaporation losses ;*
2. *Absorption losses ;* and
3. *Percolation losses* or *Reservoir leakage.*

These losses are discussed below.

18.17.1. Evaporation Losses. The evaporation losses from a reservoir depend upon several factors, such as : *water surface area, water depth, humidity, wind velocity, temperature, atmospheric pressure* and *quality of water* as discussed in article 7.34.2.3. The evaporation loss from a reservoir under the given atmospheric conditions can be easily estimated by measuring the standard pan evaporation and multiplying the same by the pan coefficient. The pan coefficients and various types of pans in use are given in article 7.34.3.4.

The evaporation losses become very significant in a hot and humid country like India ; and realistic estimation of these losses is quite important. These losses in fact vary from place to place and from season to season, and hence monthly values of these losses are usually determined. Typical average values of these losses for North and South India are given in table 18.35.

Table 18.35. Monthly Reservoir Evaporation Losses

Month	*Losses in cm*	
	North India	*South and Central India*
January	7	10
February	9	10
March	13	18
April	16	23
May	27	25
June	24	18
July	18	15
August	14	15
September	14	15
October	13	13
November	9	10
December	8	10
Total	172	182

On the basis of a review conducted on 130 sample reservoirs, the Central Water Commission, in 1990, has, however, estimated the *average annual evaporation loss to be 225 cm ;* and the total water lost from all the existing reservoirs to be 27000 Mm^3 per annum. What a tremendous waste of precious water!

In order to control such large scale wastage of water, several methods have been devised by engineers and scientists. All these methods are based upon the efforts made to reduce the evaporation rate from the surface of the water bodies by physical or chemical means, since the basic meteorological factors affecting evaporation cannot be controlled under normal conditions. *The following methods are generally used for evaporation control :*

1. Wind breakers
2. Covering of the water surface
3. Reduction of the exposed water surface
4. Use of underground storage rather than the use of surface storage
5. Integrated operation of reservoirs
6. Use of chemicals for retarding the evaporation rate from the reservoir surface.

Out of all these methods, the last method has evoked the maximum response from all over the world, and has been considered to be the only practical solution for conservation of fresh water, inspite of its various limitations and disadvantages in high cost of application in normal conditions. The use of chemicals, called **Water Evapo-Retardants** (WERs), for controlling the evaporation rate from the surface of reservoirs is therefore, discussed here in details.

A non toxic chemical, capable of forming a thin *monomolecular film* over the water surface, is generally spread over the reservoir water surface in powder, liquid or emulsion form. *The resulting film prevents energy inputs from the atmosphere, thus reducing evaporation. Such a film, however, allows the passage of enough air through it, to avoid any harmful effects on the aquatic life due to shortage of oxygen.*

Fatty alcohols of different grades like : Cetyl alcohol ($C_{16}.H_{33}.OH$) popularly called *hexa decanol*, Stearyl alcohol ($C_{18}.H_{37}.OH$) popularly called *Octadecanol*, and Behenyl alcohol ($C_{22}.H_{45}.OH$) called *docosanol*, or a mixture of these chemicals, have been generally used and found to be quite suitable. These chemicals should, however, be 99% pure for getting the desired properties of *monolayer*. National Chemical Laboratory, Pune, has developed one more compound by synthesising alkoxy ethanols.

In general, all such chemical compounds should possess the *following properties* :

(*i*) the chemical compound (WER) should be tasteless, odourless, non-toxic, non-inflammable, and should not produce any effect on the quality of water.

(*ii*) the chemical should easily spread and form an even compact cohesive and efficient monomolecular film on the water surface.

(*iii*) the thin film formed by the chemical should be pervious to oxygen and carbon dioxide, but tight enough to prevent escape of water molecules.

(*iv*) the thin film formed by the chemical should be durable, and should be able to re-seal itself, when broken due to external disturbances such as wind, waves, etc.

(*v*) the chemical and the film formed by it should not be adversely affected by the water borne bacteria, proteins and other impurities present in the water body.

The use of chemical WERs has, however not been found to be cost effective for mass scale use, and has further not been found to be suitable under the following conditions :

(*a*) when the wind velocities exceed 10 km/hr or so.

(*b*) when the temperature rises above 40°C or so.

(*c*) when the size of the water body is relatively large.

Development of cheaper WERs capable of withstanding higher wind speeds upto about 20 km/hr and having strong cohesive forces and properties of self spreading and re-uniting to maintain the monolayer in resilient state even at high wind velocities, is therefore of vital importance. Moreover, the life of the film formed, must be longer, so as to reduce the frequency of application to about 3 to 7 days from its present frequency of 24 hours. Development of such chemical WERs is the subject matter of present research.

Other long term evaporation control measures like *plantation of trees to act as wind breakers**, *reduction of exposed water surface by covers, underground storage of water, integrated operation of reservoirs*, etc. have been employed in some parts of the country. The effectiveness and economics of these methods are, however, yet to be established.

In India, the water conservation methods are presently being adopted only in draught prone and scarcity areas, since large scale use of such methods on all the reservoirs of the country is not found to be economical or practically unfeasible due to their large size and adverse meteorological factors.

18.17.2. Absorption Losses. These losses do not play any significant role in planning, since their amount, though sometimes large in the beginning, falls considerably as the pores get saturated. They certainly depend upon the type of soil forming the reservoir.

18.17.3. Percolation Losses or Reservoir Leakage. For most of the reservoirs, the banks are permeable but the permeability is so low that the leakage is of no

* Trees having lesser evapotranspiration should only be chosen and identified.

importance. But in certain particular cases, when the walls of the reservoir are made of badly fractured rocks or having continuous seams of porous strata, serious leakage may occur. Sometimes, pressure grouting may have to be used to seal the fractured rocks. The cost of grouting has to be accounted in the economic studies of the project, if the leakage is large.

18.18. Reservoir Clearance

The removal of trees, bushes and other vegetation from the reservoir area is known as reservoir clearance. It is an expensive operation and difficult to be justified on cost-benefit considerations. Non-clearance of such vegetation may lead to the following troubles :

(*i*) decay of organic material may create undesirable odours and tastes, and hence, becomes important for water supply reservoirs.

(*ii*) trees projecting above the water surface may create undesirable appearance, when the reservoir is to be used for recreation and tourism purposes.

(*iii*) bushes, trees, etc. will float and may create debris problems at the dam.

18.19. Selection of a Suitable Site for a Reservoir

It is almost impossible to select a perfect ideal reservoir site. But its selection is guided by the following factors :

(*i*) A suitable dam site is available. The cost of the dam is generally a controlling factor in the selection of a reservoir site.

(*ii*) The geological formations for the reservoir banks, walls, etc. should be such as to entail minimum leakage.

(*iii*) The geology of the catchment area should be such as to entail minimum water losses through absorption and percolation.

(*iv*) The site should be such that a deep reservoir is formed. A deep reservoir is preferred to a shallow one, because of lower land cost per unit of capacity, less evaporation loss, and less possibility of weed growth.

(*v*) The reservoir site must have adequate capacity.

(*vi*) Too much silt laiden tributaries should be avoided as far as possible.

(*vii*) The reservoir basin should have a deep narrow opening in the valley, so that the length of the dam is minimum.

18.20. Reservoir Induced Seismicity

During the recent times, a very strange phenomenon has been observed in several dams of the world. What actually has happened is that the reservoir basins or areas which were seismologically inactive, started showing seismic activities, as the reservoir was filled up with water. The magnitude of these earthquakes is found increasing with the filling of the reservoir, giving severe shocks when the reservoir is full. In all such cases, the epicentre is always seen to be located within the reservoir or along its border.

An important Indian example of such an earthquake has been in the case of Koyna dam (Maharashtra), which is situated in the seismologically inactive zone in Peninsular India. However, when the dam got ready and water started collecting (1962) in the reservoir, the earthquakes also started occurring (1963). The frequency and intensity of these earthquakes had gone on increasing, as more and more water went on collecting in the reservoir. On 10th December, 1967, the severest shock occurred with a magnitude of about 6.5, followed by three more shocks of decreasing magnitudes. The epicentres of all these shocks were traced to be within the reservoir area.

Some other examples of such earthquake sites, noticed outside India, are at *Lake Mead* (U.S.A.) ; *Grand Val Lake* (France) ; *Vorgorno Lake* (Switzerland) ; etc.

This phenomenon has been studied by various seismologists, and it has been suggested by some that in most such cases, *there must have existed some inactive faults in the reservoir basin area, which became active again due to the extra-ordinary load of the reservoir water,* thus causing displacements along these faults and consequently resulting in earthquakes. Some other seismologists have suggested that these earthquakes are caused due to *increased pore pressure* in the adjoining rocks, which lowers their shearing strength, resulting in the release of tectonic strain.

Based on these explanations, the various preventive methods suggested for preventing or reducing such earthquakes, include :

(*i*) *filling of the reservoir to a limited safe level ;*

(*ii*) *reducing pore pressure by draining out water from weaker adjoining rocks ; and*

(*iii*) *to actively explore the dam site for the absence of inactive faults before selecting the same.*

Such earthquakes, however, show a decreasing tendency with time. This phenomenon is indeed very complex and interesting, and still needs further research.

18.21. Economic Height of a Dam

The economic height of a dam is that height of the dam, corresponding to which, the cost of the dam per unit of storage is minimum. For this purpose, the estimates are prepared for construction costs, for several heights of the dam, somewhat above and below the level at which the elevation-storage curve shows a fairly high rate of increase of storage per unit rise of elevation, keeping the length of the dam moderate. The construction cost is found to increase with the dam height, as shown in Fig. 18.29.

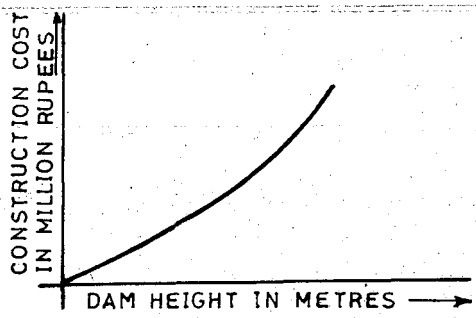


Fig. 18.29

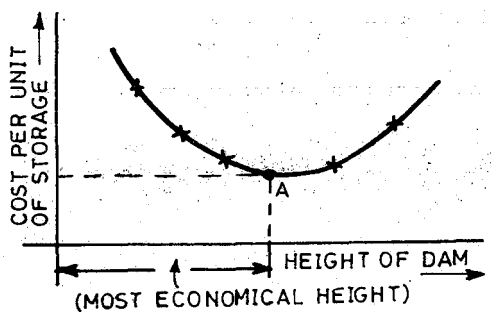


Fig. 18.30

For each dam height, the reservoir storage is known from the reservoir-capacity curve. The construction cost per unit of storage for all the possible dam heights can then be worked out and plotted, as shown in Fig. 18.30.

The lowest point A on this curve, gives the dam height for which the cost per unit of storage is minimum, and hence, most economical.

Example 18.20. *The construction costs for certain possible heights of a dam at a given site have been estimated and are tabulated in the table below. The storage capacity for all these dam heights are also given.*

S.No. (1)	*Height of the dam in metres* (2)	*Construction cost in million Rs.* (3)	*Storage in million cubic metres* (4)
1	*10*	*4*	*50*
2	*20*	*8*	*110*
3	*30*	*12*	*180*
4	*40*	*18*	*250*
5	*50*	*27*	*350*
6	*60*	*39*	*500*
7	*65*	*50*	*600*

Determine the most economical height of the dam from purely construction point of view.

Solution. The given table is extended, so as to workout the cost per million cubic metre of storage, as shown in col. (5) of Table 18.36.

Table 18.36

S.No. (1)	*Height of the dam in metres* (2)	*Construction cost in M.Rs.* (3)	*Storage in M.m*^{3} (4)	*Cost per unit of storage* $= \frac{Col.\ 3}{Col.\ 4}$ (5)
1	10	4	50	0.080
2	20	8	110	0.073
3	30	12	180	0.067
4	40	18	250	0.072
5	50	27	350	0.077
6	60	39	500	0.078
7	65	50	600	0.083

The cost per unit of storage is plotted against the height of the dam, as shown in Fig. 18.31. The most economical height is the lowest point of this curve, and it works out to be **30 metres. Ans.**

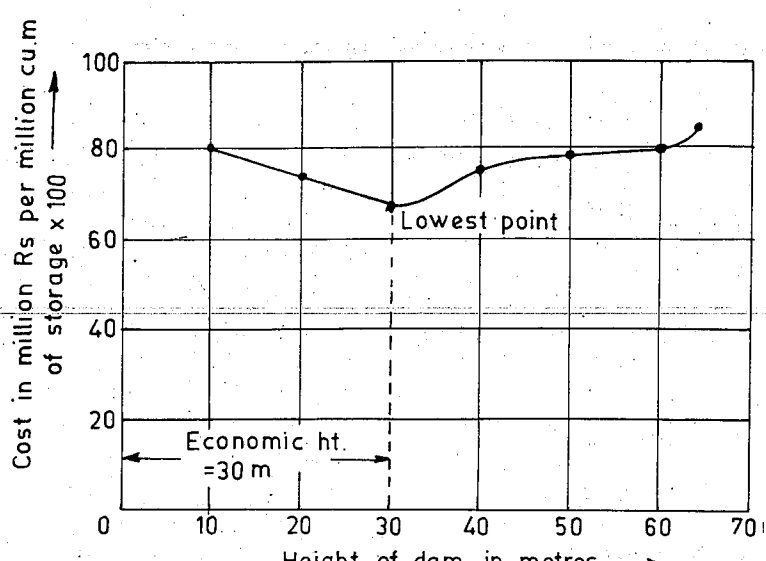


Fig. 18.31.

PROBLEMS

1. (*a*) What is meant by a 'Reservoir'? Discuss briefly the different types of reservoirs and the purpose served by each type.

(*b*) Describe briefly the techniques that are employed for computing the storage capacity of a reservoir for different water surface elevations.

2. (*a*) What is meant by a 'Flood control reservoir' ; and what are their different types ?

(*b*) Discuss with a neat sketch, the various storage zones of the dam reservoir.

(*c*) What factors you will keep in mind while selecting a suitable site for a dam reservoir?

3. (*a*) Differentiate clearly between the following :

(*i*) A flood control reservoir and a multipurpose reservoir.

(*ii*) A retarding basin and a storage reservoir.

(*iii*) Firm yield, design yield, and secondary yield of a reservoir.

(*b*) Briefly describe as to how you would fix the storage capacity of a reservoir and the height of the dam required for this storage. *(Madras University, 1976)*

4. (*a*) What is the relation between 'reservoir capacity' and 'reservoir yield'?

(*b*) How would you fix the capacity of a dam reservoir at a particular river site, provided the inflow pattern and demand pattern are known. Explain the mass curve method which is used for this purpose.

5. (*a*) Explain how the storage capacity of a reservoir is fixed.

(Madras University, 1973, 1974)

(*b*) Explain the mass curve method that can be used for determining :

(*i*) Reservoir capacity for fulfilling given demand.

(*ii*) Demand rate from a reservoir of a given capacity.

6. Discuss briefly and with necessary neat sketches, the demand patterns for the following types of reservoirs :

(*i*) Single purpose conservation reservoir.

(*ii*) Single purpose flood control reservoir.

(*iii*) Multipurpose reservoir.

7. (*a*) What is meant by 'flood routing through reservoirs'?

(*b*) Describe step by step procedure that you will adopt for flood routing computations required for reservoirs under 'trial and error method'.

8. Describe a method for routing flood water through a deep reservoir using the fundamental relation between inflow, outflow (discharge) and storage. Take $Q = \text{C.L.}\, H^{3/2}$ for the spillway.

(U.P.S.C.,Engg. Services, 1974)

9. The initial inflow in a reservoir and outflow over the spillway was 30 m^3/sec. During a storm, the following inflow rates were noted at the ends of successive half day periods : 240, 300, 240, 90, 30 and 30 m^3/sec. The relationship between discharge over the spillway (Q), and storage rate in the reservoir $\left(\frac{S}{T}\right)$ may be expressed by the equation

$$Q = \frac{1}{2} \cdot \frac{S}{T}.$$

Assuming that the average inflow rate during each half day period is equal to the average of the rates occurring at the beginning and at the end of the period, find the outflow at the end of half hour period.

Hence plot to a suitable scale the inflow and outflow hydrographs.

10. What are the factors on which the rate of silting of an impounding reservoir depends? What is trap efficiency?

Discuss the principal measures that should be undertaken to control the inflow sediment to an impounding reservoir.

An impounding reservoir had original storage capacity for 738 ha-m. The drainage area of the reservoir is 80 sq. km, from which, annual sediment discharges into the reservoir at the rate 0.1153 ha-m

per sq. km. of the drainage area. Assuming the trap efficiency as 80 per cent, find the annual capacity loss of the reservoir in per cent per year. (*U.P.S.C., Engg. Services, 1969*)

[**Ans.** 10%]

11. Write short notes on any four of the following :

(*i*) Reservoir losses (*ii*) Reservoir clearance

(*iii*) Economic height of a dam

(*iv*) Cost benefit considerations in planning dam reservoirs.

[**Note.** Please See Chapter 20 of "Hydrology and Water Resources Engineering"]

(*v*) Reservoir sedimentation and its control

(*vi*) Density currents

(*vii*) Trap efficiency

(*viii*) Estimating the life of a reservoir.

19

Design and Construction of Gravity Dams

19.1. Definition, etc.

A gravity dam has been defined as a structure which is designed in such a way that its own weight resists the external forces. This type of a structure is most durable and solid, and requires very little maintenance. Such a dam may be constructed of masonry or concrete. However, concrete gravity dams are preferred these days and mostly constructed. They can be constructed with ease on any dam site, where there exists a natural foundation strong enough to bear the enormous weight of the dam. Such a dam is generally straight in plan, although sometimes, it may be slightly curve. The line of the upstream face of the dam, or the line of the crown of the dam if the upstream face in sloping, is taken as the reference line for layout purposes, etc. and is known as the *Base line of the dam* or the '*Axis of the Dam*'. When suitable conditions are available, such dams car be constructed up to great heights. The highest gravity dam in the world is Grand Dixence Dam in Switzerland (284 m), followed by Bhakra dam in India (226 m); both are of concrete gravity type. The ratio of base width to height of all these structures is less than 1 : 1.

19.2. Typical Cross-section

A typical cross-section of a concrete gravity dam is shown in Fig. 19.1. The upstream face may by kept throughout vertical or partly slanting for some of its length, as shown. A drainage gallery is provided in order to relieve the uplift pressure exerted by the seeping water.

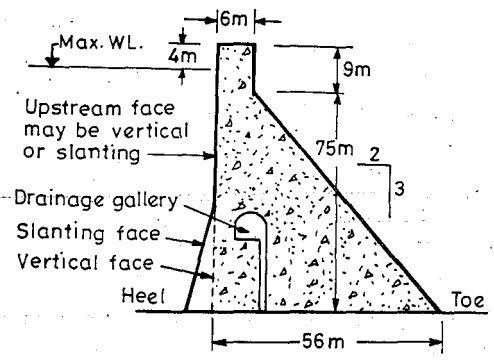


Fig. 19.1. A typical cross-section of a concrete gravity dam.

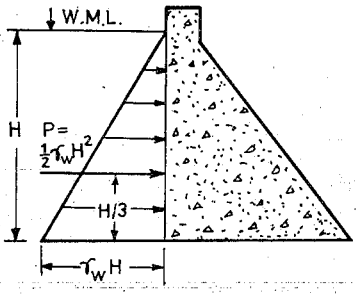
19.3. Forces Acting on Gravity Dam

The various external forces acting on a gravity dam may be :

(1) Water Pressure
(2) Uplift Pressure
(3) Pressure due to earthquake forces
(4) Silt Pressure
(5) Wave Pressure
(6) Ice Pressure
(7) The stabilising force is the weight of the dam itself.

An estimation and description of these forces is given below :

(1) **Water Pressure.** Water pressure (P) is the most major external force acting on such a dam. The horizontal water pressure, exerted by the weight of the water stored on the upstream side on the dam can be estimated from rule of hydrostatic pressure distribution ; which is triangular in shape, as shown in Fig. 19.2 (a) and (b). When the upstream face is vertical, the intensity is zero at the water surface and equal to $\gamma_w H$ at the base ; where γ_w is the unit weight of water and H is the depth of water : as shown in Fig. 19.2 (a). The resultant force due to this external water $= \frac{1}{2}\gamma_w H^2$, acting at $H/3$ from base.



Where γ_w = unit weight of water
9.81 kN/m^3 = 1000 kgf/m^3
Fig. 19.2. (a)

When the upstream face is partly vertical and partly inclined [Fig. 19.2 (b)], the resulting water force can be resolved into horizontal component (P_h) and vertical component (P_v). The horizontal component $P_h = \frac{1}{2}\gamma_w H^2$ acts at $\frac{H}{3}$ from the base ; and the vertical component (P_v) is equal to the weight of the water stored in column $ABCA$ and acts at the c.g. of the area.

Similarly, if there is tail water on the downstream side, it will have horizontal and vertical components, as shown in Fig. 19.2. (b).

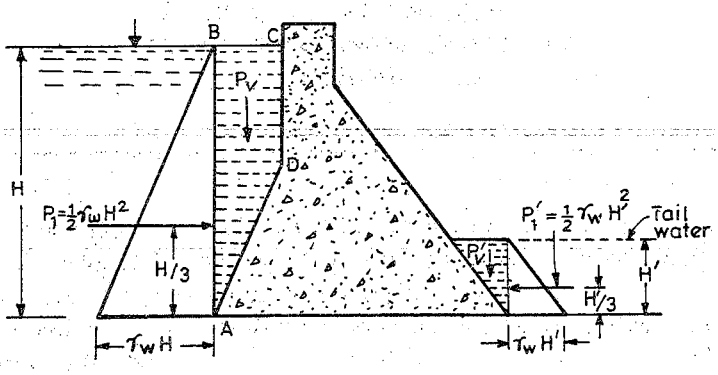


Fig. 19.2 (b)

(2) **Uplift Pressure.** Water seeping through the pores, cracks and fissures of the foundation material, and water seeping through dam body and then to the bottom through the joints between the body of the dam and its foundation at the base ; exert an uplift pressure on the base of the dam. It is the second major external force and must be accounted for in all calculations. Such an uplift force virtually reduces the downward weight of the body of the dam and hence, acts against the dam stability.

The amount of uplift is a matter of research and the present recommendations which are followed, are those suggested by United States Bureau of Reclamation (U.S.B.R.). According to these recommendations, the uplift pressure intensities at the heel and the toe should be taken equal to their respective hydrostatic pressures and joined by a

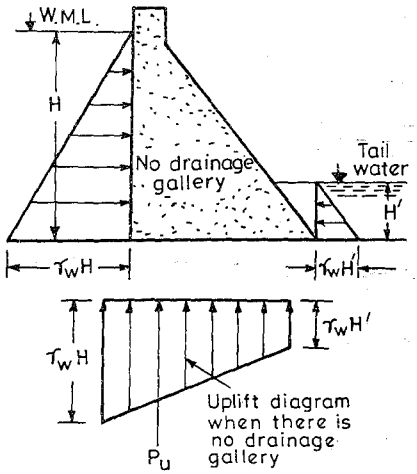


Fig. 19.3 (*a*) Uplift pressure (*U*) diagram, when no drainage gallery is provided.

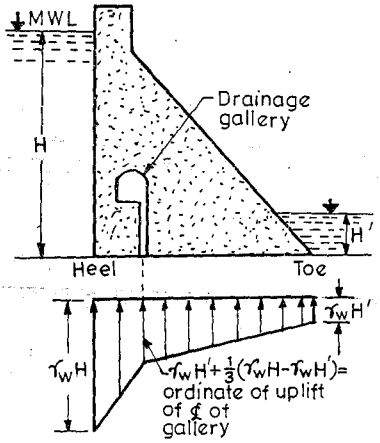


Fig. 19.3 (*b*) Uplift pressure (*U*) diagram, when drainage gallery is provided.

straight line in between, as shown in Fig. 19.3 (*a*). When drainage galleries are provided to relieve the uplift, the recommended uplift at the face of the gallery is equal to the hydrostatic pressure at toe ($\gamma_w \cdot H'$) plus $\frac{1}{3}$rd the difference of the hydrostatic pressures at the heel and the toe ; as shown in Fig. 19.3 (*b*) ; *i.e.* $\left[\gamma_w \cdot H' + \frac{1}{3}(\gamma_w \cdot H - \gamma_w \cdot H')\right]$. It is also assumed that the uplift pressures are not affected by the earthquake forces.

