17

Dams in General and a few Dams in Particular

17.1. . General

17.1.1. Definition. A dam may be defined as an obstruction or a barrier built across a stream or a river. At the back of this barrier, water gets collected, forming a pool of water. The side on which water gets collected is called the upstream side, and the other side of the barrier is called the downstream side. The lake of water which is formed upstream is often called a *reservoir.*

17.1.2. Uses. The water collected in this lake can be used for recreation, or as a reservoir of drinking water, or it can be tapped off into surrounding farm land for irrigation. The energy of this collected water can be used to turn a mill to grind wheat or to turn the blades of a tubrine to generate electrical power. And in times of floods, the dams can serve as protections for the towns and cities farther down the river.

Apart from these numerous advantages and uses (such as navigation, irrigation, electricity, flood control, etc.) of a dam, it sometimes helps us in planning war strategy and helps us in controlling the advancement of enemies and their forces. Dams have been frequently opened in times of war. The Dutch breached their dikes during Second World War to bedevil the invading Germans. Chinese used to destroy their dikes to flood out the enemy. Russian army retreating from the Nazi marauders, partly destroyed the famous Dneprostroi Dam in the Ukraine to keep its power plant from falling into the hands of Hitler's men.

17.1.3. History of Dam Construction. It is very difficult to say as where and when the first man-made dam was built. Archaeological evidences help in estimating that the very first man made dam is at least 3000—5000 years old. Whenever it was built, that first dam was almost certainly an irrigation dam. Its designer might have observed beavers at work or he might have thought of it the some other way. Beavers are mammel creatures belonging to the family of rats, mice, squirrels. They live under water and are generally 1.2 m long including more than 0.3 m of tail, and weigh upto 25 to 30 kg. These rodents produce fur. These creatures create dam type barriers to create a place where their family can live. They provide themselves with comfortable ponds to live in, by building dam type barriers across the stream with the help of trees which they themselves cut.

These elegant structures, built out of logs, buttressed with twigs and branches and sealed with mud and stone help us many a times in controlling silt entry into the stream. A beaver dam accumulates silt brought down by its stream. When a beaver dam is breached, the silted water pours through, and the fertile silt is deposited over the wide area. This creates what farmers call a *beaver's meadow,* where crops grow particularly well.

Beavers are sometimes encouraged to build dams in areas where man can not reach easily to construct man-made dams.

17.1.4. Modern Dams. The first modern dam of the world was perhaps constructed on the Nile river, in Egypt at *Aswan.* It was completed in 1902 and was a major engineering project.

This famous **Aswan Dam** was designed primarily to control the flooding of the Nile river, to promote irrigation in the Nile Valley, and to further navigation along the river.

Aswan is 1200 km from the mouth of the Nile. The site was chosen because the river at Aswan is shallow and has a granite bed, on which a firm foundation could be erected.

The first step in damming any major river is to divert the flow of water so as to permit dam construction. This was accomplished at Aswan, by constructing a circular earthen barrier around the area chosen from the dam foundations. At low river, this enclosure was pumped out. All the work had to be completed before the river flooded again. The foundation was laid round the clock with a huge labour force. So much so that 3,600 tons of masonry were put in places in a single day.

This 120′ high dam, running for about $1\frac{1}{2}$ miles from shore to shore, 100 feet thick at its base and 24′ wide at top, costed about 1.5 crore dollars, when completed in 1902. A roadway ran along the upper rim. About 1 billion tons of water was its storage capacity, and 180 sluices were constructed into this barrier. Water could be collected during the rainy season and released through the sluice gates during the summer and drought.

Roosevelt Dam. The next famous dam of the world was completed in 1911 on the Salt River of Arizona (U.S.A.) and was called Roosevelt dam.

The Aswan dam was constructed with stone, while the Roosevelt dam was constructed with solid blocks of concrete : which was of the type known as 'Solid Masonry Gravity Dam' : which simply means that it was built with solid blocks of concrete, which hold back the flow of water by sheer weight. This type of dam is one of the most ancient.

The construction was started in 1905. Despite the numerous troubles in its construction, the dam was completed in 1911 with a reservoir capacity of $\frac{1}{2}$ billion gallons of water. The completed dam, 280 feet high, was *158 feet thick* and spanning for a length of 1,125 feet. Had the dam been built without the benefit of 19th century engineering advances in design and stress control, it would have had to be 700 to *1000 feet thick* at base ; a far more expensive and cumbersome thing to build.

This dam was a fascinating advancement, but was soon overshadowed by other vastly greater dams, such as Hoover Dam, (726′ high), Bhakra Dam (740′ high), etc. All of them are 'solid concrete gravity type' dams.

17.2. Various Kinds of Dams

Before we describe some of the famous dams of the world, it is worth while to classify the various types of dams.

Most engineers recognise seven general types of dams. Three of them are ancient in origin, and four have come into general use only in the last about 100 years or so.

The three older types of dams are :

(1) Earth Dams

(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.

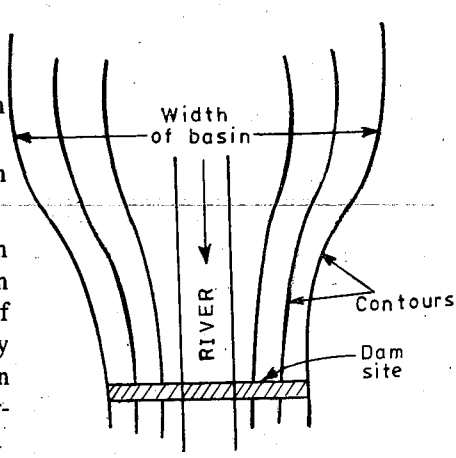

Fig. 17.1

(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.

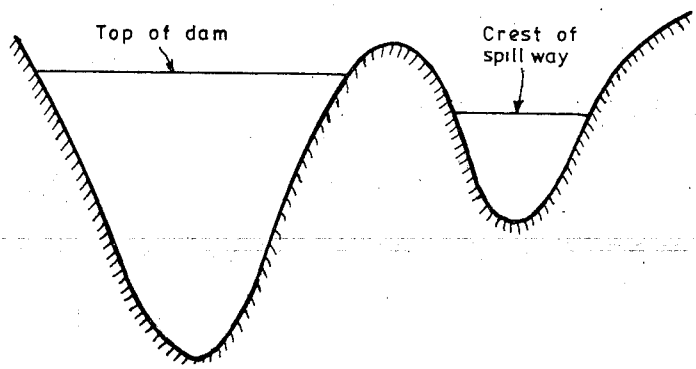

Fig. 17.2

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.

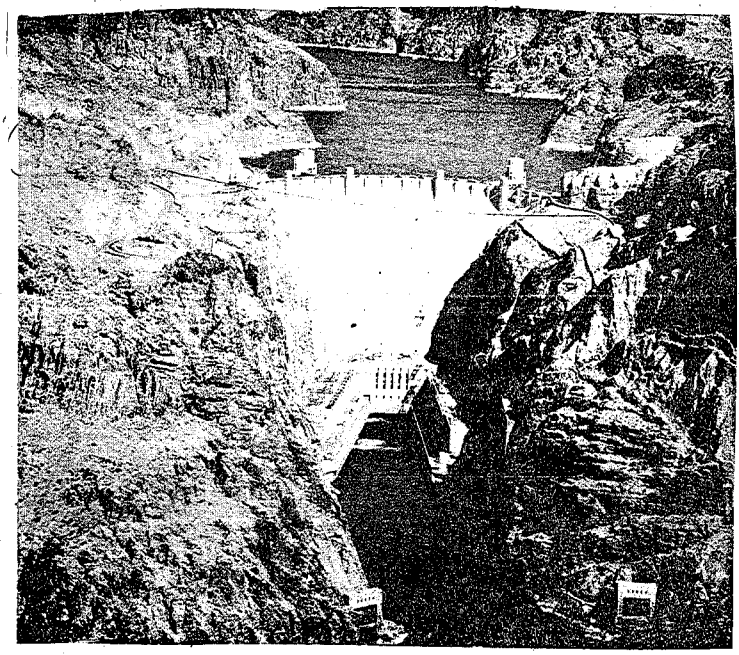

Fig. 17.3. Photo view of Hoover Dam.

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.

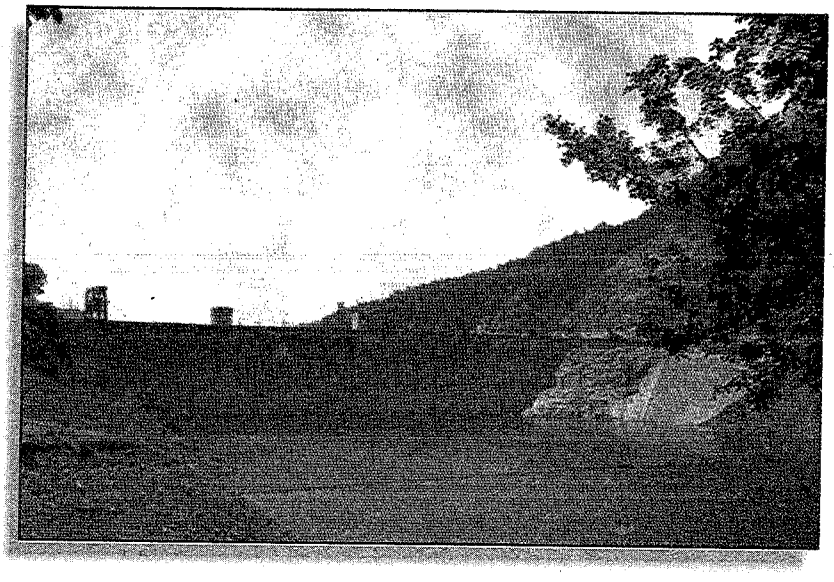

Fig. 17.4 (*a*) Upstream view of Bhakra Dam.

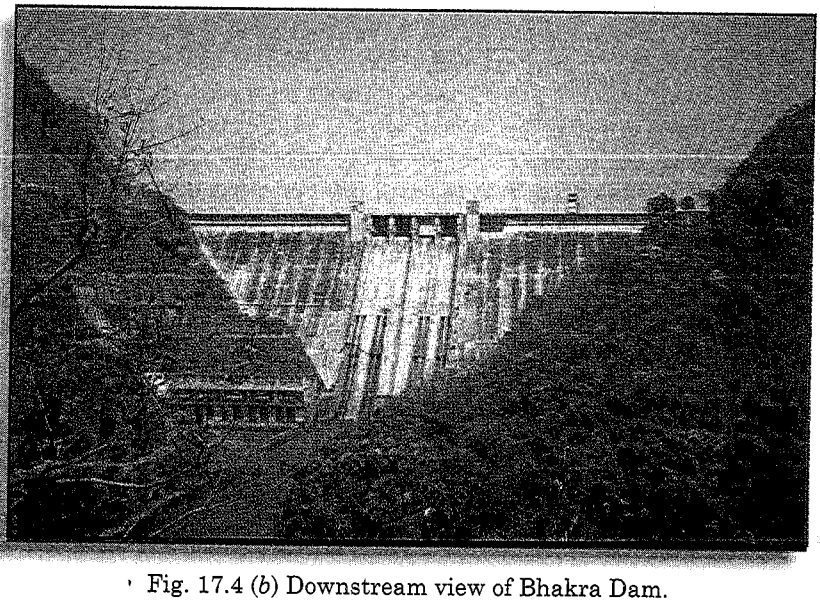

Fig. 17.4 (*b*) Downstream view of Bhakra Dam.

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

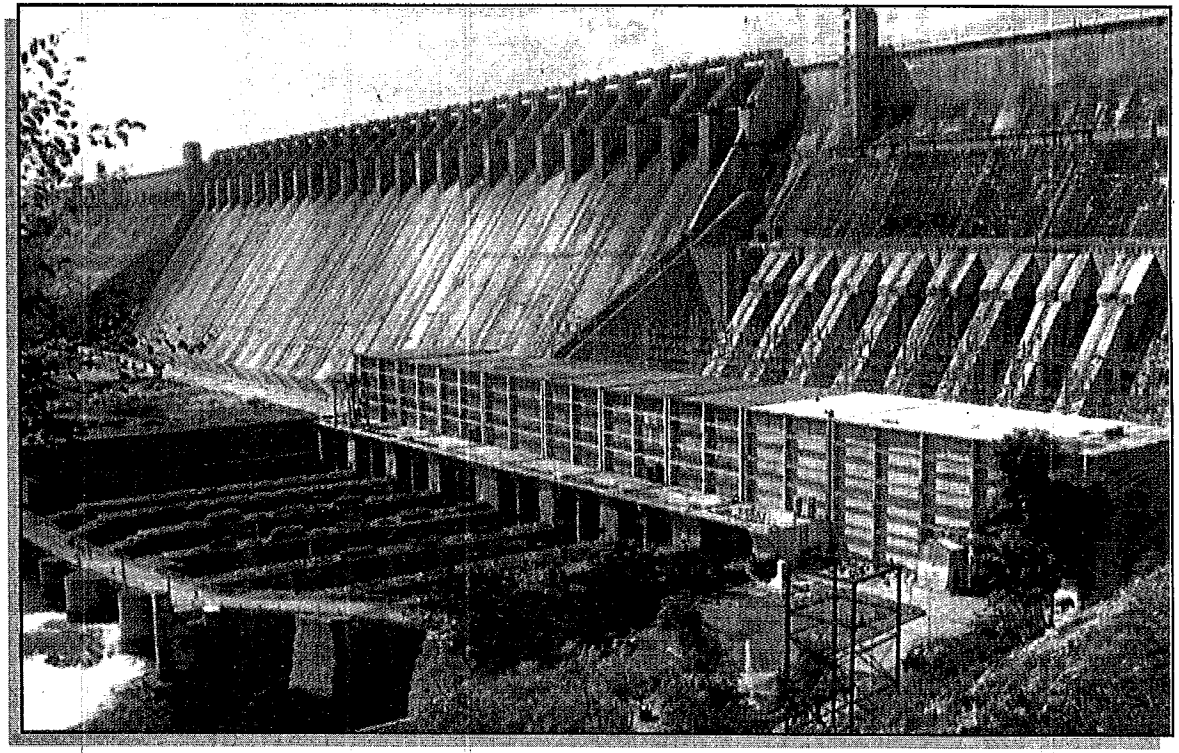

Fig. 17.5. Photoview of Nagarjuna Sagar Dam.

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.

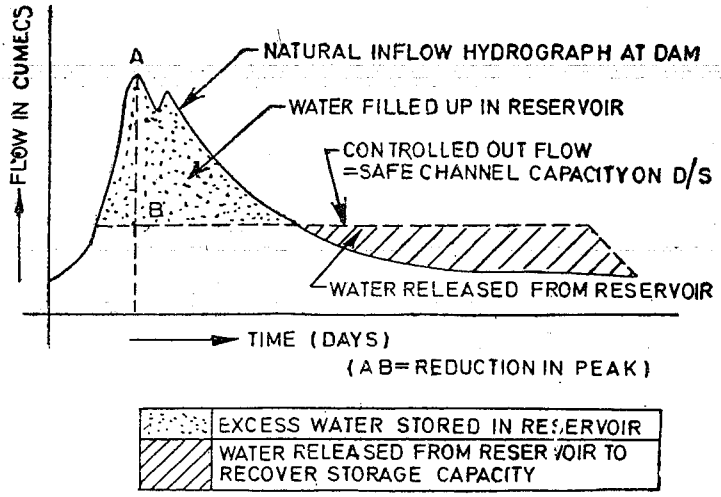

Fig. 18.1.

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

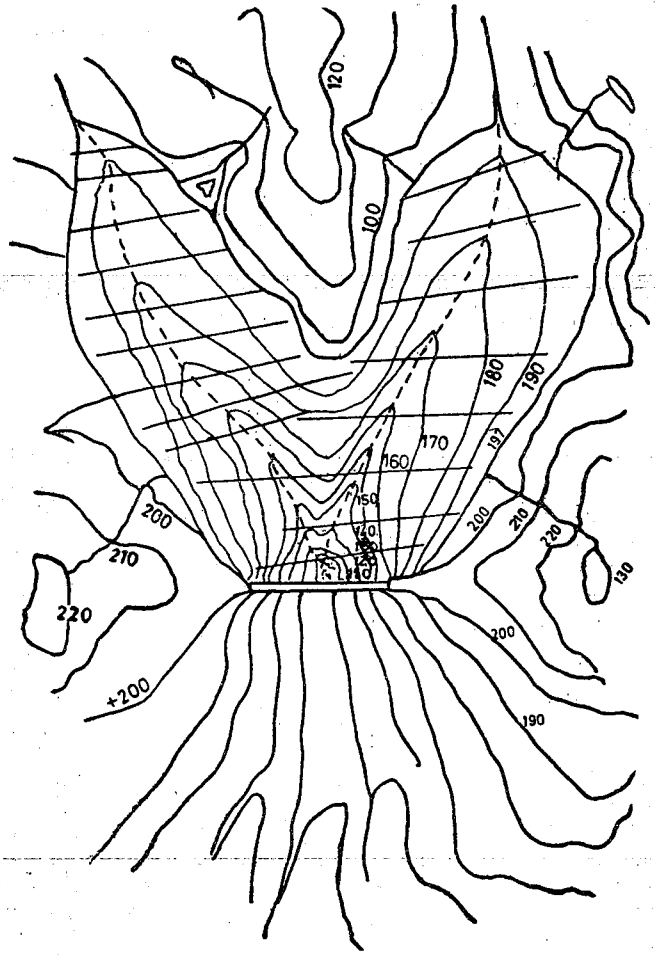

Fig. 18.2. A typical contour map of a dam site.

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

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

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

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,

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

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.

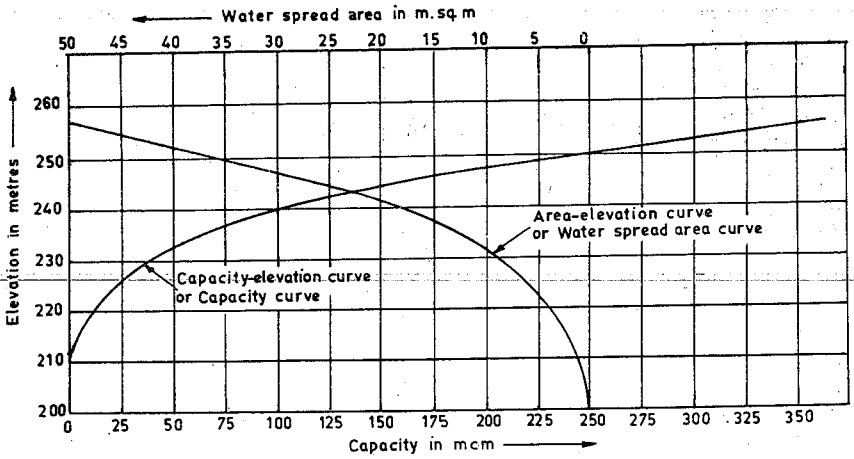

Fig. 18.3. Area elevation curve and Capacity elevation curve.

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.

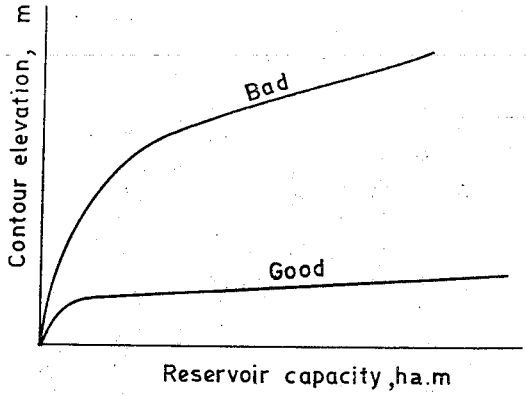

Fig. 18.4. Good and Bad capacity curves for a reservoir

18.3. Storage Zones of a Reservoir

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

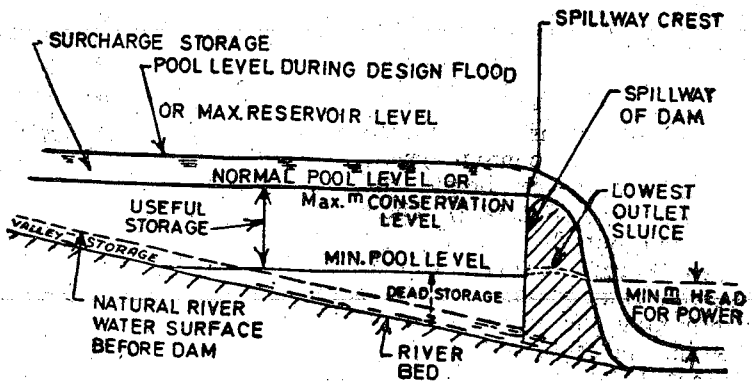

Fig. 18.5. Storage zones of a reservoir.

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 m³** **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

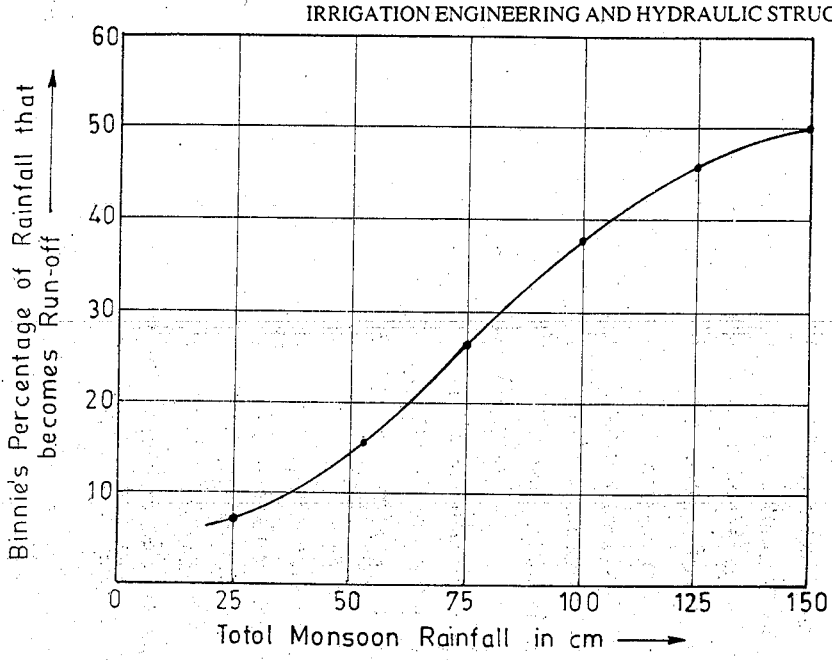

Fig. 18.6(*a*). Binnie's Monsoon Rainfall-runoff Curve.

(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

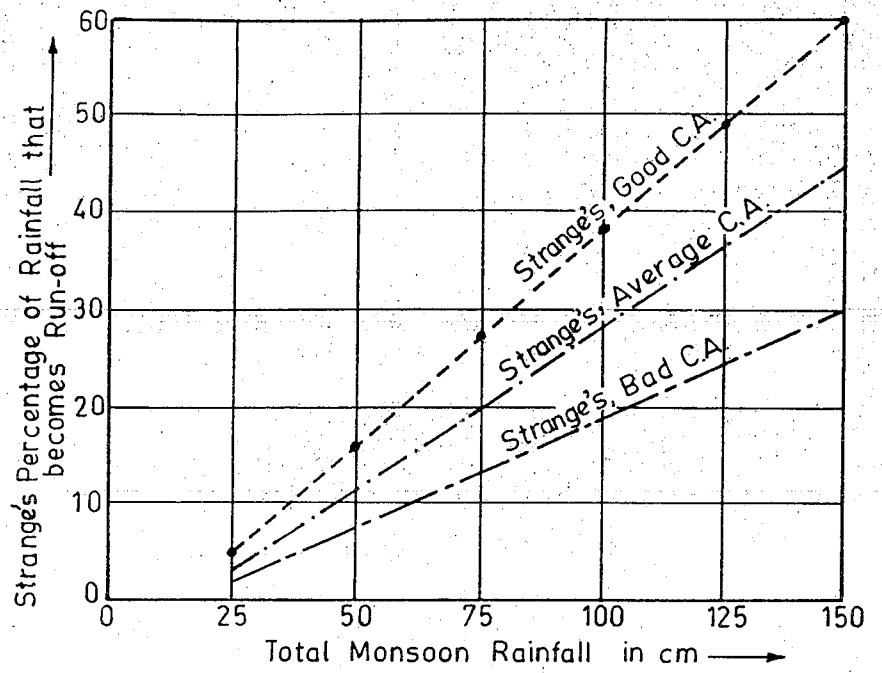

Fig. 18.6(*b*). Strange's monsoon Rainfall-Runoff curves.

(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.* 1000×10^6 m^2)

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

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 :

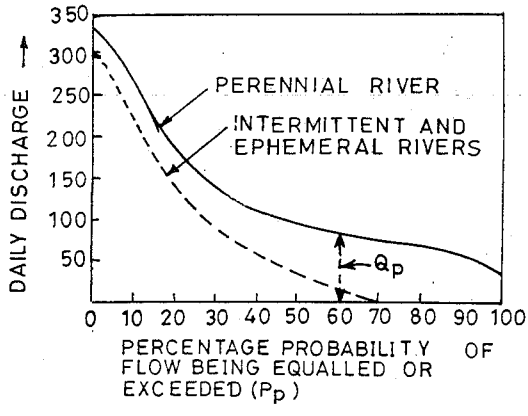

Fig. 18.7(*a*). Typical flow-duration curves on an ordinary paper.