The uplift pressures can be controlled by constructing cut-off walls under the upstream face, by constructing drainage channels between the dam and its foundation, and by pressure grouting the foundation.

(3) **Earthquake Forces.** If the dam to be designed, is to be located in a region which is susceptible to earthquakes, allowance must be made for the stresses generated by the earthquakes.

An earthquake produces waves which are capable of shaking the Earth upon which the dam is resting, in every possible direction.

The effect of an earthquake is, therefore, equivalent to imparting an acceleration to the foundations of the dam in the direction in which the wave is travelling at the moment. Earthquake wave may move in any direction, and for design purposes, it has to be resolved in vertical and horizontal components. Hence, two accelerations, *i.e.* one horizontal acceleration (α_h) and one vertical acceleration (α_v) are induced by an earthquake. The values of these accelerations are generally expressed as percentage of the acceleration due to gravity (*g*), *i.e.* $\alpha = 0.1\,g$ or $0.2\,g$, etc.

In India, the entire country has been divided into five seismic zones depending upon the severity of the earthquakes. Zone V is the most serious zone and includes Himalayan regions of North India. A map and description of these zones is available in "Physical and Engineering Geology" (1999 edition) by the same author, and can be referred to, in order to obtain an idea of the value of the α which should be chosen for designs. On an average, a value of α equal to 0.1 to 0.15 *g* is generally sufficient for high dams in seismic zones. A value equal to 0.15 *g* has been used in Bhakra dam design, and 0.2 *g* in Ramganga dam design. However, for areas not subjected to extreme earthquakes,

$\alpha_h = 0.1\,g$ and $\alpha_v = 0.05\,g$ may by used. In areas of no earthquakes or very less earthquakes, these forces may be neglected. In extremely seismic regions and in conservative designs, even a value upto 0.3 g may sometimes be adopted.

Effect of vertical acceleration (α_v). A vertical acceleration may either act downward or upward. When it is acting in the upward direction, then the foundation of the dam will be lifted upward and becomes closer to the body of the dam, and thus the effective weight of the dam will increase and hence, the stress developed will increase.

When the vertical acceleration is acting downward, the foundation shall try to move downward away from the dam body ; thus reducing the effective weight and the stability of the dam, and hence is the worst case for designs.

Such acceleration will, therefore, exert an **inertia force** given by

$$\frac{W}{g}\alpha_v \text{ (}i.e.\text{ force} = \text{Mass} \times \text{Acceleration)}$$

where W is the total weight of the dam.

$\therefore$ The net effective weight of the dam $= W - \frac{W}{g} \cdot \alpha_v$.

If $\alpha_v = k_v \cdot g$

[where k_v is the fraction of gravity adopted for vertical acceleration, such as 0.1 or 0.2, etc.].

Then, the net effective weight of the dam

$$= W - \frac{W}{g} \cdot k_v \cdot g = W[1 - k_v].$$

In other words, vertical acceleration reduces the unit weight of the dam material and that of water to $(1 - k_v)$ times their original unit weights.

Effects of horizontal acceleration (α_h). Horizontal acceleration may cause the following two forces :

(*i*) Hydrodynamic pressure ; and

(*ii*) Horizontal inertia force.

Both these forces are discussed below :

(*i*) *Hydrodynamic pressures.* Horizontal acceleration acting towards the reservoir causes a momentary increase in the water pressure, as the foundation and dam accelerate towards the reservoir and the water resists the movement owing to its inertia. The extra pressure exerted by this process is known as *hydrodynamic pressure.*

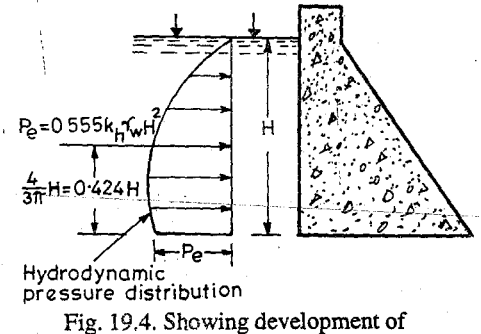


Fig. 19.4. Showing development of Hydrodynamic pressure by a horizontal earthquake moving towards the reservoir. A similar pressure will be developed on d/s tail water when the earthquake is reversed.

According to **Von-Karman**, the amount of this hydrodynamic force (P_e) is given by.

$$P_e = 0.555 \cdot k_h \gamma_w \cdot H^2 \qquad \text{...(19.1)}$$

and it acts at the height of $\frac{4H}{3\pi}$ above the base, as shown in Fig. 19.4.

where k_h is the fraction of gravity adopted for horizontal acceleration, such as 0.1, 0.2 etc.

γ_w = unit wt. of water

Moment of this force about base

$$= M_e = P_e \left(\frac{4H}{3\pi}\right) = \mathbf{0.424\, P_e \cdot H} \qquad ...(19.2)$$

Zanger has given certain big formulas for evaluating the amount of this force and its position, etc. on the vertical as well as on an inclined faces. The results of these big formulas are quite comparable to those given by Von-Karman equation and hence, for average ordinary purposes, the Von-Karman equation (19.1) is sufficient.

Zanger's formula for hydrodynamic force. According to Zanger

$$P_e = 0.726\, p_e \cdot H \qquad ...(19.3)$$

where $p_e = C_m\, k_h \cdot \gamma_w \cdot H$ ...(19.4)

$$\therefore \qquad P_e = 0.726\, C_m \cdot k_h \cdot \gamma_w \cdot H^2 \qquad ...(19.5)$$

where C_m = Maximum value of pressure co-efficient for a given constant slope

$$= 0.735\left(\frac{\theta}{90}^{\circ}\right); \qquad ...(19.6)$$

where θ is the angle in degrees, which the u/s face of the dam makes with the horizontal.

k_h = fraction of gravity adopted for horizontal acceleration (α_h) such as $\alpha_h = k_h \cdot g$

γ_w = unit w.t. of water

The moment of this force about the base is given as :

$$M_e = 0.299\, p_e \cdot H^2 \qquad ...(19.7)$$

$$= 0.299 \frac{P_e}{0.726 \cdot H} \cdot H$$

or

$$M_e = 0.412\, P_e \cdot H \qquad ...(19.8)$$

It was further stated, that if the upstream face is partly inclined (Fig. 19.5 *a*), which does not extend to more than half the depth of the reservoir, it can be taken as vertical. If the slope extends to more than half the depth (Fig. 19.5 *b*), the overall slope up to the whole height may be taken as the value of θ in equation (19.6) above.

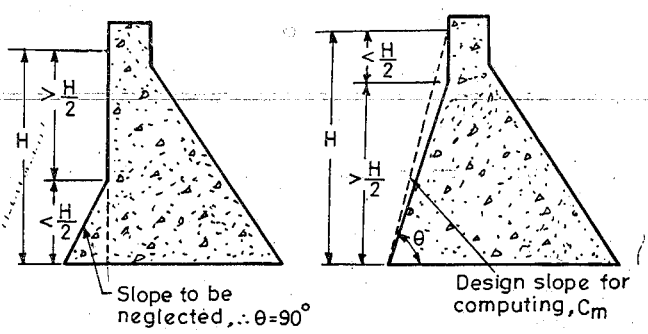


Fig. 19.5

Zanger's formulae are given in details in U.S.B.R. publication on design of small dams, which may be referred to in particular special needs.

(*ii*) *Horizontal Inertia Force.* In addition to exerting the hydrodynamic pressure, the horizontal acceleration produces an inertia force into the body of the dam. This force is generated in order to keep the body and the foundation of the dam together as one piece. The direction of the produced force will be opposite to the acceleration imparted by the earthquake.

Since an earthquake may impart either upstream or downstream acceleration, we have to choose the direction of this force in our stability analysis of dam structure, in such a way that it produces most unfavorable effects under the considered conditions.

Say for example, when the reservoir is *full*, this force would produce worst results if it additive to the hydrostatic water pressure, thus acting towards the *downstream* (*i.e.* when upstream earthquake acceleration towards the reservoir is produced). When the reservoir is *empty*, this force would produce worst results, if considered to be acting *upstream* (*i.e.* when earthquake acceleration, moving towards downstream, is produced).

Under reservoir empty conditions, earthquake forces produce effects, which may cause slight tension near the toe ; and hence stability analysis for reservoir empty case may be carried out only on the basis of wt. of the dam by ignoring earthquake forces and keeping the section free from any tension. However, for all precise designs, these forces must be fully considered, as we have done in example 19.2.

The amount of this horizontal inertial force is equal to the product of the mass of the dam and the acceleration.

∴ This horizontal Inertia force

$$=\left(\frac{W}{g}\right)\alpha_h=\frac{W}{g}\cdot k_h\cdot g=W\cdot k_h \qquad ...(19.9)$$

(where k_h is the fraction of gravity adopted for horizontal acceleration, such as 0.1, or 0.2, etc.).

This force should be considered to be acting at the centre of gravity of the mass, regardless of the shape of the cross- section, and it acts horizontally downstream in worst cases, for reservoir full case.

(4) **Silt Pressure.** It has been explained under 'Reservoir Sedimentation' in chapter 18 that silt gets deposited against the upstream face of the dam. If h is the height of silt deposited, then the force exerted by this silt in addition to external water pressure, can be represented by Rankine's formula as :

$$P_{silt}=\frac{1}{2}\cdot\gamma_{sub}\cdot h^2 K_a \text{ and it acts at } \frac{h}{3} \text{ from base} \qquad ...(19.10)$$

where K_a is the coefficient of active earth pressure of silt $=\dfrac{1-\sin\phi}{1+\sin\phi}$ where ϕ is the angle of internal friction of soil, and cohesion is neglected.

γ_{sub} = submerged unit weight of silt material.

h = height of silt deposited.

If the upstream face is inclined, the vertical weight of the silt supported on the slope also acts as a vertical force.

In the absence of any reliable data for the type of silt that is going to be deposited, U.S.B.R. recommendations may be adopted. In these recommendations, deposited silt may be taken as equivalent to a fluid exerting a force with a unit wt. equal to 3.6 kN/m^3 in the horizontal direction and a vertical force with a unit wt. of 9.2 kN/m^3. Hence, the total horizontal force will be $3.6 \frac{h^2}{2} = 1.8\, h^2$ kN/m run, and vertical force will be $9.2 \cdot \frac{h^2}{2} = 4.6\, h^2$ kN/m run.

In most of the gravity-dam designs, the silt pressure is neglected. The basis for neglecting this force is that :

Initially, the silt load is not present, and by the time it becomes significant, it gets consolidated to some extent and, therefore, acts less like a fluid. Moreover, silt deposited in the reservoir is somewhat impervious and, therefore, will help to minimise the uplift under the dam.

(5) **Wave Pressure.** Waves are generated on the surface of the reservoir by the blowing winds, which causes a pressure towards the downstream side. Wave pressure depends upon the wave height. Wave height may be given by the equation,

$$h_w = 0.032\sqrt{V \cdot F} + 0.763 - 0.271\,(F)^{3/4} \text{ for } \mathbf{F < 32\ km}, \text{ and} \quad ...(19.11)$$

$$h_w = 0.032\sqrt{V \cdot F} \text{ for } \mathbf{F > 32\ km} \quad ...(19.12)$$

where h_w = height of water from top of crest to bottom of trough in metres.

V = wind velocity in km/hr.

F = Fetch or straight length of water expanse in km.

The maximum pressure intensity due to wave action may by given by

$p_w = 2.4\,\gamma_w \cdot h_w$ and acts at $\frac{h_w}{2}$ metres above the still water surface.

The pressure distribution may be assumed to be triangular, of height $\frac{5h_w}{3}$, as shown in Fig. 19.6.

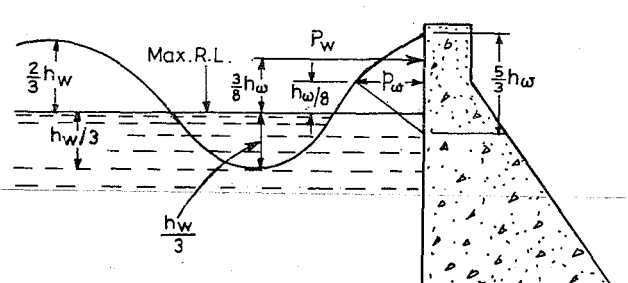


Fig. 19.6

Hence, the total force due to wave action (P_w)

$$= \frac{1}{2}(2.4\,\gamma_w \cdot h_w) \cdot \frac{5}{3} \cdot h_w$$

or $$P_w = 2 \cdot \gamma_w \cdot h_w^2 = 2 \times 9.81\, h_w^2 \text{ kN/m}$$
$$= 19.62\, h_w^2 \text{ kN/m} \quad ...(19.14)$$

This force acts at a distance $\frac{3}{8} h_w$ above the reservoir surface.

(6) **Ice Pressure.** The ice which may be formed on the water surface of the reservoir in cold countries, may sometimes melt and expand. The dam face has then to resist the thrust exerted by the expanding ice. This force acts linearly along the length of the dam and at the reservoir level. The magnitude of this force varies from 250 to 1500 kN/m^2 depending upon temperature variations. On an average, a value of 500 kN/m^2 may be allowed under ordinary conditions.

(7) **Weight of the Dam.** The weight of the dam body and its foundation is the major resisting force. In two dimensional analysis of a gravity dam, a unit length of the dam is considered. The cross-section can then be divided into rectangles and triangles. The weight of each along with their *c.gs.*, can be determined. The resultant of all these downward forces will represent the total weight of the dam acting at the *e.g.* of the dam.

Combination of forces for Designs. The design of a gravity dam should be checked for two cases, *i.e.* (*i*) when Reservoir is full ; and (*ii*) when Reservoir is empty.

(*i*) **Case I.** Reservoir full case :

When reservoir is full, the major forces acting are : weight of the dam, external water pressure, uplift pressure, and earthquake forces in serious seismic zones. The minor forces are : silt pressure, ice pressure and wave pressure. For the most conservative designs, and from purely theoretical point of view, one can say that a situation may arise when all the forces may act together. But such a situation will never arise and hence, all the forces are not generally taken together. U.S.B.R. has classified the 'normal load combinations' and 'extreme load combination, as given below :

(*a*) *Normal Load Combinations*

(*i*) Water pressure upto normal pool level, normal uplift, silt pressure and ice pressure. This class of loading is taken when ice force is serious.

(*ii*) Water pressure upto normal pool level, normal uplift, earthquake forces, and silt pressure.

(*iii*) Water pressure upto maximum reservoir level (maximum pool level), normal uplift, and silt pressure.

(*b*) *Extreme Load Combinations*

(*i*) Water pressure due to maximum pool level, extreme uplift pressure without any reduction due to drainage and silt pressure.

Case II. Reservoir empty case :

(*i*) Empty reservoir without earthquake forces to be computed for determining bending diagrams, etc. for reinforcement design, for grouting studies or other purposes.

(*ii*) Empty reservoir with a horizontal earthquake force produced towards the upstream has to be checked for non- development of tension at toe.

19.4. Modes of Failure and Criteria for Structural Stability of Gravity Dams

A gravity dam may fail in the following ways :

(1) By overturning (or rotation) about the toe.

(2) By crushing.

(3) By development of tension, causing ultimate failure by crushing.

(4) By shear failure called sliding.

The failure may occur at the foundation plane (*i.e.* at the base of the dam) or at any other plane at higher level.

(1) **Over-turning.** If the resultant of all the forces acting on a dam at any of its sections, passes outside the toe, the dam shall rotate and overturn about the toe. Practically, such a condition shall not arise, as the dam will fail much earlier by compression. The ratio of the righting moments about toe (anti clockwise) to the over turning moments about toe (clock-wise) is called the factor of safety against overturning. Its value, generally varies between 2 to 3.

(2) **Compression or crushing.** A dam may fail by the failure of its materials, *i.e.* the compressive stresses produced may exceed the allowable stresses, and the dam-material may get crushed. The vertical direct stress distribution at the base is given by the equation :

$$p = \text{Direct stress} + \text{Bending stress.}$$

$$\therefore \quad p_{\substack{max \\ min}} = \frac{\Sigma V}{B} \pm \frac{M}{I} y = \frac{\Sigma V}{B} \pm \frac{\Sigma V \cdot e}{B^2/6} = \frac{\Sigma V}{B}\left[1 \pm \frac{6e}{B}\right]$$

or

$$p_{\substack{max \\ min}} = \frac{\Sigma V}{B}\left[1 \pm \frac{6e}{B}\right] \qquad ...(19.15)$$

where e = Eccentricity of the resultant force from the centre of the base.

ΣV = Total vertical force.

B = Base width.

Note. Resultant is nearer the toe and hence, maximum compressive stress is produced at the toe (Reservoir full case)

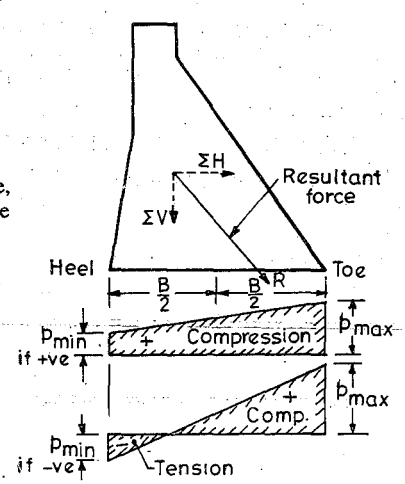


Fig. 19.7. (*a*) Vertical Stress Distribution for Reservoir Full case.

The maximum stress, *i.e.* p_{max}, will be produced on the end which is nearer to the resultant, as shown in Fig. 19.7 (*a*) and (*b*).

Note. The resultant is nearer the heel and hence, maximum compressive stress (+ve stress) is produced at the heel (Reservoir empty with horizontal earthquake wave moving away from reservoir-case).

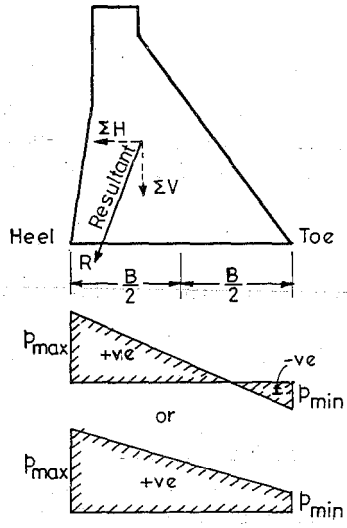


Fig. 19.7. (*b*) Vertical Stress Distribution for Reservoir Empty case.

If p_{min} comes out to be negative, it means that tension shall be produced at the appropriate end.

If p_{min} exceeds the allowable compressive stress of dam material [generally taken as 3000 kN/m^2 (30 kg/cm^2) for concrete]; the dam may crush and fail by crushing.

(3) **Tension.** Masonry and concrete gravity dams are usually designed in such a way that no tension is developed anywhere, because these materials cannot withstand sustained tensile stresses. If subjected to such stresses, these materials may finally crack. However, for achieving economy in designs of very high gravity dams, certain amount of tension may be permitted under severest loading condition. This may be permitted because of the fact that such worst loading conditions shall occur only momentarily for a little time and would neither last long nor occur frequently. The maximum permissible tensile stress for high concrete gravity dams, under worst leadings, may be taken as 500 kN/m^2 (5 kg/cm^2).

Effect produced by tension cracks. In a dam, when such a tension crack develops, say at the heel, crack width (or strictly speaking crack-area) looses contact with the bottom foundations, and thus, becomes ineffective.

Hence, the effective width B (considering unit length) of the dam base will be reduced. This will increase p_{max} at the toe.

Moreover, the uplift pressure diagram gets modified due to crack formation, as shown in Fig. 19.8, resulting in an increase in the uplift. Since the uplift increases and the net effective downward force reduces, the resultant will shift more towards the toe and thus further increasing the compressive stress at the toe and further lengthening the crack due to further tension development. The process continues ; the effective base width goes on reducing and compressive stress at the toe goes on increasing ; finally leading to the failure of the toe by direct compression. *Hence, a tension crack by itself does not fail the structure, but it leads to the failure of the structure by producing excessive compressive stresses.*

ABC = old uplift diagram
$A'B'C'$ = New uplift diagram after the crack AA', has devloped.

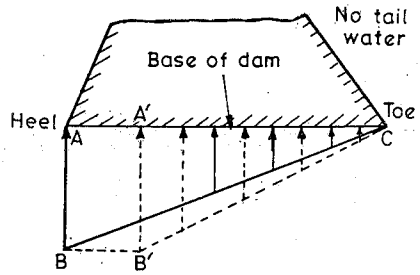


Fig. 19.8

In order to ensure that no tension is developed anywhere, we must ensure that p_{min} is at the most equal to zero.

Since $$p_{\substack{max\\min}} = \frac{\Sigma V}{B}\left[1 \pm \frac{6e}{B}\right] \quad ...(19.15)$$

$$p_{min} = \frac{\Sigma V}{B}\left[1 - \frac{6e}{B}\right]$$

If $$p_{min} = 0,$$

$$\frac{\Sigma V}{B}\left[1 - \frac{6e}{B}\right] = 0$$

or $$1 - \frac{6e}{B} = 0$$

or $$e = \frac{B}{6}.$$

Hence, maximum value of eccentricity that can be permitted on either side of the centre is equal to $\frac{B}{6}$; which leads to the famous statement : *the resultant must lie within the middle third.*

(4) **Sliding.** Sliding (or shear failure) will occur when the net horizontal force above any plane in the dam or at the base of the dame exceeds the frictional resistance developed at that level.

The friction developed between two surfaces is equal to $\mu\Sigma V$., (Fig. 19.9) where ΣV is the algebraic sum of all the vertical forces whether upward or downward, and μ is the coefficient of friction between the two surfaces. In order that no sliding takes place, the external horizontal forces (ΣH) must be less than the shear resistance $\mu \cdot \Sigma V$.

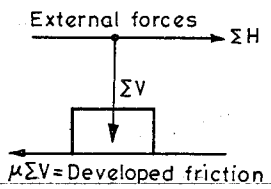


Fig. 19.9

or $$\Sigma H < \mu\Sigma V.$$

or $$\frac{\mu\Sigma V}{\Sigma H} > 1$$

$\frac{\mu \cdot \Sigma V}{\Sigma H}$ represents nothing but the factor of safety against sliding, which must be greater than unity.

$$\therefore \text{ F.S.S. (Factor of safety against sliding)} = \frac{\mu \cdot \Sigma V}{\Sigma H}.$$

In low dams, the safety against sliding should be checked only for friction, but in high dams, for economical precise designs, the shear strength of the joint, which is an additional shear resistance, must also be considered. If this shear resistance of the joint is also considered, then the equation for factor of safety against sliding which is measured by shear friction factor (S.F.F.) becomes

$$\text{S.F.F.} = \frac{\mu \Sigma V + B \cdot q}{\Sigma H} \qquad ...(19.16)$$

where B = width of the dam at the joint,

q = Average shear strength of the joint which varies from about 1400 kN/m^2 (14 kg/cm^2) for poor rocks to about 4000 kN/m^2 (40 kg/cm^2) for good rocks.

The value of μ generally varies from 0.65 to 0.75.

Attempts are always made to increase this shear strength (q) at the base and at other joints. For this purpose, foundation is stepped at the base, as shown in Fig. 19.10 and measures are taken to ensure a better bond between the dam base and the rock-foundation.

During the construction of a dam, horizontal joints have to be left as shown in Fig. 19.10. The shear strength of these joints should be made as good as possible by ensuring better bond between the two surfaces. For this purpose, the lower surface must be thoroughly cleaned and a layer of neat cement or rich cement mortar should be spread before pouring the standard concrete mix for the upper layer. If these precautions of quality control are not adhered to in the filed, the assumption made in accounting for this shear strength in the design, will not be justified. That is why, for small dams, where quality control is less, this shear strength of the joint is not taken into account at all, while determining the shear friction factor or factor of safety against sliding.

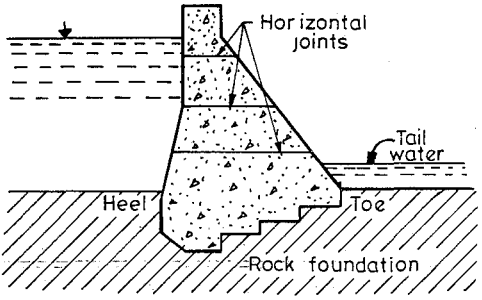


Fig. 19.10

19.4.1. Principal and Shear Stresses. The vertical stress intensity, p_{max} or p_{min} determined from the equation (19.15) is not the maximum direct stress produced anywhere in the dam. The maximum normal stress will, in fact, be the major principal stress that will be generated on the major principal plane. When the reservoir is full, the vertical direct stress [given by equation (19.15), and represented by p_v in future] is maximum at the toe as the resultant is nearer to the toe. To study the principal stresses that will develop near the toe, let us consider a small element ABC [See Fig. 19.11 (a) and (b)] near the toe of

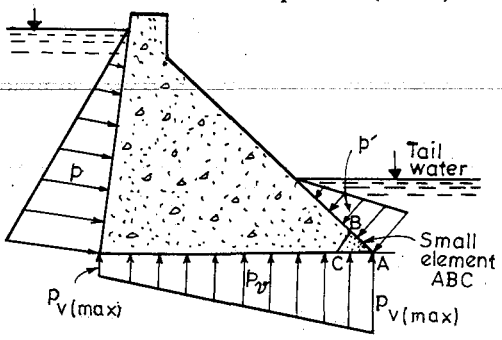


Fig. 19.11 (a)

the dam. The element is so small that the stress intensities may be assumed to be uniform on its faces.