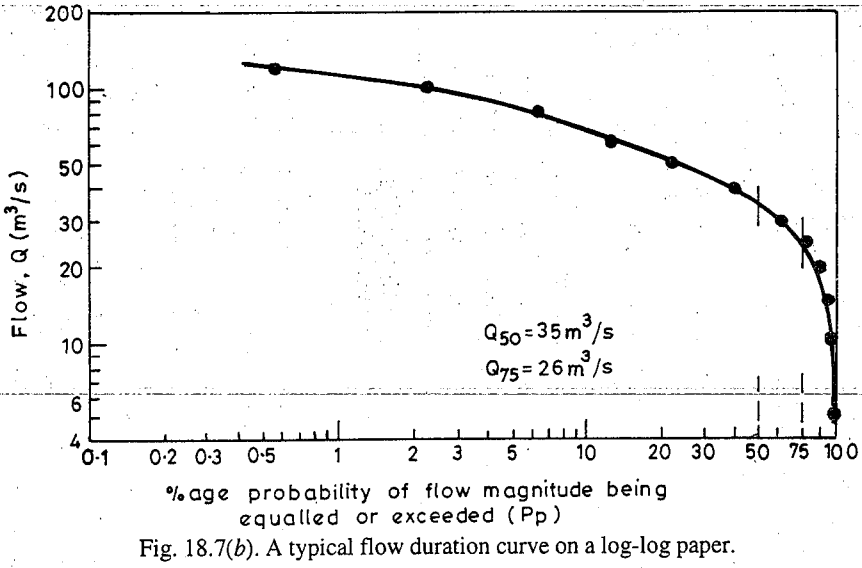

Fig. 18.7(*b*). A typical flow duration curve on a log-log paper.

* 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*).

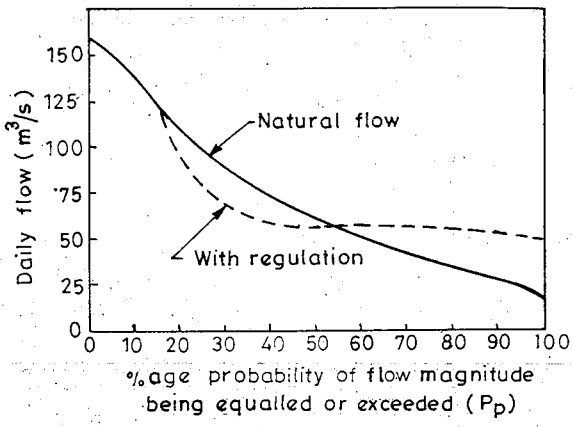

Fig. 18.7. (*c*) Reservoir regulation effect on the flow-duration curve of a stream.

(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.

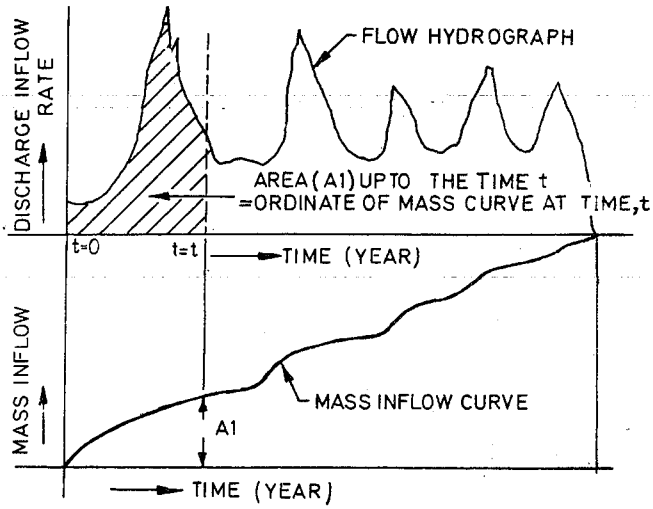

Fig. 18.8 (*a*) and (*b*).

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*)].

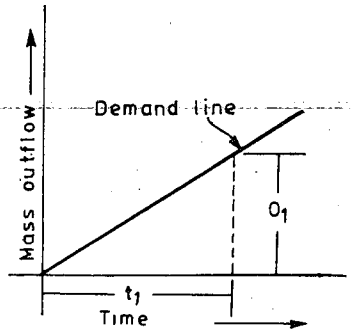

Fig. 18.9 (*a*)

O_1/t_1 = Slope of the line = Demand rate

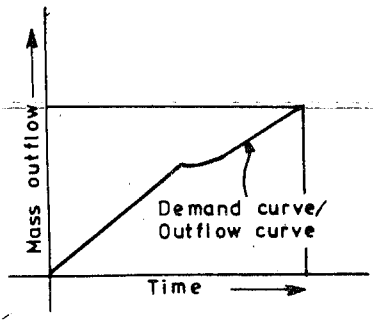

Fig. 18.9 (*b*).

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.

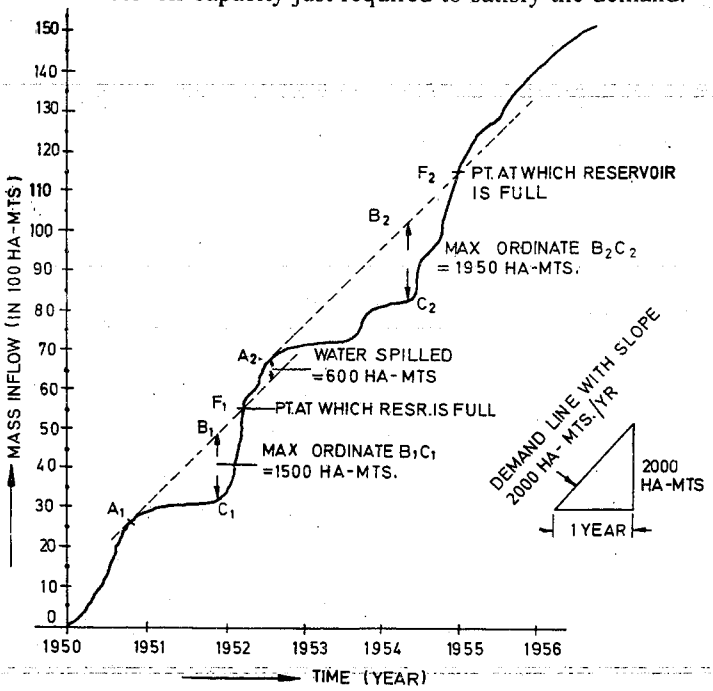

Fig. 18.10.

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.

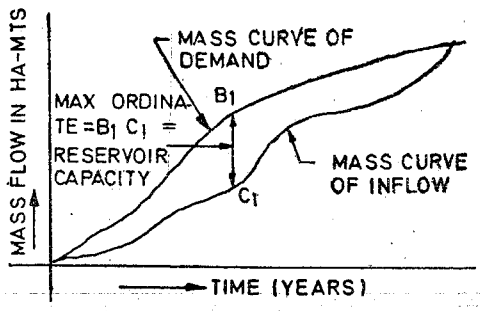

Fig. 18.11.

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 :

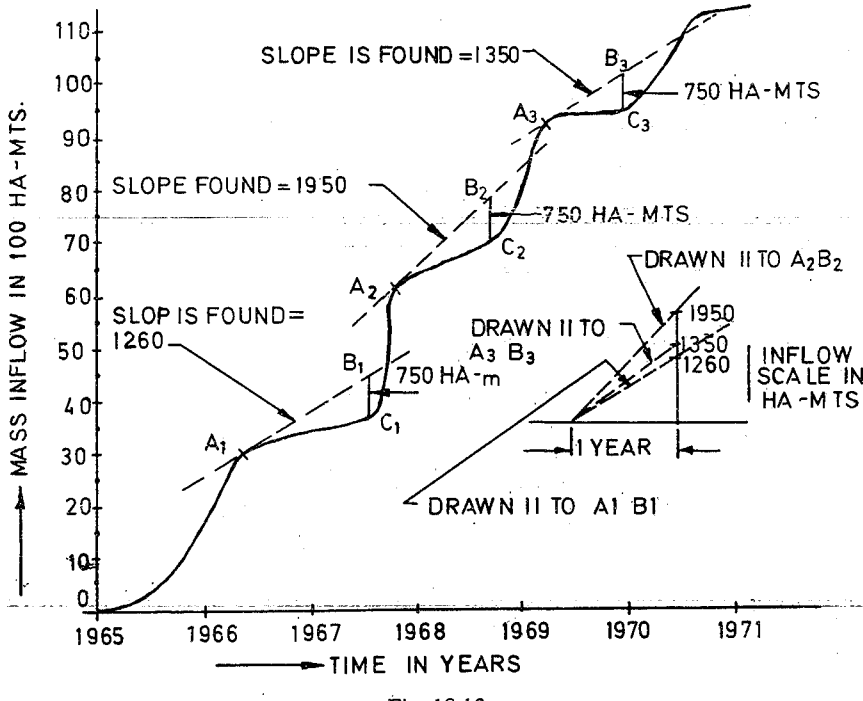

Fig. 18.12.

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 (1)	*Yearly runoff (cm)* (2)	*Runoff as volume in* $\text{M.m}^3 = \frac{\text{Col. (2)}}{100} \times 1675$ (3)	*Cumulative runoff as volume in* M.m^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.

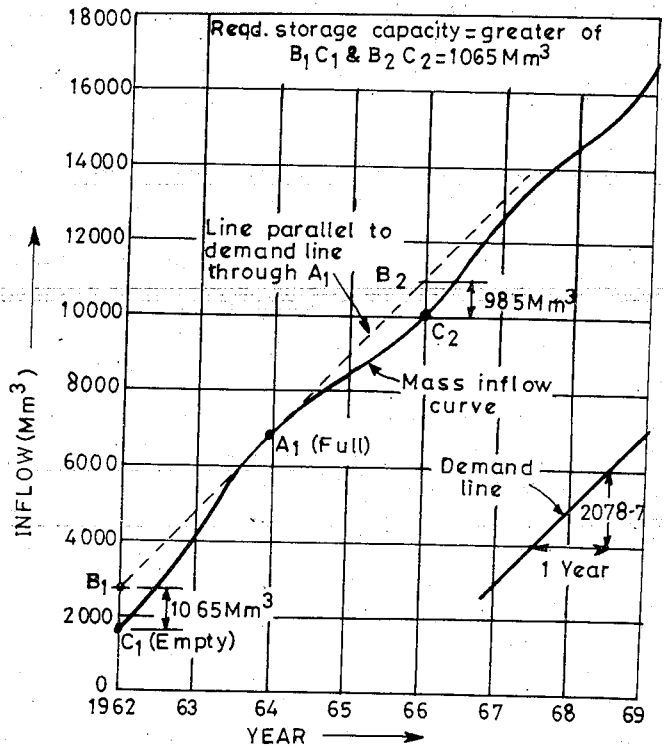

Fig. 18.13.

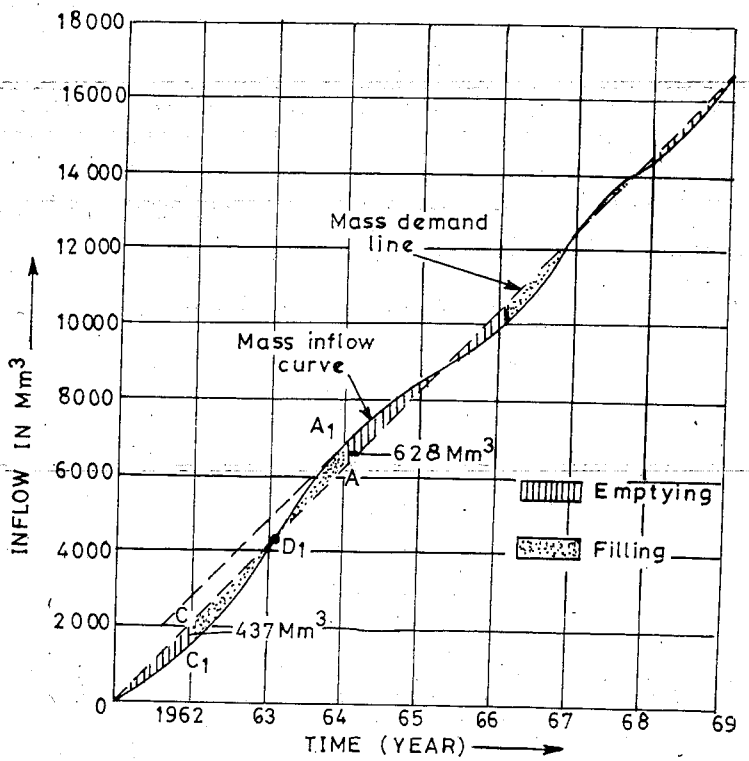

Fig. 18.14.

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

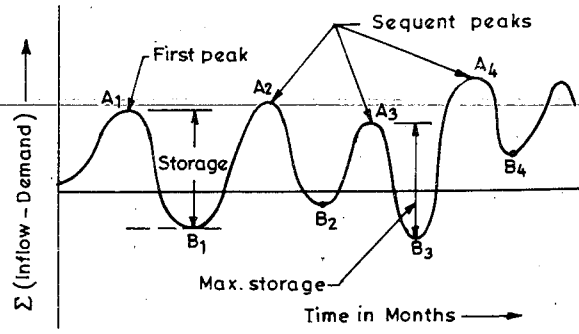

Fig. 18.15. Sequent Peak Algorithm.

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.**

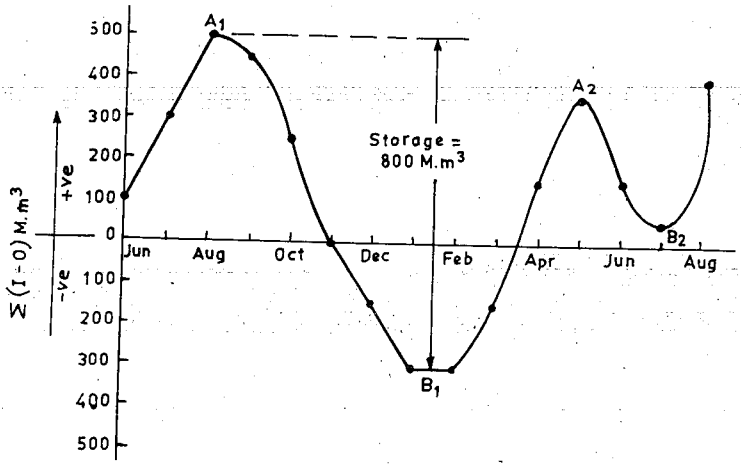

Fig. 18.16. Sequent peak algorithm for Example 18.8.

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.**

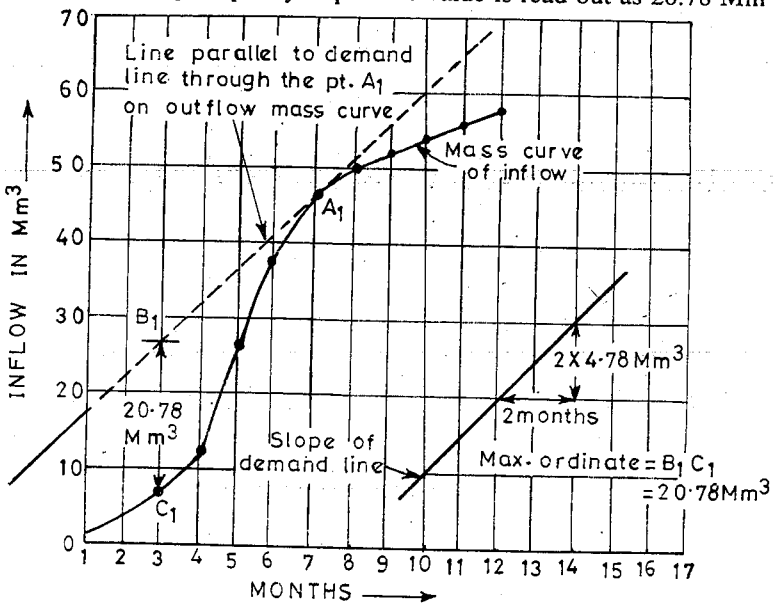

Fig. 18.17.

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.

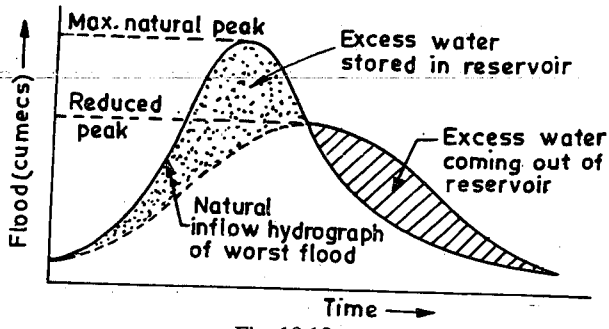

Fig. 18.18.

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

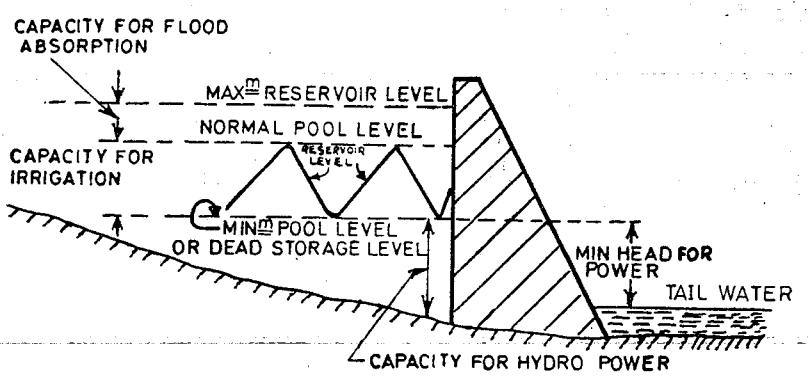

Fig. 18.19.

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.*

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

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.

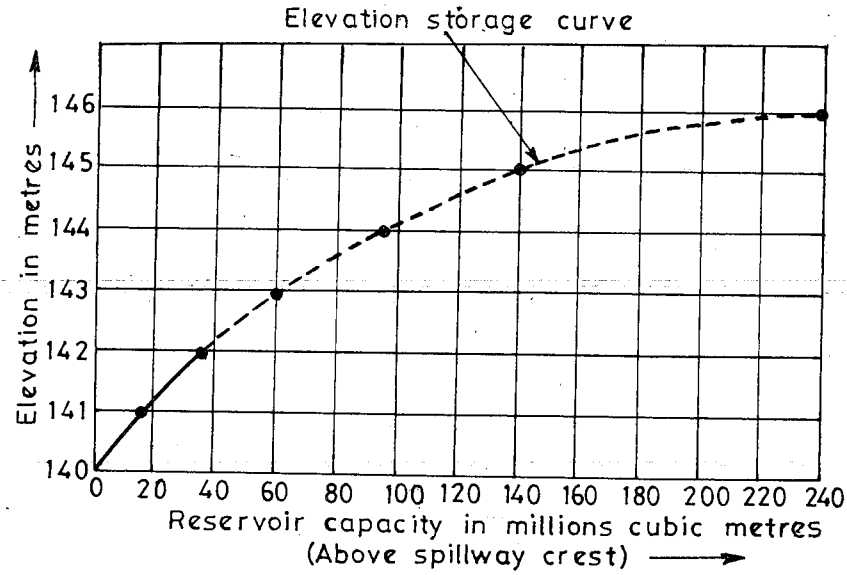

Fig. 18.20. Elevation Storage curve for example 18.13.

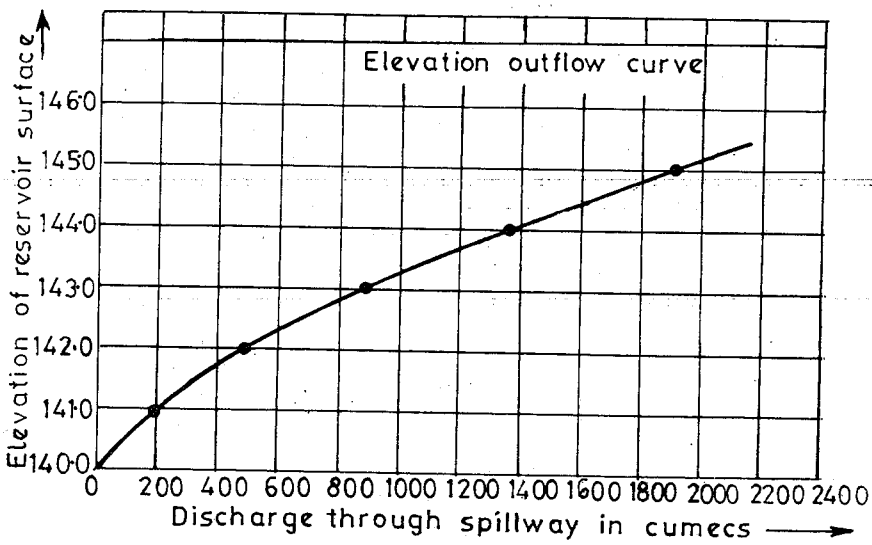

Fig. 18.21. Elevation-Outflow curve for example 18.13.

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	$(S_2 - S_1)$	$O_{(V)} + \Delta S$	
Hrs.	Hrs.	m	m^3/s	m^3/s	$M.m^3$	m^3/s	m	m^3/s	$M.m^3$	reckoned over spillway crest $M.m^3$	M.m3	*i.e.* (ΔS) $M.m^3$	$M.m^3$	Whether $(O_{(V)} + \Delta S)$ 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

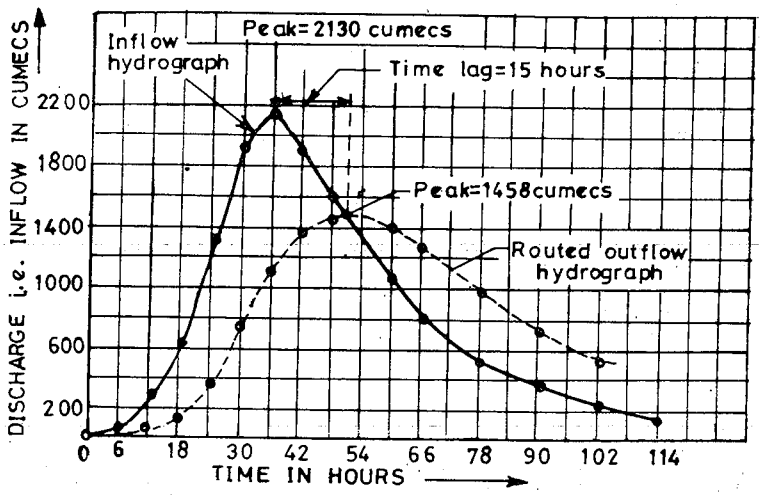

Fig. 18.22. Hydrograph of the given flood and that of the routed flood for example 18.10.

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

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

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- 081144047-6
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

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

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

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

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

(*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 $S_1 = 0$

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

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

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.

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

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

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

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

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.

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* 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.

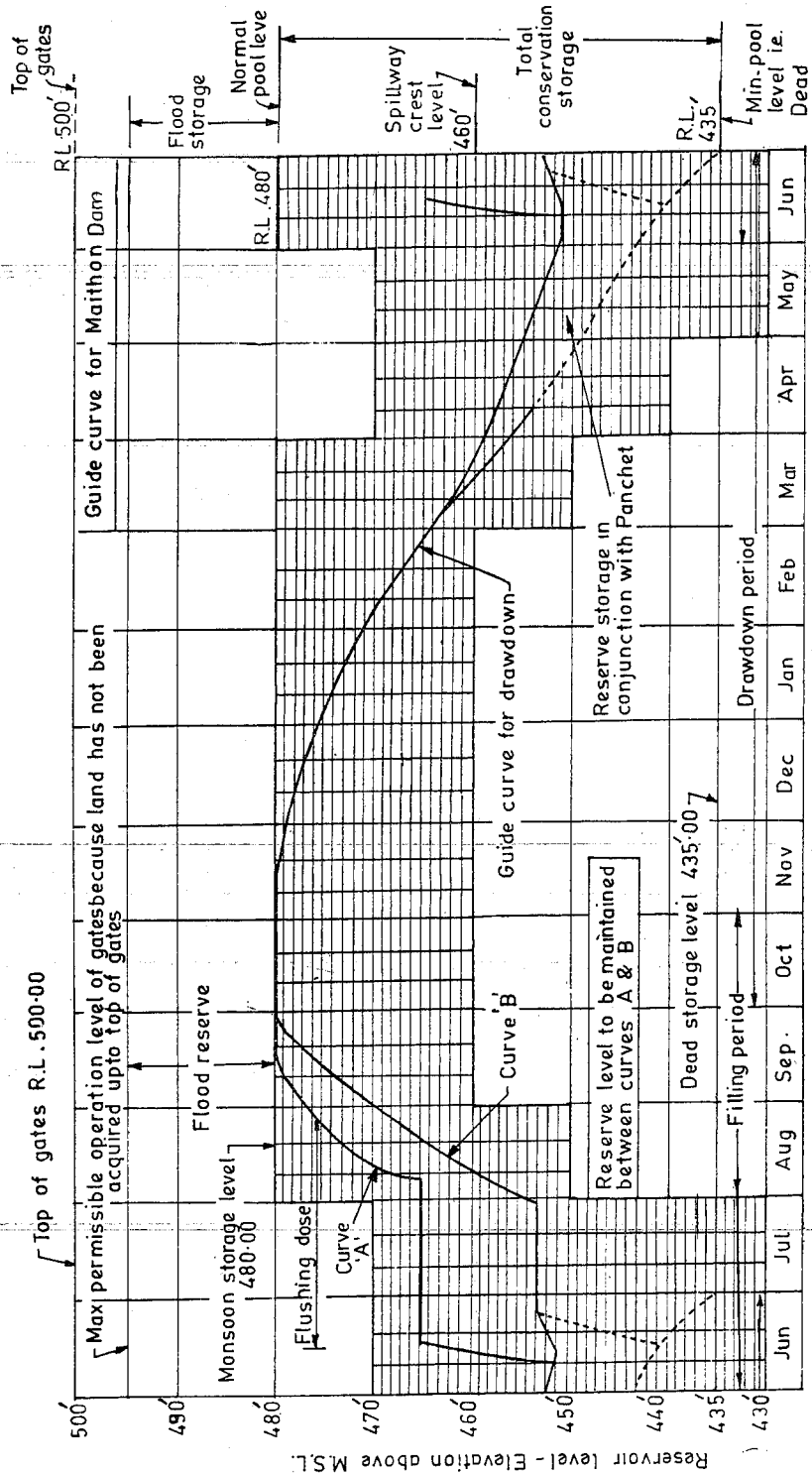


Fig. 18.23. A typical operating guide for Maithon Dam Reservoir.