Let the downstream face of the dam be inclined at an angle α to the vertical.

This face of the dam will act as a principal plane because the water pressure p' acts at right angles to the face, and also there is no shear stress acting on this plane. Since the principal planes are at right angles to each other ; the plane BC drawn at right angles to the face AB will be the second principal plane. Let the stress acting on this plane be σ.

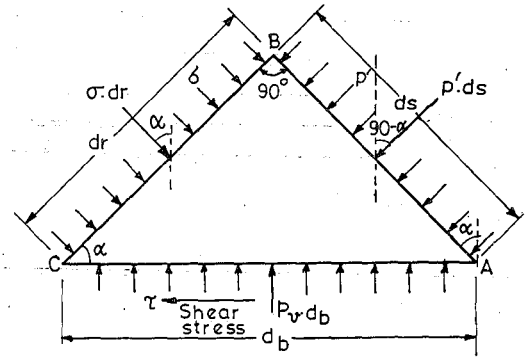


Fig. 19.11. (*b*) Enlarged view of small element *ABC* of Fig. 19.11 (*a*).

Let ds, dr and db be the lengths of AB, BC and CA respectively. p' is the intensity of water pressure on face AB and p_v is the intensity of vertical pressure on face AC, and σ is the intensity of normal stress (principal stress) on face BC. Considering unit length of the dam, the forces acting on the faces AB, BC and CA are $p'\,ds$, $\sigma\,dr$ and $p_v \cdot db$ respectively.

Resolving all the forces in the vertical direction, we get

$$p' \cdot ds \cdot \sin\alpha + \sigma \cdot dr \cdot \cos\alpha = p_v \cdot db,$$

Now $$\frac{ds}{db} = \sin\alpha, \quad \text{or } ds = db \cdot \sin\alpha.$$

$$\frac{dr}{db} = \cos\alpha, \quad \text{or } dr = db \cdot \cos\alpha.$$

$\therefore$ $$p' \cdot (db \cdot \sin\alpha) \cdot \sin\alpha + \sigma \cdot (db \cdot \cos\alpha) \cos\alpha = p_v \cdot db$$

or $$p' \cdot \sin^2 a + \sigma \cdot \cos^2\alpha = p_v.$$

or $$\sigma = \frac{p_v - p' \cdot \sin^2\alpha}{\cos^2\alpha}$$

or $$\boxed{\sigma = p_v \cdot \sec^2\alpha - p' \tan^2\alpha} \qquad \text{...(19.17)}$$

For σ to be maximum, p' should be zero, *i.e.* when there is no tail water ; then in such a case

$$\sigma = p_v \cdot \sec^2\alpha \qquad \text{...[19.17 (a)]}$$

Since $\sec^2\alpha$ is always more than 1, it follows, that σ will be more than p_v. *This value of normal stress, which is the maximum produced anywhere in the body of the dam, must be calculated and should not be allowed to exceed the maximum allowable compressive stress of dam material.*

If the hydrodynamic pressure (p_e') exerted by the tail water during an earthquake moving towards the reservoir is also considered, then the net pressure on the face AB will be ($p' - p_e'$), because the effect of this earthquake will be to reduce the tail water pressure.

The principal stress (σ) can then be given by

or $$\boxed{\sigma_{at\ toe} = p_v \cdot \sec^2 \alpha - (p' - p_e') \tan^2 \alpha} \quad ...(19.18)$$

The equation for σ, derived above for the element at the toe is also applicable to the element at the heel. The equation at the heel is, therefore, given as :

$$\sigma_1 = \sigma_{at\ heel} = p_v \cdot \sec^2 \phi - (p + p_e) \tan^2 \phi \quad ...(19.19)$$

where ϕ is the angle which the u/s face makes with vertical.

But at the heel, the pressure of water p is always more than σ, and hence, p will be the minor principal stress at the heel.

Shear stress on the horizontal plane near the toe. A shear stress τ will act on the face *CA* on which the vertical stress is acting. Resolving all the forces [Fig. 19.11 (*b*)] in the horizontal direction, we get

$$\sigma \cdot dr \sin \alpha - p' \cdot ds \cdot \cos \alpha = \tau_0 \cdot db$$

or $$\sigma \cdot (db \cdot \cos \alpha) \sin \alpha - p' (db \cdot \sin \alpha) \cos \alpha = \tau_0 \cdot db$$

or $$\sigma \cdot \sin \alpha \cos \alpha - p' \sin \alpha \cos \alpha = \tau_0$$

or $$\tau_0 = (\sigma - p') \sin \alpha \cos \alpha$$

Substituting the value of σ from equation (19.17), we get

$$\tau_0 = \left[p_v \sec^2 \alpha - p' \tan^2 \alpha - p' \right] \sin \alpha \cos \alpha$$

$$\tau_0 = \left[p_v \sec^2 \alpha - p' (1 + \tan^2 \alpha) \right] \sin \alpha \cos \alpha = \left[(p_v - p') \sec^2 \alpha \right] \sin \alpha \cos \alpha$$

or $$= [(p_v - p') \sec^2 \alpha \cdot \sin \alpha \cdot \cos \alpha]$$

or $$\boxed{\tau = (p_v - p') \tan \alpha} \quad ...(19.20)$$

Neglecting tail water, shear stress is given by

$$\tau_0 = p_v \cdot \tan \alpha \quad ...[19.20\ (a)]$$

If the effect of hydrodynamic pressure produced by an earthquake moving towards the reservoir, is also considered, the equation for shear stress on a horizontal plane near the toe becomes,

$$\tau_0 = \left[p_v - (p' - p_e') \right] \tan \alpha \quad ...(19.21)$$

Similarly, shear stress at heel

$$= \tau_{0\,(heel)} = \left[p_v - (p + p_e) \right] \tan \phi$$

–ve sign shows that the direction is reversed.

Example 19.1. *Fig. 19.12 (a) shows the section of a gravity dam (non-overflow portion) built of concrete.*

Calculate (neglecting earthquake effects)

(*i*) *The maximum vertical stresses at the heel and toe of the dam.*

(*ii*) *The major principal stress at the toe of the dam.*

(*iii*) *The intensity of shear stress on a horizontal plane near the toe.*

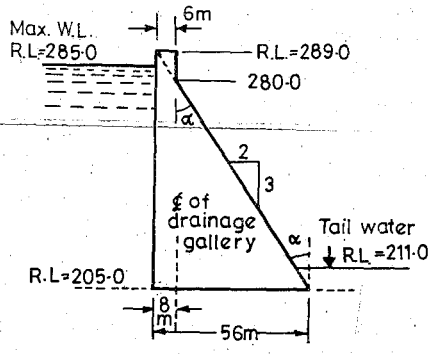


Fig. 19.12 (*a*)

Assume weight of concrete = 23.5 kN/m³; and unit length of dam. Allowable stress in concrete may be taken 2500 kN/m²

Solution. Assuming $\gamma_w = 9.81\ \text{kN/m}^3$; the various forces acting on the dam are drawn in Fig. 19.12 (*b*).

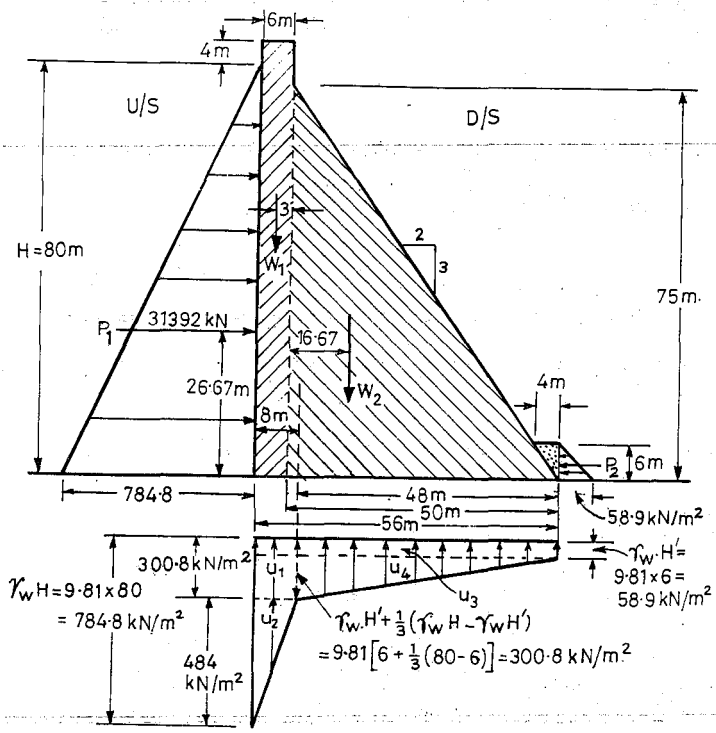


Fig. 19.12 (*b*)

Consider 1 m length of the dam.

The various forces and their moments about the toe are then calculated and tabulated in Table 19.1. From this table, we have

Distance of resultant from the toe

$$(\bar{x}) = \frac{\Sigma M}{\Sigma V}$$

$$= \frac{7,77,639\ \text{kN} \cdot m}{43050\ \text{kN}} = 18.06\ \text{m}$$

$$\text{Eccentricity} = e = \frac{56}{2} - 18.06 = 28 - 18.06 = 9.94\ \text{m}$$

Vertical stress p_v is given as :

$$p_v = \frac{\Sigma V}{B}\left[1 \pm \frac{6e}{B}\right]$$

$$\therefore \qquad p_v = \frac{43,050\ \text{kN}}{56\ \text{m}}\left[1 \pm \frac{6 \times 9.94}{56}\right] = 768.8\ (1 \pm 1.065)$$

Table 19.1

Name of the force	*Designation if given*	*Magnitude in kN*	*Lever arm in m*	*Moments about toe in kN.m*
Vertical forces				
Downward weight of the dam	W_1	(+) $84 \times 6 \times 1 \times 23.5 = 11{,}844$	53.0	(+) 6,27,732
	W_2	(+) $\frac{1}{2} \times 50 \times 75 \times 1 \times 23.5 = 44{,}063$	33.33	(+) 14,68,620
Weight of water supported on d/s face	—	(+) $\frac{1}{2} \times 4 \times 6 \times 1 \times 9.81 = 118$	1.33	(+) 157
		$\Sigma V_1 = 56{,}025$		$\Sigma M_1 = (+)\ 20{,}96{,}509$
Uplift pressures	U_1	(–) $300.8 \times 8 \times 1 = 2406$	52.0	(–) 1,25,112
—	U_2	(–) $\frac{1}{2} \times 484 \times 8 \times 1 = 1936$	53.33	(–) 1,03,247
	U_3	(–) $58.9 \times 48 \times 1 = 2827$	24.0	(–) 67,848
	U_4	(–) $\frac{1}{2} \times 241.9 \times 48 \times 1 = 5806$	32.0	(–) 1,85,792
		$\Sigma V_2 = (-)\ 12{,}975$		$\Sigma M_2 = (-)\ 4{,}81{,}999$
		$\Sigma V = 56{,}025 - 12{,}975 = \mathbf{43050}$		
Horizonal *Water pressure*				
On u/s face	P	$\frac{1}{2} \times 784.8 \times 80 \times 1 = 31{,}392$	26.67	(–) 8,37,225
On d/s face	P'	(–) $\frac{1}{2} \times 58.9 \times 6 = (-)\ 177$	2.0	(+) 354
		ΣH (towards downstream) = 31,215		$\Sigma = (-)\ 8{,}36{,}871$
ΣM = Net (+) moment = 20,96,509 – 4,81,999 – 8,36,871 = 7,77,639 kN-m				

$\therefore$ Max. vertical stress $= p_{max}$ at toe $= 768.8 \times 2.065 = \mathbf{1587.6\ kN/m^2}$

$\therefore$ Min. vertical stress $= p_{min}$ at heel $= 768.8 \times (-)\ 0.065 = \mathbf{(-)\ 49.97\ kN/m^2}$ **Ans.**

(*ii*) Major principal stress at toe (σ) is given by Eq. (19.17) as :

$$\sigma = p_{v\,(toe)} \sec^2 \alpha - p' \cdot \tan^2 \alpha$$

$$\text{here } p_{v\,(toe)} = 1587.6\ \text{kN/m}^2$$

$$p' = 58.9\ \text{kN/m}^2$$

$$\tan \alpha = \frac{2}{3}$$

$$\sec \alpha^2 = 1 + \tan^2 \alpha = 1 + \frac{4}{9} = \frac{13}{9}$$

$$\therefore \quad \sigma = 1587.6 \times \frac{13}{9} = 58.9 \times \frac{4}{9}$$

$$= \mathbf{2267\ kN/m^2 < 2500\ kN/m^2\ (OK)} \quad \textbf{Ans.}$$

(*iii*) Intensity of shear stress on a horizontal plane near toe is given by equation (19.20)

$$\therefore \quad \tau_0 = \left[p_{v(toe)} - p'\right] \tan \alpha$$

$$= 1587.6 - 58.9)\ \tfrac{2}{3} = \mathbf{1019.1\ kN/m^2}. \quad \textbf{Ans.}$$

STABILITY ANALYSIS

The stability of a gravity dam can be approximately and easily analysed by two dimensional gravity method and can be precisely analysed by three dimensional methods such as slab analogy method, trial load twist method, or by experimental studies on models. Two dimensional gravity method is discussed below :

19.5. Gravity Method or Two Dimensional Stability Analysis

The preliminary analysis of all gravity dams can be made easily by isolating a typical cross-section of the dam of a unit width. This section is assumed to behave independently of the adjoining sections. In other words, the dam is considered to be made up of a number of cantilevers of unit width each, which act independently of each other. This assumption of independent functioning of each section, disregards the beam action in the dam as a whole.

If the vertical transverse joints of the dam are not grouted or keyed together, this assumption is nearly true. Hence, for wide *U*-shaped valleys, where transverse joints are not generally grouted, this assumption is nearly satisfied. But for narrow V- shaped valleys, where the transverse joints are generally keyed together and the entire length of the dam acts monolithically as a single body, this assumption may involve appreciable errors. In such cases, preliminary designs may be done by gravity method and precise final designs may be carried out by any of the available three dimensional methods.

The description of the three dimensional methods is beyond the scope of this book, and only the two dimensional analysis has been used for the design of gravity dams in this chapter.

Assumptions. The various assumptions made in the two dimensional designs of gravity dams are summarised below :

(*i*) The dam is considered to be composed of a number of cantilevers, each of which is 1 m thick and each of which acts independent of the other.

(*ii*) No loads are transferred to the abutments by beam action.

(*iii*) The foundation and the dam behave as a single unit ; the joint being perfect.

(*iv*) The materials in the foundation and body of the dam are isotropic and homogeneous.

(*v*) The stresses developed in the foundation and body of the dam are within elastic limits.

(*vi*) No movements of the foundations are caused due to transference of loads.

(*vii*) Small openings made in the body of the dam do not affect the general distribution of stresses and they only produce local effects as per St. Venant's principle.

Procedure. Two dimensional analysis can be carried out analytically or graphically.

(*a*) *Analytical Method.* The stability of the dam can be analysed in the following steps :

(*i*) Consider unit length of the dam.

(*ii*) Work out the magnitude and directions of all the vertical forces acting on the dam and their algebraic sum, *i.e.* ΣV.

(*iii*) Similarly, work out all the horizontal forces and their algebraic sum, *i.e.* ΣH.

(*iv*) Determine the lever arm of all these forces about the toe.

(*v*) Determine the moments of all these forces about the toe and find out the algebraic sum of all those moments, *i.e.* ΣM.

(*vi*) Find out the location of the resultant force by determining its distance from the toe,

$$\bar{x} = \frac{\Sigma M}{\Sigma V}.$$

(*xii*) Find out the eccentricity (e) of the resultant (R) using $e = \frac{B}{2} - \bar{x}$. It must be less than $B/6$ in order to ensure that no tension is developed anywhere in the dam.

(*viii*) Determine the vertical stresses at the toe and heel using Eq. (19.15), *i.e.*

$$p_v = \frac{\Sigma V}{B}\left[1 \pm \frac{6e}{B}\right] \qquad ...(19.15)$$

Sometimes stresses are found by ignoring uplift.

(*ix*) Determine the maximum normal stresses, *i.e.* principal stresses at the toe and the heel using equations (19.18) to (19.20). They should not exceed the maximum allowable values. The crushing strength of concrete varies between 1500 to 3000 kN/m^2 depenting upon its grade M15 to M3O.

(*x*) Determine the factor of safety against overturning as equal to $\frac{\Sigma \text{ Stabilising moment } (+)}{\Sigma \text{ Disturbing moment } (-)}$; +ve sign is used for anti-clockwise moments and −ve sign is used for clockwise moments.

(*xi*) Determine the factor of safety against sliding, using Sliding factor $= \frac{\mu \Sigma V}{\Sigma H}$.

$$\text{Shear friction factor (S.F.F.)} = \frac{\mu \Sigma V + bq}{\Sigma H}.$$

Sliding factor must be greater than unity and S.F.F. must be greater than 3 to 5. The analysis should be carried out for reservoir full case as well as for reservoir empty case. The entire procedure has been illustrated in example 19.2.

(*b*) *Graphical method.* In the graphical method, the entire dam section is divided into a number of horizontal sections at some suitable intervals, particularly at the places where the slope changes, as shown in Fig. 19.13. For each section, the sum of the vertical forces (ΣV) and the sum of all the horizontal forces (ΣH) acting above that particular section, are worked out and the resultant force (R) is drawn, graphi-

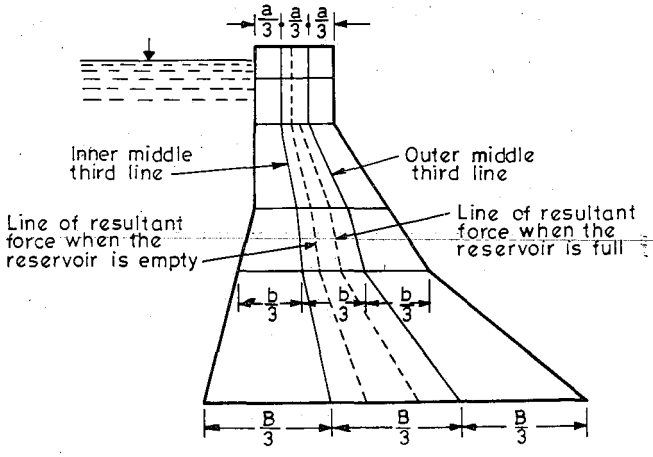


Fig. 19.13

cally. This is done for each section and a line joining all the points where the individual resultants cut the individual sections, is drawn. This line represents the resultant force and should lie within the middle third, for no tension to develop. The procedure should be carried out for reservoir full case as well as for reservoir empty case. The resultant in both cases must show non-development of tension in the dam body.

19.6. Elementary Profile of a Gravity Dam

The elementary profile of a dam, subjected only to the external water pressure on the upstream side, will be a right-angled triangle, having zero width at the water level and a base width (B) at bottom *i.e.*, the point where the maximum hydrostatic water pressure acts. In other words, the shape of such a profile is similar to the shape of the hydrostatic pressure distribution (Fig. 19.14).

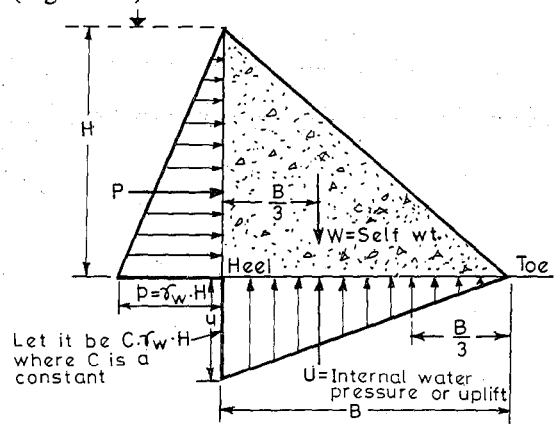


Fig. 19.14

When the reservoir is empty, the only single force acting on it is the self-weight (W) of the dam and it acts at a distance $B/3$ from the heel. This is the maximum possible innermost position of the resultant for no tension to develop. Hence, such a line of action of W is the most ideal, as it gives the maximum possible stabilising moment about the toe without causing tension at toe, when the reservoir is empty. *The vertical stress distribution at the base, when the reservoir is empty,* is given as :

$$P_{max/min} = \frac{\Sigma V}{B}\left[1 \pm \frac{6e}{B}\right]$$

Here $\Sigma V = W$

$$e = \frac{B}{6}.$$

$$\therefore \qquad p_{max/min} = \frac{W}{B}\left[1 \pm \frac{6 \cdot B}{B \cdot 6}\right]$$

or $$p_{max} = \frac{2W}{B}$$

and $$p_{min} = 0.$$

Hence, the maximum vertical stress equal to $\frac{2W}{B}$ will act at the heel ($\because$ the resultant is nearer the heel) and the vertical stress at toe will be zero.

When the reservoir is full, the base width is governed by :

(*i*) The resultant of all the forces, *i.e.* P, W and U (Fig. 19.14) passes through the outer most middle third point (*i.e.* lower middle third point).

(*ii*) The dam is safe in sliding.

(*i*) For the 1st condition to be satisfied, we proceed as follows : Taking moments of all the forces (Fig. 19.14) about the lower middle third point (*i.e.* the point through which resultant is passing), we get

$$W\left(\frac{B}{3}\right) - U\left(\frac{B}{3}\right) - P\frac{H}{3} = R \times 0$$

or
$$(W - U)\frac{B}{3} - P\frac{H}{3} = 0$$

But
$$W = \frac{1}{2} \times B \times H \times 1 \times S_c \times \gamma_w$$

where S_c = Sp. gravity of concrete, *i.e.* that of the material of the dam.

γ_w = unit wt. of water = 9.81 kN/m^3

Let the uplift at the heel be $C \cdot \gamma_w \cdot H$., where C is a constant which according to U.S.B.R. recommendation is taken equal to 1.0 in calculation and will be equal to zero when no uplift is considered.

$\therefore$
$$U = \left(\frac{1}{2} C \cdot \gamma_w \cdot H\right) B$$

and
$$P = \frac{1}{2} \gamma_w \cdot H \cdot H = \frac{\gamma_w H^2}{2}$$

$\therefore$ Equation $(W - U)\frac{B}{3} - P\frac{H}{3} = 0$, becomes

$$\left[\frac{1}{2} B \cdot H \cdot S_c \cdot \gamma_w - \frac{1}{2} C \cdot \gamma_w \cdot H \cdot B\right]\frac{B}{3} - \frac{\gamma_w H^2}{2} \cdot \frac{H}{3} = 0$$

or
$$\frac{B}{3} \times \frac{1}{2} B \cdot H \cdot \gamma_w \cdot [S_c - C] = \frac{\gamma_w H^3}{6}$$

or
$$B^2 (S_c - C) = H^2$$

or
$$\boxed{B = \frac{H}{\sqrt{S_c - C}}} \qquad ...(19.22)$$

Hence, if B is taken equal to or greater than $\frac{H}{\sqrt{S_c - C}}$, no tension will be developed at the heel with full reservoir,

when
$$C = 1$$

$$B = \frac{H}{\sqrt{S_c - 1}} \qquad ...[19.22\ (a)]$$

If uplift is not considered, $B = \frac{H}{\sqrt{S_c}}$ $(\because C = 0)$ ...[19.22 (*b*)]

(*ii*) For the II condition (*i.e.* dam is safe in sliding) to be satisfied ; the frictional resistance $\mu \Sigma V$ or $\mu (W - U)$ should be equal to or more than the horizontal forces $\Sigma H = P$.

or $$\mu (W - U) \geq P$$

or $$\mu \left(\frac{1}{2} BH \cdot S_c \cdot \gamma_w - \frac{1}{2} C \cdot \gamma_w \cdot H \cdot B. \right) \geq \frac{\gamma_w H^2}{2}$$

or $$\mu (S_c - C) \frac{1}{2} \cdot B \cdot H \cdot \gamma_w \geq \frac{\gamma_w H^2}{2}$$

or $$\mu (S_c - C) B \geq H$$

or $$B \geq \frac{H}{\mu (S_c - C)}$$

Under limiting condition

or $$\boxed{B = \frac{H}{\mu (S_c - C)}} \quad ...(19.23)$$

If $C = 1$ $$B = \frac{H}{\mu (S_c - 1)} \quad ...[19.23(a)]$$

If $C = 0$, *i.e.* no uplift is considered, then

$$B \geq \frac{H}{\mu S_c} \quad ...[19.23(b)]$$

The value of B chosen should be greater of the two values given by Equations (19.22) and (19.23).