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.

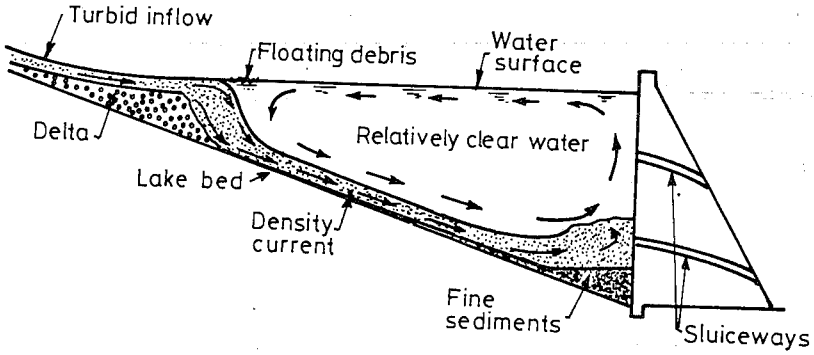


Fig. 18.24. Sediment accumulation in a typical reservoir.

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.

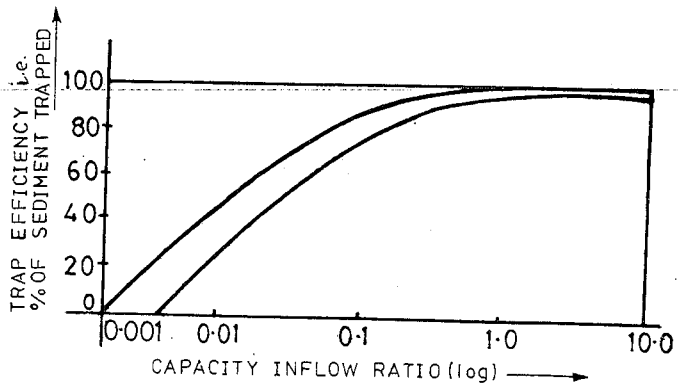


Fig. 18.25.

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. $5 \times \frac{1}{6}$ 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. $10\% \times 5 \text{ Mcum} = 0.5 \text{ Mcum}$.

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 :

$$K = \frac{1}{100}\left[\, 2.1\, M^{1.14}\,(10^{-4})\,(12-a) + 3.25\,(b-2) + 2.5\,(c-3)\,\right] \qquad ...(18.19)$$

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.

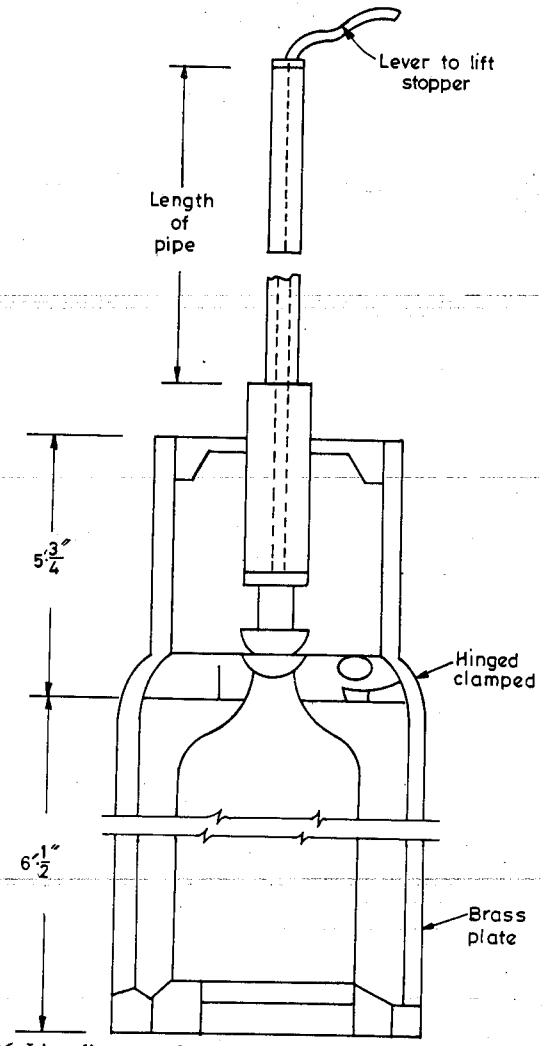


Fig. 18.26. Line diagram of a typical sediment sampler (Punjab bottle sampler).

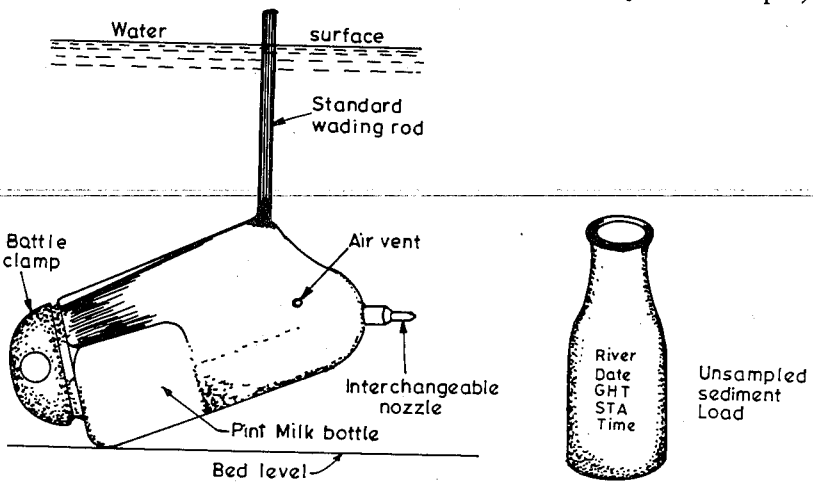


Fig. 18.27. AUS.DH-48 depth-integrating hand sampler for small streams.

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.

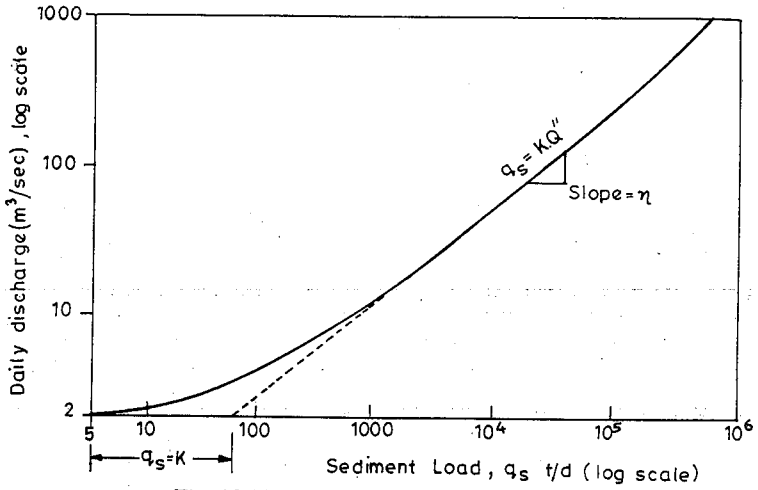


Fig. 18.28. A typical sediment rating curve.

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. $=$ **426.3 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

$$^* \; q_s \text{ (Total sediment in Mm}^3) = \frac{q_s \text{ in M} \cdot \text{t}}{1.2 \text{ t/m}^3} \times (1.1)^{**} = \left(col\,(3) \times \frac{1.1}{1.2} \right)$$

** 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 · m³/100 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 · m³ per 100 sq. km of catchment

A = catchment area in km².

(*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 · m³	*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.

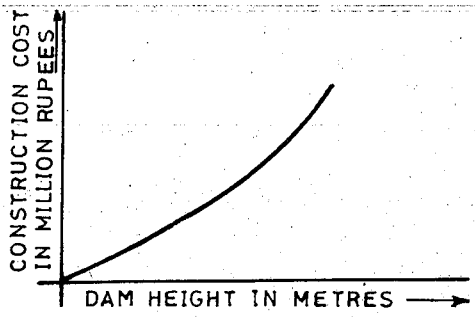


Fig. 18.29

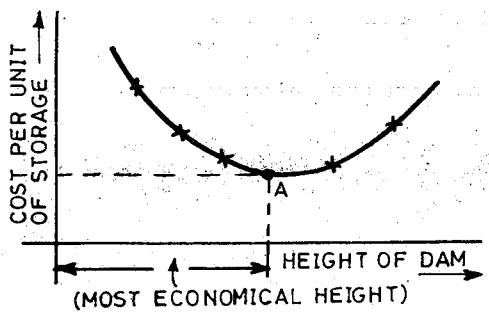


Fig. 18.30

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.**

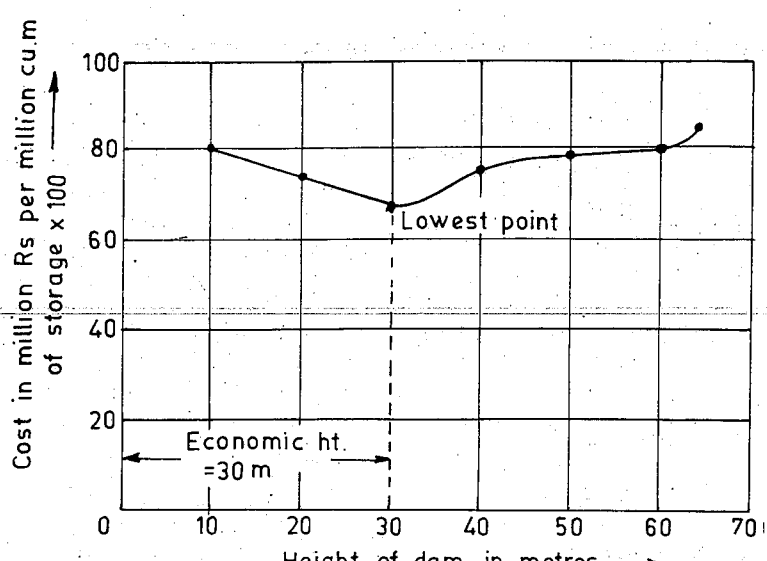


Fig. 18.31.

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.

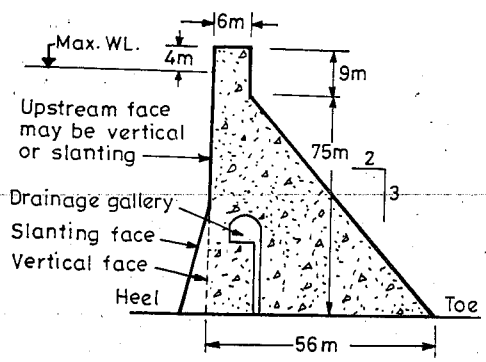


Fig. 19.1. A typical cross-section of a concrete gravity dam.

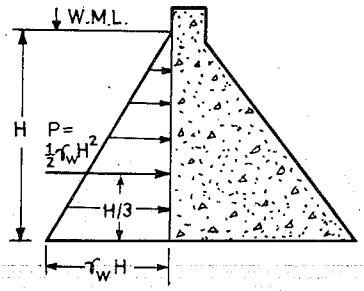
19.3. Forces Acting on Gravity Dam

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).

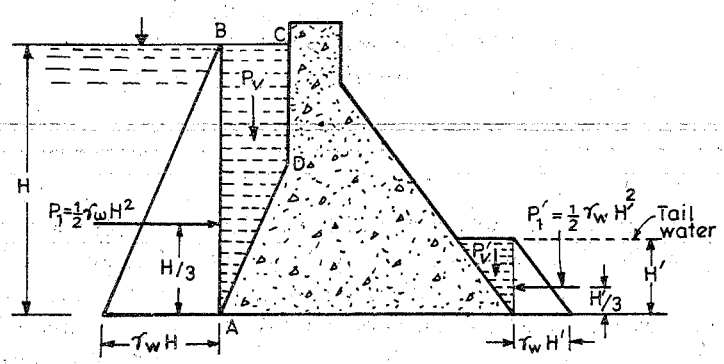


Fig. 19.2 (b)

(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

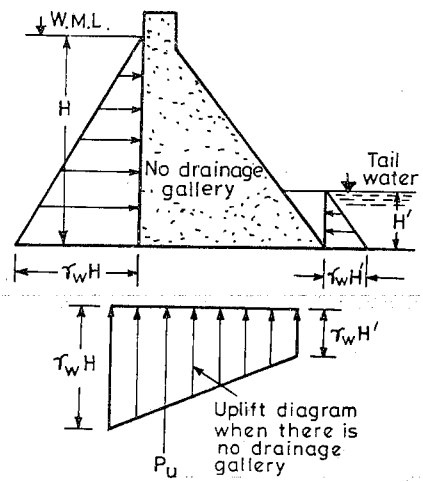


Fig. 19.3 (*a*) Uplift pressure (*U*) diagram, when no drainage gallery is provided.

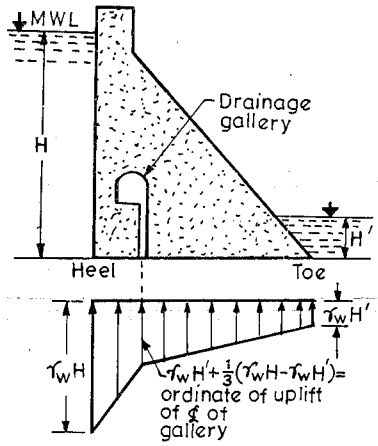


Fig. 19.3 (*b*) Uplift pressure (*U*) diagram, when drainage gallery is provided.

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.*

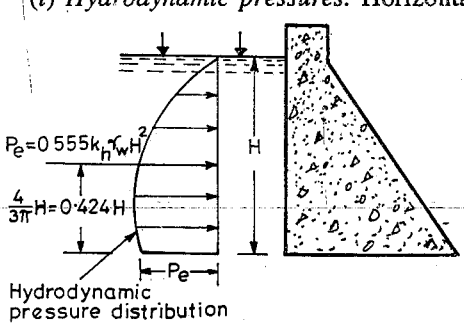


Fig. 19.4. Showing development of Hydrodynamic pressure by a horizontal earthquake moving towards the reservoir. A similar pressure will be developed on d/s tail water when the earthquake is reversed.

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.

(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.

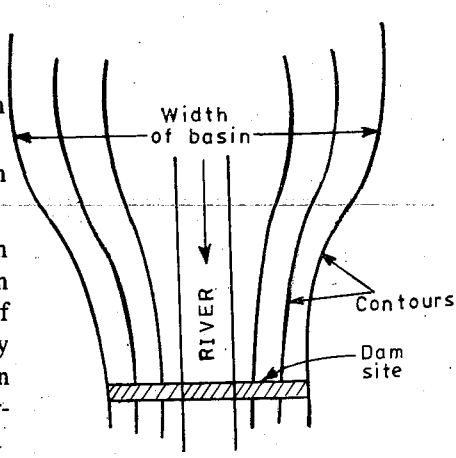


Fig. 17.1

(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.

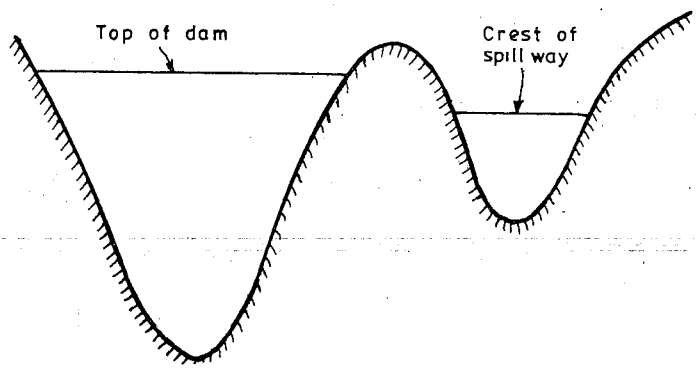


Fig. 17.2

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.

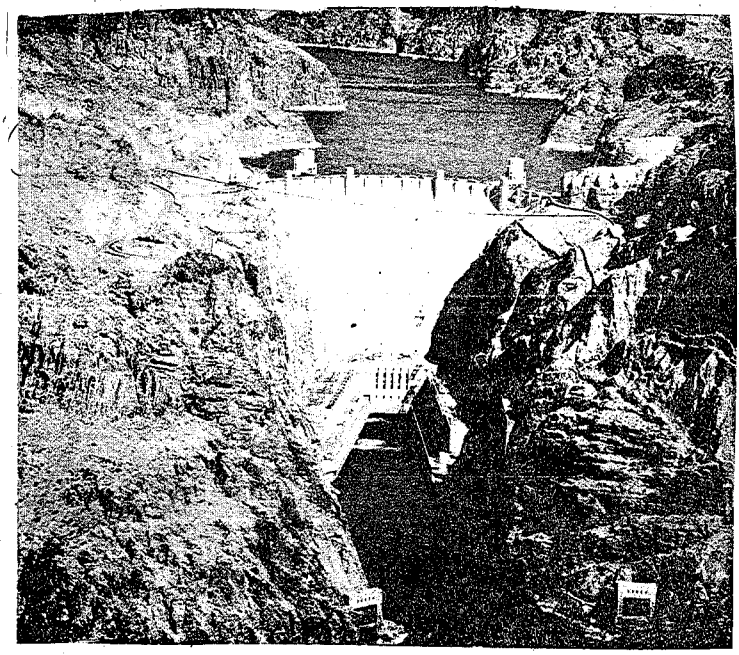


Fig. 17.3. Photo view of Hoover Dam.

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.

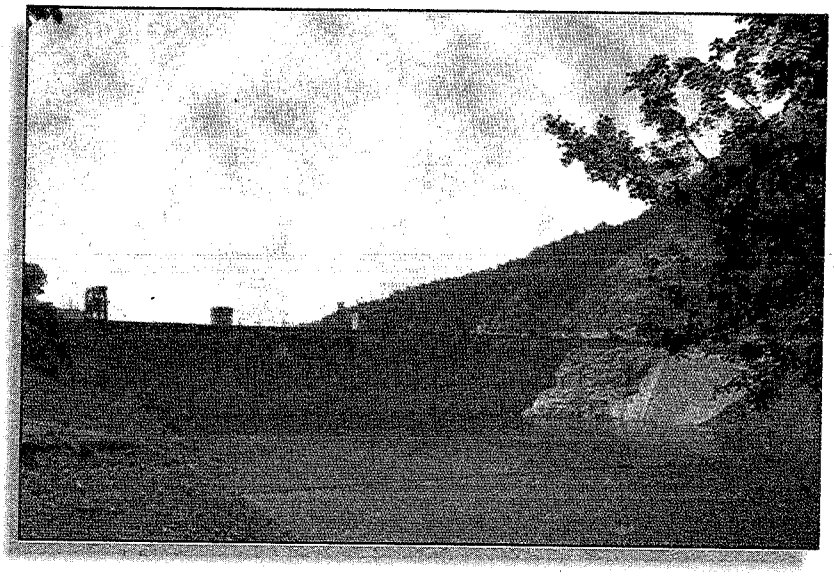


Fig. 17.4 (*a*) Upstream view of Bhakra Dam.

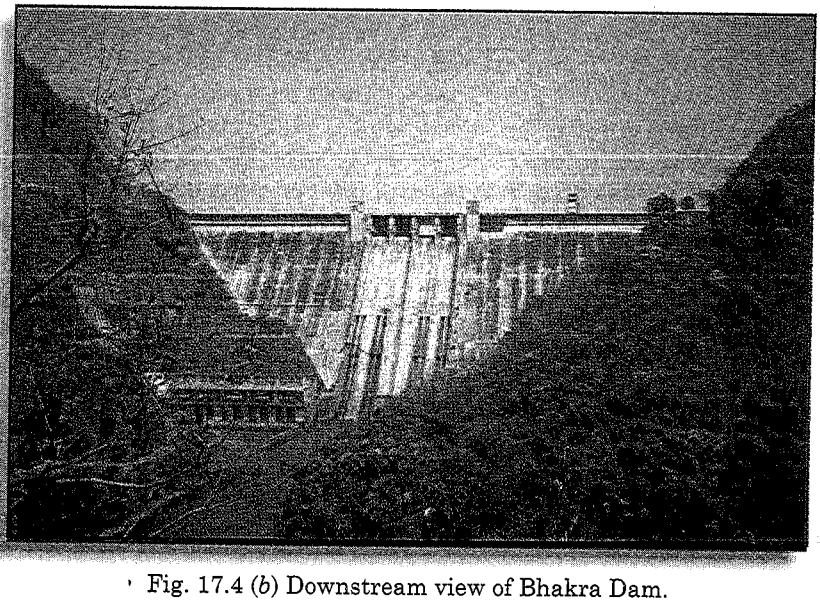


Fig. 17.4 (*b*) Downstream view of Bhakra Dam.

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

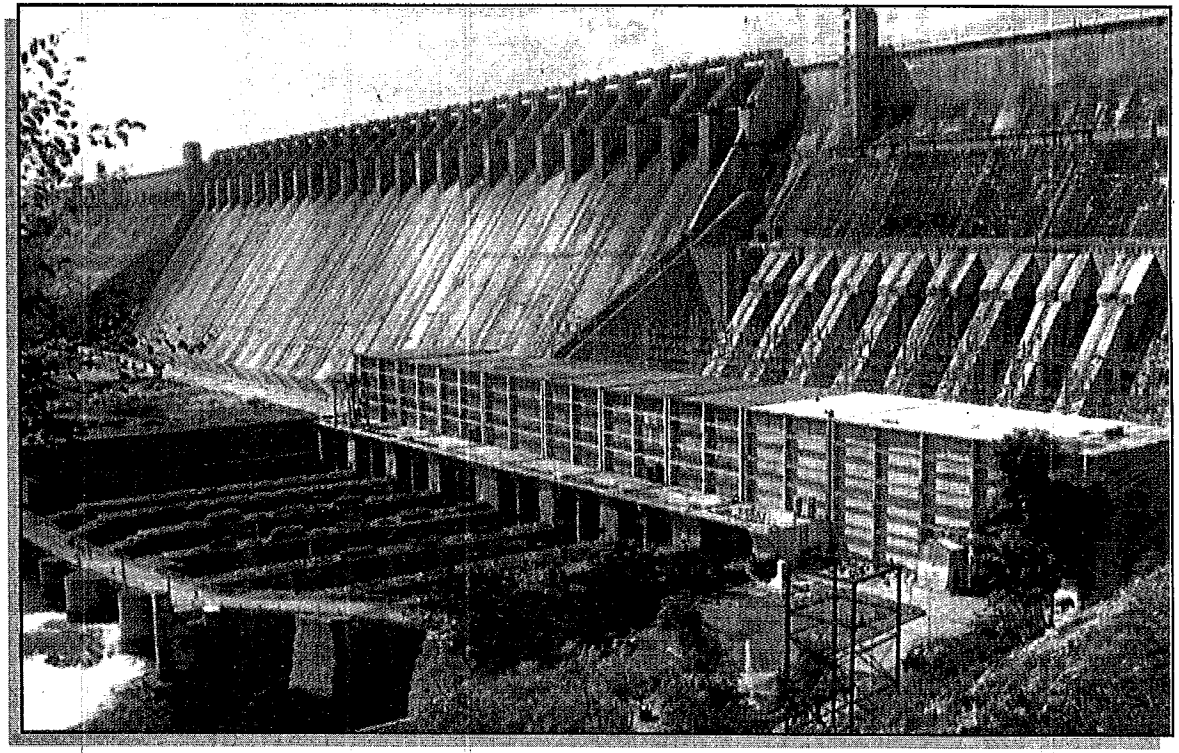


Fig. 17.5. Photoview of Nagarjuna Sagar Dam.

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.

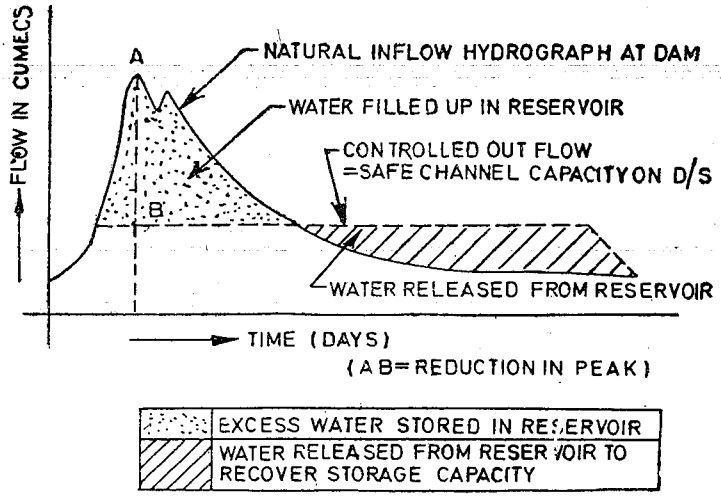


Fig. 18.1.