Using $S_c = 2.4$ and $\mu = 0.7$ and $C = 0$, we get

$$B \text{ (by Equation 19.22)} = \frac{H}{\sqrt{2.4 - 0}} = \frac{H}{\sqrt{2.4}}$$

$$B \text{ (by Equation 19.23)} = \frac{H}{0.7 \times (2.4 - 0)} = \frac{H}{0.7 \times 2.4} = \frac{H}{1.68}$$

But $\frac{H}{1.68}$ is less than $\frac{H}{\sqrt{2.4}}$

$\therefore$ For all practical purposes, the base width may be taken as $\frac{H}{\sqrt{S_c}}$

The vertical stress distribution when reservoir is full is given as :

$$p_{max/min} = \frac{\Sigma V}{B} \left[1 \pm \frac{6e}{B} \right]$$

where $\Sigma V = W - U$

$$= \left(\frac{1}{2} B \cdot H \cdot 1 \cdot S_c \cdot \gamma_w - \frac{1}{2} C \cdot \gamma_w \cdot H \cdot B \right)$$

$$= \frac{1}{2} B \cdot \gamma_w \cdot H \cdot [S_c - C]$$

$$e = \frac{B}{6}$$

$$\therefore \quad p_{max/min} = \frac{\frac{1}{2} \cdot B \cdot \gamma_w \cdot H (S_c - C)}{B} \left[1 \pm \frac{6B}{6B}\right]$$

maximum stress will occur at toe, because the resultant is near the toe.

$$\therefore \quad p_{max} \text{ at toe} = \frac{1}{2} \gamma_w \cdot H (S_c - C)\, 2.0 = \gamma_w H (S_c - C)$$

$$\boxed{p_v \text{ at toe} = \gamma_w H (S_c - C)} \qquad ...(19.24)$$

$$P_{\min} \text{ at heel} = 0.$$

The principal stress near the toe (σ) which is the maximum normal stress in the dam, is given by Equation (19.17)

$$\sigma = p_v \sec^2 \alpha - p' \tan^2 \alpha$$

when there is no tail water *i.e.*, $p' = 0$

$$\sigma = p_v \sec^2 \alpha$$

σ at toe, with full reservoir in elementary profile

$$= \gamma_w H (S_c - C) \sec^2 \alpha$$

$$= \gamma_w H (S_c - C) [1 + \tan^2 \alpha]$$

$$= \gamma_w H (S_c - C) \left[1 + \frac{B^2}{H^2}\right]$$

But $B = \dfrac{H}{\sqrt{S_c - C}}$ from Eq. (19.22)

or $\dfrac{B^2}{H^2} = \dfrac{1}{S_c - C}$

$$\therefore \quad \sigma = \gamma_w H (S_c - C) \left[1 + \frac{1}{S_c - C}\right]$$

or

$$\boxed{\sigma = \gamma_w H (S_c - C + 1)} \qquad ...(19.25)$$

when $C = 1$, $\quad S_c = 2.4$

$$\sigma = \gamma_w H \left(\frac{2.4 - 1 + 1}{2.4 - 1}\right) = \frac{2.4}{1.4} \gamma_w H$$

$$= \mathbf{1.71\, \gamma_w\, H.}$$

The shear stress τ_0 at a horizontal plane near the toe is given by the equation (19.20) as :

$$\tau_0 = (p_v - p') \tan \alpha$$

If $\quad p' = 0$

$$\tau_0 = p_v \tan \alpha$$

But $p_v = \gamma_w H (S_c - C)$ from Eq. (19.24)

$$\therefore \quad \tau_0 = \gamma_w H (S_c - C) \tan \alpha$$

or

$$\tau_0 = \gamma_w \cdot H (S_c - C) \frac{B}{H}$$

$$= \gamma_w H (S_c - C) \frac{1}{\sqrt{S_c - C}}$$

or
$$\boxed{\tau_0 = \gamma_w H \sqrt{S_c - C}} \quad ...(19.26)$$

19.7. High and Low Gravity Dams

The principal stress calculated for an elementary profile is given by Equation (19.25), *i.e.* $\sigma = \gamma_w H (S_c - C + 1)$. The value of principal stress calculated above varies only with H, as all other factors are fixed.

To avoid dam failure by crushing, the value of σ should be less than or at the most equal to the maximum allowable compressive stress of dam material. If f represents the allowable stress of the dam material, then the maximum height (H_{max}) which can be obtained in an elementary profile, without exceeding the allowable compressive stresses of the dam material, is given as :

$$f = \gamma_w H (S_c - C + 1)$$

or
$$H = \frac{f}{\gamma_w (S_c - C + 1)}$$

The lowest value of H will be obtained when $C = 0$, *i.e.* when uplift is neglected. Hence, for determining the limiting height and to be on a safer side, uplift is neglected.

H_{max} *i.e.* maximum possible height is given as :

$$H_{max} = \frac{f}{\gamma_w (S_c + 1)} \quad ...(19.27)$$

Hence, if the height of a dam having an elementary profile of a triangle, is more than that given by the Equation (19.27), the maximum compressive stress generated will exceed the allowable value. In order to keep it safe within limits, extra slopes on the upstream as well as on the downstream, below the limiting height will have to be given, as shown in Fig. 19.15.

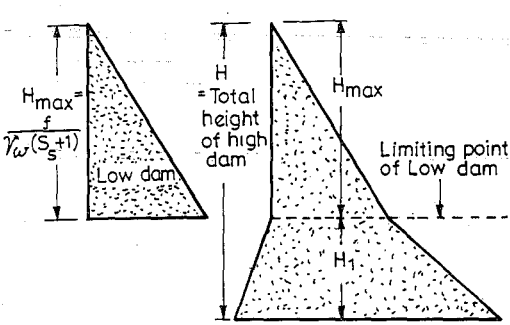


Fig. 19.15

This limiting height (H_{max}) given by Equation (19.27), draws a dividing line between a low gravity dam and a high gravity dram, which are purely technical terms to differentiate between them.

Hence, a low gravity dam is the one whose height is less than that given by Equation (19.27). If the height of the dam is more than this, it is known as a high gravity dam.

The limiting height of a low concrete gravity dam, constructed in concrete having strength equal to 3000 kN/m^2 is thus given :

$$H_{max} = \frac{f}{\gamma_w (S_c + 1)}$$

where $\gamma_w = 9.81 \text{ kN/m}^3$

$S_c = 2.4$

$f = 3000 \text{ kN/m}^2$

$$\therefore \quad H_{max} = \frac{f}{\gamma_w (S_c + 1)} = \frac{3000}{9.81\,(2.4 + 1)} = \mathbf{90\ m}$$

19.8. Profile of a Dam from Practical Considerations

The elementary profile of a gravity dam, (*i.e.* a triangle with maximum water surface at apex) is only a theoretical profile. Certain changes will have to be made in this profile in order to cater to the practical needs. These needs are : (*i*) providing a straight top width, for a road construction over the top of the dam ; (*ii*) providing a free-board above the top water surface, so that water may not spill over the top of the dam due to wave action, etc.

The additions of these two provisions, will cause the resultant force to shift towards the heel. The resultant force, when the reservoir is empty, was earlier passing through the inner middle third point. This will, therefore, shift more towards the heel, crossing the inner middle third point and consequently, tension will be developed at the toe. In order to avoid the development of this tension, some masonry or concrete will have to be added to the upstream side, as shown in Fig. 19.16, which shows the typical section along with the possible dimensions that can be adopted for a low gravity dam section. It should, however, be checked for stability analysis.

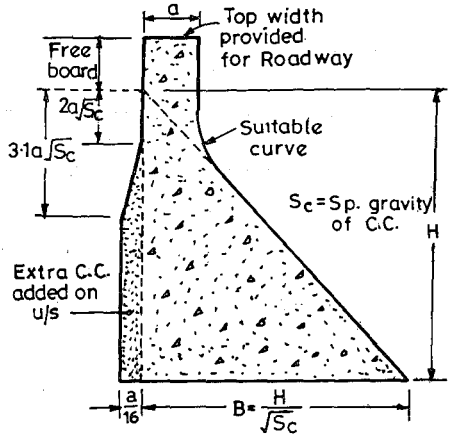


Fig. 19.16.
Typical section of a low gravity dam.

19.9. Design Considerations and Fixing the Section of a Dam

The free-board and top width for roadway should be selected as follows :

(*i*) **Freeboard.** The margin between the maximum reservoir level and top of the dam is known as freeboard. This must be provided in order to avoid the possibility of water spilling over the dam top due to wave action. This can also help as a safety for unforeseen floods, higher than the designed flood.

The freeboard is generally provided equal to $\frac{3}{2}h_w$. where h_w is given by Equations (19.11) and (19.12). However, these days, a *free-board equal to 4 to 5% of the dam height is provided.*

(*ii*) **Top width.** The effects produced by the addition of top width at the apex of an elementary dam profile, and their remedies are explained below :

In Fig. 19.17, let *AEF* be the triangular profile of dam of height H_1. Let the element *ABQA* be added at the apex for providing a top width *a* for a road construction. Let M_1 and M_2 be the inner third and outer third points on base. Thus, AM_1 and AM_2 are the inner third and outer third lines. The weight of the element (W_1) will act through the *e.g.* of this triangle, *i.e.* along *C.M.* Let *CM* and AM_1 cross at *H*, and *CM* and AM_2 cross at *K*.

Reservoir Empty Case. We know that in the elementary profile, the resultant of the forces passes through the inner third point when the reservoir is empty. In such a case, the addition of W_1 above the plane *GHD* will shift this initial resultant towards the downstream side because W_1 lies downstream to the earlier resultant. Similarly, the addition of ΔW_1 'below the plane *GHD* will shift this initial resultant towards the upstream side, causing tension on the downstream. Hence, an upstream batter *GF'* will have to be added below the plane *GHD* as shown in Fig. 19.17.

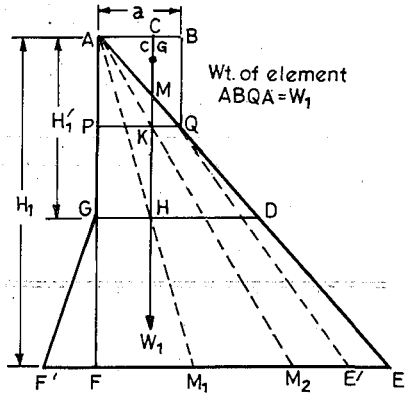


Fig. 19.17

The height H_1' below which this upstream batter is required can be worked out as below :

The ΔAGD, is governed by Equation (19.22), *i.e.*

$$B = \frac{H}{\sqrt{S_c - C}}$$

$$\therefore \qquad B_1' = \frac{H_1'}{\sqrt{S_c - C}}$$

But $$GH = \frac{B_1'}{3}$$

$$\therefore \qquad GH = \frac{B_1'}{3} = \frac{H_1'}{3\sqrt{S_c - C}}$$

Also $$GH = AC = \frac{2}{3}a$$

$$\therefore \qquad \frac{2a}{3} = \frac{H_1'}{3 \cdot \sqrt{S_c - C}}$$

or $$H_1' = 2a \cdot \sqrt{S_c - C} \qquad \text{...(19.28)}$$

Thus, for height greater than H_1', u/s batter is necessary.

Reservoir full case. When the reservoir is full, the resultant of all the forces acting on the elementary profile passes through the outer third point. When W_1 is added to this initial resultant at any plane below the plane *PKQ*, final resultant will shift towards the upstream side because W_1 lies upstream of the initial resultant. In order to bring the resultant back to the outer third point from economy point of view, the slope of the d/s face may be flattened from *QE* to *QE'*.

Thus, an increases in top width, will increase the masonry in the added element and increase it on u/s face, but shall reduce it on d/s face. The most economical top width, without considering earthquake forces has been found by Creager to be equal to 14% of the dam height. Its useful value varies between 6 to 10 m and is generally taken approximately equal to $\sqrt{H}$, where H is the height of max. water level above the bed.

19.10. Design of Gravity Dams

The section of gravity dam should be chosen in such a way that it is the most economical section and satisfies all the conditions and requirements of stability. Hence, after the section of the dam has been arrived at, the stability analysis for the dam must be carried out.

To Decide whether the Dam is Low or High. First of all, the height of the dam to be constructed, should be checked so as to ensure whether it is a low gravity dam or a high gravity dam. If the height of the dam is less than that given by

$$\frac{f}{\gamma_w (S_c + 1)}$$

(where f is the permissible compressive stress of the dam material and S_c is the Sp. gravity of the dam material) ;

then the dam will be a low gravity dam, otherwise it will be a high gravity dam.

Even for a high gravity dam, the upper height $H_1 = \dfrac{f}{\gamma_w (S_c + 1)}$ can be designed as a low gravity dam and the remaining lower portion can be designed as explained a little later.

Design of Low Dam. The economical section of low gravity dam of height H_1 after deciding the top width a and freeboard, can be drawn as shown in Fig. 19.18.

The base width B_1 of the ΔAIJ can be chosen as given by Equation (19.22) as :

$$B_1 = \frac{H_1}{\sqrt{S_c - C}}$$

or by Equation (19.23), as : $B_1 = \dfrac{H_1}{\mu (S_c - C)}$

The upstream face can be kept vertical up to a height H_1' to be determined by trial, and whose approximate 1st value may be chosen by Equation (19.28), as :

$$H_1' = 2a \cdot \sqrt{S_c - C}.$$

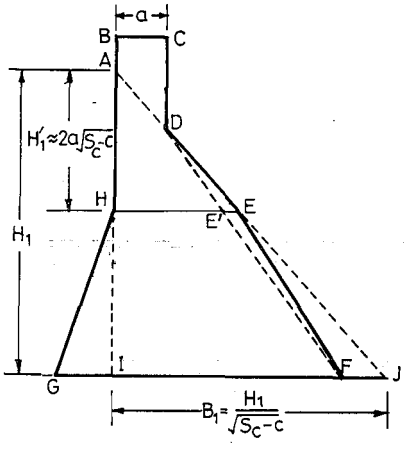


Fig. 19.18

Below this height H_1', the upstream face as well as the downstream face are sloped in such a manner that no tension is developed any-where in the dam, and the resultant forces remain as close to the outer third and inner third points as possible, for reservoir full and reservoir empty cases, respectively. This is accomplished by hit and trial method, and when it is so accomplished, all the stability requirements will be satisfied. The final shape of the dam will then be *ABCDEFGHA*.

The d/s face can be brought inward from the point D itself rather than from the point E (explained earlier). In such a case, the final dam shape will be *ABCDE′FGHA*. Sometimes, in practical conservative designs, the d/s slope is not at all brought inward and is kept the same as is obtained in an elementary profile. The final dam shape, in such a case, will be *ABCDEJGHA*.

Note. However, in most of the practical conservative designs, the low gravity dam is designed as per the provision of Fig. 19.16 and the d/s slope is not brought inward.

Design of High Dam. When the height of the dam exceeds H_1 given by $\frac{f}{\gamma_w (S_c - 1)}$; then its upper height equal to H_1 can be designed as low gravity dam as explained earlier, and its remaining height can be designed by dividing it into a number of suitable strips as shown in Fig. 19.19. The design of each strip can be carried out as per the formulas given below. The basis of these formula is that, the maximum normal stress (*i.e.* principal stress) should not exceed the allowable value (f), and at the same time, the section should be as economical as possible.

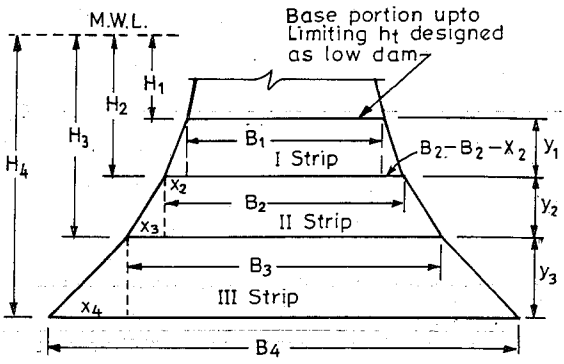


Fig. 19.19

Design of I Strip

The total base width required at the bottom of the 1st strip (B_2) is given by

$$B_2 = \sqrt{\frac{\gamma_w \cdot H_2^3}{f}\left[1 + \frac{\gamma_w^2 \cdot H_2^4}{4 \cdot W_2^2}\right]} \quad \text{...[19.29 (a)]}$$

where B_1 = Base width of low dame, *i.e.* the base width at top of 1st strip

B_2 = Base width required at the bottom of 1st strip

H_2 = Height of dam portion from M.W.L. to the bottom of I strip

γ_w = Unit weight of water

f = Allowable compressive stress of the dam material

W_1 = Total vertical weight of dam and water, above the top of I strip

W_2 = Total wt. of dam portion and water above the bottom of I strip.

The increase of base width required on the upstream side, at the bottom of I strip (say X_2), is given by the equation

$$\frac{\gamma_w \cdot S_c \cdot y_1}{24}\left[3 \cdot B_1^2 - B_2^2 + 6 \cdot X_2 (B_1 + B_2) + 2B_1B_2\right]$$
$$- \frac{\gamma_w \cdot X_2}{12}\left[H_1 + H_2\right]\left[2 \cdot B_2 - 3X_2\right] - W_1\left[\frac{B_2 - B_1}{3} - X_2\right] = 0 \quad \text{...[19.30 (a)]}$$

The increase in width required on u/s, *i.e.* X_2 can be determined from this equation. Total width B_2 is already known. Hence, the increase required on d/s can be calculated as $(B_2 - B_1 - X_2)$.

Design of II Strip

Equation [19.29 (*a*)] for the I strip can be changed to Equation [19.29 (*b*)] for the II strip by changing suffixes as below

$$B_3 = \sqrt{\frac{\gamma_w \cdot H_3^3}{f}\left[1 + \frac{\gamma_w^2 \cdot H_3^4}{4W_3^2}\right]} \qquad \text{...[19.29 (b)]}$$

where H_3 = Height of the dam portion from M.W.L. to the bottom of the II strip

W_3 = Total vertical wt. of the dam and water above the bottom of II strip.

Similarly, Equation [19.30 (*a*)] can be written as

$$\frac{\gamma_w \cdot S_c \cdot y_2}{24}\left[3 \cdot B_2^2 - B_3^2 + 6 \cdot X_3 (B_2 + B_3) + 2 \cdot B_2 B_3\right]$$
$$-\frac{\gamma_w \cdot X_3}{12}\left[H_2 + H_3\right]\left[2 \cdot B_3 - 3X_3\right] - W_2\left[\frac{B_3 - B_2}{3} - X_3\right] = 0 \qquad \text{...[19.30 (b)]}$$

Total width B_3 required is known from Equation [19.29 (*b*)] and the increase required on u/s (X_3) is also known from equation [19.30 (*b*)]. The increase required on d/s can be worked out as equal to ($B_3 - B_2 - X_3$).

All the remaining strips are designed in this way till the base of the dam is reached.

Example 19.2. *Fig. 19.20 (a) shows the section of a gravity dam built of concrete. Examine the stability of this section at the base.*

The earthquake forces may be taken as equivalent to 0.1 g for horizontal forces and 0.05 g for vertical forces. The uplift may be taken as equal to the hydrostatic pressure at the either ends and is considered to act over 60% of the area of the section.

A tail water depth of 6 m is assumed to be present when the reservoir is full and there is no tail water when the reservoir is empty.

Also indicate the values of various kinds of stresses that are developed at heel and toe. Assume the unit wt. of concrete as 24 kN/m³ ; and unit wt. of water = 10 kN/m³.

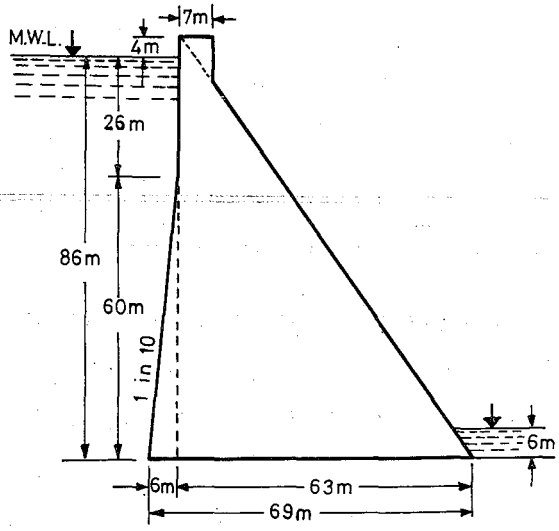


Fig. 19.20 (*a*)

Solution. The stability analysis shall be carried out for both the cases, *i.e.* (1) Reservoir Empty, and (2) Reservoir Full.

Case (I) Reservoir Empty. Consider 1 m length of the dam.

When the reservoir is empty, the various forces are worked out in Table 19.2 (*a*) with reference to Fig. 19.20 (*b*). Horizontal earthquake forces acting towards upstream are considered. Stability is examined for two sub-cases, *i.e.* (*a*) When vertical earthquake

forces are additive to the weight of the dam ; (*b*). When vertical earthquake forces are subtractive to the dam weight.

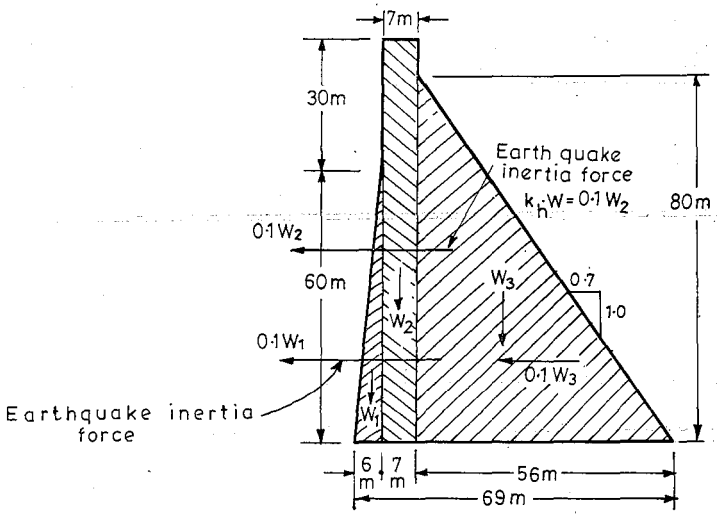


Fig. 19.20 (*b*). Reservoir empty case.

Table 19.2 (*a*)

Name of the force	*Designation if any*	*Magnitude of force in kN.*		*Lever arm m*	*Moments about the toe anti-clockwise (+ve) in kN.m.*
		Vertical	*Horizontal*		
Downward wt. of dam	W_1	(+) $\frac{1}{2} \times 6 \times 60 \times 24 = 4{,}320$		65.0	(+) 2,80,400
	W_2	(+) $7 \times 90 \times 24 = 15{,}110$		59.5	(+) 8,99,000
	W_3	(+) $\frac{1}{2} \times 56 \times 80 \times 24 = 53700$		37.33	(+) 20,00,000
		$\Sigma V_1 = 73{,}130$			$\Sigma M_1 =$ (+) 31,79,400
Horizontal earthquake forces	P_{w1}		$0.1\ W_1 = 0.1 \times 4320 = 432$	20.0	(+) 8640
	P_{w2}		$0.1\ W_2 = 0.1 \times 1{,}511 = 1511$	45.0	(+) 68000
	P_{W3}		$0.1\ W_3 = 0.1 \times 5{,}370 = 5370$	26.67	(+) 1,43,200
			$\Sigma H = 7313$		$\Sigma M_2 = 2{,}19{,}840$
Vertical earthquake forces		$\Sigma V_2 = 0.05 \times \Sigma V_1 = 0.05 \times 73130 = 3{,}657$			$\Sigma M_2 = 0.05 \times \Sigma M_1 = 0.05 \times 31{,}79{,}400 = 1{,}58{,}970$

Case (I). (*a*) *Reservoir empty and vertical earthquake forces are acting downward.*

From table 19.2 (*a*), we have $\Sigma M = \Sigma M_1 + \Sigma M_2 + \Sigma M_3$

$$= 31,79,400 + 2,19,840 + 1,58,970 = \mathbf{35,58,210\ kN \cdot m}$$

Also, $\Sigma V = \Sigma V_1 + \Sigma V_2 = 73,130 + 3,657 = 76,787$ kN

$$\bar{x} = \frac{\Sigma M}{\Sigma V} = \frac{35,58,210}{76,787} = 47.3 \text{ m}$$

$$e = \frac{B}{2} - \bar{x} = \frac{69}{2} - 46.3 = 34.5 - 46.3 = -\mathbf{11.8\ m} > \frac{B}{6}, \textit{ i.e. } 11.5 \text{ m.}$$

Resultant acts near the heel and slight tension will develop at toe.

$$p_{max/min} = \frac{\Sigma V}{B}\left[1 \pm \frac{6e}{B}\right]$$

$$\therefore \quad p_{max/min} = \frac{76,787}{69}\left[1 \pm \frac{6 \times 11.8}{69}\right] = 1114\ [1 \pm 1.026]$$

p_v at heel $= 1114 \times 2.026 = 2260$ kN/m^2; which is ≤ 3000 (safe)

p_v at toe $= 1114 \times (-0.026) = -29$ kN/m^2 ; which is < 420 (*safe*)

Average vertical stress

$$= \frac{\Sigma V}{B} = \frac{76787}{69} = 1114 \text{ kN/m}^2 \text{ ; which is} < 3000 \text{ (safe)}$$

Principal stress at toe,

$$\sigma = p_v \sec^2 \alpha \text{ ; } (\tan \alpha = 0.7)$$

$$= -29\ (1 + 0.49) = -29 \times 1.49 = -43 \text{ kN/m}^2 \text{ ; which is} < 420 \text{ (safe)}$$

Principle stress at heel

$$\sigma_1 = p_{v \cdot (heel)} \sec^2 \phi \qquad \text{where } \tan \phi = 0.1$$

$$\text{or} \quad \sec^2 \phi = 1 + \tan^2 \phi = 1 + 0.01 = 1.01.$$

or $\sigma_1 = 2260 \times 1.01 = \mathbf{2280\ kN/m^2}$; which is < 3000 (safe).

Shear stress at toe

$$\tau_{0(toe)} = p_{v(toe)} \tan \alpha$$

$$= -29 \times 0.7 = -20.3 \text{ kN/m}^2 \text{ ; which is} < 420 \text{ (safe)}$$

Shear stress at heel

$$\tau_{0(heel)} = p_{v \cdot (heel)} \tan \phi$$

$$= 2260 \times 0.1 = 226 \text{ kN/m}^2 \text{ ; which is} < 3000 \text{ (safe).}$$

Case I. (*b*) *Reservoir empty and vertical earthquake forces are acting upward.*

Then $\Sigma V = \Sigma V_1 - \Sigma V_3$

$$= 73,130 - 3657 = 69473 \text{ kN}$$

$$\Sigma M = \Sigma M_1 + \Sigma M_2 - \Sigma M_3$$

$$= 31,79,400 + 2,19,840 - 1,58,970 = 32,40,270 \text{ kN} \cdot \text{m.}$$

$$\bar{x} = \frac{\Sigma M}{\Sigma V} = \frac{32,40,270}{69473} = \mathbf{46.7\ m.}$$

$$e = \frac{B}{2} - \bar{x} = 34.5 - 46.7 = (-)\ 12.2 \text{ m} < \frac{B}{6}$$

[– ve sign shows that resultant lies near the heel and, therefore, tension will develop at toe.]

Average vertical stress

$$= \frac{\Sigma V}{B} = \frac{69{,}473}{69} = 1004 \text{ kN/m}^2$$

$$p_{max/min} = \frac{\Sigma V}{B}\left[1 \pm \frac{6e}{B}\right]$$

$$= \frac{69473}{69}\left[1 \pm \frac{6 \times 12.2}{69}\right] = 1004\,[1 \pm 1.06]$$

p_v at heel $= 1004 \times 2.06 = 2070 \text{ kN/m}^2 < 3000$ (safe)

p_v at toe $= (-)\ 1004 \times 0.06 = -60.3 \text{ kN/m}^2 < 420$ (safe)

Principal stress at toe

$$= \sigma = p_{v(toe)} \sec^2 \alpha$$

$$= -60.3\,(1 + 0.49) = -60.3 \times 1.49 = 90 \text{ kN/m}^2$$

Shear stress at toe

$$= \tau_0 = p_{v(toe)} \tan \alpha = -60.3 \times 0.7$$

$$= \mathbf{-42.21\ kN/m^2}\text{ ; which is} < 420 \text{ (safe)}$$

stresses at heel remain critical in this 1st case.

Case II. When the reservoir is full

Horizontal earthquake moving towards the reservoir causing upstream acceleration, and thus producing horizontal forces towards downstream is considered, as it is the worst case for this condition. Similarly, a vertical earthquake moving downward and thus, producing forces upward, *i.e.* subtractive to the weight of the dam is considered.

The uplift coefficient C is taken as equal to 0.6, as given in the equation, and thus uplift pressure diagram as shown in Fig. 19.20 (c), is developed.

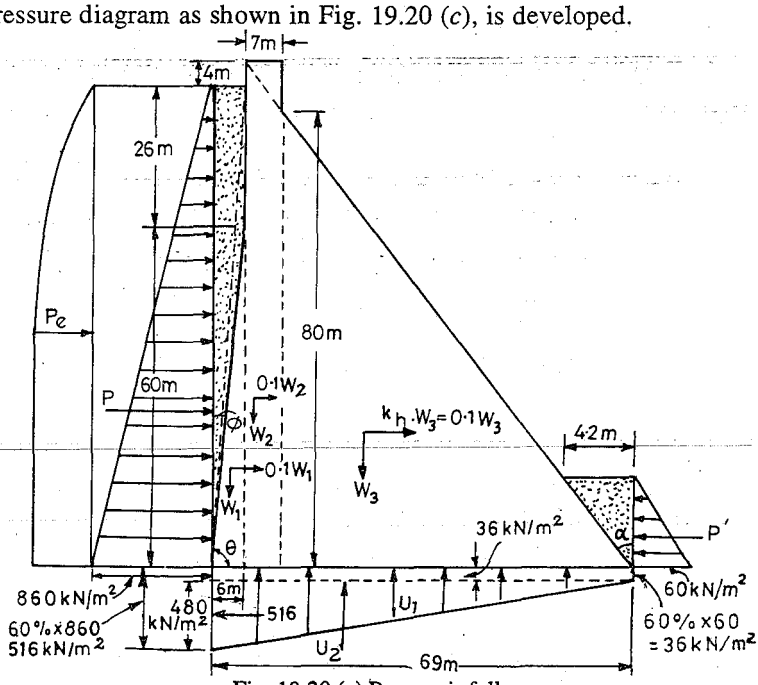


Fig. 19.20 (c) Reservoir full case.

The various forces acting in this case are :

(*i*) Hydrostatic pressures P and P'.

(*ii*) Hydrodynamic pressure P_e. (P_e' is neglected as it is very small and neglection is on conservative side.)

(*iii*) Uplift forces U_1 and U_2

(*iv*) Weight of the dam, W_1, W_2 and W_3.

(*v*) Horizontal inertial earthquake forces acting towards downstream, equal to 0.1 W_1, 0.1 W_2 and 0.1 W_3 at *c.gs.* of these weights W_1, W_2 and W_3 respectively.

(*vi*) A vertical force equal to 0.05 W or (0.05 ΣV_1) acting upward.

Calculation of P_e

P_e and the moment due to this hydrodynamic force is calculated, and then all the forces and their moments are tabulated in Table 19.2 (*b*).

Calculation of P_e from Zanger's formulas

$$P_e = 0.726\, p_e H \qquad ...(19.3)$$

$$\text{where } p_e = C_m \cdot K_h \cdot \gamma_w \cdot H \qquad ...(19.4)$$

$$\text{and} \quad C_m \doteq 0.735 \frac{\theta}{90°}$$

Since the u/s inclined face is extended for more than half the depth, the overall slope up to the whole height may be taken.

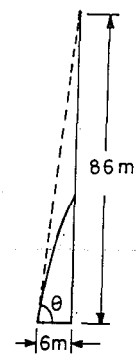


Fig. 19.20 (*d*)

$$\therefore \quad \tan\theta = \frac{86}{6} = 14.33$$

$$\theta = 81.9°$$

$$\therefore \quad C_m = 0.735 \times \frac{81.9°}{90°} = 0.668.$$

$$p_e = 0.668 \times 0.1 \times 10 \times 86 = 57.5$$

$$P_e = 0.726 \times 57.5 \times 86 = \mathbf{3580\ kN.}$$

$$M_e = 0.412 \cdot P_e \cdot H = 0.412 \times 3580 \times 86 = \mathbf{1,26,500\ kN.m.}$$

Case 2 (*a*) Reservoir full with all forces including uplift

$$\Sigma M = [31,79,400 + 2,23,380 - 8,47,500 - 1,58,970 - 10,59,730 - 1,26,500 - 2,19,840]$$

$$= \mathbf{34,02,780 - 24,12,540 = 9,90,240\ kN/m.}$$

$$\Sigma V = 73130 + 3486 - 19030 - 3657 = 53929\ \text{kN}$$

$$\bar{x} = \frac{\Sigma M}{\Sigma V} = \frac{9,90,240}{53,929} = 18.36\ \text{m}$$

$$e = \frac{B}{2} - \bar{x} = 34.5 - 18.36 = 16.14 > \frac{B}{6}$$

The resultant is nearer the toe and tension is developed at the heel.

Average vertical stress

$$= \frac{\Sigma V}{B} = \frac{53929}{69} = 782\ \text{kN/m}^2.$$

$$p_{max/min} = \frac{\Sigma V}{B}\left[1 \pm \frac{6e}{B}\right]$$

Table 19.2 (*b*)

Name of force	*Designation if any*	*Magnitude of force in kN* — *Vertical forces Downward = +ve Upward = −ve*	*Magnitude of force in kN* — *Horizontal forces Towards Upstream = +ve Towards Downstream = −ve*	*Lever arm in m*	*Moments about toe in kN. Anticlock wise (+ve) and clockwise (− ve) in kN.m*
(1)	(2)	(3)	(4)	(5)	(6)
Weight of Dam	W_1	(+) $\frac{1}{2} \times 6 \times 60 \times 1 \times 24 = 4320$		65.0	(+) 2,80,400
	W_2	(+) $7 \times 90 \times 1 \times 24 = 15,110$		59.5	(+) 8,99,000
	W_3	(+) $\frac{1}{2} \times 56 \times 80 \times 1 \times 24 = 53,700$		37.33	(+) 20,00,000
		$\Sigma V_1 = (+)\ 73,130$			$\Sigma M_1 = 31,79,400$
Weight of water supported on u/s slope water on d/s slope.	—	(+) $26 \times 6 \times 1 \times 10 = 1560$		66.0	(+) 1,02,800
	—	(+) $\frac{1}{2} \times 60 \times 6 \times 1 \times 10 = 1800$		67.0	(+) 1,10,400
	—	(+) $\frac{1}{2} \times 6 \times 4.2 \times 1 \times 10 = 126$		1.4	(+) 180
		$\Sigma V_2 = (+)\ 3486$			$\Sigma M_2 = (+)\ 2,23,380$
Uplift forces	U_1	(−) $69 \times 3.6 \times 10 = 2,480$		34.5	(−) 85,500
	U_2	(−) $\frac{1}{2} \times 69 \times 48 \times 10 = 16,550$		46.0	(−) 7,62,000
		$\Sigma V_3 = (-)\ 19,030$			$\Sigma M_3 = (-)\ 8,47,500$
Upward vertical earthquake forces 0.05 W		$\Sigma V_4 = (-)\ 0.05 \cdot \Sigma V_1$ $= (-)\ 0.05 \times 73,130$ $= (-)\ 3,657$			$= (-)\ 0.05 \cdot \Sigma M_1$ $= (-)\ 0.05 \times 31,79,400$ $\Sigma M_4 = (-)\ 1,58,970$
Horizontal hydrostatic pressure	P		(−) $\frac{1}{2} \times 10 \times 86 \times 86 \times 1$ $= (-)\ 36,980$	28.67	(−) 10,60,090
	P'		(+) $\frac{1}{2} \times 10 \times 6 \times 6 \times 1 = (+)\ 180$	2.0	(−) 360
			$\Sigma H_1 = (-)\ 36,800$		$\Sigma M_5 = (-)\ 10,59,730$
Horizontal hydro-dynamic pressure	P_e		Calculated separately earlier : $= (-)\ 3,580$		$\Sigma M_e = (-)\ 1,26,500$ (calculated separately earlier
			$\Sigma H_2 = (-)\ 3,580$		
Horizontal inertia forces due to earthquake	P_{w_1}		(−) $0.1\ W_1 = (-)\ 432$	20.0	(−) 8,640
	P_{W_2}		(−) $0.1\ W_2 = (-)\ 1,511$	45.0	(−) 68,000
	P_{W_3}		(−) $0.1\ W_3 = (-)\ 5,370$	26.67	(−) 1,43,200
			$\Sigma H_3 = (-)\ 7,313$		$\Sigma M_7 = (-)\ 2,19,840$

$$\Sigma H = \Sigma H_1 + \Sigma H_2 + \Sigma H_3 = (-)36,800 - 3580 - 7313 = (-)\ 47,693$$

$$= \frac{53929}{69}\left[1 \pm \frac{6 \times 18.32}{69}\right] = 782\,[1 \pm 1.595]$$

p_v (at toe) $= 782 \times 2.595 =$ **2030 kN/m²** ; which is < 3000 kN/m² (∴ Safe)

p_v (at heel) $= -782 \times 0.405$

$= -$ **316.7 kN/m²** ; which is < 420 kN/m² (∴ Safe)

Since the tensile stress developed is less than the safe allowable value, the dam is safe even when examined with seismic forces, under reservoir full condition.

Principal stress at toe

$$= \sigma = p_v \cdot \sec^2 \alpha - p' \tan^2 \alpha \quad \textit{i.e.}\ \text{Eq. (19.17)}$$

where $\tan \alpha = 0.7$, $p' = 60$ kN/m² ; $p_v = 2030$ kN/m²

$$\sigma = 2030\,(1 + \tan^2 \alpha) - p' \tan^2 \alpha$$
$$= 2030\,(1 + 0.49) - 60 \times 0.49 = 2030 \times 1.49 - 29$$
$$= 3025 - 29 = \mathbf{2996\ kN/m^2 : which\ is < 3000}$$ (just Safe)

Principal stress at heel is

$$\sigma_1 = p_{v(heel)} \sec^2 \phi - (p + p_e) \tan^2 \phi \quad \textit{i.e.}\ \text{Eq. (19.19)}$$

where ϕ is the angle which the upstream face makes with the vertical

$\tan \phi = 0.1$

$$\therefore \quad \sigma_1 = -316.7\,[1 + (0.1)^2] - (860 + 57.5)\,(0.1)^2$$
$$= -316.7 \times 1.01 - 917.5 \times 0.01 = -319.9 - 9.2$$
$$= -\mathbf{329.1\ kN/m^2}\ ;\ \text{which is} < 420\ \text{kN/m}^2$$ (Hence, safe)

Shear stress at toe

$$\tau_{0(toe)} = (p_{v(toe)} - p')\tan \alpha = (2030 - 60)\,0.7$$
$$= 1970 \times 0.7 = \mathbf{1379\ kN/m^2}.$$

Shear Stress at heel

$$\tau_{0(heel)} = -[p_{v(heel)} - (p + p_e)]\tan \phi$$
$$= -[-329.1 - (860 + 57.5)]\,0.1$$
$$= -[-329.1 - 917.5]\,0.1 = +1246.6 \times 0.1 = \mathbf{124.7\ kN/m^2}$$

Factor of safety against overturning

$$= \frac{\Sigma M\,(+)}{\Sigma M\,(-)} = \frac{34{,}02{,}780}{24{,}12{,}540} = 1.41\ ;\ \text{which is} < 1.5$$ (Hence, Unsafe)

Factor of safety against sliding

$$= \frac{\mu \cdot \Sigma V}{\Sigma H}$$

where $\mu = 0.7$

$\Sigma V = 53{,}929$

$\Sigma H = \Sigma H_1 + \Sigma H_2 + \Sigma H_3$

$= -36800 - 3580 - 7313 = -47{,}693$ kN

$$\textit{Sliding factor} = \frac{0.7 \times 53929}{47693} = 0.79, \text{ which is} < 1 \qquad \text{(Hence, Unsafe)}$$

Shear friction factor

$$\text{S.F.F.} = \frac{\mu \cdot \Sigma V + B \cdot q}{\Sigma H}$$

$$= \frac{0.7 \times 53929 + 69 \times 1400}{47693}$$

$$= 2.81 \text{ ; which is less than 3} \qquad \text{(Hence, slightly unsafe)}$$

Case 2 (*b*). Reservoir full, without uplift

Sometimes, values of stresses at toe and heel are worked out when there is no uplift, as the vertical downward forces are maximum in this case. For this case, we shall calculate ΣM and ΣV by ignoring the corresponding values of ΣV_3 and ΣM_3 caused by uplift.

$$\therefore \quad \Sigma M = \Sigma M_1 + \Sigma M_2 + \Sigma M_4 + \Sigma M_5 + \Sigma M_6 + \Sigma M_7$$

$$= 31{,}79{,}400 + 2{,}23{,}380 - 1{,}58{,}970 - 10{,}59{,}730 - 1{,}26{,}500 - 2{,}19{,}840$$

$$= 34{,}02{,}780 - 15{,}65{,}040 = 18{,}37{,}740$$

$$\Sigma V = \Sigma V_1 + \Sigma V_2 + \Sigma V_4 = 73130 + 3486 - 3657 = 72{,}959 \text{ kN}$$

$$\bar{x} = \frac{\Sigma M}{\Sigma V} = \frac{18{,}37{,}740}{72{,}959} = 25.19 \text{ m}$$

$$e = \frac{B}{2} - \bar{x} = 34.5 - 25.9 = 9.31 \text{ m} > \frac{B}{6} \quad i.e. \quad \frac{69}{6} = 11.5 \text{ m}$$

Resultant is nearer the toe and no tension is developed any where.

$$p_{max/min} = \frac{\Sigma V}{B}\left[1 \pm \frac{6e}{B}\right]$$

$$= \frac{72{,}959}{69}\left[1 \pm \frac{6 \times 9.31}{69}\right] = 1057\,[1 \pm 0.81]$$

$$p_v \text{ at toe} = 1057 \times 1.81 = \mathbf{1913\ kN/m^2} < 3000 \qquad (\therefore \text{ Safe})$$

$$p_v \text{ at heel} = 1057 \times 0.19 = \mathbf{201\ kN/m^2} < 3000 \qquad (\therefore \text{ Safe})$$

$$\text{Principal stress at toe} = \sigma = p_v \cdot \sec^2 \alpha - p' \tan^2 \alpha \qquad \text{...(19.17)}$$

$$p' = 60, \tan \alpha = 0.7$$

$$\therefore \quad \sigma = 1913\,(1 + 0.49) - 60 \times 0.49 = 1913 \times 1.49 - 29 = 2821 \text{ kN/m}^2 < 3000$$

(Hence, Unsafe)

Principal stress at heel

$$\sigma_1 = p_{v(heel)} \sec^2 \phi - (p + p_e) \tan^2 \phi \qquad \text{...(19.19)}$$

where $\tan \phi = 0.1$

$$\therefore \quad \sigma_1 = 201(1 + 0.01) - (860 + 57.5) \times 0.01$$

$$= 203 - 9 = 194 \text{ kN/m}^2 < 420 \quad \text{(Safe)}$$

Shear stress at toe

$$\tau_0 = (p_v - p') \tan \alpha \qquad i.e. \text{ Eq. (19.20)}$$

$$= (1913 - 60)\,0.7$$

$$= 1853 \times 0.7 = 1297 \text{ kN/m}^2 < 1400 \qquad (\therefore \text{ safe})$$

Note. Shear friction factor, etc. are not worked out here as they were more critical in the 1st case, *i.e.* in 'Reservoir full with uplift' case.

Conclusion. The dam is unsafe only in sliding and S.F.F., for which shear key etc. can be provided.

Example 19.3. *Examine the stability of the dam section given in the previous example, if there are no seismic forces acting on the dam. Also state the magnitude of maximum compressive stress and maximum shear stress that may develop under any conditions of loading in the dam and also state whether tension is developed anywhere or not.*

Solution. The figures calculated earlier in Table 19.2 (*a*) and (*b*) shall be used here.

Case I. When the reservoir is empty

$$\Sigma V = \Sigma V_1 \text{ from Table 10.2 } (a) = 73130$$

$$\Sigma M = \Sigma M_1 \text{ from Table 19.2 } (b) = 3179400$$

$$\therefore \quad \bar{x} = \frac{\Sigma M}{\Sigma V} = \frac{3179400}{73130} = 43.4 \text{ m}$$

$$e = \frac{B}{2} - \bar{x} = 34.5 - 43.4 = -8.9 \text{ m}$$

–ve sign means that the resultant is towards left side, *i.e.* nearer to the heel, and since $e < \frac{B}{6}$, no tension is developed

$$p_{\max/\min} = \frac{\Sigma V}{B}\left[1 \pm \frac{6e}{B}\right]$$

$$= \frac{73130}{69}\left[1 \pm \frac{6 \times 8.9}{69}\right]$$

$$= 1060\,[1 \pm 0.774]$$

$$p_v \text{ at heel} = 1060(1 + 0.774) = 1060 \times 1.774 = \mathbf{1880\ kN/m^2}$$

$$p_v \text{ at toe} = 1060(1 - 0.774) = 1060 \times 0.226 = \mathbf{239\ kN/m^2}$$

Average vertical stress

$$= \frac{\Sigma V}{B} = \frac{73130}{69} = \mathbf{1060\ kN/m^2}$$

Principal stress at toe

$$\sigma = p_{v(toe)} \sec^2 \alpha$$

$$= 239(1 + 0.49) = 239 \times 1.49 = \mathbf{357\ kN/m^2}$$

Principal stress at heel,

$$\sigma = p_{v(heel)} \sec^2 \phi$$

where $\tan \phi = 0.1$

$$= 1880(1 + 0.01) = 1880 \times 1.01 = \mathbf{1896\ kN/m^2}$$

Shear stress at toe

$$\tau_0 = p_{v(toe)} \tan \alpha$$

$$= 239 \times 0.7 = 167.3 \text{ kN/m}^2$$

Shear stress at heel

$$\tau_{0(heel)} = p_{v(i.e.\cdot l)} \tan \phi$$
$$= 1880 \times 0.1 = 188 \text{ kN/m}^2.$$

Case II (*a*). When the reservoir is full with full uplift

The values of forces, and moments etc. shall be used from the already calculated values in Table 19.2 (*b*).

$$\Sigma V = \Sigma V_1 + \Sigma V_2 + \Sigma V_3$$

(Seismic forces absent mean ΣV_4 and ΣM_4 are zero)

$$= 73130 + 3486 - 19030 = 76616 - 19030 = 57586 \text{ kN}$$

$$\Sigma M = \Sigma M_1 + \Sigma M_2 + \Sigma M_3 + \Sigma M_5$$
$$= 31,79,400 + 2,23,380 - 8,47,500 - 10,59,730$$
$$= 34,02,780 - 19,07,230 = 14,95,550 \text{ kN–m.}$$
$$\Sigma H = \Sigma H_1 = -36800 \text{ kN}$$
$$\bar{x} = \frac{\Sigma M}{\Sigma V} = \frac{14,95,550}{57,586} = \mathbf{26\ m}$$
$$\frac{B}{2} - \bar{x} = 34.5 - 26 = \mathbf{8.5\ m}$$

The resultant is nearer the toe and e is less than $\frac{B}{6}$, and hence, no tension is developed anywhere

$$\therefore \quad p_{max/min} = \frac{\Sigma V}{B}\left[1 \pm \frac{6e}{B}\right]$$
$$= \frac{57586}{69}\left[1 \pm \frac{6 \times 8.5}{69}\right] = 835[1 \pm 0.738]$$
$$p_v \text{ at toe} = 835(1 + 0.738) = 835 \times 1.738 = \mathbf{1451\ kN/m^2}$$
$$p_v \text{ at heel} = 835(1 - 0.738) = 835 \times 0.262 = \mathbf{219\ kN/m^2}$$

Principal stress at toe

$$\sigma = p_{v(toe)} \sec^2 \alpha - p' \tan^2 \alpha \qquad \text{...(19.17)}$$
$$= 1451(1 + 0.49) - 60 \times 0.49 = 1451 \times 1.49 - 29$$
$$= 2162 - 29 = \mathbf{2133\ kN/m^2.}$$

Principal stress at heel

$$\sigma_1 = p_{v(heel)} \sec^2 \phi - p \tan^2 \phi$$

where $\sec \phi = 0.1$

$$\sigma_1 = 219 \times (1 + 0.01) - (860) \times 0.01 = 221.2 - 8.6 = \mathbf{212.6\ kN/m^2.}$$

Note. This is the minor principal stress and major principal stress is equal to p, *i.e.* 860 kN/m^2.