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

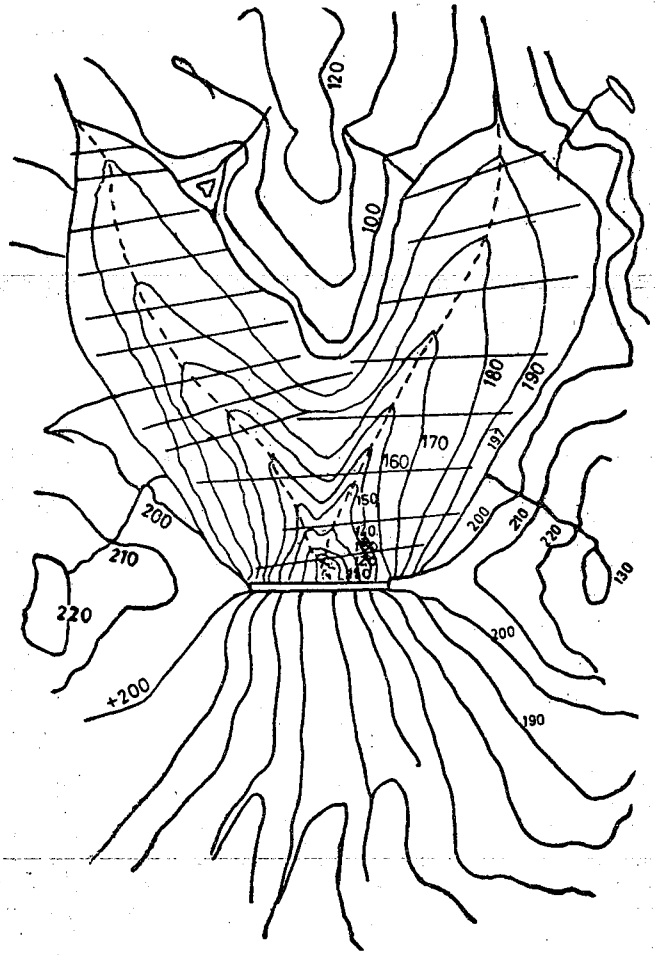


Fig. 18.2. A typical contour map of a dam site.

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

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

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

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,

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

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.

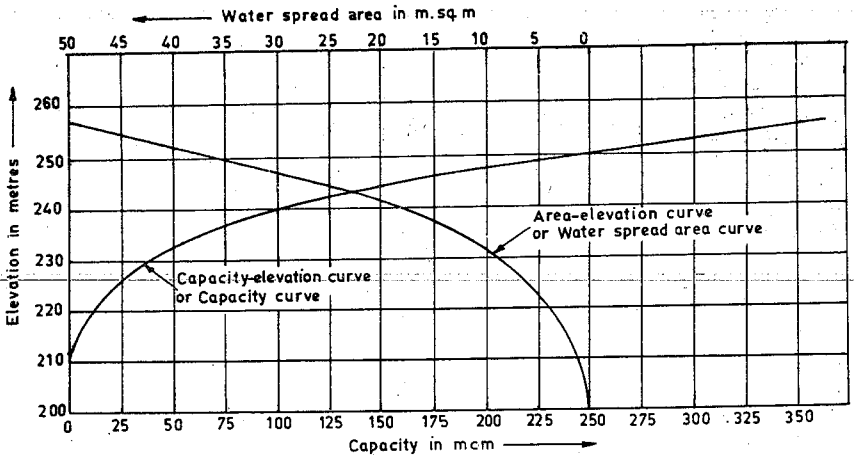


Fig. 18.3. Area elevation curve and Capacity elevation curve.

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.

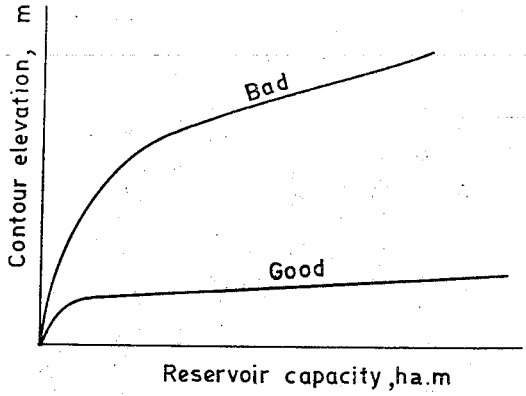


Fig. 18.4. Good and Bad capacity curves for a reservoir

18.3. Storage Zones of a Reservoir

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

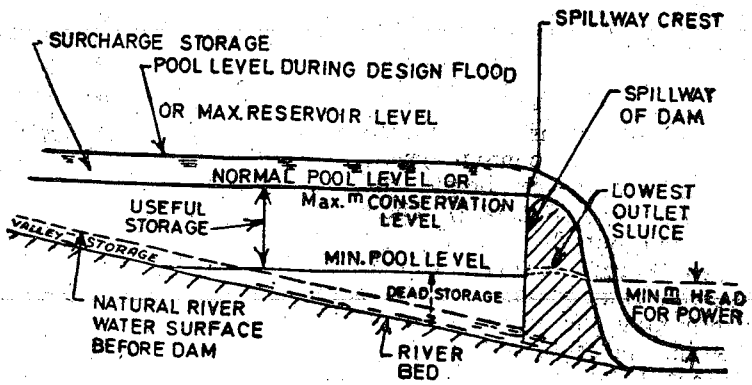


Fig. 18.5. Storage zones of a reservoir.

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 m³** **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

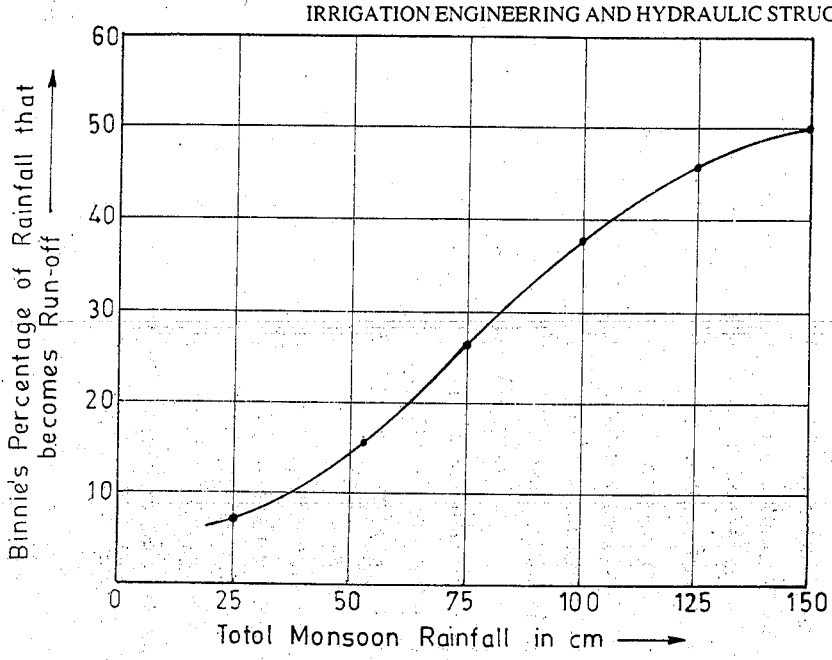


Fig. 18.6(*a*). Binnie's Monsoon Rainfall-runoff Curve.

(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

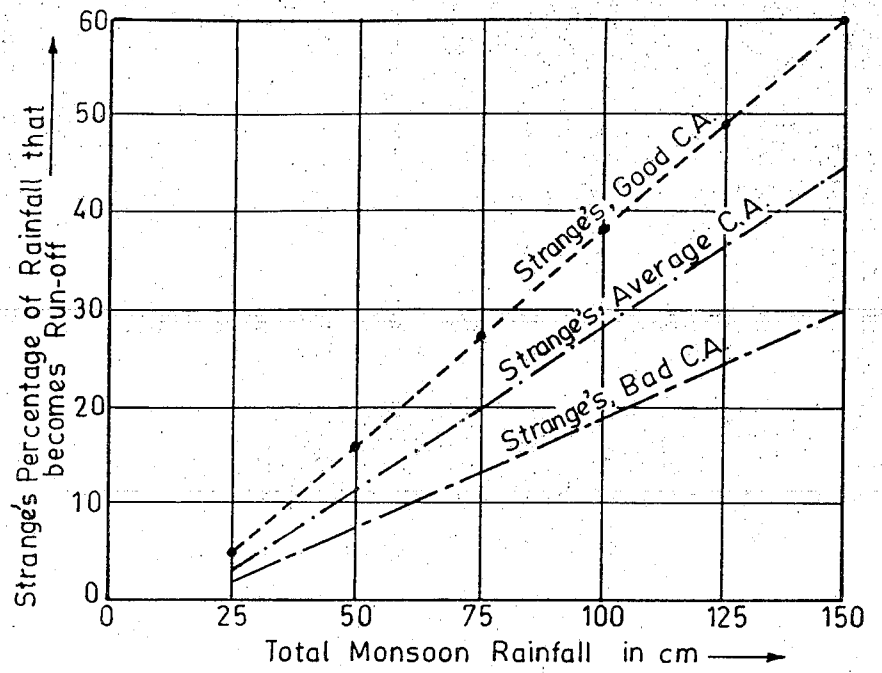


Fig. 18.6(*b*). Strange's monsoon Rainfall-Runoff curves.

(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.* 1000×10^6 m^2)

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

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 :

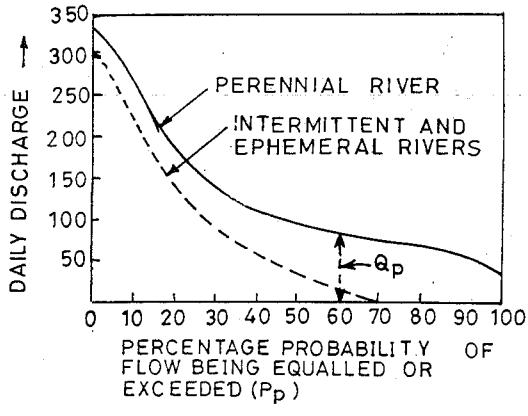


Fig. 18.7(*a*). Typical flow-duration curves on an ordinary paper.

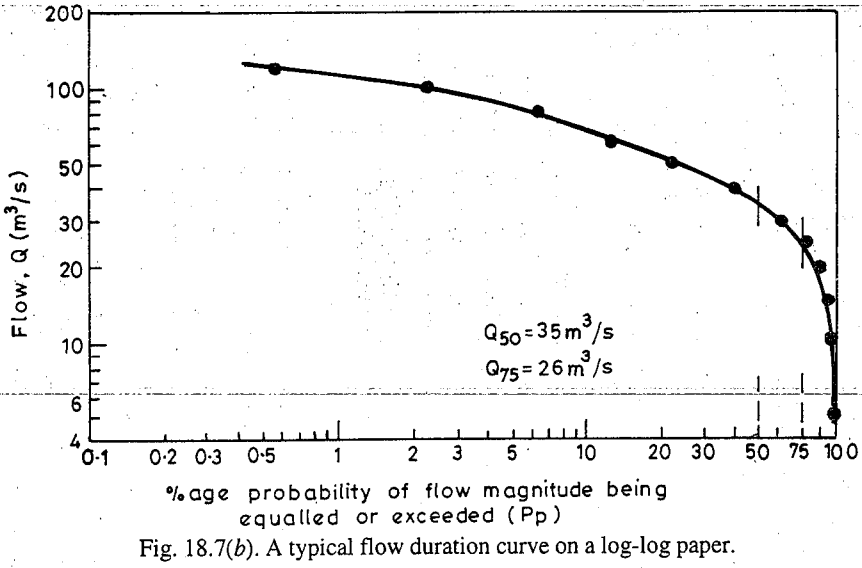


Fig. 18.7(*b*). A typical flow duration curve on a log-log paper.

* 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*).

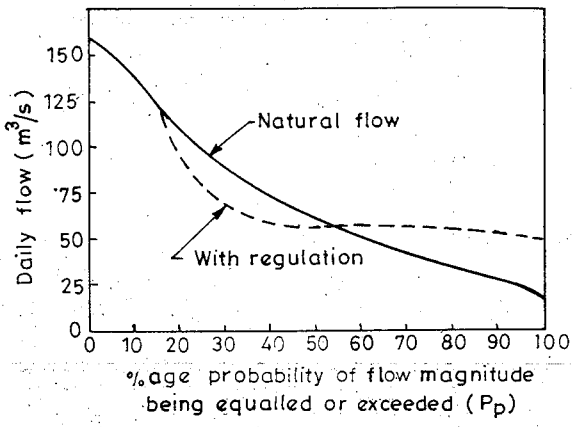


Fig. 18.7. (*c*) Reservoir regulation effect on the flow-duration curve of a stream.

(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.

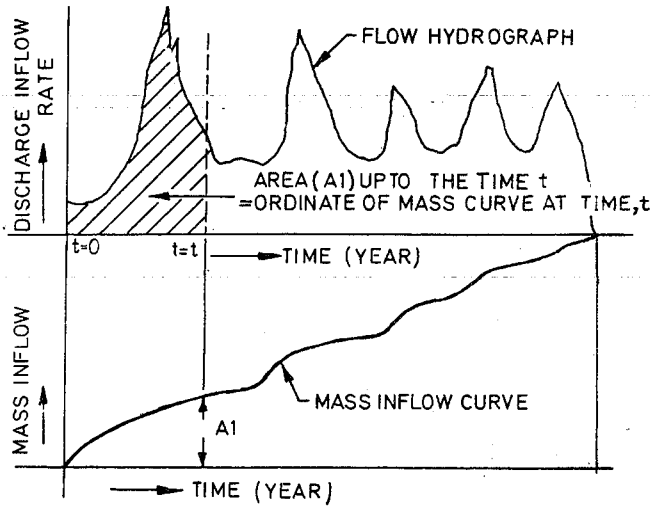


Fig. 18.8 (*a*) and (*b*).

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*)].

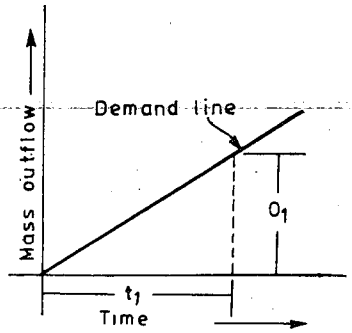


Fig. 18.9 (*a*)

O_1/t_1 = Slope of the line = Demand rate

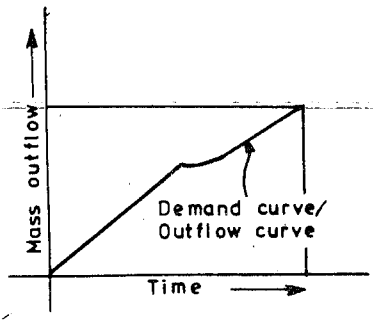


Fig. 18.9 (*b*).

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.

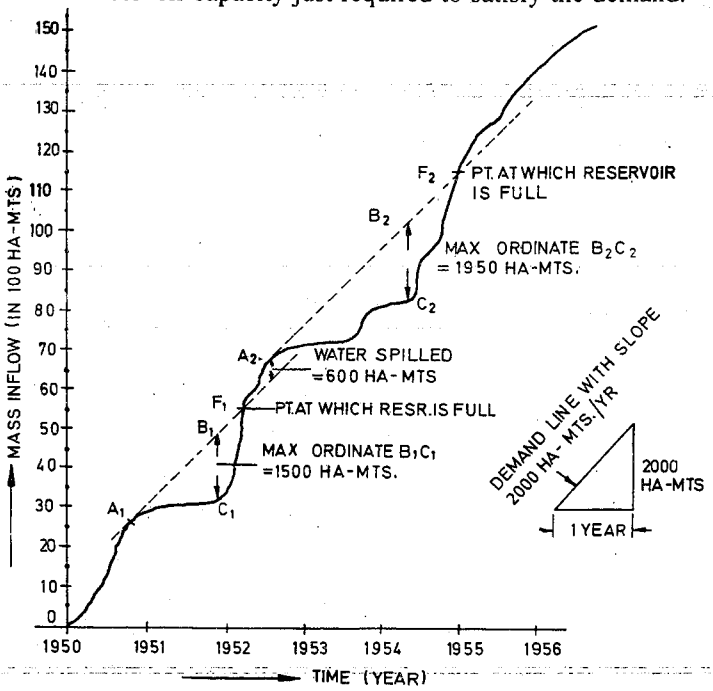


Fig. 18.10.

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.

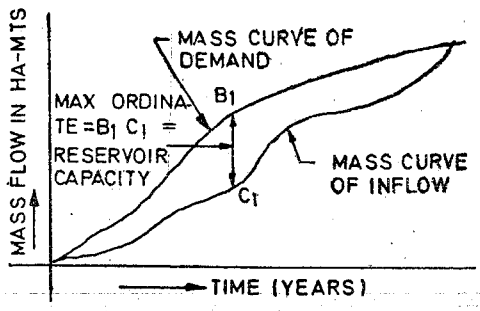


Fig. 18.11.

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 :

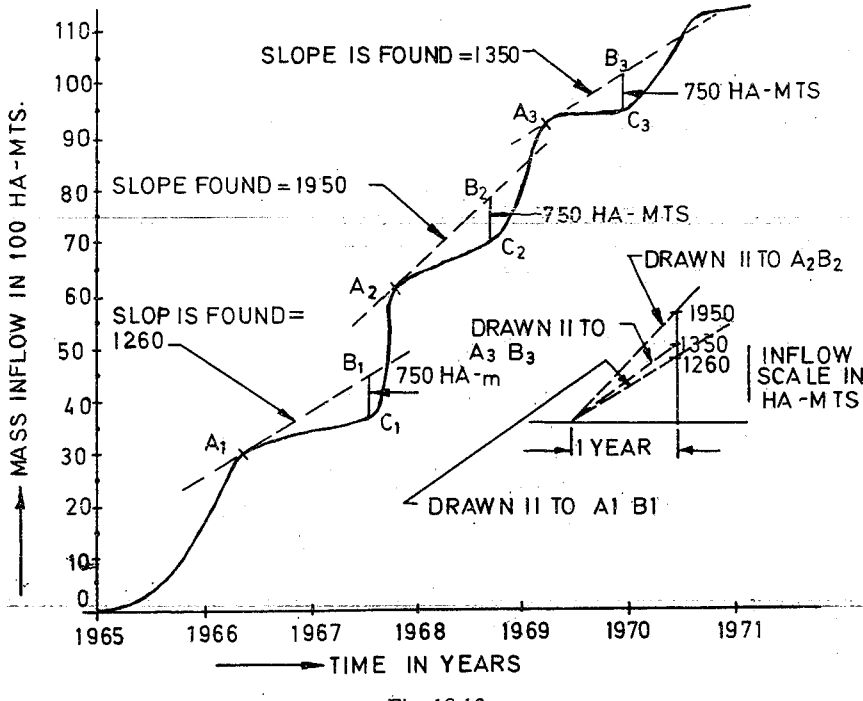


Fig. 18.12.

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 (1)	*Yearly runoff (cm)* (2)	*Runoff as volume in* $\text{M.m}^3 = \frac{\text{Col. (2)}}{100} \times 1675$ (3)	*Cumulative runoff as volume in* M.m^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.

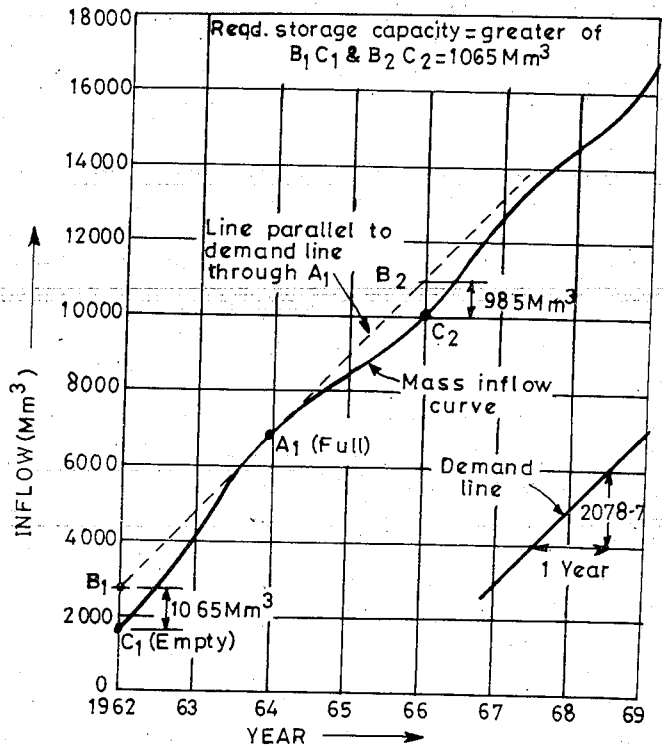


Fig. 18.13.

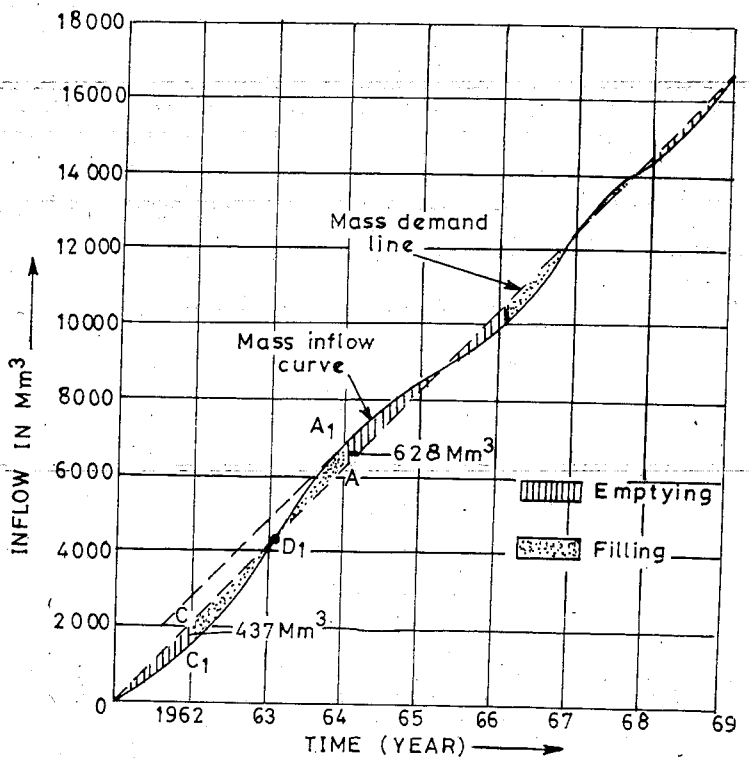


Fig. 18.14.

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

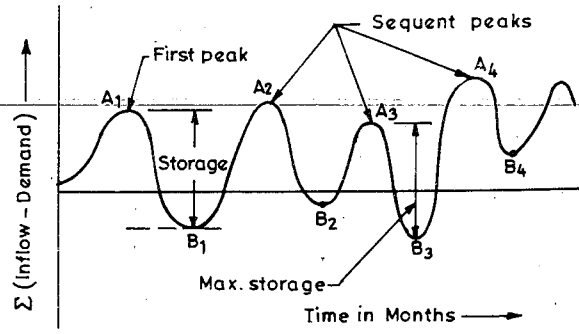


Fig. 18.15. Sequent Peak Algorithm.

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.**

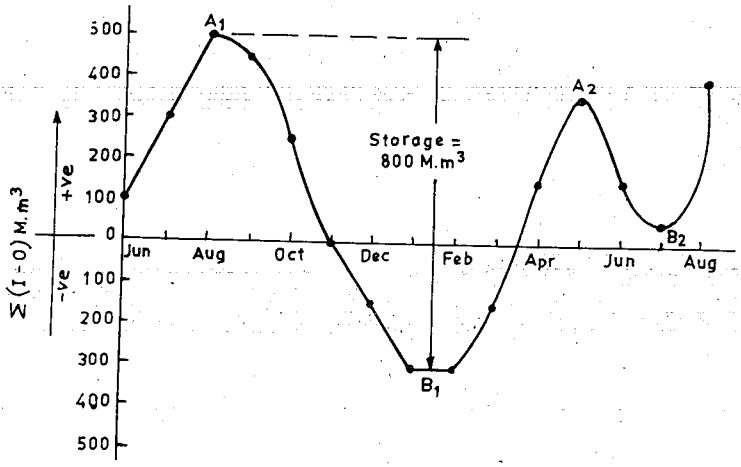


Fig. 18.16. Sequent peak algorithm for Example 18.8.

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.**

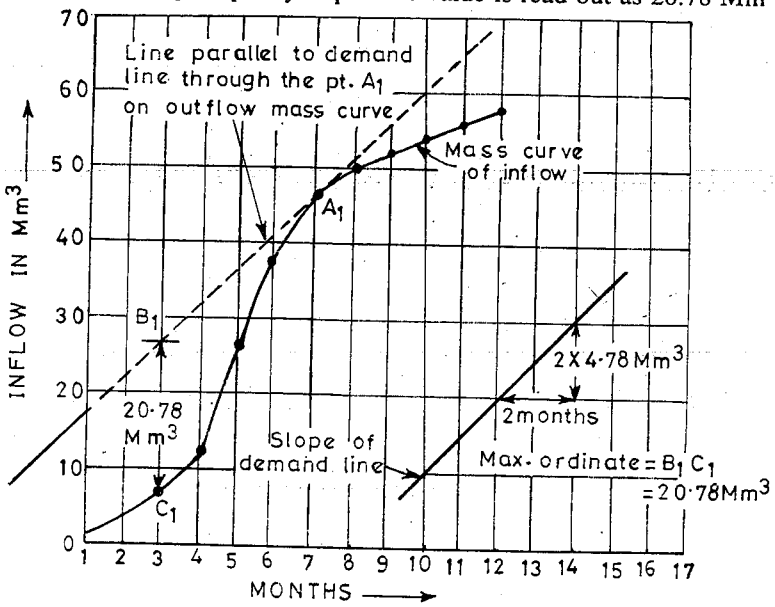


Fig. 18.17.