Shear stress at toe

$$\tau_0 = \left[p_{v(toe)} - p'\right] \tan \alpha = (1451 - 60)\ 0.7 = 974 \text{ kN/m}^2.$$

Shear stress at heel

$$\tau_{0(heel)} = -(p_v - p)\tan\phi$$
$$= -(219 - 860)0.1 = 541 \times 0.1 = 54.1 \text{ kN/m}^2;$$

Factor of safety against overturning

$$= \frac{\Sigma M(+)}{\Sigma M(-)} = \frac{3402780}{1907230} = 1.78 > 1.5 \qquad (\therefore \text{ Safe})$$

Factor of safety against sliding

$$= \frac{\mu \Sigma V}{\Sigma H} = 0.7 \times \frac{57586}{36800} = 1.10 > 1 \qquad (\therefore \text{ Safe})$$

Shear friction factor

$$\text{S.F.F.} = \frac{\mu \Sigma V + B \cdot q}{\Sigma H} = \frac{0.7 \times 57586 + 69 \times 1400}{36800}$$
$$= 3.72 < 4 \text{ to } 5 \qquad (\therefore \text{ Unsafe})$$

Case II (*b*). When the reservoir is full without uplift

$$\Sigma V = \Sigma V_1 + \Sigma V_2 = 73130 + 3486 = 76616 \text{ kN}$$
$$\Sigma M = \Sigma M_1 + \Sigma M_2 + \Sigma M_5 = 3179400 + 223380 - 1059730$$
$$= 3402780 - 1059730 = 2343050 \text{ kN–m}$$
$$\Sigma H = \Sigma H_1 = (-)\ 36800 \text{ kN}$$
$$\bar{x} = \frac{\Sigma M}{\Sigma V} = \frac{2343050}{76616} = 30.6 \text{ m}$$
$$e = \frac{B}{2} - \bar{x} = 34.5 - 30.6 = \mathbf{3.9\ m}$$

Resultant is nearer the toe and $e < \frac{B}{6}$; hence, no tension is developed anywhere

$$p_{max/min} = \frac{\Sigma V}{B}\left[1 \pm \frac{6e}{B}\right]$$
$$= \frac{76616}{69}\left[1 \pm \frac{6 \times 3.9}{69}\right]$$
$$= 1110\ [1 \pm 0.339]$$
$$p_v \text{ at toe} = 1110 \times 1.339 = \mathbf{1490\ kN/m^2}$$
$$p_v \text{ at heel} = 1110 \times 0.661 = \mathbf{734\ kN/m^2}$$

Average vertical stress

$$= \frac{\Sigma V}{B} = \frac{76619}{69} = \mathbf{1110\ kN/m^2}$$

Principal stress at toe,

$$\sigma = p_{v(toe)} \sec^2\alpha - p' \tan^2\alpha$$
$$= 1490\ (1 + 0.49) - 60 \times 0.49$$
$$= 2220 - 29 = \mathbf{2191\ kN/m^2}$$

Principal stress at heel,

$$\sigma_1 = p_{v(heel)} \sec^2\phi - p \tan^2\phi$$

$$= 734\,(1 + 0.01) - 860 \times 0.01$$
$$= 742 - 9 = \mathbf{733\ kN/m^2}$$

Shear stress at toe

$$\tau_0 = [p_{v(toe)} - p'] \tan \alpha$$
$$= (1490 - 60)\ 0.7 = 1430 \times 0.7 = 1001\ \text{kN/m}^2$$

Shear stress at heel

$$= -\,[p_{v(heel)} - p] \tan \phi$$
$$= -\,[734 - 860] \times 0.7 = 126 \times 0.7 = \mathbf{88.2\ kN/m^2}$$

Conclusions. We find that the dam is safe throughout except that the S.F.F. is equal to 3.72, while generally it should be between 4 to 5. The dam thus remains slightly unsafe in S.F.F. even when the seismic forces are not considered.

The results of stability analysis are given below :

The maximum shear stress developed in dam	$= 1001\ \text{kN/m}^2$.	**Ans.**
Maximum compressive stress developed in dam	$= 2191\ \text{kN/m}^2$	
No tension is developed anywhere.		
Factor of safety against sliding	$= 1.10$	
S.F.F.	$= 3.72$	
Factor of safety against overturning	$= 1.78$	

Example 19.4. *Design a concrete gravity dam for the following data :*

Maximum allowable compressive stress in concrete	$= 3000\ kN/m^2$.
Maximum reservoir level	$= 200.0\ m$.
R.L. of bottom of dam	$= 100.0\ m$
Specific gravity of concrete	$= 2.4$.
Unit wt. of water	$= 10\ kN/m^3$.

Solution. The practical profile of dam will have a freeboard of about say 3 to 4% of dam height. Use 3 m as freeboard. The freeboard cannot be calculated as the wave height, etc. are not given.

The R.L. of the top of the dam

$$= 200.0 + 3.0 = 203.0\ \text{m}.$$

The height of the low gravity dam

$$= H_1 = \frac{f}{\gamma_w\,(S_c + 1)} \qquad \text{...(19.26)}$$

In this question,

$$f = 3000\ \text{kN/m}^2\ (i.e.\ 30\ \text{kg/cm}^2)$$
$$\gamma_w = 10\ \text{kN/m}^3$$
$$S_c = 2.4$$

$$\therefore \qquad H_1 = \frac{3000}{10\,(2.4 + 1)} = 88.2\ \text{m},\ < \text{Height of dam.}$$

Therefore, it is a high gravity dam.

Hence, the dam from RL 200.0 m to RL 200 – 88.2 = 111.8 m shall be designed as a low gravity dam, and the remaining bottom height of the dam from RL 111.8 m to 100.0 m shall be designed on the principles of high gravity dam.

Design of low dam between RL 200.0 m to RL 111.8

A section for this dam can be chosen as per the provisions of Fig. 19.16.

$$\text{Top width required} = a = \sqrt{\frac{H_1}{3.28}} = \sqrt{\frac{88.2}{3.28}} = \sqrt{26.9} = 5.18 \text{ m}$$

$$\text{Base width required} = \frac{H}{\sqrt{S}} = \frac{88.2}{\sqrt{2.4}} = 56.8 \text{ m.}$$

The upstream projection from the vertical face required

$$= \frac{a}{16} = \frac{5.18}{16} = 0.33 \text{ m (say)}$$

Total base width (B_1) provided

$$= 56.8 + 0.33 = 57.13 \text{ m.}$$

The u/s. batter starts at a depth of

$$2a\sqrt{S_c} = 2 \times 5.18 \times \sqrt{2.4} = 16.1 \text{ m.}$$

from below the M.W.L. and it ends at a depth of

$$3.1\, a\sqrt{S_c} = 3.1 \times 5.18\sqrt{2.4} = 24.9 \text{ m}$$

below the M.W.L. The section of this portion of dam is shown in Fig. 19.21 (*a*).

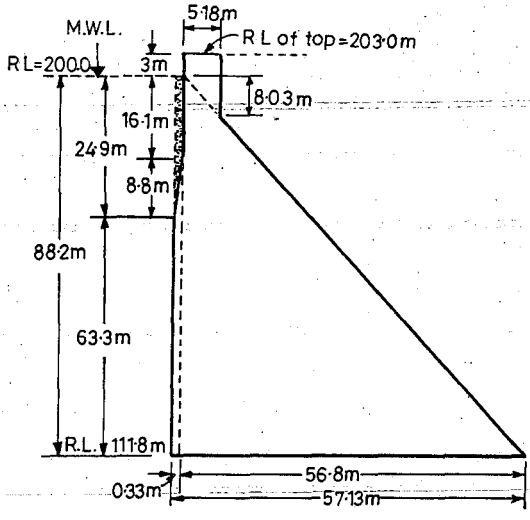


Fig. 19.21 (*a*). Low dam portion from RL 200 to RL 111.8.

Design of the Dam from RL 111.8 m to RL 100 m

Let us divide this length of 11.8 m into 3 strips. Let the depth of the I strip be 3.8 m and the depths of II and III strips be 4.0 m each.

Design of the 1st strip

Consider 1 m length of the dam.

$$= 734\,(1 + 0.01) - 860 \times 0.01$$
$$= 742 - 9 = \mathbf{733\ kN/m^2}$$

Shear stress at toe

$$\tau_0 = [p_{v(toe)} - p'] \tan \alpha$$
$$= (1490 - 60)\ 0.7 = 1430 \times 0.7 = 1001\ \text{kN/m}^2$$

Shear stress at heel

$$= -\,[p_{v(heel)} - p] \tan \phi$$
$$= -\,[734 - 860] \times 0.7 = 126 \times 0.7 = \mathbf{88.2\ kN/m^2}$$

Conclusions. We find that the dam is safe throughout except that the S.F.F. is equal to 3.72, while generally it should be between 4 to 5. The dam thus remains slightly unsafe in S.F.F. even when the seismic forces are not considered.

The results of stability analysis are given below :

The maximum shear stress developed in dam	$= 1001\ \text{kN/m}^2$.	**Ans.**
Maximum compressive stress developed in dam	$= 2191\ \text{kN/m}^2$	
No tension is developed anywhere.		
Factor of safety against sliding	$= 1.10$	
S.F.F.	$= 3.72$	
Factor of safety against overturning	$= 1.78$	

Example 19.4. *Design a concrete gravity dam for the following data :*

Maximum allowable compressive stress in concrete	$= 3000\ kN/m^2$.
Maximum reservoir level	$= 200.0\ m$.
R.L. of bottom of dam	$= 100.0\ m$
Specific gravity of concrete	$= 2.4$.
Unit wt. of water	$= 10\ kN/m^3$.

Solution. The practical profile of dam will have a freeboard of about say 3 to 4% of dam height. Use 3 m as freeboard. The freeboard cannot be calculated as the wave height, etc. are not given.

The R.L. of the top of the dam

$$= 200.0 + 3.0 = 203.0\ \text{m}.$$

The height of the low gravity dam

$$= H_1 = \frac{f}{\gamma_w\,(S_c + 1)} \qquad \text{...(19.26)}$$

In this question,

$$f = 3000\ \text{kN/m}^2\ (i.e.\ 30\ \text{kg/cm}^2)$$
$$\gamma_w = 10\ \text{kN/m}^3$$
$$S_c = 2.4$$
$$\therefore \quad H_1 = \frac{3000}{10\,(2.4 + 1)} = 88.2\ \text{m},\ < \text{Height of dam.}$$

Therefore, it is a high gravity dam.

Hence, the dam from RL 200.0 m to RL 200 – 88.2 = 111.8 m shall be designed as a low gravity dam, and the remaining bottom height of the dam from RL 111.8 m to 100.0 m shall be designed on the principles of high gravity dam.

Design of low dam between RL 200.0 m to RL 111.8

A section for this dam can be chosen as per the provisions of Fig. 19.16.

$$\text{Top width required} = a = \sqrt{\frac{H_1}{3.28}} = \sqrt{\frac{88.2}{3.28}} = \sqrt{26.9} = 5.18 \text{ m}$$

$$\text{Base width required} = \frac{H}{\sqrt{S}} = \frac{88.2}{\sqrt{2.4}} = 56.8 \text{ m.}$$

The upstream projection from the vertical face required

$$= \frac{a}{16} = \frac{5.18}{16} = 0.33 \text{ m (say)}$$

Total base width (B_1) provided

$$= 56.8 + 0.33 = 57.13 \text{ m.}$$

The u/s. batter starts at a depth of

$$2a\sqrt{S_c} = 2 \times 5.18 \times \sqrt{2.4} = 16.1 \text{ m.}$$

from below the M.W.L. and it ends at a depth of

$$3.1\, a\sqrt{S_c} = 3.1 \times 5.18\sqrt{2.4} = 24.9 \text{ m}$$

below the M.W.L. The section of this portion of dam is shown in Fig. 19.21 (*a*).

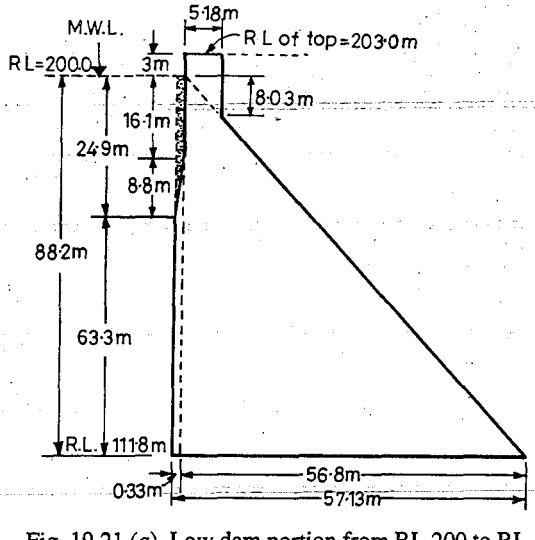


Fig. 19.21 (*a*). Low dam portion from RL 200 to RL 111.8.

Design of the Dam from RL 111.8 m to RL 100 m

Let us divide this length of 11.8 m into 3 strips. Let the depth of the I strip be 3.8 m and the depths of II and III strips be 4.0 m each.

Design of the 1st strip

Consider 1 m length of the dam.

Weight of dam section up to RL 111.8 m. (i.e. weight of low dam)

$$= 24 \times 1 \left[\frac{1}{2} \times 8.8 \times 0.33 + 63.3 \times 0.33 + \frac{1}{2} + 56.8 \times 88.2 + 3 \times 5.18 + \frac{1}{2} \times 8.03 \times 5.18 \right]$$

$$= 24\,[1.5 + 20.9 + 2{,}505 + 15.5 + 20.8]$$

$$= 24\,[2{,}563.7] = \mathbf{61{,}600\ kN.}$$

Approximate width of bottom of 1st strip (say B_2') is obtained by drawing a horizontal line at RL 108 m and by producing the already provided d/s face and u/s face of low dam, as shown in Fig. 19.21 (*b*).

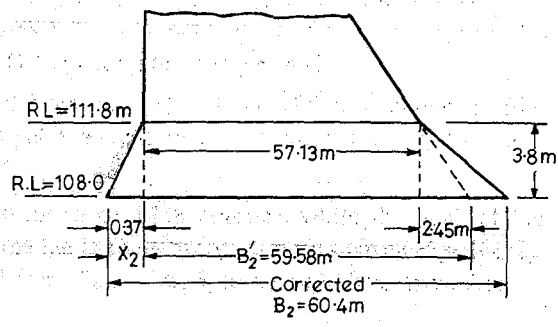


Fig. 19.21 (*b*)

$$B_2' = 57.13 + \frac{56.8}{88.2} \times 3.8$$

$$= 57.13 + 2.45 = 59.58 \text{ m}$$

Approximate weight of 1st strip

$$= 24 \times 1 \left[\frac{57.13 + 59.58}{2} \right] \times 3.8$$

$$= 24 \times 58.35 \times 3.8 = 5320 \text{ kN}$$

Weight of water resting on the u/s face

$$= 10 \times 1 \left[16.1 \times 0.33 + \frac{1}{2} \times 8.8 \times 0.33 \right]$$

$$= 10 \times 0.33 \times 20.5 = 67.7\text{ ; } \mathbf{say\ 68\ kN.}$$

Total weight of dam and water at top of 1st strip, *i.e.* at base of small dam

$$= W_1 = 61{,}600 + 68 = 61{,}668 \text{ kN ; } \mathbf{Say\ 61670\ kN}$$

Total approximate weight of dam and water at base of 1st strip

$$= W_2 = 61{,}670 + \text{App. weight of 1st strip}$$

$$= 61{,}670 + 5320 = \mathbf{66{,}990\ kN.}$$

The correct base width B_2, which shall keep the maximum compressive stresses within the allowable limits is given by equation [19.29 (*a*)], *i.e.*

$$B_2 = \sqrt{\frac{\gamma_w \cdot H_2^3}{f} \left[1 + \frac{\gamma_w^2 \cdot H_2^4}{4 \cdot W_2^2} \right]}$$

where $H_2 = 92$ m

$W_2 = 66{,}990$ kN

$f = 3000$ kN/m^2

$\gamma_w = 10$ kN/m^3.

Substituting, we get

$$B_2 = \sqrt{\frac{10 \times (92)^3}{3000} \left[1 + \frac{10^2 \cdot (92)^4}{4 \times (66{,}990)^2} \right]}$$

$$= \sqrt{2596\,(1.4)} = \sqrt{3{,}634} = \mathbf{60.4\ m}$$

Now let us find out the projection X_2 of base B_1 on the u/s side. It is given by equation [19.30 (*a*)], *i.e.*

$$\frac{\gamma_w \cdot S_s \cdot y_1}{24}\left[3B_1^2 - B_2^2 + 6X_2(B_1 + B_2) + 2B_1B_2\right]$$
$$-\frac{\gamma_w \cdot X_2}{12}[H_1 + H_2][2B_2 - 3X_2] - W_1\left[\frac{B_2 - B_1}{3} - X_2\right] = 0.$$

where $y_1 = 3.8$ m

$B_1 = 57.13$ m

$B_2 = 60.4$ m

$H_1 = 88.2$ m

$H_2 = 92$ m

$W_1 = 61{,}670$ kN

Substituting, we get

$$\frac{10 \times 2.4 \times 3.8}{24}[3(57.13)^2 - (60.4)^2 + 6X_2(57.13 + 60.4) + 2 \times 57.13 \times 60.4]$$
$$-\frac{10 \cdot X_2}{12}[88.2 + 92][2 \times 60.4 - 3X_2] - 61{,}670\left[\frac{60.4 - 57.13}{3} - X_2\right] = 0.$$

or $\quad 3.8\,[9{,}780 - 3{,}640 + 705X_2 + 6{,}900] - 10X_2[1{,}815 - 45X_2] - 61{,}670\,[1.09 - X_2]$

or $\quad 4.960 + 268X_2 - 1{,}815X_2 + 45X_2^2 - 6{,}720 + 6{,}167X_2 = 0$

or $\quad 45X_2^2 + 4{,}620X_2 - 1{,}760 = 0$

Solving this equation, we get

$$\mathbf{X_2 = 0.37\ m\ (say).}$$

Hence, out of 60.4 m base width at base of 1st strip, 0.37 m shall be provided on u/s side as shown in Fig. 19.21 (*b*). This will ensure that the resultant shall pass through the inner third point when the reservoir is empty and no tension will develop at toe.

Design of II Strip. It shall be designed on the same lines on which 1st strip has been designed.

Corrected weight of 1st strip

$$= 24 \times 1\left[\frac{57.13 + 60.4}{2}\right] \cdot 3.8 = 1.2 \times 117.53 \times 3.8 = 5370\text{ kN}.$$

Weight of water resting on the u/s face of first strip

$$= 10 \cdot \left[88.2 \times 0.37 + \frac{1}{2} \times 3.8 \times 0.37\right]$$
$$= 3.7\,[88.2 + 1.9] = 383\text{ kN}$$

Total corrected weight at the base of 1st strip

$$W_2 = 61{,}670 + 5370 + 383$$
$$= 67{,}423\text{ kN. } \textbf{Say 67,420 kN.}$$

Depth of II strip $= y_2 = 4.0$ m.

Approximate base width B_3' of the bottom of II strip

$$= 60.4 + \frac{56.8}{88.2} \times 4.0 + \frac{0.37}{3.8} \times 4.0$$

$$= 60.4 + 2.58 + 0.39 = 63.37\text{ ; say } \mathbf{63.4\ m.}$$

Approximate weight of II strip

$$= 24 \times 1 \left[\frac{60.4 + 63.4}{2}\right] \times 4 = 5940 \text{ kN}$$

Approximate weight of water resting on the u/s face of II strip, if the u/s slope given to 1st strip is extended in the II strip

$$= \left[\frac{92 + 96}{2} \times 0.39\right] 10 = \mathbf{370\ kN}$$

Hence, the total approximate weight of dam and water at base of II strip

$$= W_3 = 67{,}420 + .5940 + 370 = \mathbf{73{,}730\ kN}$$

The corrected base width B_3 at bottom of II strip

$$B_3 = \sqrt{\frac{\gamma_w^2 \cdot H_3^3}{f}\left[1 + \frac{\gamma_w^2 H_3^4}{4W_3^2}\right]}$$

$$B_3 = \sqrt{\frac{10 \times (96)^3}{3000}\left[1 + \frac{10^2\,(96)^4}{4 \times (73{,}730)^2}\right]}$$

$$= \sqrt{2{,}949\,(1 + 0.39)} = \sqrt{4{,}100} = \mathbf{64.0\ m.}$$

Now X_3 is obtained from the equation [19.30 (*b*)], *i.e.*

$$\frac{\gamma_w \cdot S_s \cdot y_2}{24}\left[3B_2^2 - B_3^2 + 6X_3\,(B_2 + B_3) + 2B_2B_3\right]$$

$$- \frac{\gamma_w \cdot X_3}{12}\,[H_2 + H_3]\,[2B_3 - 3X_3] - W_2\left[\frac{B_3 - B_2}{3} - X_3\right] = 0$$

where $y_2 = 4.0$ m

$B_2 = 60.4$ m

$B_3 = 64.0$ m

$H_2 = 92$ m

$H_3 = 96$ m

$W_2 = 6{,}742$ tonnes

$\gamma_w = 10$ kN/m^3

Substituting, we get

$$\frac{10 \times 2.4 \times 4.0}{24}\left[3\,(60.4)^2 - (64.0)^2 + 6 \cdot X_3\,(60.4 + 64.0) + 2 + 60.4 \times 64.0\right]$$

$$- \frac{10 \cdot X_3}{12}\left[92 + 96\right]\left[2 \times 64.0 - 3X_3\right] - 67{,}420\left[\frac{64.0 - 60.4}{3} - X_3\right] = 0$$

or $0.4\,[10{,}920 - 4{,}100 + 747\,X_3 + 7{,}750] - X_3[2010 - 47X_3] = 6{,}742\,[1.20 - X_3] = 0.$

or $5{,}828 + 299X_3 - 2{,}010X_3 + 47X_3^2 - 8{,}091 + 6{,}742X_3 = 0$

or $47X_3^2 + 5{,}031X_3 - 2{,}263 = 0$

or $X_3^2 + 107X_3 - 48.2 = 0$

Solving this equation, we get, $\mathbf{X_3 = 0.45\ m}$

Hence, out of 64.2 m of base width the the base of II strip, 0.45 m shall be provided on u/s side as shown in Fig. 19.21 (*c*).

Design of last, *i.e.* third strip

Corrected weight of II strip

$$= 24 \times 1 \left[\frac{60.4 + 64.0}{2}\right] \times 4.0 = 5980 \text{ kN}$$

Weight of water resting on the u/s face of II strip

$$= 10\left[92 \times 0.45 + \frac{1}{2} \times 4 \times 0.45\right] = 500 \text{ kN.}$$

Total corrected weight at the base of II strip

$$W_3 = 67{,}420 + 5980 + 500 = \mathbf{73{,}900\ kN}$$

Depth of III strip = 4.0 m.