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.

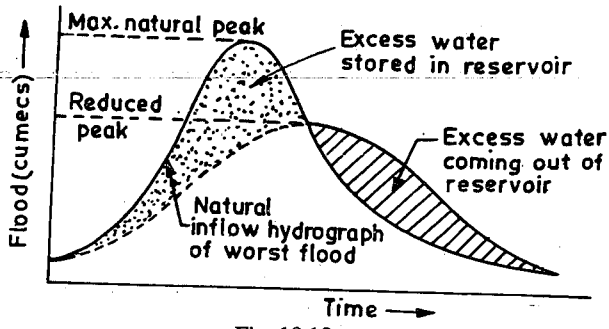


Fig. 18.18.

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

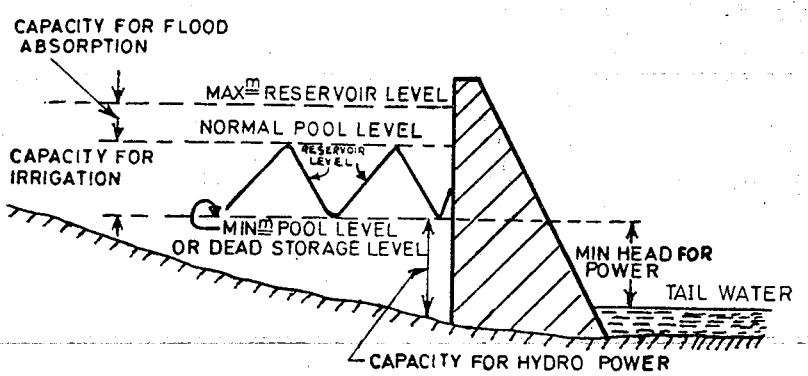


Fig. 18.19.

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.*

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

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
Discharge 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.

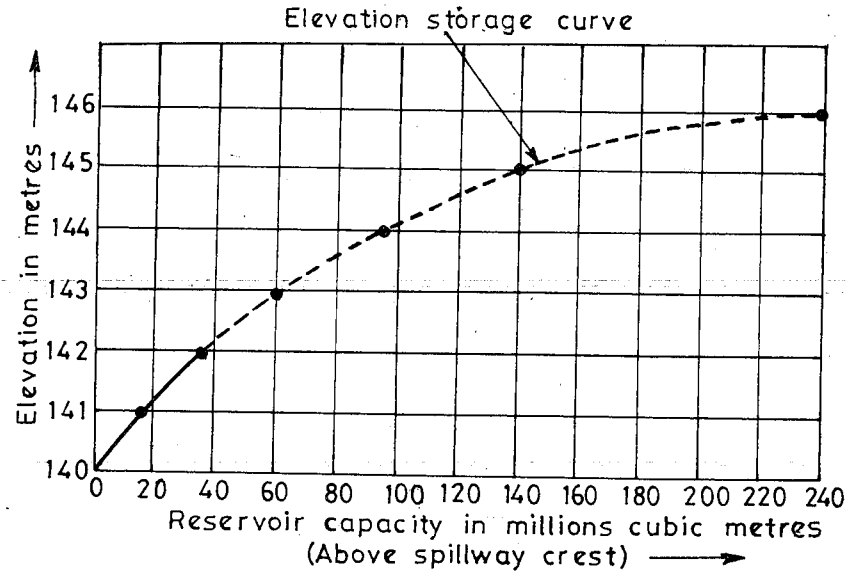


Fig. 18.20. Elevation Storage curve for example 18.13.

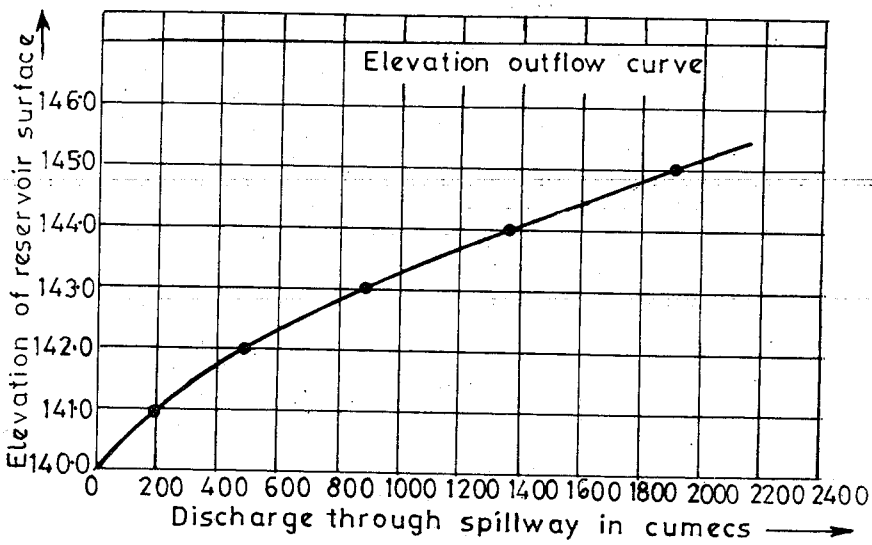


Fig. 18.21. Elevation-Outflow curve for example 18.13.

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	$(S_2 - S_1)$	$O_{(V)} + \Delta S$	
Hrs.	Hrs.	m	m^3/s	m^3/s	$M.m^3$	m^3/s	m	m^3/s	$M.m^3$	reckoned over spillway crest $M.m^3$	M.m3	*i.e.* (ΔS) $M.m^3$	$M.m^3$	Whether $(O_{(V)} + \Delta S)$ 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

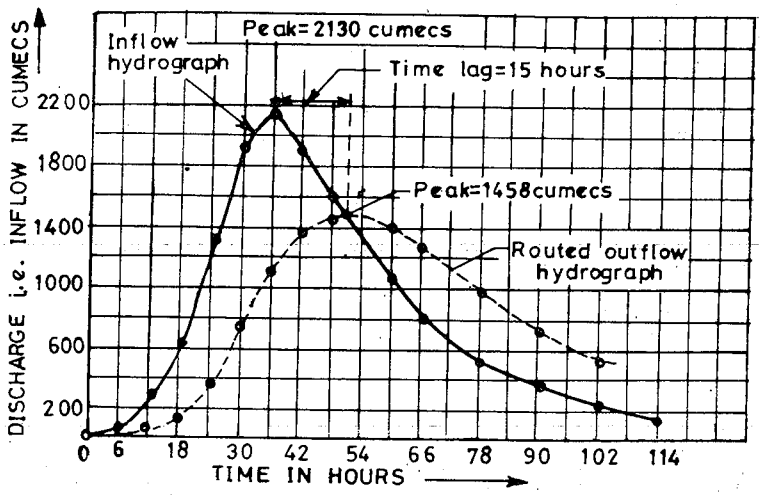


Fig. 18.22. Hydrograph of the given flood and that of the routed flood for example 18.10.

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

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

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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

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

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

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

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

(*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 $S_1 = 0$

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

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

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.

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

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

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

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

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.

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* 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.

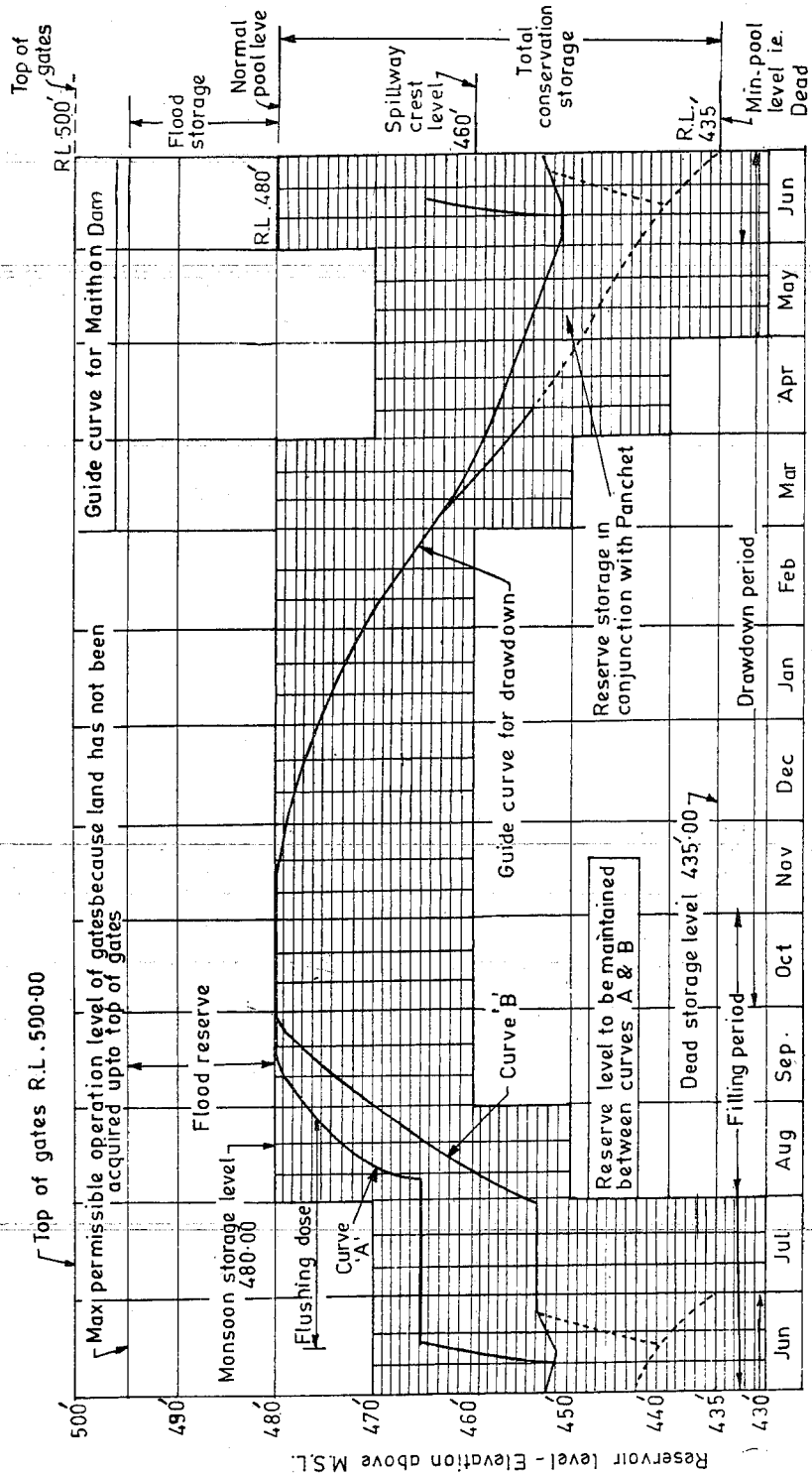


Fig. 18.23. A typical operating guide for Maithon Dam Reservoir.

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.

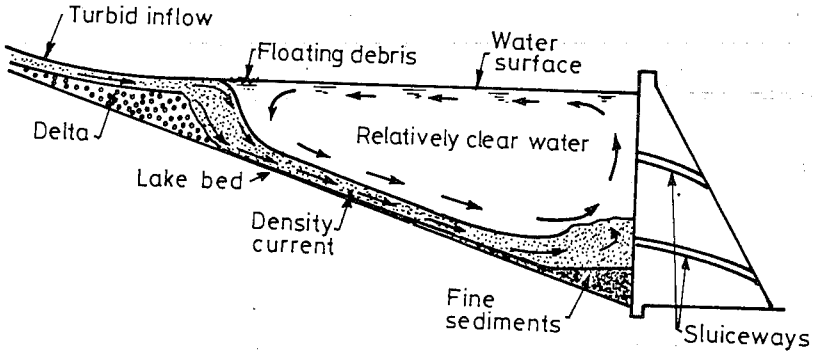


Fig. 18.24. Sediment accumulation in a typical reservoir.

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.

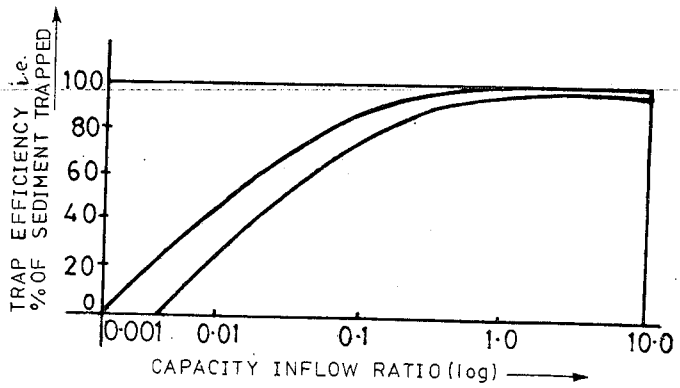


Fig. 18.25.

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. $5 \times \frac{1}{6}$ 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. $10\% \times 5 \text{ Mcum} = 0.5 \text{ Mcum}$.

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 :

$$K = \frac{1}{100}\left[\, 2.1\, M^{1.14}\,(10^{-4})\,(12-a) + 3.25\,(b-2) + 2.5\,(c-3)\,\right] \qquad ...(18.19)$$

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.

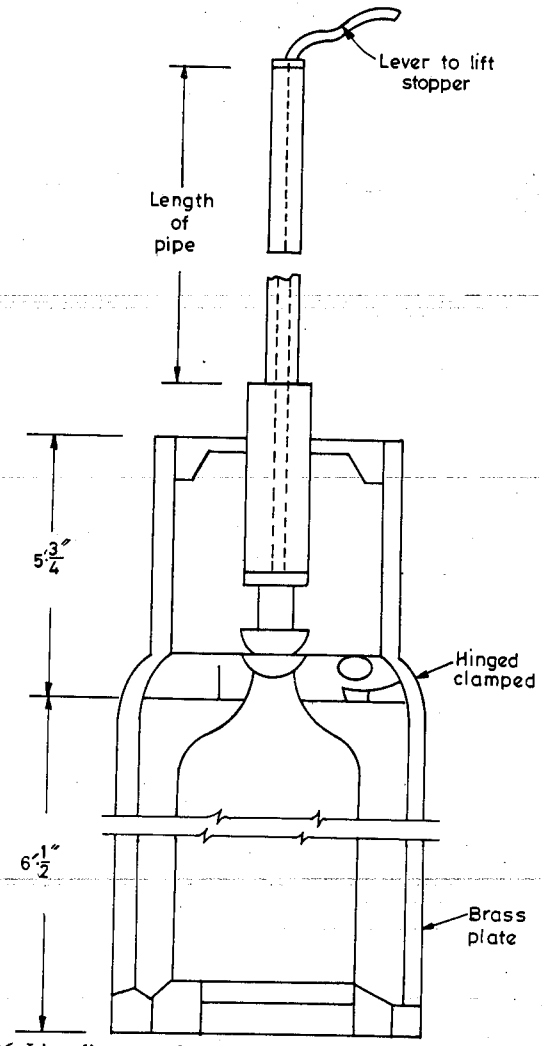


Fig. 18.26. Line diagram of a typical sediment sampler (Punjab bottle sampler).

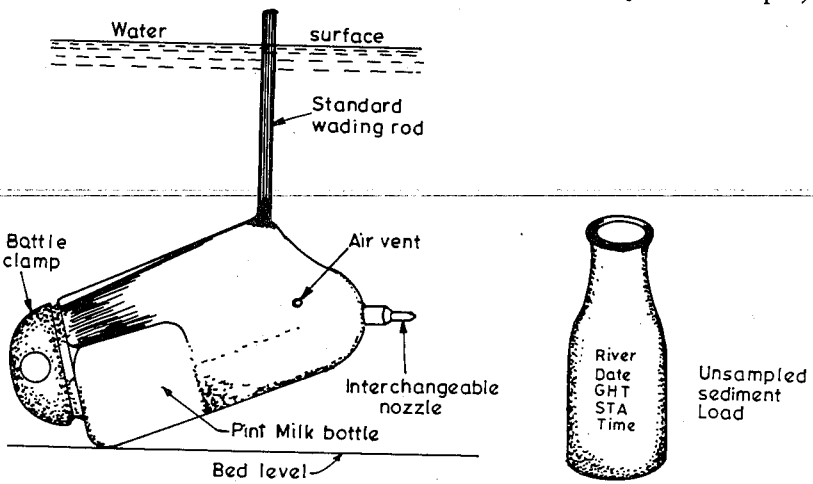


Fig. 18.27. AUS.DH-48 depth-integrating hand sampler for small streams.

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.

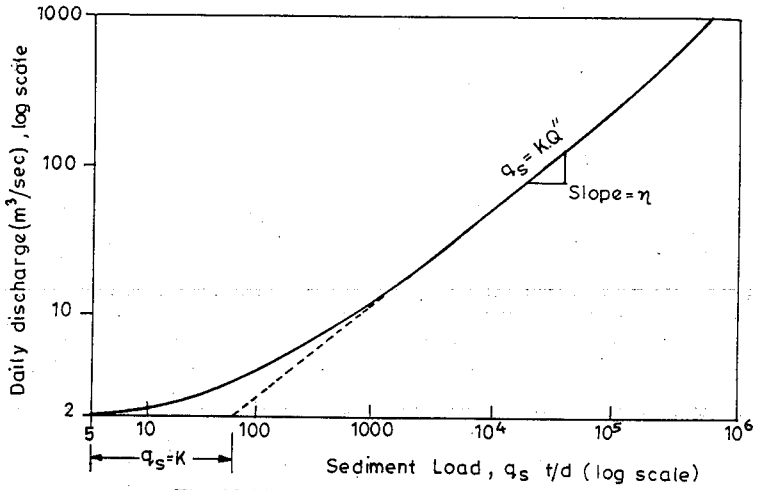


Fig. 18.28. A typical sediment rating curve.

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. $=$ **426.3 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

$$^* \; q_s \text{ (Total sediment in Mm}^3) = \frac{q_s \text{ in M} \cdot \text{t}}{1.2 \text{ t/m}^3} \times (1.1)^{**} = \left(col\,(3) \times \frac{1.1}{1.2} \right)$$

** 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 · m³/100 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 · m³ per 100 sq. km of catchment

A = catchment area in km².

(*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 · m³	*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.

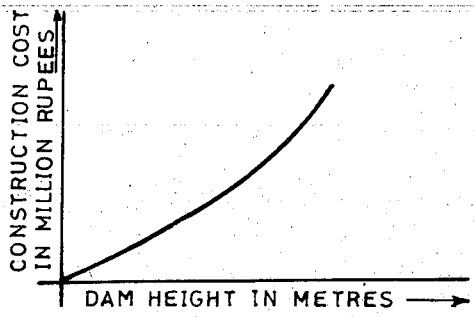


Fig. 18.29

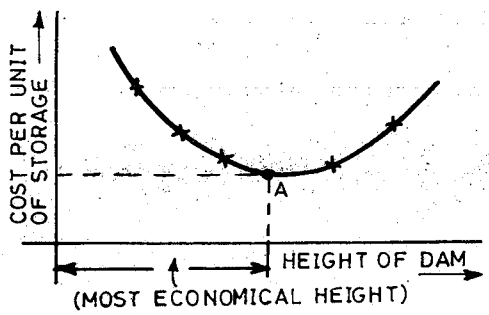


Fig. 18.30

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.**

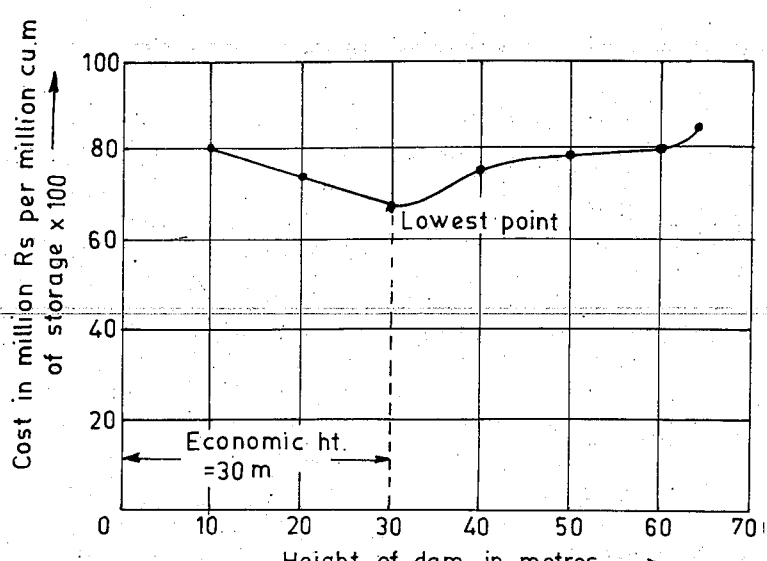


Fig. 18.31.

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.

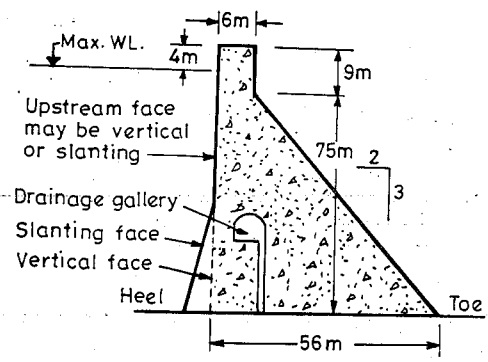


Fig. 19.1. A typical cross-section of a concrete gravity dam.

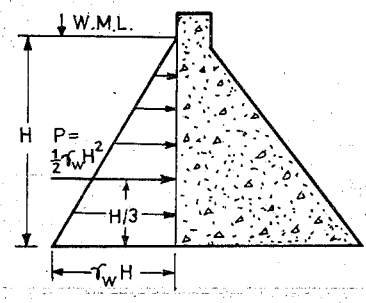
19.3. Forces Acting on Gravity Dam

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).

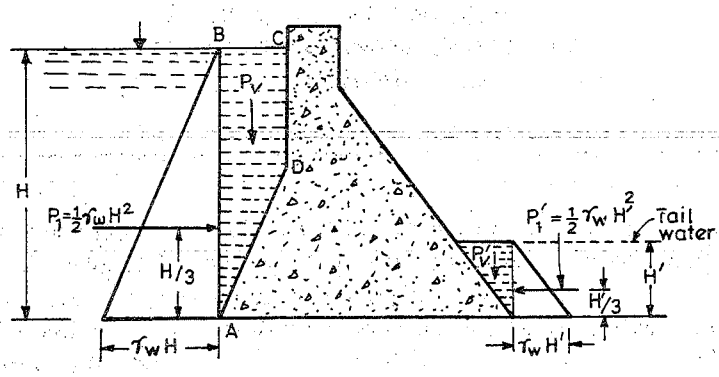


Fig. 19.2 (b)

(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

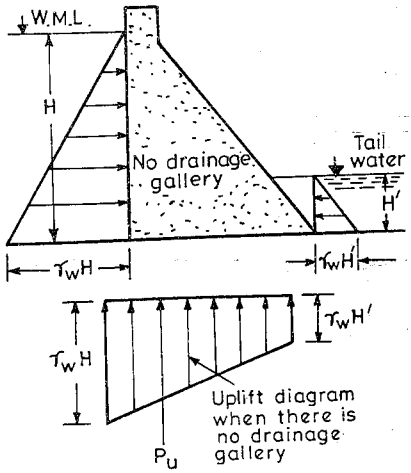


Fig. 19.3 (*a*) Uplift pressure (*U*) diagram, when no drainage gallery is provided.

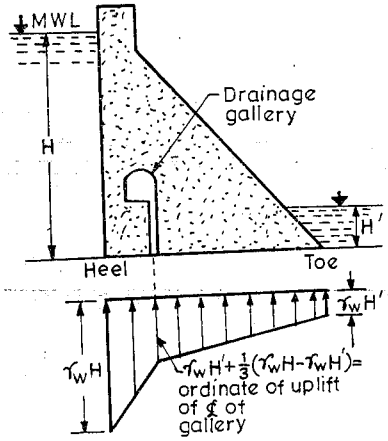


Fig. 19.3 (*b*) Uplift pressure (*U*) diagram, when drainage gallery is provided.

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

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

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.*

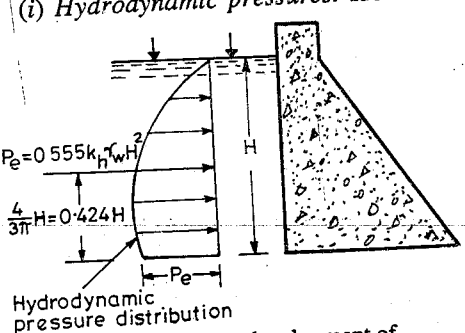


Fig. 19.4. Showing development of Hydrodynamic pressure by a horizontal earthquake moving towards the reservoir. A similar pressure will be developed on d/s tail water when the earthquake is reversed.

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

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

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

$$P_e = 0.726\, p_e \cdot H \qquad ...(19.3)$$

$$\text{where } p_e = C_m \cdot k_h \cdot \gamma_w \cdot H \qquad ...(19.4)$$

$$\therefore \quad P_e = 0.726\, C_m \cdot k_h \cdot \gamma_w \cdot H^2 \qquad ...(19.5)$$

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.

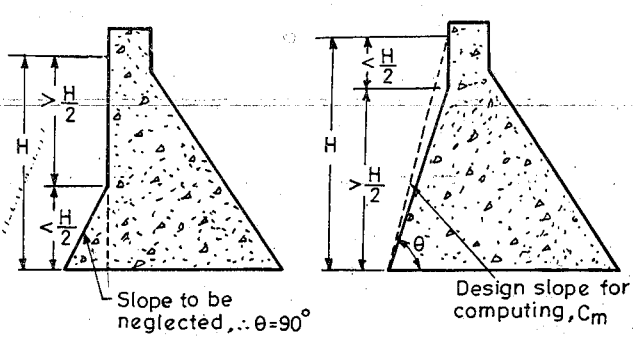


Fig. 19.5

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 $= \frac{1 - \sin\phi}{1 + \sin\phi}$ 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.

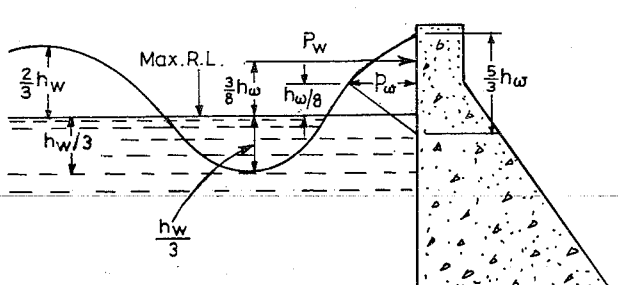


Fig. 19.6

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
$$P_w = 2 \cdot \gamma_w \cdot h_w^2 = 2 \times 9.81\, h_w^2 \text{ kN/m}$$
$$= 19.62\, h_w^2 \text{ kN/m} \quad \text{...(19.14)}$$

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)

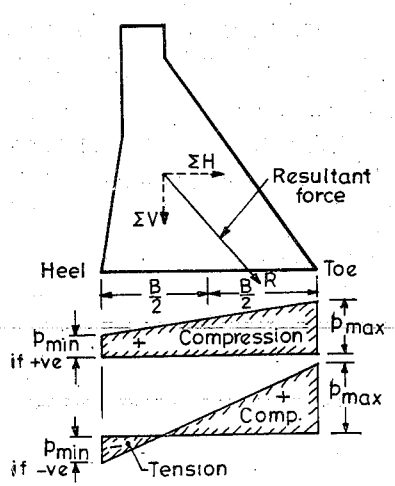


Fig. 19.7. (*a*) Vertical Stress Distribution for Reservoir Full case.

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).

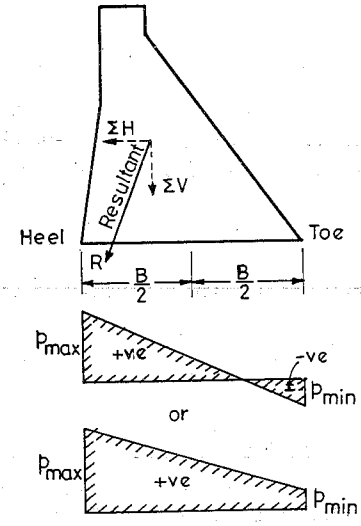


Fig. 19.7. (*b*) Vertical Stress Distribution for Reservoir Empty case.

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.

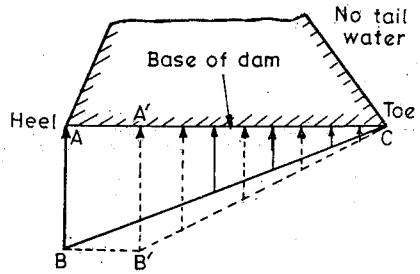


Fig. 19.8

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 $\mu \cdot \Sigma V$.

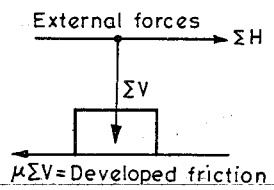


Fig. 19.9

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.

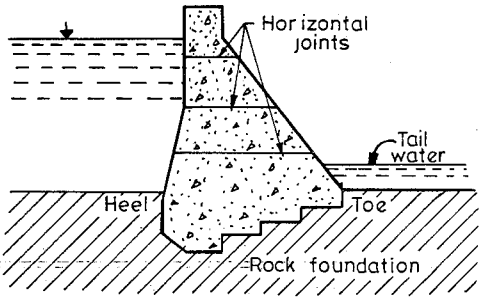


Fig. 19.10

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

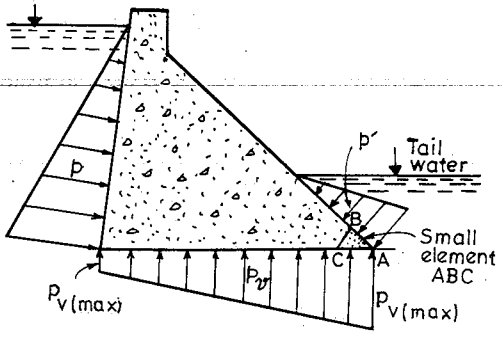


Fig. 19.11 (a)

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.

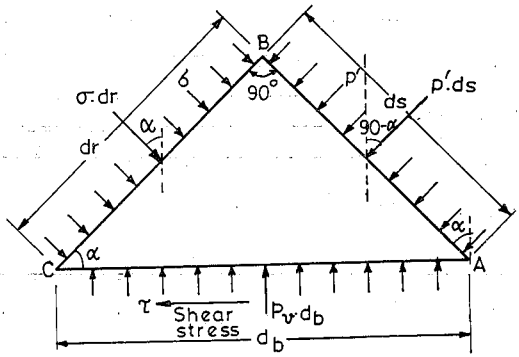


Fig. 19.11. (*b*) Enlarged view of small element *ABC* of Fig. 19.11 (*a*).

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 $$\frac{ds}{db} = \sin\alpha, \quad \text{or } ds = db \cdot \sin\alpha.$$

$$\frac{dr}{db} = \cos\alpha, \quad \text{or } dr = db \cdot \cos\alpha.$$

∴ $$p' \cdot (db \cdot \sin\alpha) \cdot \sin\alpha + \sigma \cdot (db \cdot \cos\alpha) \cos\alpha = p_v \cdot db$$

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

$$\tau_0 = \left[\overline{p_v \sec^2 \alpha - p' \tan^2 \alpha} - p' \right] \sin \alpha \cos \alpha$$

$$\tau_0 = \left[p_v \sec^2 \alpha - p' (1 + \tan^2 \alpha) \right] \sin \alpha \cos \alpha = \left[(p_v - p') \sec^2 \alpha \right] \sin \alpha \cos \alpha$$

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.*

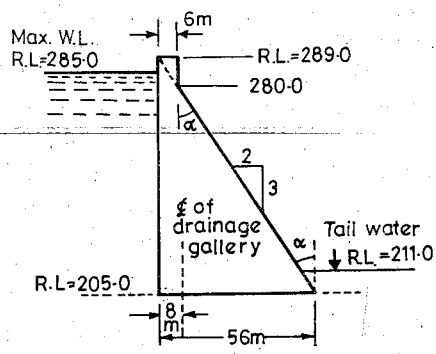


Fig. 19.12 (*a*)

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*).

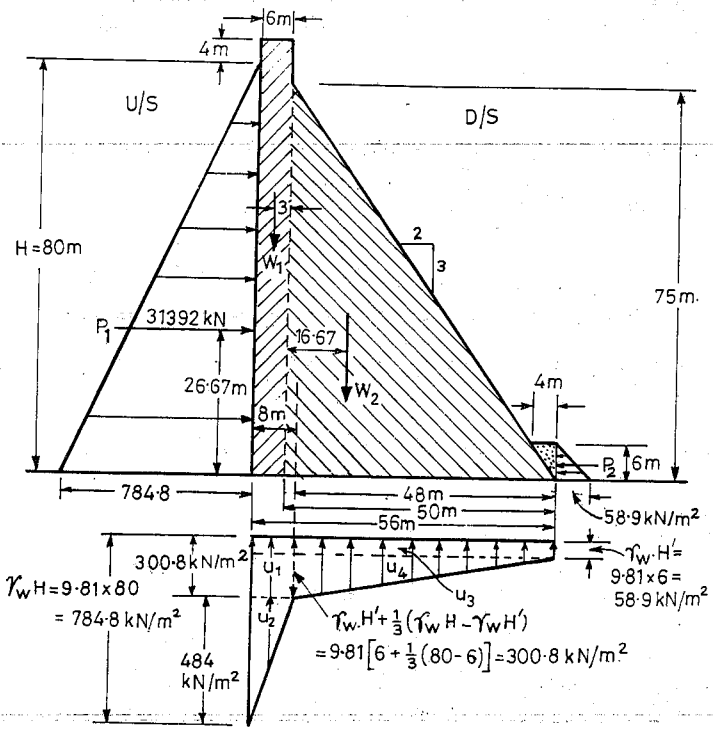


Fig. 19.12 (*b*)

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
		$\Sigma V = 56{,}025 - 12{,}975 =$ **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

$\Sigma M =$ Net (+) moment = 20,96,509 – 4,81,999 – 8,36,871 = 7,77,639 kN-m

$\therefore$ Max. vertical stress $= p_{max}$ at toe $= 768.8 \times 2.065 =$ **1587.6 kN/m²**
$\therefore$ Min. vertical stress $= p_{min}$ at heel $= 768.8 \times (-)\, 0.065 =$ **(–) 49.97 kN/m²** **Ans.**

(*ii*) Major principal stress at toe (σ) is given by Eq. (19.17) as :

$$\sigma = p_{v\,(toe)} \sec^2 \alpha - p' \cdot \tan^2 \alpha$$

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-

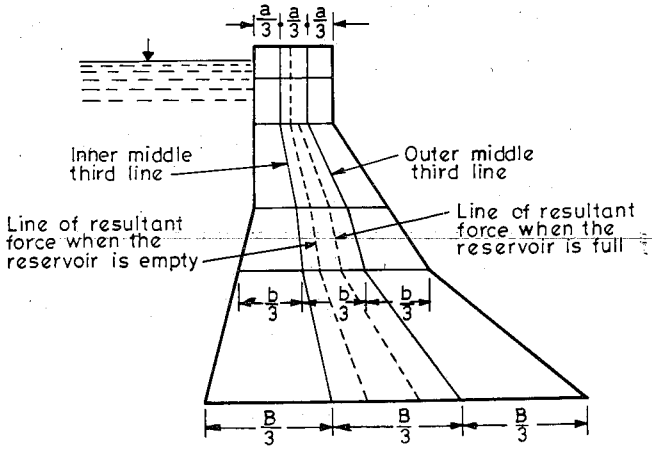


Fig. 19.13

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).

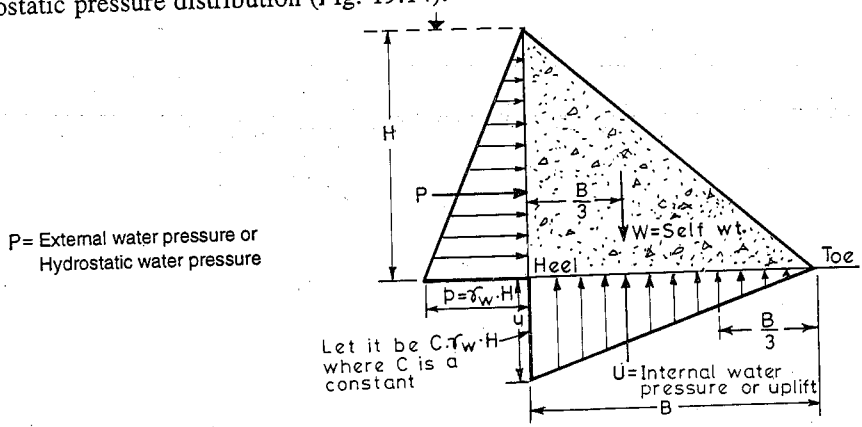


Fig. 19.14

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.

$$\therefore \quad p_{max} \text{ at toe} = \frac{1}{2} \gamma_w \cdot H (S_c - C)\, 2.0 = \gamma_w H (S_c - C)$$

$$\boxed{p_v \text{ at toe} = \gamma_w H (S_c - C)} \qquad \text{...(19.24)}$$

$$P_{\min} \text{ at heel} = 0.$$

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

$$\boxed{\sigma = \gamma_w H (S_c - C + 1)} \qquad \text{...(19.25)}$$

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.

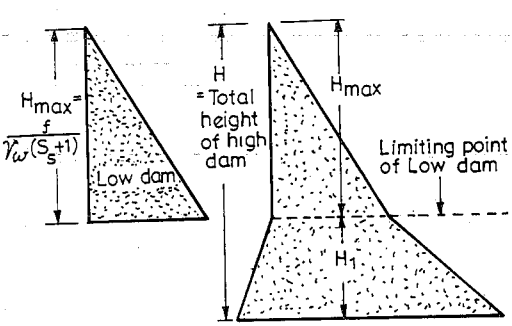


Fig. 19.15

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.

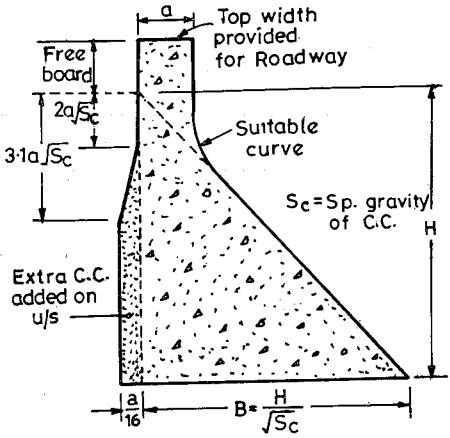


Fig. 19.16.
Typical section of a low gravity dam.

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.

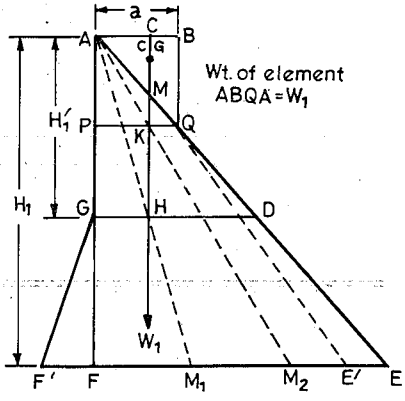


Fig. 19.17

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 :

$$H_1' = 2a \cdot \sqrt{S_c - C}.$$

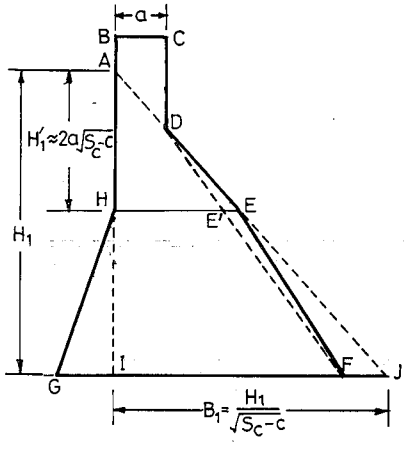


Fig. 19.18

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.

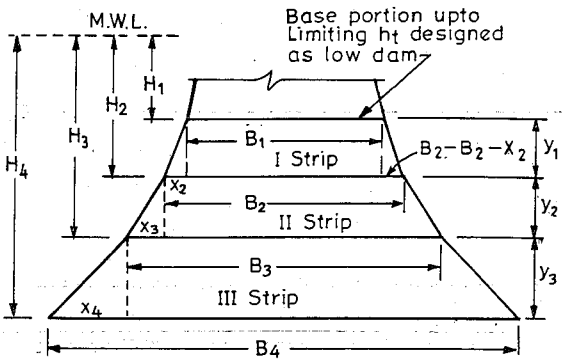


Fig. 19.19

Design of I Strip

The total base width required at the bottom of the 1st strip (B_2) is given by

$$B_2 = \sqrt{\frac{\gamma_w \cdot H_2^3}{f}\left[1 + \frac{\gamma_w^2 \cdot H_2^4}{4 \cdot W_2^2}\right]} \quad ...[19.29\ (a)]$$

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

$$\frac{\gamma_w \cdot S_c \cdot y_1}{24}\left[3 \cdot B_1^2 - B_2^2 + 6 \cdot X_2 (B_1 + B_2) + 2B_1B_2\right]$$
$$-\frac{\gamma_w \cdot X_2}{12}\left[H_1 + H_2\right]\left[2 \cdot B_2 - 3X_2\right] - W_1\left[\frac{B_2 - B_1}{3} - X_2\right] = 0 \quad ...[19.30\ (a)]$$

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

$$B_3 = \sqrt{\frac{\gamma_w \cdot H_3^3}{f}\left[1 + \frac{\gamma_w^2 \cdot H_3^4}{4W_3^2}\right]} \quad \text{...[19.29 (b)]}$$

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

$$\frac{\gamma_w \cdot S_c \cdot y_2}{24}\left[3 \cdot B_2^2 - B_3^2 + 6 \cdot X_3 (B_2 + B_3) + 2 \cdot B_2 B_3\right]$$
$$-\frac{\gamma_w \cdot X_3}{12}\left[H_2 + H_3\right]\left[2 \cdot B_3 - 3X_3\right] - W_2\left[\frac{B_3 - B_2}{3} - X_3\right] = 0 \quad \text{...[19.30 (b)]}$$

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.

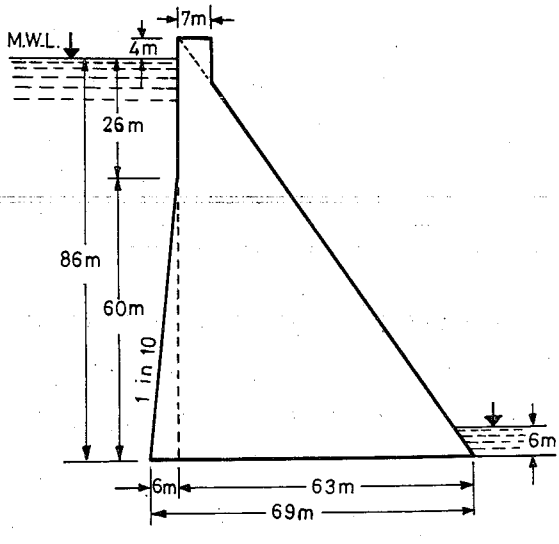


Fig. 19.20 (*a*)

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.

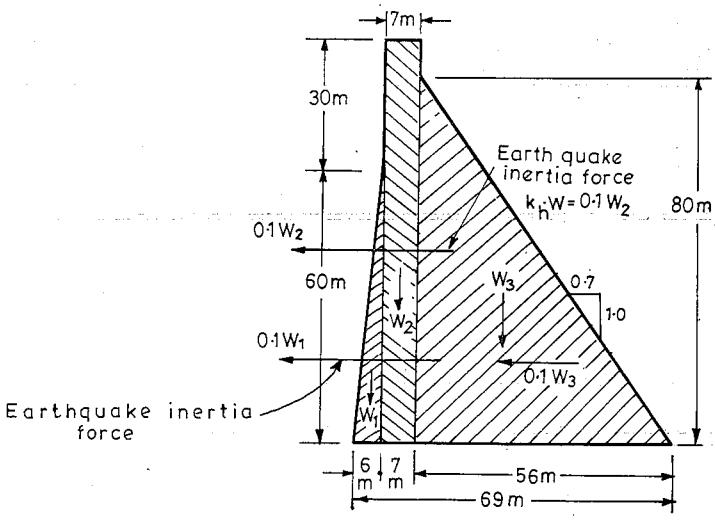


Fig. 19.20 (*b*). Reservoir empty case.

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_{w_1}		$0.1\ W_1 = 0.1 \times 4320 = 432$	20.0	(+) 8640
	P_{w_2}		$0.1\ W_2 = 0.1 \times 1{,}511 = 1511$	45.0	(+) 68000
	P_{W_3}		$0.1\ W_3 = 0.1 \times 5{,}370 = 5370$	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 $= 1004 \times 2.06 = 2070$ kN/m^2 < 3000 (safe)

p_v at toe $= (-)\ 1004 \times 0.06 = -60.3$ 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.

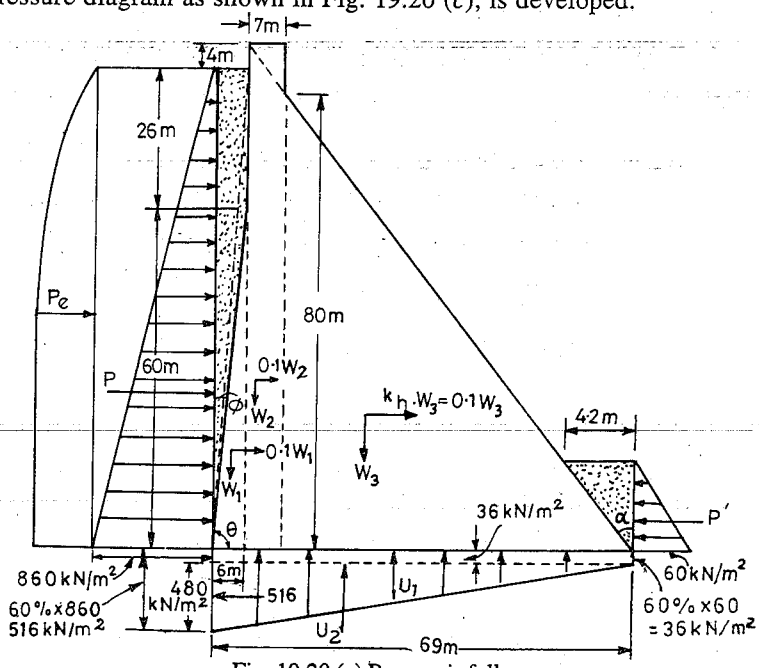


Fig. 19.20 (c) Reservoir full case.

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.g.s.* 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.

$$\therefore \quad \tan\theta = \frac{86}{6} = 14.33$$

$$\theta = 81.9°$$

$$\therefore \quad C_m = 0.735 \times \frac{81.9°}{90°} = 0.668.$$

$$p_e = 0.668 \times 0.1 \times 10 \times 86 = 57.5$$

$$P_e = 0.726 \times 57.5 \times 86 = \mathbf{3580\ kN.}$$

$$M_e = 0.412 \cdot P_e \cdot H = 0.412 \times 3580 \times 86 = \mathbf{1,26,500\ kN.m.}$$

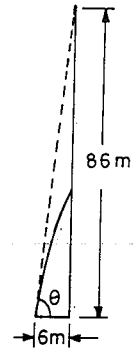


Fig. 19.20 (*d*)

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*		*Lever arm in m*	*Moments about toe in kN. Anticlock wise (+ve) and clockwise (– ve) in kN.m*
		Vertical forces Downward = +ve Upward = –ve	*Horizontal forces Towards Upstream = +ve Towards Downstream = –ve*		
(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

Sliding factor $= \dfrac{0.7 \times 53929}{47693} = 0.79$, which is < 1 (Hence, Unsafe)

Shear friction factor

$$\text{S.F.F.} = \frac{\mu \cdot \Sigma V + B \cdot q}{\Sigma H}$$

$$= \frac{0.7 \times 53929 + 69 \times 1400}{47693}$$

$= 2.81$; which is less than 3 (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 at toe $= 1057 \times 1.81 = \mathbf{1913\ kN/m^2} < 3000$ (∴ Safe)

p_v at heel $= 1057 \times 0.19 = \mathbf{201\ kN/m^2} < 3000$ (∴ Safe)

Principal stress at toe $= \sigma = p_v \cdot \sec^2 \alpha - p' \tan^2 \alpha$ ...(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

$$\sigma_1 = p_{v(heel)} \sec^2 \phi - (p + p_e) \tan^2 \phi \qquad ...(19.19)$$

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

$$\tau_0 = (p_v - p') \tan \alpha \qquad i.e. \text{ Eq. (19.20)}$$

$$= (1913 - 60)\,0.7$$

$$= 1853 \times 0.7 = 1297 \text{ kN/m}^2 < 1400$$

(∴ 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

$$\tau_{0(heel)} = p_{v(i.e.,l)} \tan \phi$$
$$= 1880 \times 0.1 = 188 \text{ kN/m}^2.$$

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 kN/m^2.	**Ans.**
Maximum compressive stress developed in dam	= 2191 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*).

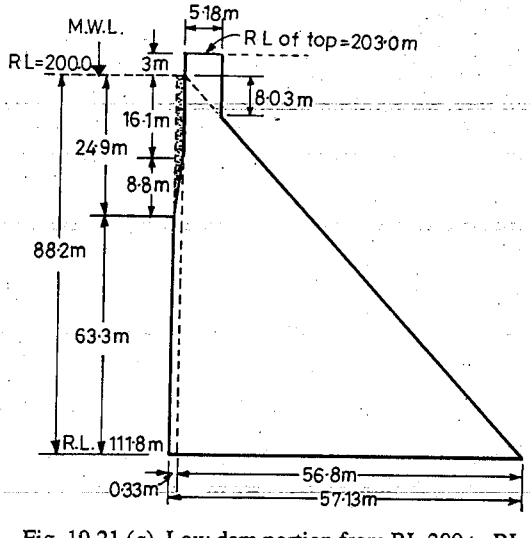


Fig. 19.21 (*a*). Low dam portion from RL 200 to RL 111.8.

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.

$$= 734\,(1 + 0.01) - 860 \times 0.01$$
$$= 742 - 9 = \mathbf{733\ kN/m^2}$$

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 kN/m^2.	**Ans.**
Maximum compressive stress developed in dam	= 2191 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*).

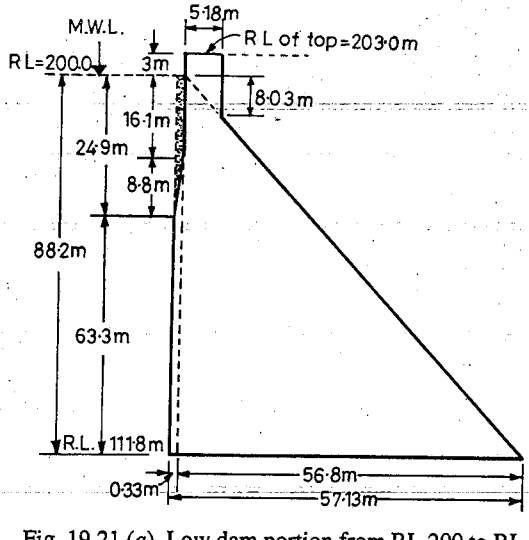


Fig. 19.21 (*a*). Low dam portion from RL 200 to RL 111.8.