Approximate base width B_4' at the bottom of III strip

$$= 64.0 + \frac{56.8}{88.2} - 4.0 + 0.45$$

$$= 64.0 + 2.58 + 0.45 = 67.03 \text{ m.}$$

Approximate weight of III strip

$$= 24 \times 1 \left[\frac{64.0 + 67.03}{2}\right] \times 4.0 = 6280 \text{ kN}$$

Approximate weight of water resting on the u/s face of III strip if the u/s slope given to II strip is extended in III strip

$$= 10\left[\frac{96 + 100}{2} \times 0.45\right] = 442 \text{ kN}$$

Hence, the total approximate weight of dam and water at base of III strip

$$= W_4 = 73{,}980 + 6280 + 442 = 80{,}620 \text{ kN}$$

The corrected base width B_4 at the bottom of III strip is given by

$$B_2 = \sqrt{\frac{\gamma_w \cdot H_4^3}{f}\left[1 + \frac{\gamma_w^2 \cdot H_4^4}{4W_4^2}\right]}$$

where $f = 3000 \text{ kN/m}^2$

$H_4 = 100 \text{ m}$

$W_4 = 80{,}620 \text{ kN.}$

$$\therefore \qquad B_4 = \sqrt{\frac{10 \times (100)^3}{3000}\left[1 + \frac{10^2 \times (100)^4}{4 \times (80620)^2}\right]}$$

$$= \sqrt{3{,}333\,[1 + 0.385]} = \sqrt{3{,}333\,(1.385)} = \sqrt{4{,}620} = 68.0 \text{ m.}$$

Now X_4 is obtained from the equation

$$\frac{\gamma_w \cdot S_s \cdot y_3}{24}\left[3B_3^2 - B_4^2 + 6X_4\,(B_3 + B_4) - 2B_3B_4\right]$$

$$- \frac{\gamma_w \cdot X_4}{12}\left[H_3 + H_4\right]\left[2B_4 - 3X_4\right] - W_3\left[\frac{B_4 - B_3}{3} - X_4\right] = 0,$$

where $y_3 = 4.0 \text{ m}$

$B_3 = 64.0 \text{ m}$

$$B_4 = 68.0 \text{ m}$$
$$H_3 = 96 \text{ m}$$
$$H_4 = 100 \text{ m}$$
$$W_3 = 7{,}390 \text{ tonnes.}$$

Substituting, we get

$$\frac{10 \times 2.4 \times 4.0}{24}\left[3\,(64.0)^2 - (68.0)^2 + 6 \cdot X_4\,(64.0 + 68.0) + 2 \times 64.0 \times 68.0\right]$$
$$-\frac{10 \cdot X_4}{12}\left[96 + 100\right]\left[2 \times 68.0 - 3X_4\right] - 73{,}900\left[\frac{68.0 - 64.0}{3} - X_4\right] = 0$$

or $4\,[12{,}288 - 4{,}624 + 792X_4 + 8.704] - 10X_4[2{,}220 - 49X_4] - 73{,}900\,[1.33 - X_4] = 0$

or $65{,}470 + 3170X_4 - 22{,}200X_4 + 490X_4^2 - 98{,}530 + 73{,}900X_4 = 0$

or $6{,}547 + 317x_4 - 2{,}220X_4 + 49X_4^2 - 9{,}853 + 7{,}390X_4 = 0$

or $49X_4^2 + 5{,}487X_4 - 3{,}406 = 0$

or $X_4^2 + 112X_4 - 69.5 = 0$

Solving, we get $\mathbf{X_4 = 0.62}$ **m.**

Hence, out of base width of 68.0 m, at the base of III strip, 0.62 m shall be provided on the u/s side, and the remaining shall be provided on d/s side as shown in Fig. 19.21 (*c*).

The design of all the bottom strips is shown in Fig. 19.21 (*c*).

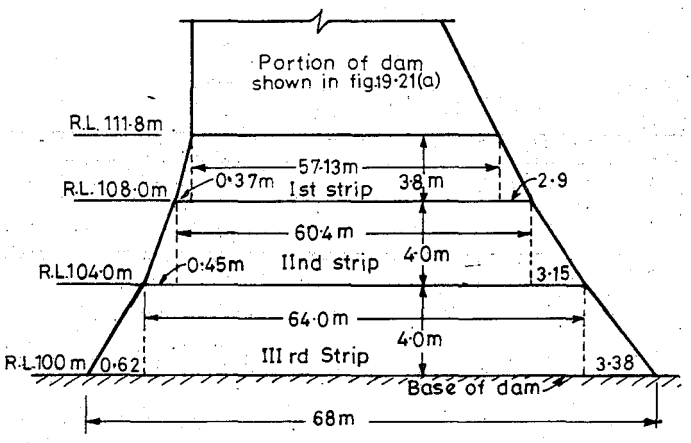


Fig. 19.21 (*c*)

CONSTRUCTION OF GRAVITY DAMS

19.11. Diversion Problem in Dams Construction

Before the actual construction of a dam can start in a river channel, the water of the river channel must be temporarily diverted. It is advantageous to schedule the construction of the lower portion of the dam during normal periods of low flow so as to minimise the diversion problem. The diversion of river water can be accomplished in either of the following two ways :

(*i*) **Provision of a Diversion Tunnel.** If geological and topographical conditions are favourable, a diversion tunnel or a diversion open channel may be constructed to carry the entire flow around the dam site as shown in Fig. 19.22. The area in which construction work has to take place, is closed by coffer-dams. The diversion tunnel or channel will start from upstream of the upstream coffer-dam and will join the river again on the downstream of the downstream coffer-dam, as shown in Fig. 19.22.

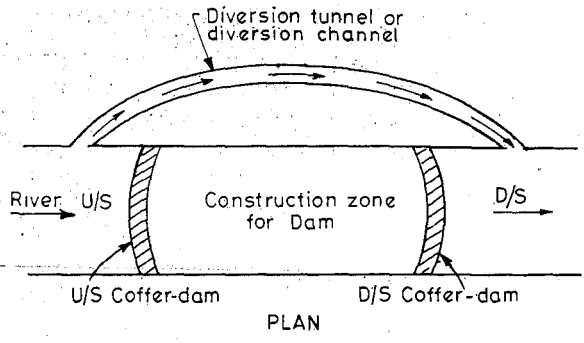


Fig. 19.22

(*ii*) **By constructing the dam in two stages.** The dam is sometimes constructed in two stages. In such a case, the flow is, first of all, diverted and confined to one side of the channel by constructing a semi-circular type of a coffer-dam as shown in Fig. 19.23. The construction work can be taken up in the water-free zone. When the work on the lower portion of the dam on half of its length in one side of the channel gets completed, the remaining half width of the channel is closed by a coffer-dam, as shown in Fig. 19.24. The flow is diverted through the dam outlets or sometimes it may even be allowed to overtop the already constructed portion of the dam. The work will continue in the water-free zone.

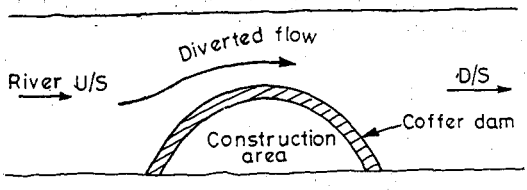


Fig. 19.23

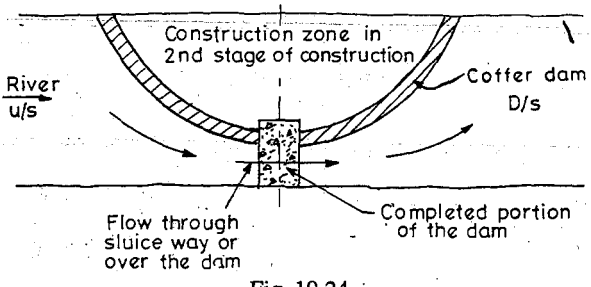


Fig. 19.24

19.12. Construction of Galleries in Gravity Dams

Galleries are the horizontal or sloping openings or passages left in the body of the dam. They may run longitudinally (*i.e.* parallel to dam axis) or transversely (*i.e.* normal to dam axis) and are provided at various elevations. All the galleries are interconnected by steeply sloping passages or by vertical shafts fitted with stairs or mechanical lifts. The size of a gallery will depend upon the size of the dam and the function of the gallery.

Functions and Types of Galleries in Dams

(1) **Foundation Gallery.** A gallery provided in a dam may serve one particular purpose or more than one purpose. For example, a gallery provided near the rock

foundations, serves to drain off the water which percolates through the foundations. This gallery is called a *foundation gallery* or a *drainage gallery*. It runs longitudinally and is quite near to the upstream face of the dam. Its size usually varies from1.5 m × 2.2 m to 1.8 m × 2.4 m. Drain holes are drilled from the floor of this gallery after the foundation grouting has been completed. Seepage is collected through these drain holes. The size of the gallery should be sufficient to accommodate atleast a drilling machine. Besides draining off seepage water, it may be helpful for drilling and grouting of the foundations, when this can not be done from the surface of the dam.

(2) **Inspection Galleries.** The water which seeps through the body of the dam is collected by means of a system of galleries provided at various elevations (say at heights of 15 m or so) and interconnected by vertical shafts, etc. All these galleries, besides draining off seepage water, serve inspection purposes. They provide access to the interior of the dam and are, therefore, called *Inspection galleries*. However, galleries in dams are seldom provided for purely inspection purposes. They generally serve other purposes along with this purpose. Their main functions are summarised below :

(*i*) They intercept and drain off the water seeping through the dam body.

(*ii*) They provide access to dam interior for observing and controlling the behaviour of the dam.

(*iii*) They provide enough space for carrying pipes, etc. during artificial cooling of concrete, (see next article).

(*iv*) They provide access for grouting the contraction joints when this cannot be done from the face of the dam.

(*v*) They provide access to all the outlets and spillway gates, valves, etc. by housing their electrical and mechanical controls. All these gates, valves, etc. can hence, be easily controlled by men, from inside the dam itself.

(*vi*) They provide space for drilling and grouting of the foundations, then it cannot be done from the surface of the dam. Generally, the foundation gallery is used for this purpose.

Foundation gallery is generally differently named from inspection galleries, although strictly speaking, it can also be used for inspection purposes and may be called as inspection gallery.

19.12.1. Cross-sections of Dam Galleries. Dam galleries are formed as the concrete is placed and its size depends upon the function of the gallery and also upon the size of the dam. Certain important shapes of the commonly used galleries are shown below in Fig. 19.25.

The provision of a gallery in a dam body, changes the normal pattern of stresses in the body of the dam. Stress concentration may, therefore, occur at corners, and hence, in order to minimise this stress concentrations, the corners mast be rounded smoothly. Tension and compression zones may be worked out and proper reinforcements, etc. are provided to counteract them.

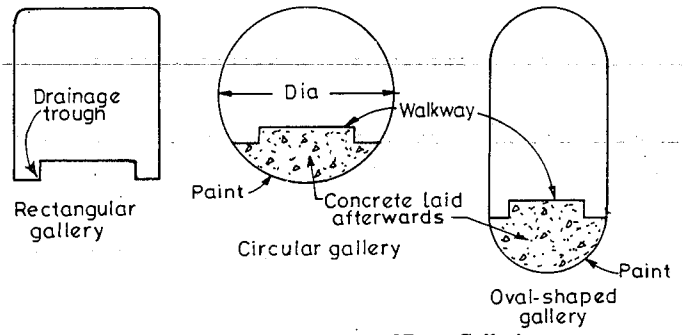


Fig. 19.25. Various Types of Dam Galleries.

19.13. Cracking of Concrete in Concrete Gravity Dams

When concrete sets, a tremendous amount of heat is liberated (due to heat of hydration of cement), which will raise the temperature inside the body of the dam. But the temperature outside the dam remains equal to the atmospheric temperature. Due to these temperature differences, temperature stresses get developed in the dam body. Besides, *due to shrinkage of concrete as it cools, shrinkage stresses get developed.* These temperature stresses and shrinkage stresses will cause the concrete to crack unless remedial measures are undertaken. Various measures, generally adopted in concrete gravity dams, to avoid this cracking are :

(*i*) Using minimum amount of cement in a given mix of specified strength. The quantity of cement can be decreased by better grading the aggregates.

(*ii*) 'Low lifts' should be used for concrete. When concrete is poured, it is poured up to a certain height in the first attempt. This height is called 'lift'. Generally, 1.5 m lift is used in modern dams. If this lift is reduced, more horizontal joints will get developed and also sufficient cooling time between two successive pours shall be obtained, thus reducing cracking.

(*iii*) By providing suitably spaced contraction joints, in addition to the normal construction joints.

(*iv*) Special low heat cements may be used.

(*v*) The materials which go into the concrete, may be cooled before mixing.

(*vi*) Further cooling is accomplished by circulating cold water through pipes embedded in concrete. This is quite an expensive measure and is adopted only for large gravity dams.

19.14. Joints in a Gravity Dam

The concreting of the dam is usually placed in blocks. The size of blocks will depend upon the size of dam and necessity of contraction joints required from the considerations of cracking of concrete. Their width is usually not more than 15 m or so, in large dams. Maximum height of a single pour of concrete (called 'lift') is usually about 1.5 m or so. The alternate blocks of the very first layer which is laid immediately over the rock foundation is taken as 0.75 m deep, as shown in Fig. 19.26. Dam sections are poured alternately so that each block is permitted to stand serveral days before another layer is poured on top of the first. Care must be taken to protect the individual dam sections from the drying action of air by ensuring full-fledged curing. After the shutterings are removed, the lateral surfaces of each section are painted with an asphaltic emulsion paint so as to prevent its adherence to the adjoining sections and thus to serve as construction joints to reduce cracking in concrete. The *Vertical joints* so developed are called **the transverse joints** and they run through the entire height and extend through the full width of the dam section. Transverse joints will thus divide the dam length into a number of vertical cantilevers each of which is independent of the other ; justifying the two dimensional analysis of gravity dams.

The *horizontal joints* called **longitudinal joints** are developed at each lift height and will extend through the entire width of the dam section. They shall run through the entire length of the dam but shall be staggered between transverse joints, as shown in Fig. 19.26.

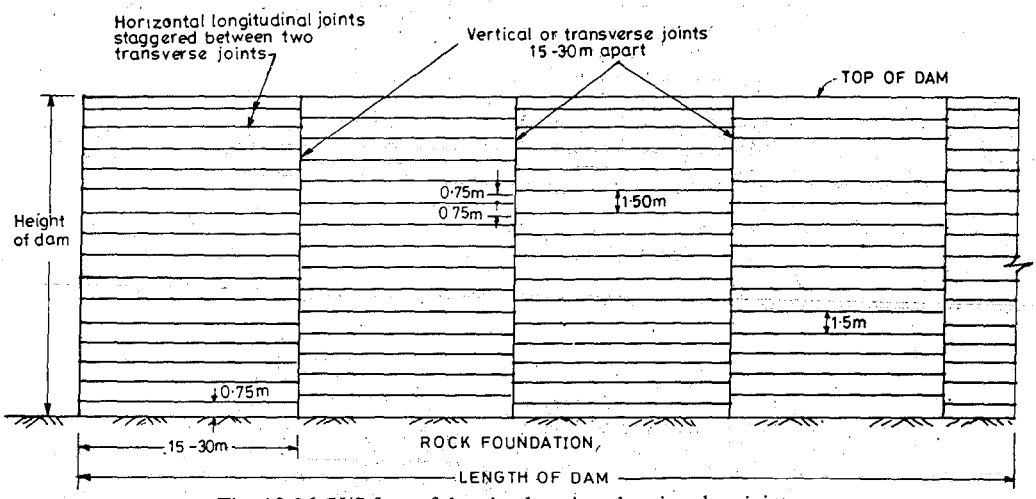


Fig. 19.26. U/S face of dam in elevation showing dam joints.

Construction joints and Contraction joints. Truly speaking, every joint whichsoever is left in the dam, shall be a construction joint and every *construction joint* will oppose contraction stresses, and hence will be a *contraction joint.* Therefore, there should be no difference between the two. But whether the joint left was needed from the considerations of practical difficulties in laying the concrete in a single stretch or it was left intentionally for making provisions for shrinkage and temperature stresses, sometimes defines the limits of these two terms. In other words, horizontal joints which were a must from considerations of lift, are sometimes called the construction joints ; while the joints which are mainly left for shrinkage of concrete are called the contraction joints. Although most of the times, these two terms are used to mean one and the same thing.

19.14.1. Shear Keys or Key ways. Where foundation conditions are such that undesirable differential settlements or displacements between adjacent concrete blocks may occur, shear keys are formed in transverse vertical joints between sections to carry the shear from one section to the adjacent one and make the dam act as a monolithic structure. These shear keys between the transverse joints are seldom provided on good foundations and in U-shaped valleys, as they will unjustify the two dimensional design of the dam.

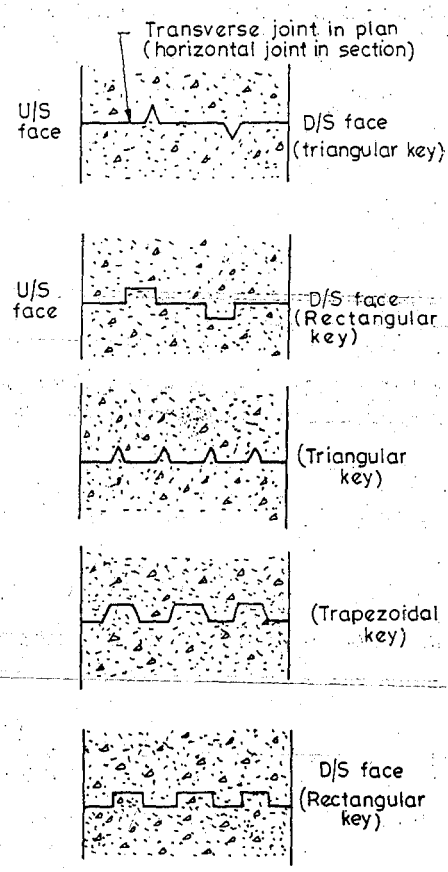


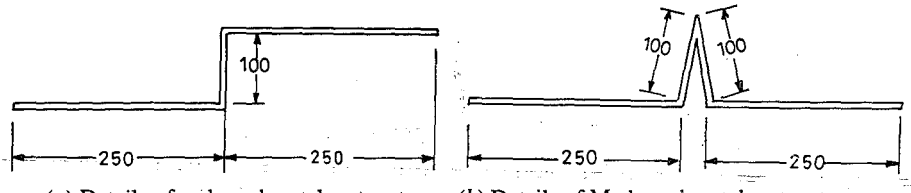
Fig. 19.27. Various types of key-ways.

Shear keys are however, always provided between the horizontal- longitudinal joints in order to transmit vertical shearing stresses across the section. After the concrete has undergone its shrinkage, these longitudinal horizontal joints are grouted through pipes embedded in the blocks before hand. The transverse vertical joints may also be keyed and grouted if the monolithic behaviour is desired as is generally done on poor foundations and in narrow V-shaped gorges. Various types of key ways used these days are shown in Fig. 19.27.

19.14.2. Water Stops. Water stops or water bars, as used in R.C.C. aqueducts (discussed in article 14.6), are also required to be provided in the transverse as well as horizontal joints in concrete adjacent to the upstream face of dam. The openings of the vertical transverse joints as well as of the longitudinal horizontal joints in a dam body (Fig. 19.26), if not sealed properly with water stops, will provide passage for seepage of water through the dam body. To stop this leakage, water stops consisting of metal strips, rubber or PVC are installed across the joints adjacent to the upstream face. *Asphalt water stops* are used as secondary water stops to the metal and rubber/PVC water stops.

In **metal water stops**, copper or stainless steel strips of not less than 1.5 mm thickness and 300 mm width are used. Z-shape and M-shape metal stops, as shown in Fig. 19.28 (*a*) and (*b*), are generally used.

Rubber water stops are made from natural rubber confirming to the requirements given in table 19.3. The shape of such water stops may vary, depending upon the water head, etc. The usual minimum width of rubber water stops for dams is 300 mm.



(*a*) Details of z-shaped metal water stop (*b*) Details of M-shaped metal water stop

Fig. 19.28. Various types of metal waterstops.

PVC waterstops should be fabricated from plastic compounds, the basin resin of which shall be polyvinyl chloride. The compound shall contain any additional resins, plasticizers, inhibitors or other material such that when the material is compounded, it shall meet the requirements given in table 19.3.

Table 19.3. Performance Requirements of Rubber or PVC Water Stops

S. No.	*Characteristics*	*Unit*	*Values*
(*i*)	Tensile strength	N/mm^2	11.5, min
(*ii*)	Ultimate strength	%	80, min
(*iii*)	Tear resistance	N/mm^2	4.9, min
(*iv*)	Stiffness in flexure	N/mm^2	2.46, min
(*v*)	Accelerated extraction		
	(*a*) Tensile strength	N/mm^2	10.5, min
	(*b*) Ultimate alongation	%	250, min
(*vi*)	Effect of alkali : 7 days		
	(*a*) Weight increase	%	0.10, max
	(*b*) Weight decrease	%	0.10, max.
	(*c*) Hardness change	Point	± 5
(*vii*)	Effect of alkali : 28 days		
	(*a*) Weight increase	%	0.40, max
	(*b*) Weight decrease	%	0.90, max
	(*c*) Dimension change	%	± 1

Certain typical shapes in which rubber/PVC water stops, suitable for concrete gravity dams, may be made are shown in Fig. 19.29. The shape recommended by IS : 12200-1987 for dams is shown in Fig. 19.30.

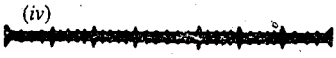
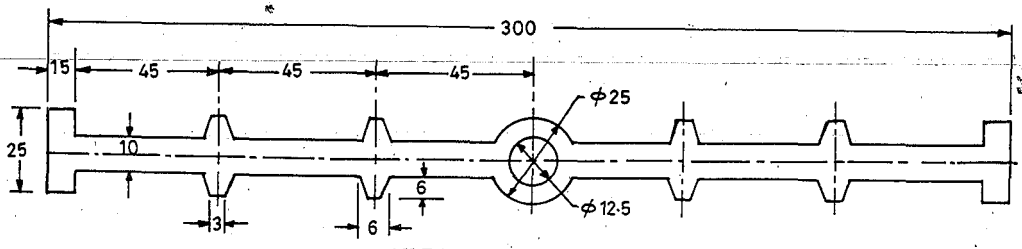
Shape of PVC or Rubber Waterstop	*Type*	*Category*	*Width*	*Thickness*	*Hydraulic Heads*	*Safe Stability*
(*i*) 	230XA	Dumb Bell with Central Bulb	230	10	50	Used where large movement due to expansion/contraction is expected. Also for settlement joints and crumple sections
(*ii*) 	230G	Serrated with Central bulb	230	4-5	25	For thicker concrete walls and with lower hydrostatic pressures for expansion joints.
(*iii*) 	240H	Serrated with Central bulb	240	8-10	50	For thicker concrete wall and for higher hydrostatic pressure for expansion and construction joints.
(*iv*) 	240RS	Ribbed and serrated	240	5	25	For construction joints
(*v*) 	305F	Serrated with Central bulb	305	8-12	60	For very high hydrostatic pressure and where higher grading of concrete aggregates used for expansion joints in Dams etc.
(*vi*) 	300N	Ribbed with Central bulb	300	10-12	60	Same as 305F but slightly thicker, specially designed for Narmada Project.
(*vii*) 	230KD	Kicker	230		25	Externally placed where re-inforcement in R.C.C. elements congested. Used for construction joints.
(*viii*) 	230KE	Kicker with Central bulb	230	5	25	Externally placed in R.C.C. elements congested. Used for movement joints.
(*ix*) 	320KZ	Kicker	320	5	25	Surface waterstops. Used for construction joints of long lengths.

Fig. 19.29. Certain typical shapes in which rubber/PVC waterstops may be made to suit individual dam project.



All Dimensions in mm

Fig. 19.30. Detailed dimensions of a rubber/PVC waterstop recommended by IS : 12200-1987.

Metal and rubber/PVC water stops are often used together to reduce leakage through the joints in a dam as shown in Fig. 19.31 to 19.34.

An **Asphalt water stop** is constructed by forming a well of square opening across the contraction joints and filling the opening with asphaltic compound. The well may be fitted in advance with a steam pipe or an electrical heat conductor for reliquefying the asphalt. The asphalt to be used should confirm to the specifications shown in table 19.4 below.

Table 19.4. Recommended Specifications of Asphalt as per IS : 12200-1987 for Asphalt Joints

S. No.	*Property*	*Recommended Value*
1.	Density	1015–1065 kg/m^3
2.	Penetration at 25°C	200–300 mm
3.	Softening point (Ring and ball test)	80–90°C
4.	Brittleness test on 22 mm^2 specimen at 5°C, (energy absorbed)	0.97 kgm

The location, shape and dimensions of an asphalt water stop (square hole filled with asphalic compound) are shown within a typical sectional plan of a contraction joint in Fig. 19.31.

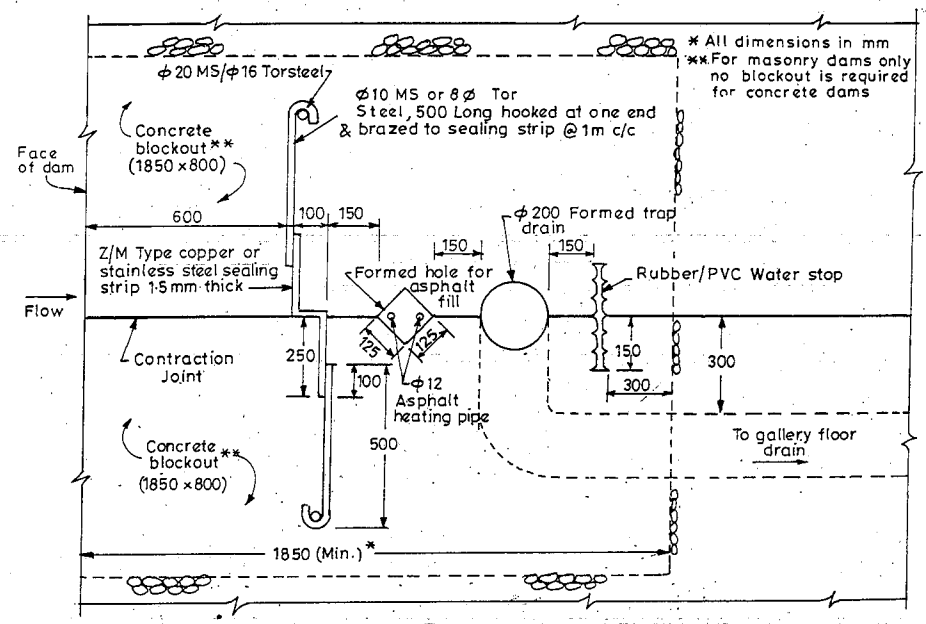


Fig. 19.31. Sectional plan at a contraction joint (square hole shows location of asphalt water stop)
Note : Dowel bars between concrete/masonry face of the blockout not shown.

19.14.2.1. Installation of water stops

(1) The metal waterstops shall be erected with the help of anchor rods.

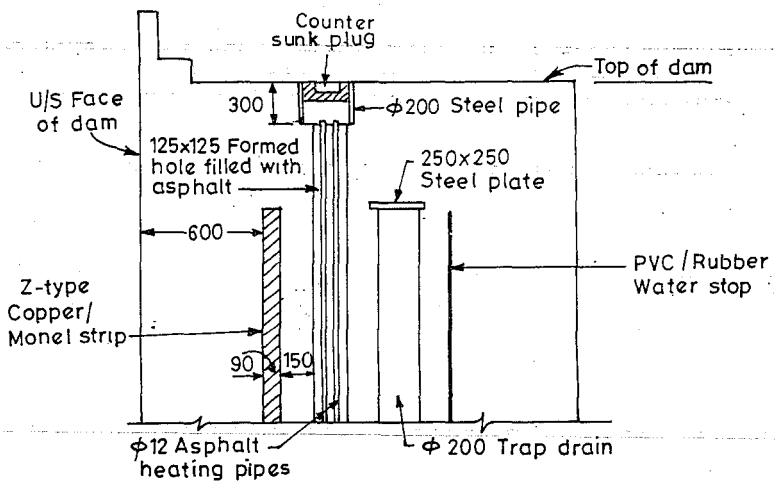
(2) In the case of masonry dams, the surface adjacent to the blockouts (shown by dotted lines in Fig. 19.30) shall be irregular and the joints in the masonry shall be raked out when mortar is green, with some stone protruding beyond dotted lines regularly in both directions. No such blockouts shall be provided in concrete dams where concreting on either side of the water seals is done along with the concreting of the rest of the concreting block.

(3) 25 mm dia dowel bars, 1500 mm long (500 mm in concrete and 1000 mm in masonry) 500 mm c/c in both directions shall be provided at the concrete/masonry interface of the blockout in case of a masonry dam to prevent shrinkage crack at the interface.

(4) The blockout may be concreted in lifts not more than 1.5 m. Minimum grade of concrete to be used in the blockout shall be M20.

(5) The blockout of one block may be concreted first and the joint face given a coat of coaltar black paint conforming to IS : 290-1961 and then only the blockout of the second block should be concreted, so as to have a clear contraction joint.

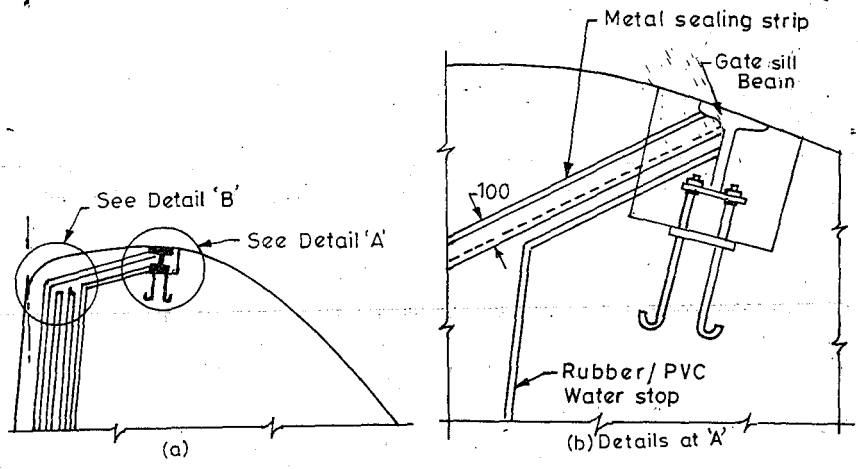
(6) Typical details of waterstop arrangement (at contraction joints between two monoliths of a dam) near the top of a non-overflow section are shown in Fig. 19.32 ; near the crest of an overflow section in Fig. 19.33 and near the bottom of the dam in Fig. 19.34.

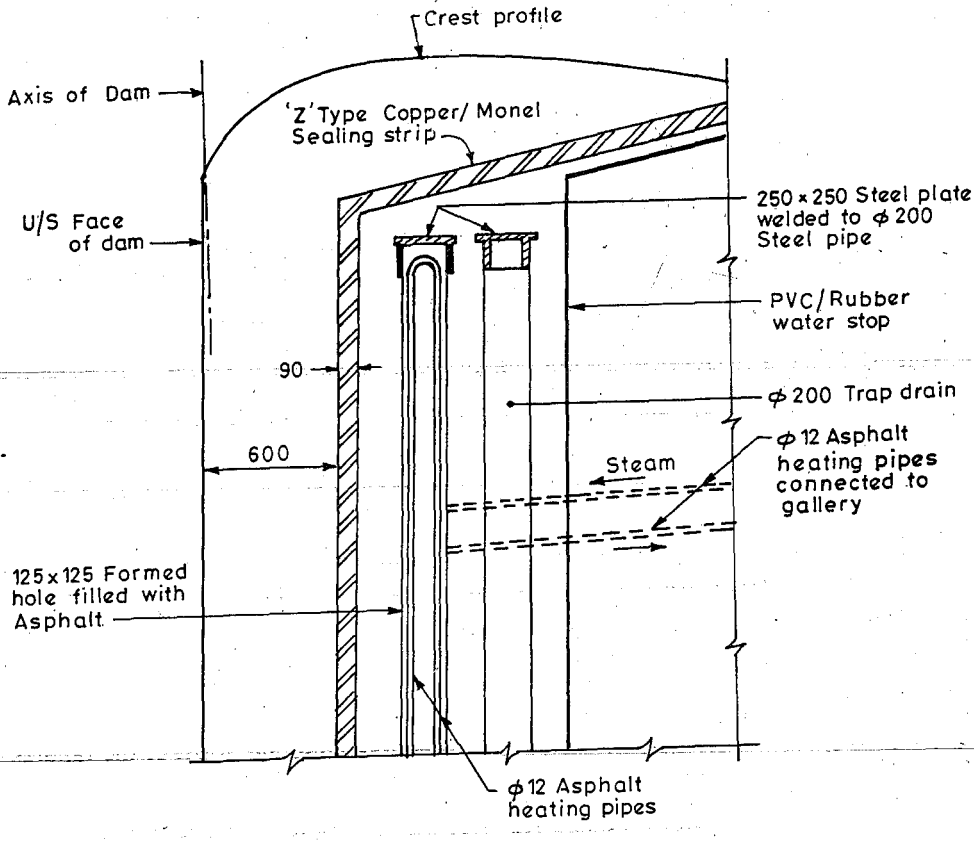


All Dimensions in mm

Fig. 19.32. Typical waterstop details near the top of non-overflow section of a dam.

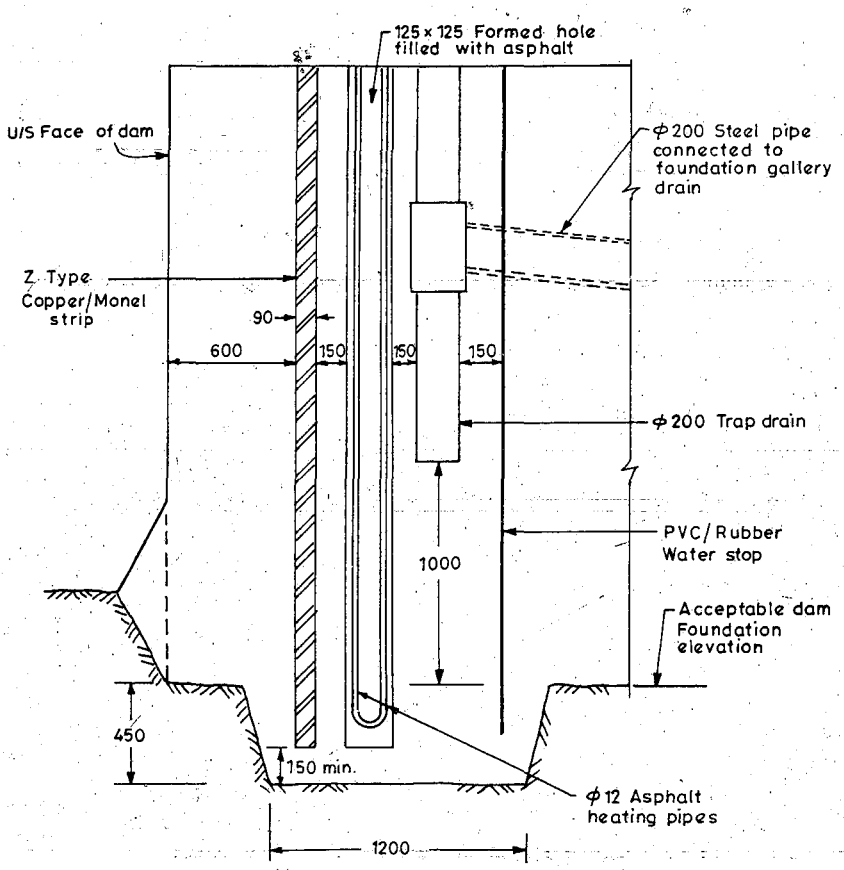
*IS code on "Methods of Testing Plastics".





(*c*) Details at 'B'
All dimensions in mm

Fig. 19.33. Typical waterstop details near the crest of overflow section (spillway).



All dimensions in mm

Fig. 19.34. Typical waterstop details near the bottom of the dam.

Rubber/PVC waterstops shall be provided around galleries/adits at the contraction joints between two monoliths of a dam, as shown in Fig. 19.35.

19.15. Foundation Treatment for Gravity Dams

The material underlying the base of a dam, *i.e.* the foundations of the dam, must be strong enough and capable to withstand the foundation pressure exerted on it under various conditions of loading and in dry as well as wet condition. Most of the failures of the dams have occurred because of the failure of their underlying strata. A concrete gravity dam of California called St. Francis dam was about 62 m high and about 210 m long and failed soon after its completion. The cause of failure was found to be the

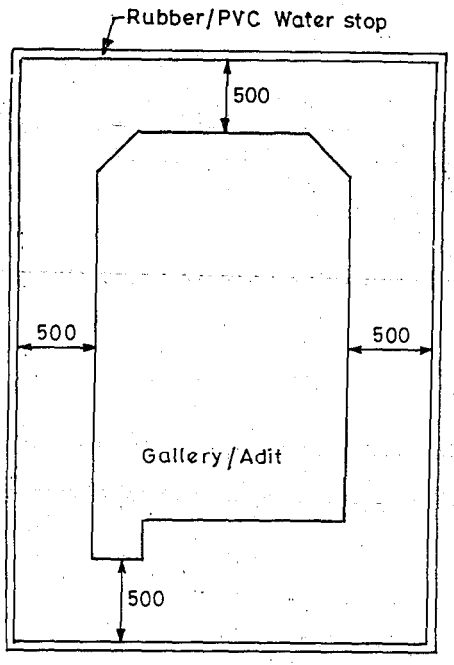


Fig. 19.35. Rubber/PVC waterstop around Dam Gallery/Adit at contraction joints.

presence of conglomerate in one abutment, which was weakened after exposure to moisture from the reservoir. Similarly, the failure of a 60 m high arch dam in France (called Malpasset dam) was attributed to the presence of a clay seam in the rock at one of the abutments. Austin Dam on the Colorado river in Texas failed in the year 1900, because large cavities had been dissolved in its limestone foundation.

All these examples have been quoted just to stress upon the readers and the designers, the importance of foundations and the need for their thorough investigation and remedial treatment, if any thing special is found adverse. Besides the special remedial measures in particular cases, the foundation treatment commonly adopted for all foundations can be divided into two steps :

(1) *Preparing the surface* ; and

(2) *Grouting the foundations.*

These treatments are briefly discussed below :

(1) **Preparing the Surface.** The surface preparation consists in removing the entire loose soil till a sound bed rock is exposed. The excavation should be carried out in such a way that the underlying rock is not damaged. The final surface obtained above is stepped, so as to increase the frictional resistance of the dam against sliding. The stepping of the foundation and provision of a shear key is shown in Fig. 19.36. The shear key may sometimes be provided in the centre but is generally provided at the heel.

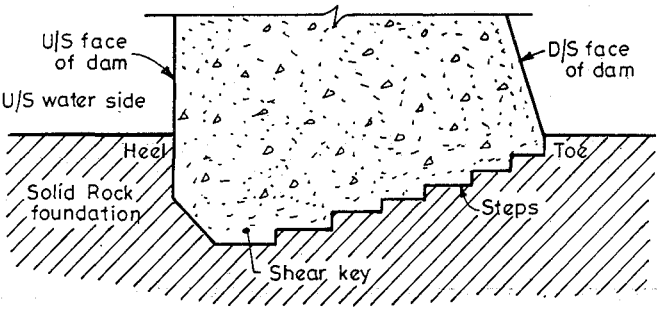


Fig. 19.36. Stepping of dam foundation and provision of shear key.

If faults, seams, or shattered rock zones are detected in the exploratory geological investigations, special steps and remedies must be taken to ensure their removal. They may have to be entirely excavated and back-filled with concrete grouting. The treatment will depend upon the specific needs.

The top foundation surface is thoroughly cleaned with wet sand blasting and washing before the concreting for dam section is started to be laid.

(2) **Grouting the Foundation.** The foundation grouting can be divided into :

(*a*) Consolidation grouting ; and

(*b*) Curtain grouting.

(*a*) *Consolidation grouting.* The entire foundation of the dam is consolidated by grouting. For this purpose, shallow holes (called *B* holes) are drilled through the foundation rock. The depths of these holes generally vary between 10 to 15 m. They are situated at about 5 to 20 m apart, in the general area of the heel of the dam. After the holes have been drilled, mixture of cement and water (called grout) is forced into the holes at low pressure of about 30 to 40 N/cm^2. This is accomplished, before any concreting for the dam section is laid. This low pressure grouting will result in a general consolidation of the foundations. These low pressure grout holes will later serve the purpose of a cut-off against leakage of high pressure grout, which is to be used after some concreting of the dam has taken taken place.

(*b*) *Curtain Grouting.* It helps in forming the principal barrier or a curtain against the seepage through the foundations, and thus reduce the uplift pressures. To accomplish this high pressure grouting, relatively deeper holes (called *A* holes) are drilled near the heel of the dam. The spacing of the holes may vary from 1.2 to 1.5 m. Holes are first of all, drilled and grouted at about 10 to 12 m apart, and then the intermediate holes are drilled and grouted. The depths of the holes vary from 30 to 40% of the total upstream water head for strong rock foundations, and may be as much as 70% of the water head for poor rocks. After the holes have been drilled, a mixture of cement and water (*i.e.* grout) is forced into the holes under high pressure. The grouting pressure may be kept as high as possible without lifting the foundation strata. Usually, the foundation pressure used in this high pressure grouting is equal to 2.5 D N/cm^2, where D is the depth of grouting in metres below the surface. This grouting is generally done in stages of depth equal to 15 m or so, and carried out only after some portion of the dam section has been laid.

This grouting may have to be accomplished from the foundation gallery or from other galleries within the dam. It may also be done from the upstream face of the dam, if possible. In certain special cases, this grouting may have to be accomplished from tunnels driven into the foundation rock below the dam.

PROBLEMS

1. (*a*) What is meant by gravity dams ?

(*b*) What are the main points to be considered while selecting a site for a gravity dam construction? *(Madras University, 1975)*

(*c*) Explain briefly with neat sketches the different forces that may act on a gravity dam. Indicate their magnitudes, directions and locations. *(Madras University, 1974)*

2. (*a*) Explain in details the various forces causing instability in a gravity dam. *(Madras University, 1974)*

(*b*) What is meant by the elementary profile of a gravity dam ; and how is it deduced ? What should be the maximum depth of elementary profile of a dam if the safe limit of stress on the masonry should not exceed 1500 kN per m^2 ? *(Madras University, 1975)*

[**Ans.** 44 m, assuming unit weight of masonry = 24 kN/m^3]

3. Explain briefly how grout curtain and drainage affect uplift pressures in gravity dams.

Fig. 19.37 shows the section of a gravity dam (non-overflow portion) built of concrete. Calculate (neglecting earthquake effects):

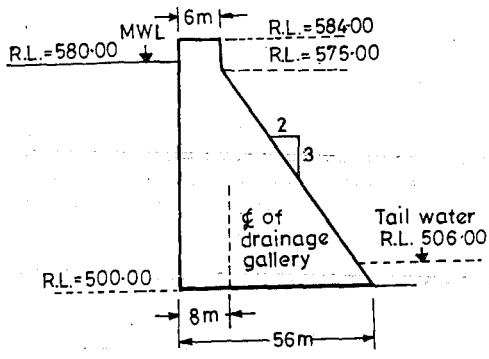


Fig. 19.37

(*i*) The maximum vertical stresses at the heel and toe of the dam.

(*ii*) The major principal stress at the toe of the dam.

(*iii*) The intensity of shear stress on a horizontal plane near the toe.

Assume weight of concrete = 24 kN/m^3.

[**Hint :** Follow example 19.1]

4. (*a*) What do you mean by the elementary profile of a gravity dam ? *(Madras University, 1976)*

(*b*) What are the advantages and disadvantages of a gravity dam over the other types ? *(Madras University, 1976)*

(*c*) What are the methods adopted to reduce uplift in masonry dams ? *(Madras University, 1976)*

5. (*a*) What is meant by the term 'low dam' ? Determine the dimensions of the elementary profile of a low gravity dam. *(Madras University, 1973)*

(*b*) Explain the criteria that govern the design of a high gravity dam in different zones of its cross-section. (*Madras University, 1976*)

6. (*a*) What is meant by "concrete gravity dams" ? Draw a neat typical cross-section of such a dam. Name the highest dam of the world as well as that of India.

(*b*) What are the different ways by which a concrete gravity dam may fail, and how will you ensure its safety against each type of failure ?

7. (*a*) Differentiate between a 'low gravity dam' and a 'high gravity dam'.

(*b*) How does the practical profile of a low gravity dam differs from that of the theoretical one, and why ?

(*c*) Discuss step by step the analytical procedure that you will adopt for analysing the stability (two dimensional analysis) of gravity dams.

8. Write detailed notes on any two of the following :

(*i*) Forces acting on gravity dams.

(*ii*) Stability analysis of gravity dams.

(*iii*) Elementary profile of a gravity dam.

(*iv*) Design considerations and fixing the section of a gravity dam.

(*v*) Design criteria for the design of high gravity dams.

9. Write short notes on :

(*i*) Uplift force.

(*ii*) Drainage gallery.

(*iii*) Grout curtain.

(*iv*) Construction joint.

(*v*) Earthquake forces on dams. (*Madras University, 1973*)

10. (*a*) Derive an expression for the limiting height of a low dam. (*Madras University, 1974*)

(*b*) Briefly explain the functions of the following :

(*i*) Drainage gallery.

(*ii*) Construction joints in a dam.

(*iii*) Ogee spillway. (*Madras University, 1974*)

[**Hint :** For (*iii*) Please refer chapter 21]

11. A concrete dam can be assumed to be trapezoidal in section having a top width of 2 m and bottom width of 10 m. Its height is 12 m and the upstream face has a batter of 1 : 10. Give an analysis of the stability of the dam for the base section for overturning and sliding in the full reservoir condition assuming no free-board allowance but allowing for uplift pressures. Assume uplift intensity factor as 100%. Also determine the compressive stresses at the toe and the heel, and major principal and shear stress developed at the toe. Assume weight of concrete to be 24 kN/m^3, unit shear strength of concrete to be 1400 kN/m^3, and the coefficient of friction between concrete and foundation soil to be 0.7.

12. The following data refer to the non-overflow section of a gravity dam :

R.L. of top of the dam	= 315 m
R.L. of bottom of the dam	= 260 m
Full reservoir level	= 312 m
Top width of the dam	= 12 m.

Upstream face is vertical. Downstream face is vertical upto R.L. 304 m ; and thereafter, the downstream face slopes at 0.7 (*H*) : 1 (*V*) up to base.

Drainage holes are located 8 m away from the upstream face.

Unit weight of masonry = 23 kN/m^3

Reduction of uplift at drainage hole = 50%

Coefficient of friction between masonry and foundation material = 0.8.

Determine (*i*) factor of safety against overturning ; (*ii*) factor of safety against sliding ; (*iii*) maximum pressure on foundation, and (*iv*) maximum principal stress in the masonry of the dam, at the

base. Consider only the forces due to water thrust, uplift, earthquake (inertial forces due to weight of masonry only) and the self-weight.

13. (*a*) Discuss the evolution of the final profile of a gravity dam from its elementary triangular profile, and explain the main principles of its design.

(*b*) Write a brief note on the necessity and method of foundation treatment of dams.

14. (*a*) Explain how uplift considerations affect the design of gravity dams. What measures can be adopted to reduce the undesirable effects due to uplift in such cases ?

(*b*) Write short notes on :

(*i*) Dam galleries.

(*ii*) Cracking of concrete during the construction of concrete gravity dams, and remedial measures.

(*iii*) Provision of keyways in concrete gravity dams.

15. (*a*) Differentiate between rigid dams and non-rigid dams.

(*b*) How are pore pressures and uplift pressures controlled in rigid dams ?

(*c*) Describe briefly the investigations that are necessary, and the treatment which is commonly given to the foundations of gravity dams, so as to avoid their failures in general.

16. The cross-section of a low gravity dam is shown in Fig. 19.38.

Assuming the reservoir to be full, determine

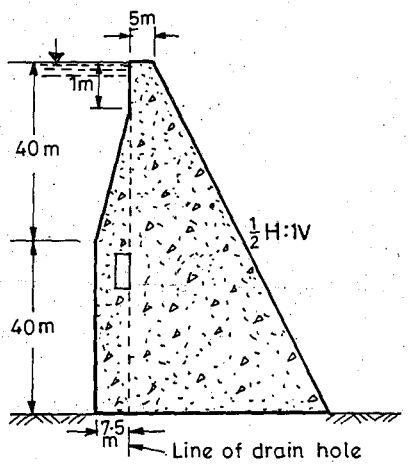


Fig. 19.38

(*i*) The normal stress ;

(*ii*) The principal stress ;

(*iii*) The shear friction factor at base.

Count full uplift as per U.S.B.R. recommendations. Neglect earthquake forces, wave pressure and silt pressure.

17. Design the practical profile of a gravity dam made of stone masonry given the following data :

R.L. of base of dam	= 198 m.
R.L. of HFL of reservoir	= 228 m.
Specific gravity of masonry	= 2.4.

Safe compressive stress in masonry = 1200 kN/m^2.

18. Fig. 19.39 shows the section of a concrete gravity dam. Check the stability of this dam section at the base. Assume any data not given and needed.

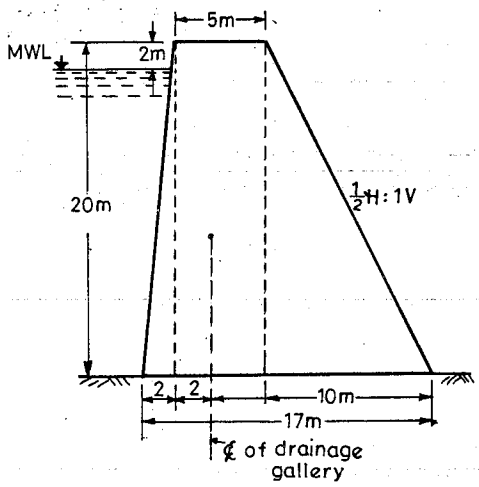


Fig. 19.39