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*).

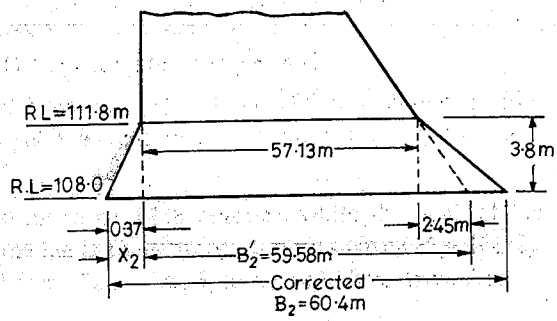


Fig. 19.21 (*b*)

$$B_2' = 57.13 + \frac{56.8}{88.2} \times 3.8$$

$$= 57.13 + 2.45 = 59.58 \text{ m}$$

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 $$3.8[9{,}780 - 3{,}640 + 705X_2 + 6{,}900] - 10X_2[1{,}815 - 45X_2] - 61{,}670[1.09 - X_2]$$

or $$4.960 + 268X_2 - 1{,}815X_2 + 45X_2^2 - 6{,}720 + 6{,}167X_2 = 0$$

or $$45X_2^2 + 4{,}620X_2 - 1{,}760 = 0$$

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 $f = 3000 \text{ kN/m}^2$

$H_4 = 100$ m

$W_4 = 80{,}620$ kN.

$$\therefore \quad B_4 = \sqrt{\frac{10 \times (100)^3}{3000}\left[1 + \frac{10^2 \times (100)^4}{4 \times (80620)^2}\right]}$$

$$= \sqrt{3{,}333\,[1 + 0.385]} = \sqrt{3{,}333\,(1.385)} = \sqrt{4{,}620} = 68.0 \text{ m.}$$

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 $y_3 = 4.0$ m

$B_3 = 64.0$ 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*).

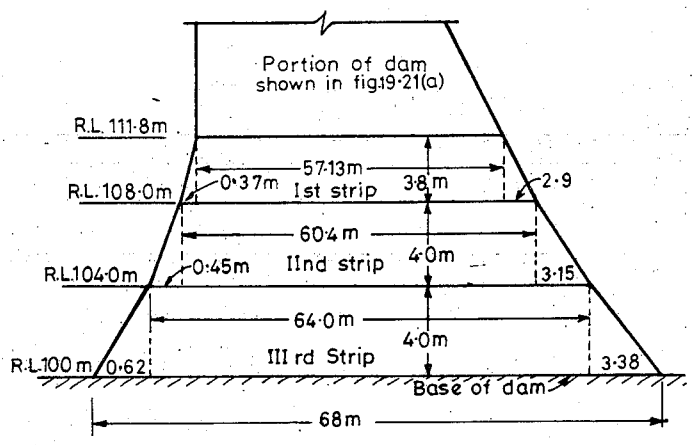


Fig. 19.21 (*c*)

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.

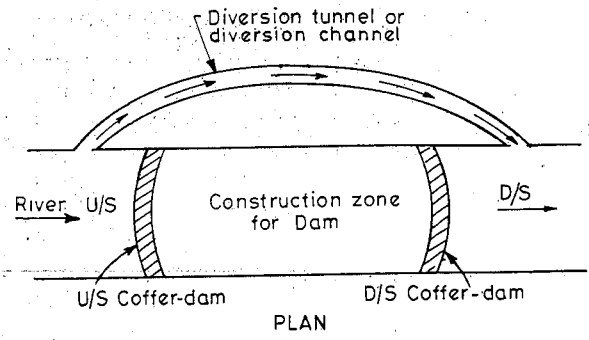


Fig. 19.22

(*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.

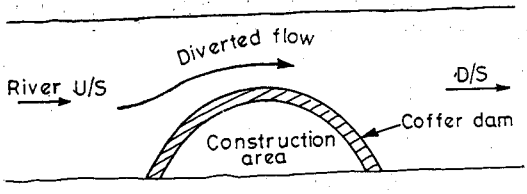


Fig. 19.23

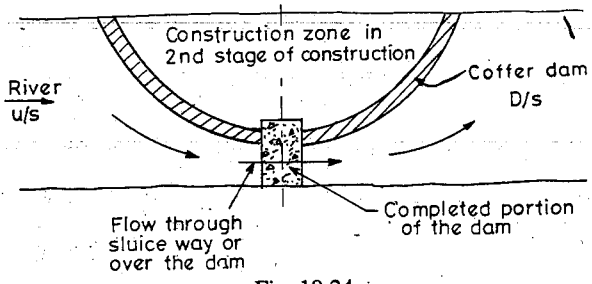


Fig. 19.24

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.

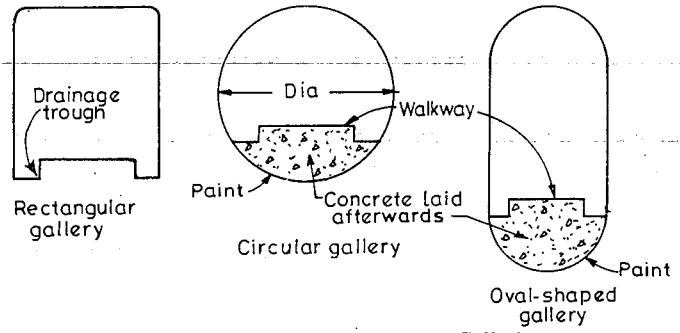


Fig. 19.25. Various Types of Dam Galleries.

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.

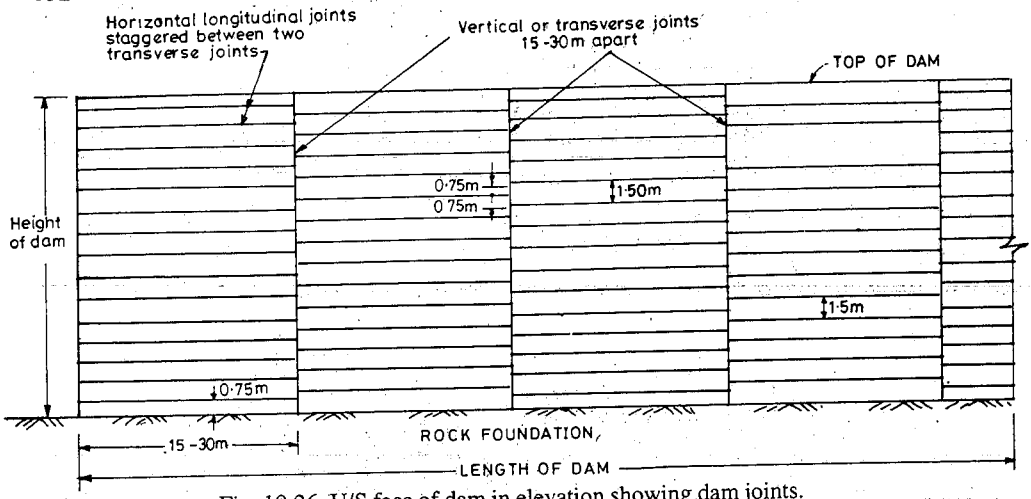


Fig. 19.26. U/S face of dam in elevation showing dam joints.

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.

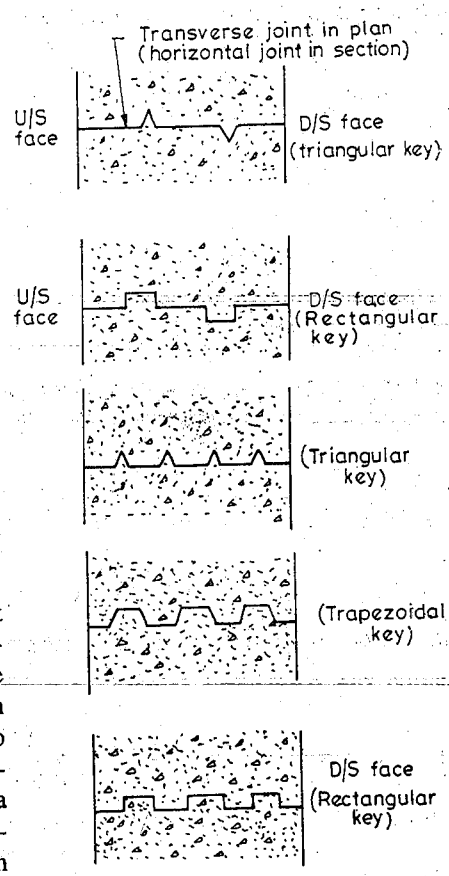


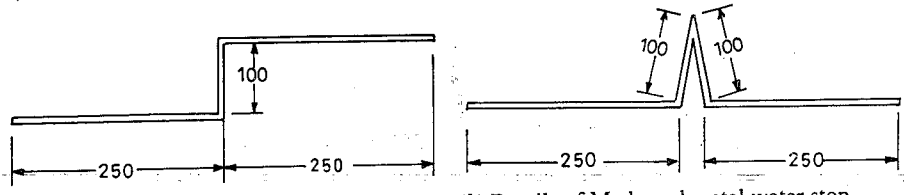
Fig. 19.27. Various types of key-ways.

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.

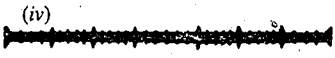
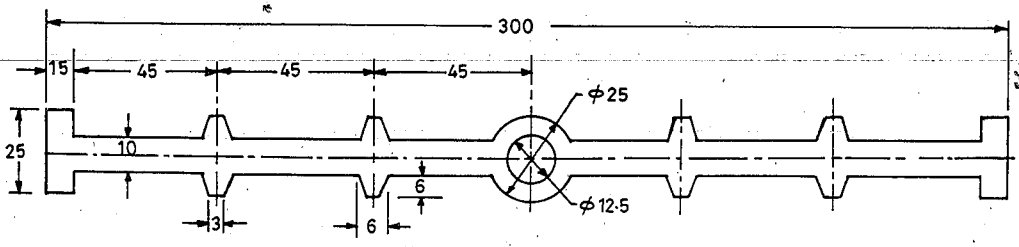
Shape of PVC or Rubber Waterstop	*Type*	*Category*	*Width*	*Thickness*	*Hydraulic Heads*	*Safe Stability*
(*i*)	230XA	Dumb Bell with Central Bulb	230	10	50	Used where large movement due to expansion/contraction is expected. Also for settlement joints and crumple sections
(*ii*)	230G	Serrated with Central 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.

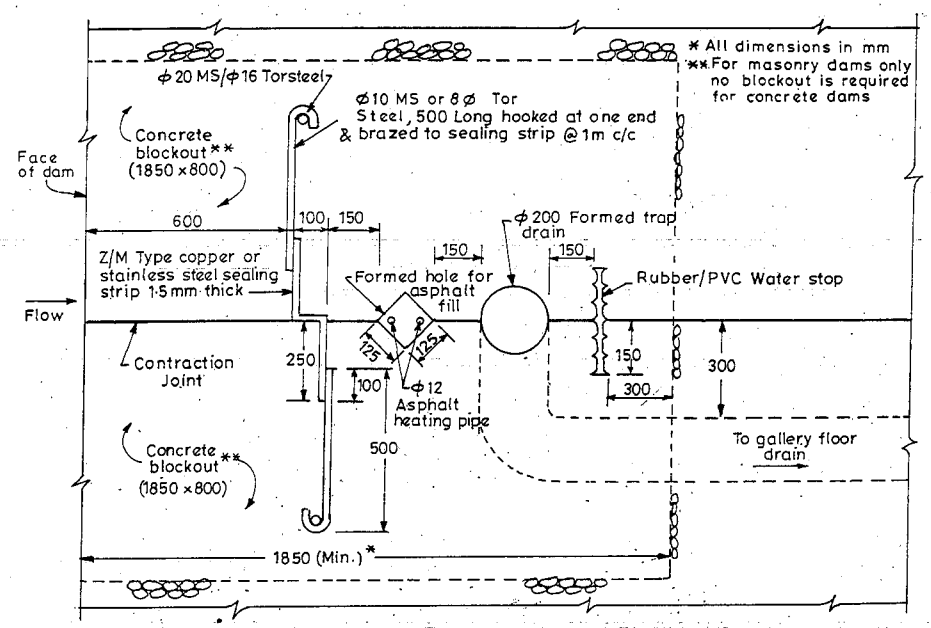


Fig. 19.31. Sectional plan at a contraction joint (square hole shows location of asphalt water stop)
Note : Dowel bars between concrete/masonry face of the blockout not shown.

19.14.2.1. Installation of water stops

(1) The metal waterstops shall be erected with the help of anchor rods.

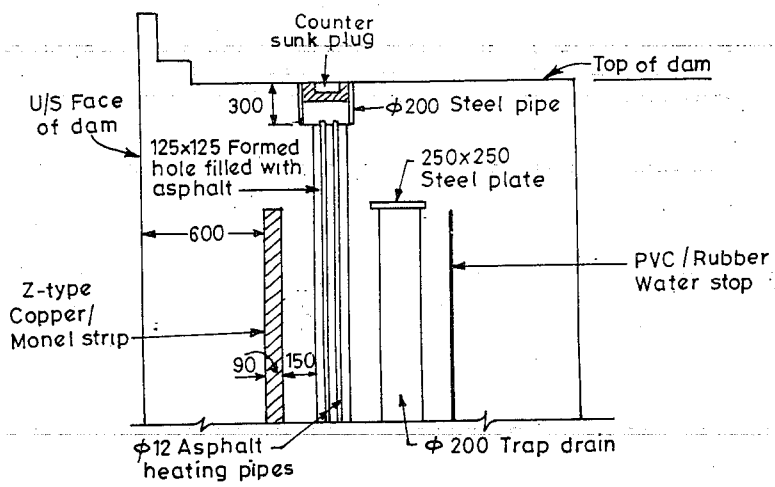
(2) In the case of masonry dams, the surface adjacent to the blockouts (shown by dotted lines in Fig. 19.30) shall be irregular and the joints in the masonry shall be raked out when mortar is green, with some stone protruding beyond dotted lines regularly in both directions. No such blockouts shall be 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.

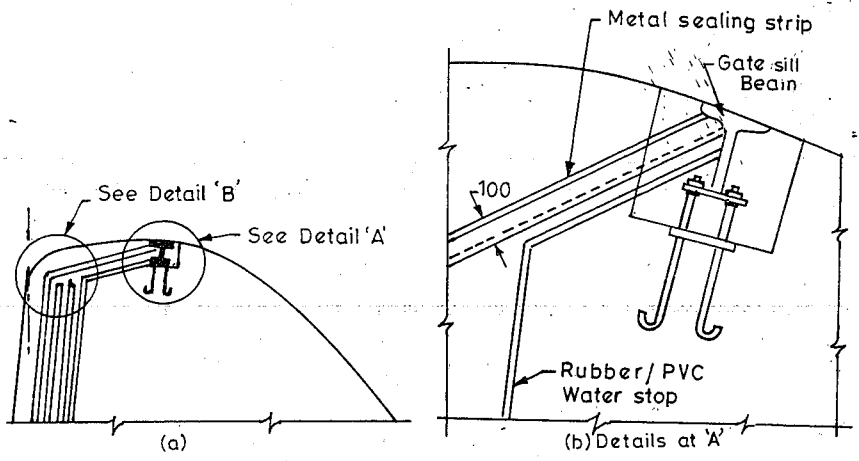
(6) Typical details of waterstop arrangement (at contraction joints between two monoliths of a dam) near the top of a non-overflow section are shown in Fig. 19.32 ; near the crest of an overflow section in Fig. 19.33 and near the bottom of the dam in Fig. 19.34.

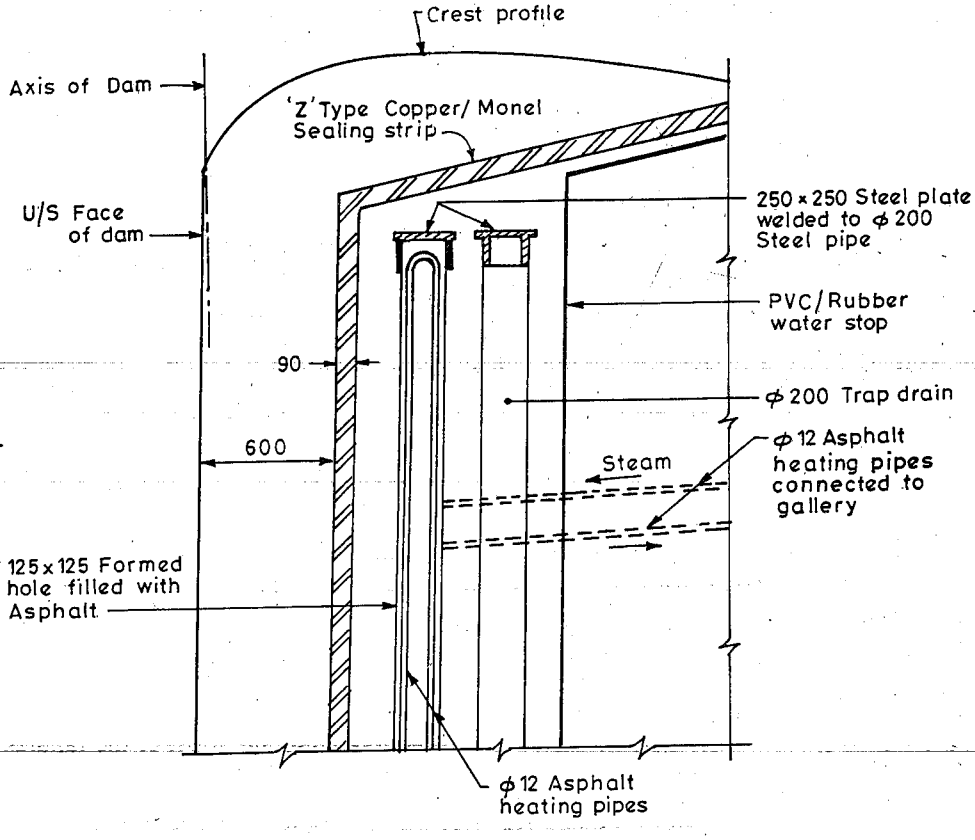


All Dimensions in mm

Fig. 19.32. Typical waterstop details near the top of non-overflow section of a dam.

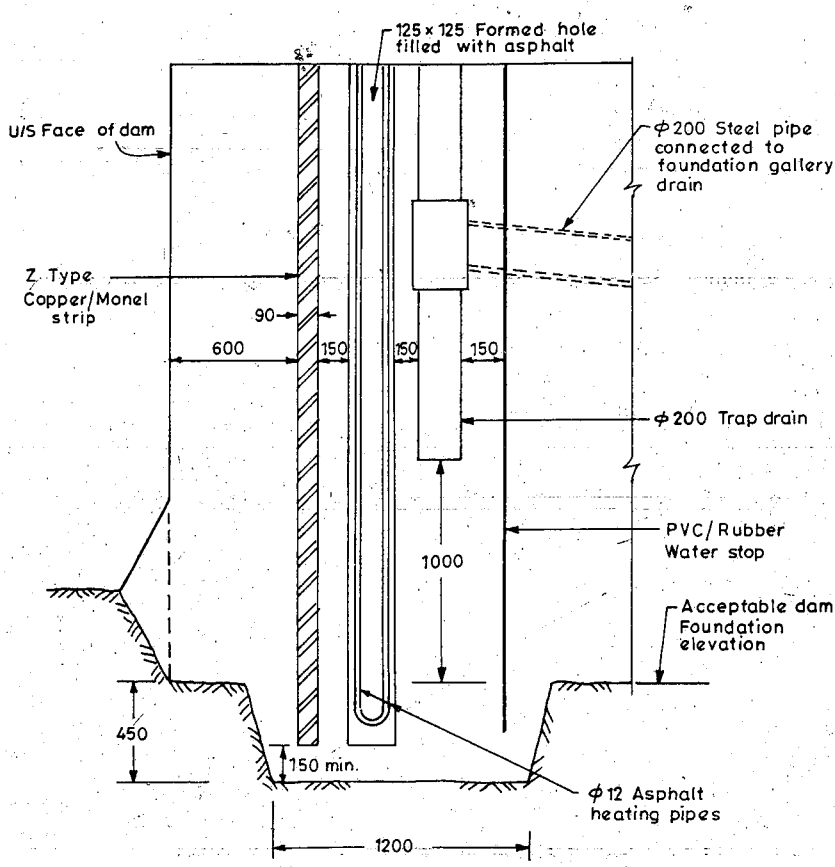
*IS code on "Methods of Testing Plastics".





(*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

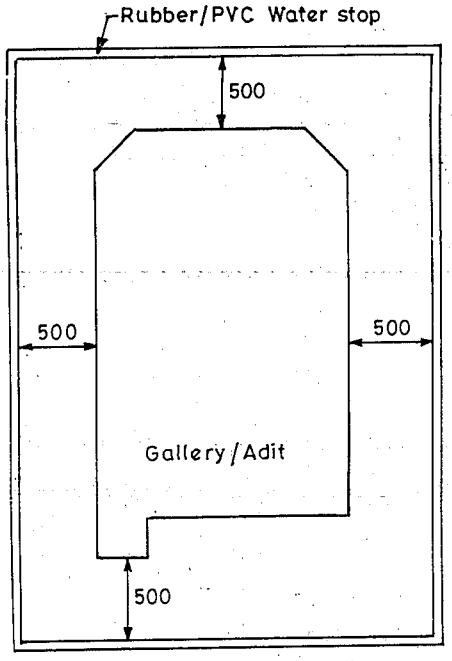


Fig. 19.35. Rubber/PVC waterstop around Dam Gallery/Adit at contraction joints.

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.

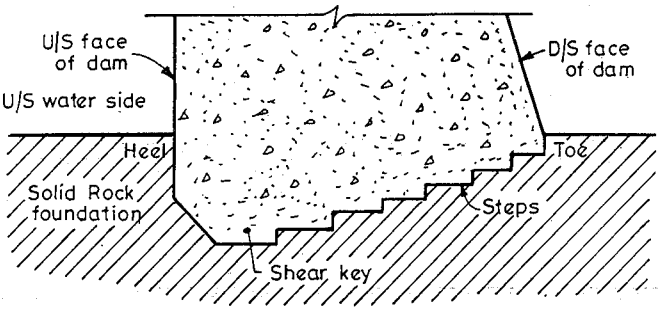


Fig. 19.36. Stepping of dam foundation and provision of shear key.

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):

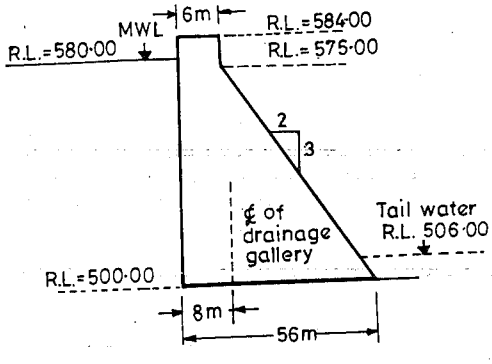


Fig. 19.37

(*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

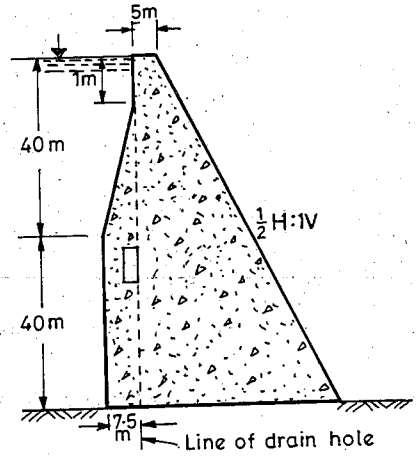


Fig. 19.38

(*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.

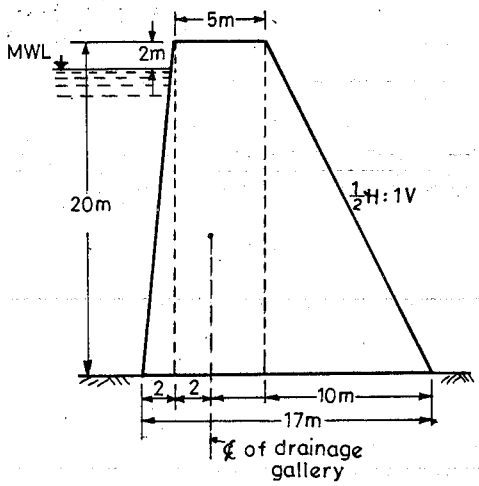


Fig. 19.39

20

Earthen Dams and Rock Fill Dams

20.1. Introduction

Earthen dams and earthen levees are the most ancient type of embankments, as they can be built with the natural materials with a minimum of processing and primitive equipment. But in ancient days, the cost of carriage and dumping of the dam materials was quite high. However, the modern developments in earth moving equipments have considerably reduced the cost of carriage and laying of the dam materials. The cost of gravity dams on the other hand, has gone up because of an increase in the cost of concrete, masonry, etc. Earthen dams are still cheaper as they can utilise the locally available materials, and less skilled labour is required for them.

Gravity dams and arch dams require sound rock foundations, but earthen dams can be easily constructed on earth foundations. However, earth dams are more susceptible to failure as compared to rigid gravity dams or arch dams. Before the development of the subject of Soil-Mechanics, these dams were being designed and constructed on the basis of experience, as no rational basis for their design was available. This led to the failure of various such earthen embankments. However, in these days, these dams can be designed with a fair degree of theoretical accuracy, provided the properties of the soil placed in the dam, are properly controlled. This condition makes the design and construction of such dams, thoroughly interdependent. Continuous field observations of deformations and pore water pressures have to be made during the construction of such dams. Suitable modifications in the design, are then made during construction, depending upon these field observations.

20.2. Types of Earthen Dams

The earthen dam can be of the following three types :

1. *Homogeneous Embankment type*
2. *Zoned Embankment type*
3. *Diaphragm type.*

These three types of dams are described below :

(1) **Homogeneous Embankment Type.** The simplest type of an earthen embankment consists of a single material and is homogeneous throughout. Sometimes, a blanket of relatively impervious material may be placed on the upstream face. A purely homogeneous section is used, when only one type of material is economically or locally available. Such a section is used

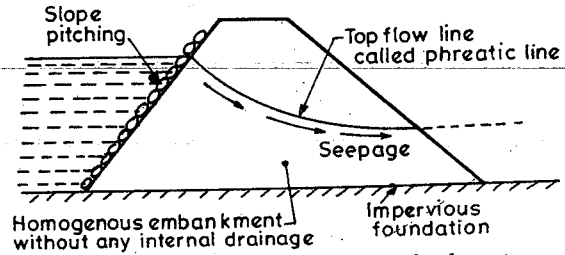


Fig. 20.1 (*a*). Homogeneous type embankment.

for low to moderately high dams and for levees. Large dams are seldom designed as homogeneous embankments.

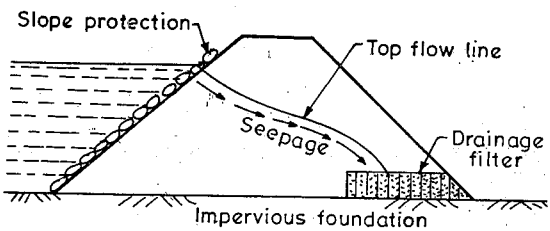


Fig. 20.1 (*b*). Homogeneous embankment provided with a drainage filter.

A purely homogeneous section poses the problems of seepage, and huge sections are required to make it safe against piping, stability, etc. Due to this, a homogeneous section is generally added with an internal drainage system : such as a horizontal drainage filter [Fig. 20.1 (*b*)], rock toe, etc. The internal drainage system keeps the phreatic line (*i.e.* top seepage line) well within the body of the dam, and steeper slopes and thus, smaller sections can be used. the internal drainage is, therefore, always provided in almost all types of embankments.

(2) **Zoned Embankment Type.** Zoned embankments are usually provided with a central impervious core, covered by a comparatively pervious transition zone, which is finally surrounded by a much more pervious outer zone (Fig. 20.2).

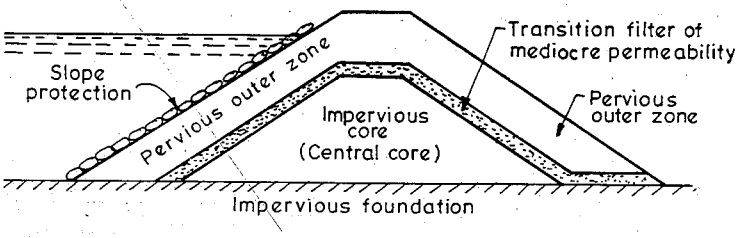


Fig. 20.2. Zoned type embankment.

The central core checks the seepage. The transition zone prevents piping through cracks which may develop in the core. The outer zone gives stability to the central impervious fill and also distribute the load over a large area of foundations.

This type of embankments are widely constructed and the materials of the zones are selected depending upon their availabilities. *Clay, inspite of it being highly impervious, may not make the best core, if it shrinks and swells too much. Due to this reason, clay is sometimes mixed with fine sand or fine gravel, so as to use it as the most suitable material for the central impervious core. Silts or silty clays may be used as the satisfactory central core materials. Freely draining materials, such as coarse sands and gravels, are used in the outer shell.* Transition filters are provided between the inner zone and the outer zone, as shown in Fig. 20.2. This type of transition filters are always provided, whenever there is an abrupt change of permeability from one zone to the other.

(3) **Diaphragm Type Embankments.** Diaphragm type embankments have a thin impervious core, which is surrounded by earth or rock fill. The impervious core, called diaphragm, is made of impervious soils, concrete, steel, timber or any other material. It acts as a water barrier to prevent seepage through the dam. The diaphragm may be placed either at the centre as a central vertical core or at the upstream face as a blanket. The diaphragm must also be tied to the bed rock or to a very impervious foundation material, if excessive under-seepage through the existing previous foundations has to be avoided (Fig. 20.3).

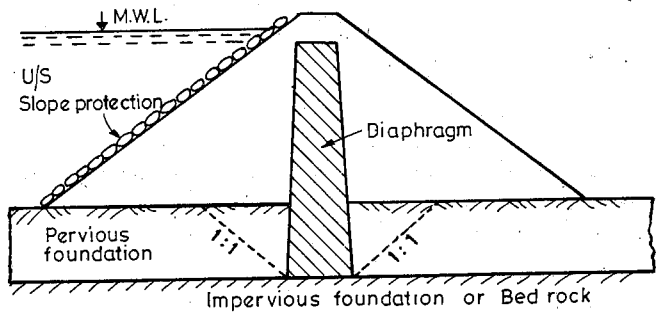


Fig. 20.3. Diaphragm type embankment.

The diaphragm type of embankments are differentiated from zoned embankments, depending upon the thickness of the core. If the thickness of the diaphragm at any elevation is less than 10 metres or less than the height of the embankment above the corresponding elevation, the dam embankment is considered to be of 'Diaphragm Type'. But if the thickness equals or exceeds these limits, it is considered to be of zoned embankment type.

20.3. Methods of Construction

There are two methods of constructing earthen dams :

(1) *Hydraulic-fill Method* ; and

(2) *Rolled-fill Method.*

(1) **Hydraulic-fill Method.** In this method of construction, the dam body is constructed by excavating and transporting soils by using water. Pipes called flumes, are laid along the outer edge of the embankment. The soil materials are mixed with water and pumped into these flumes. The slush is discharged through the outlets in the flumes at suitable intervals along their lengths. The slush, flowing towards the centre of the bank, tends to settle down. The coarser particles get deposited soon after the discharge near the outer edge, while the fines get carried and settle at the centre, forming a zoned embankment having a relatively impervious central core.

Since the fill is saturated when placed, high pore pressures develop in the core material, and the stability of the dam must be checked for these pressures. This type of embankment is susceptible to settlement over long periods, because of slow drainage from the core.

Hydraulic-fill method is, therefore, seldom adopted these days. Rolled-fill method for constructing earthen dams is, therefore, generally and universally adopted in these modern days.

(2) **Rolled-fill Method.** The embankment is constructed by placing suitable soil materials in thin layers (15 to 30 cm) and compacting them with rollers. The soil is brought to the site from burrow pits and spread by bulldozers, etc. in layers. These layers are thoroughly compacted by rollers of designed weights. Ordinary road rollers can be used for low embankments (such as for levees or bunds) ; while power-operated rollers are to be used for dams. The moisture content of the soil fill must be properly controlled. The best compaction can be obtained at a moisture content somewhere near the optimum moisture content. (The optimum moisture content is the moisture required for obtaining optimum density in the fill). Compaction of coarse gravels cannot be properly done by rolling and is best done by vibrating equipment. Detail of rolling and compactiing different types of soils are available in "Soil Mechanics and Foundation Engineering" by the same author.

20.4. Shearing Strength of Soils

Before we describe the causes of failures of earthen dams and the criteria for their safe design ; we shall review a few important conceptions of soil-mechanics and their importance in the design of earth dams.

Coulomb's Law. The resistance offered by a soil mass against the shearing forces, is known as the shear strength of the soil. In order to ensure that the foundation soil and the soil of the body of the dam are stable, we must ensure that the shearing strength of this soil mass is sufficiently greater than the shear stresses developed within the soil mass by external forces. According to Coulomb's Law, the shear strength of any soil mass is provided by two factors, *i.e.*

(*i*) Cohesion between the soil grains.

(*ii*) Internal friction between the soil particles.

Thus, the unit shear strength (τ_f) of a soil is given by

$$\tau_f = c + \sigma' \cdot \tan\phi \ (\text{kN/m}^2) \qquad ...(20.1)$$

where c = Unit cohesion of soil in (kN/m^2)

σ' = intergranular compressive stress, called the effective vertical stress (kN/m^2)

ϕ = Angle of internal friction of soil.

The constant c and ϕ depend upon the type of soil and the moisture content in it. Sands and gravels are cohesionless soils and $c \approx 0$ for them ; while ϕ is approximatrely zero for saturated cohesive impervious clays.

20.5. Various Kinds of Densities and Their Relations

Every soil has pores or voids in it, which remain filled either with air or with water or with both, depending upon the void ratio (e) and the moisture content (w) of the soil. The void ratio, porosity, moisture content, etc. of a soil are defined as follows :

$$\text{Void ratio } (e) = \frac{\text{Volume of voids in a soil mass}}{\text{Volume of solids in the soil mass}} = \frac{V_v}{V_s}$$

$$\text{Porosity of soil } (n) = \frac{\text{Volume of voids}}{\text{Total volume of soil}} = \frac{V_v}{V}$$

$$= \frac{V_v}{V_s + V_v} = \frac{\frac{V_v}{V_s}}{1 + \frac{V_v}{V_s}} = \frac{e}{1+e}$$

or $$e = \frac{1+e}{n}$$

$$\text{or Water content of soil } (w) = \frac{\text{Mass of moisture in soil mass}}{\text{Mass of solids in soil mass}}$$

$$= \frac{M_w}{M_s}$$

where M_w = Mass of water in the soil mass.

M_s = Mass of solids in the soil mass.

The unit weight of soil (γ) called bulk unit weight, is defined as the weight per unit volume of soil mass. If W is the total weight of soil, then

$$\gamma = \frac{W}{V} = \frac{W_s + W_w}{V_s + V_v} \text{ (air does not have any weight)}$$

or
$$\gamma = \frac{W_s\left(1+\frac{W_w}{W_s}\right)}{V_s\left(1+\frac{V_v}{V_s}\right)} \quad ...(A)$$

Now $\frac{W_s}{V_s} = \gamma_s = S_s \cdot \gamma_w$

where γ_s = unit weight of solids

γ_w = unit weight of water, and

S_s = specific gravity of solid soil particles $= \frac{\gamma_s}{\gamma_w}$

Substituting in (A), we get

$$\therefore \quad \gamma = \frac{W_s}{V_s}\left[\frac{1+\frac{W_w}{W_s}}{1+\frac{V_v}{V_s}}\right] = S_s \cdot \gamma_w \left[\frac{1+w}{1+e}\right]$$

Hence, $$\gamma = S_s \cdot \gamma_w \cdot \left[\frac{1+w}{1+e}\right] \quad ...(20.2)$$

If the soil mass is dry, $w = 0$

$$\therefore \quad \gamma_{dry} = \frac{S_s \cdot \gamma_w}{1+e} \quad ...(20.3)$$

$$\therefore \quad \gamma = \gamma_{dry}(1+w)$$

Degree of saturation (S) is defined as

$$S = \frac{V_w}{V_v} = \frac{\text{Volume of water}}{\text{Volume of voids}}$$

Now $$e = \frac{V_v}{V_s} = \frac{V_v}{V_s} \times \frac{V_w}{V_w} = \frac{V_v}{V_w} \times \frac{V_w}{V_v}$$

$$= \frac{1}{S}\left[\frac{\frac{M_w}{\gamma_w}}{\frac{M_s}{S_s \cdot \gamma_w}}\right] = \frac{1}{S} \cdot \frac{M_w \cdot S_s}{M_s} = \frac{1}{S} \cdot w \cdot S_s$$

$$\boxed{e = \frac{wS_s}{S}} \quad ...(20.4)$$

For saturated soils $S = 1$,

$$\therefore \quad e = w \cdot S_s$$

$$\therefore \quad \boxed{\text{Moisture content, } w = \frac{e}{S_s} \text{ for saturated soil}} \quad ...(20.5)$$

Now, if unit weight of saturated soil is designated as γ_{sat}, then

$$\gamma = \frac{S_s \gamma_w (1+w)}{(1+e)} \quad \text{from Eq. (20.2)}$$

$$\gamma_{sat} = \frac{S_s \cdot \gamma_w \left(1 + \frac{e}{S_s}\right)}{1+e} = \frac{S_s \cdot \gamma_w (S_s + e)}{S_s (1+e)}$$

or

$$\boxed{\gamma_{sat} = \frac{\gamma_w (S_s + e)}{(1+e)}} \quad ...(20.6)$$

Submerged unit weight of soil is defined as the unit weight of saturated soil minus the unit weight of water, and is designated as γ_{sub}.

$$\gamma_{sub} = \gamma_{sat} - \gamma_w = \frac{\gamma_w (S_s + e)}{1+e} - \gamma_w$$

$$\gamma_{sub} = \gamma_w \left[\frac{S_s - 1}{1+e}\right] \quad ...(20.7)$$

20.6. Pore-Water Pressure and its Significance in the Design of Earth Dams

We know that every soil has some voids or pores which are partly or fully filled with water. Let us consider a soil mass below the water-table *BB* in Fig. 20.4.

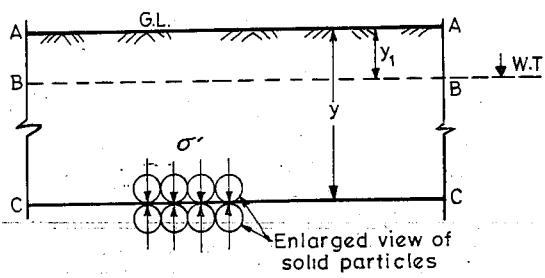


Fig. 20.4

The soil below *BB* is fully saturated and all its pores between the solid particles of soil, are full of water. The soil below the water-table is, therefore, subjected to hydrostatic uplift. Hence, at any level (say *CC*), the total downward normal pressure (σ) exerted by the weight of the soil above this level, shall be supported partly by the intergranular pressure (σ′ or σ_{eff}) developed between dry particles of the soil, and partly by the hydrostatic pressure (u) due to the water present in the pores. Hence

$$\sigma = \sigma' + u \quad ...(20.8)$$

where, σ is the total normal pressure on soil.

σ′ = Total effective pressure, *i.e.* the intergranular pressure, or the pressure which is transmitted from grain to grain of soil.

u = Pore water pressure or *neutral pressure*. (It is the hydrostatic pressure due to presence of water in the soil pores.)

In Fig. 20.4, if the pressures are considered at a level say *CC* at a depth y from the ground, we can easily write,

Total normal pressure (σ) at level *CC*

$$= \gamma_{dry}\, y_1 + \gamma_{sat}\,(y - y_1)$$

where y_1 is the depth of watertable below the ground level.

$$\therefore \quad \sigma = \gamma_{dry}\, y_1 + \gamma_{sat.}\,(y - y_1)$$

Pore water pressure $= u = \gamma_w\,(y - y_1)$

Hence, the net effective pressure (σ') is given by

$$\sigma' = \sigma - u = \gamma_{dry}\, y_1 + \gamma_{sat}\,(y - y_1) - \gamma_w\,(y - y_1)$$
$$= \gamma_{dry}\, y_1 + \gamma_{sub}\,(y - y_1)$$

It, therefore, becomes evident that the effective normal stress is much less than the total normal stress, as a part of the total stress gets consumed by water as pore pressure. The effective stress is dependent on the submerged unit weight of soil.

The water pressure or pore pressure acts equally in all directions. It does not press the soil grains against one another, and, therefore, does not lead to compression of the soil or an increase in its frictional resistance, that is why, it is called a 'neutral stress'.

When the pore pressure is considered, the Coulomb's law will become

Unit Shear Strength of soil $(\tau_f) = c + \sigma' \cdot \tan\phi$

or

$$\boxed{\tau_f = c + (\sigma - u)\tan\phi} \qquad ...(20.9)$$

This clearly indicates that the shearing strength of a soil gets reduced due to the presence of pore pressure. The pore water pressure gets developed in the body of the earthen dam when the seepage takes place through the body of the dam, thus reducing the shear strength of the soil.

The shear stress (τ) developed at any plane in an earth structure is given by

$$\tau = \frac{\sigma_1 - \sigma_2}{2} \cdot \sin 2\theta \qquad ...(20.10)$$

where $\sigma_1 =$ major principal stress

$\sigma_2 =$ minor principal stress

$\theta =$ is the angle between the plane considered for shear and the plane on which σ_1 acts.

It is apparent from the Equation (20.10), that the value of shearing stress remains unaltered, whether σ_1 and σ_2 are used or their effective components $[(\sigma_1 - u)$ and $(\sigma_2 - u)]$ are used. But the shear strength of the soil gets reduced when effective component $(\sigma - u)$ is used in place of σ.

Hence, the stability of the dam against shear failure must be checked when the maximum pore water pressure is present

Consolidation. The development of pore pressure is important, even during the time of construction of an earth dam. When the fully or partly saturated soil is placed and rolled in the body of the dam, the entire applied external load is taken up by the water immediately, and transferred to the soil afterwards. The pore water thus gets compressed, and if it is unable to drain out freely (due to low permeability of the soil), the pore pressure rises. *This rise in pore pressure during compaction is known as* **'hydrostatic excess' pressure in pore water.** It further reduces the shearing strength of the soil and hence, the stability of the soil. As the excess water drains out, more and more consolidation will take place, as this pressure will be transferred to soil grains and the

shear strength will tend to achieve its normal value. *Hence, the pore pressure temporarily reduces the shear strength of the soil during compaction* by preventing full compaction. But the shear strength gets recovered after the compaction is over, as the pore water is ultimately squeezed out.

In highly compressible soils, having low coefficient of permeability and moisture content above its optimum moisture content, this condition of 'hydrostatic excess' becomes very serious. Such soils are, therefore, more liable to fail during construction.

Similarly, appreciable consolidation of soil may takes place in fine grained compressible soils like clay, even after the construction is over, though sufficient compaction was done during construction.

Due to these reasons, pore pressure observations are often made during the construction period of an earth dam. If the hydrostatic excess of pore pressure rises to a dangerous amount, the construction may be stopped for some time till the excess water drains out and full natural compaction takes place. The construction may be restarted after this 'excess' is either fully dissipated or reduced to a safe value. The shear failure of the soil of the dam or its foundation is, therefore, very much connected with the development of pore pressure in the body of the dam and in the foundation, and must be properly checked and accounted for.

20.7. Causes of Failure of Earthen Dams

Earth dams are less rigid and hence more susceptible to failure. Every past failure of such a dam has contributed to an increase in the knowledge of the earth dam designers. Earthen dams may fail, like other engineering structures, due to improper designs, faulty constructions, lack of maintenance, etc. The various causes leading to the failure of earth dams can be grouped into the following three classes :

(1) Hydraulic failures
(2) Seepage failures
(3) Structural failures.

These causes are described below in details :

20.7.1. Hydraulic failures. About 40% of earth dam failures have been attributed to these causes. The failure under this category, may occur due to the following reasons:

(*a*) *By over topping.* The water may overtop the dam, if the design flood is underestimated or if the spillway is of insufficient capacity or if the spillway gates are not properly operated. Sufficient freeboard should, therefore, be provided as an additional satety measure.

(*b*) *Erosion of upstream face.* The waves developed near the top water surface due to the winds, try to notch-out the soil from the upstream face and may even, sometimes, cause the slip of the upstream slope. Upstream stone pitching or riprap should, therefore, be provided to avoid such failures.

(*c*) *Cracking due to frost action.* Frost in the upper portion of the dam may cause heaving and cracking of the soil with dangerous seepage and consequent failure. An additional freeboard allowance upto a maximum of say 1.5 m should, therefore, be provided for dams in areas of low temperatures.

(*d*) *Erosion of downstream face by gully formation.* Heavy rains falling directly over the downstream face and the erosive action of the moving water, may lead to the formation of gullies on the downstream face, ultimately leading to the dam failure. This can be avoided by proper maintenance, filling the cuts from time to time especially

during rainy season, by grassing the slopes and by providing proper berms at suitable heights (Fig. 20.5), so that the water has not to flow for considerable distances. The proper drainage arrangements are made for the removal of the rain water collected on the horizontal berms. Since the provision of berms ensures the collection and removal of water before it acquires high downward velocities, the consequent erosion caused by the moving water (run off) is considerably reduced.

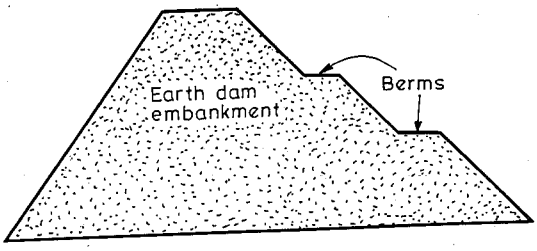


Fig. 20.5

(*e*) *Erosion of the d/s toe.* The d/s toe of the earth dam may get eroded due to two reasons, *i.e.* (*i*) the erosion due to cross currents that may come from the spillway buckets; and (*ii*) the erosion due to tail water. This erosion of the toe can be avoided by providing a downstream slope pitching or a riprap up to a height slightly above the normal tail water depth. Side walls of the spillway (called diaphragm walls) must be of sufficient height and length, as so to prevent the possibility of the cross flow towards the earthen embankment.

20.7.2. Seepage Failures. Controlled seepage or limited uniform seepage is inevitable in all earth dams, and ordinarily it does not produce any harm. However, uncontrolled or concentrated seepage through the dam body or through its foundation may lead to piping or sloughing and the subsequent failure of the dam. Piping is the progressive erosion and subsequent removal of the soil grains from within the body of the dam or the foundation of the dam. Sloughing is the progressive removal of soil from the wet downstream face. More than 1/3rd of the earth dams have failed because of these reasons.

(*a*) *Piping through foundations.* Sometimes, when highly permeable cavities or fissures or strata of coarse sand or gravel are present in the foundation of the dam, water may start seeping at a huge rate through them (Fig. 20.6). This concentrated flow at a high gradient, may erode the soil. This leads to increased flow of water and soil, ultimately resulting in a rush of water and soil, thereby creating hollows below the foundation. The dam may sink down into the hollow so formed, causing its failure.

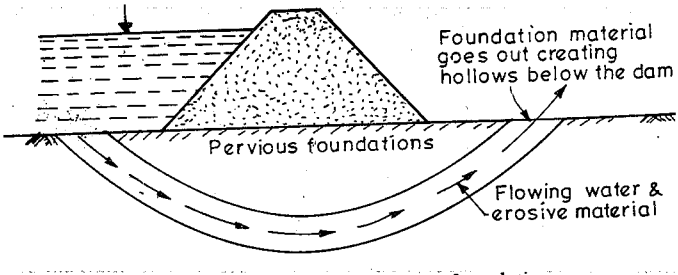


Fig. 20.6. Piping through the dam foundation.

(*b*) *Piping through the dam body.* When the concentrated flow channels get developed in the body of the dam, (Fig. 20.7) soil may be removed in the same manner as was explained in foundation piping, leading to the formation of hollows in the dam body, and subsequent subsidence of the dam. These flow channels may develop due to faulty construction, insufficient compaction, cracks developed in embankment due to

foundation settlement, shrinkage cracks, animal burrows, etc. All these causes can be removed by better construction and better maintenance of the dam embankments.

Piping through the dam body, generally get developed near the pipe conduits passing through the dam body. Contact seepage along the outer side of conduits may either develop into piping, or seepage through leaks in the conduits may develop into piping. This can be avoided by thoroughly and properly compacting the soils near the outlet conduits and by preventing the possibilities of leakage through conduits, but preventing the formation of cracks in the conduits. These cracks in the conduits are caused by differential settlement and by overloading from the embankment. When these factors are controlled, automatically, the possibility of piping due to leakage through the conduits is reduced.

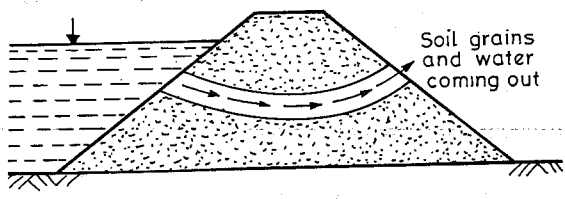


Fig. 20.7. Piping through the dam body.

(*c*) *Sloughing of D/S Toe.* The process behind the sloughing of the toe is somewhat similar to that of piping. The process of failure due to sloughing starts when the downstream toe becomes saturated and get eroded, producing a small slump or a miniature slide. The miniature slide leaves a relatively steep face which becomes saturated by the seepage from the reservoir and slumps again, forming a more unstable surface. The process continues till the remaining portion of the dam is too thin to withstand the horizontal water pressure, leading to the sudden failure of the dam.

20.7.3. Structural failures. About 25% of the dam failures have been attributed to structural failures. Structural failures are generally caused by shear failures, causing slides.

(*a*) *Foundation slide.* (*i.e.* overall stability of the dam). When the foundation of the earth dams are made of soft soils, such as fine silt, soft clay, etc., the entire dam may slide over the foundation. Sometimes, seams of fissured rocks, shales or soft clay, etc. may exits under the foundation, and the dam may slide over some of them, causing its failure. In this type of failure, the top of embankment gets cracked and subsides, the lower slope moves outward forming large mud waves near the heel, as shown in Fig. 20.8.

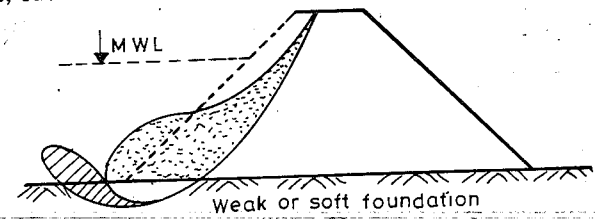


Fig. 20.8. Sliding due to soft or weak foundation.

Excessive pore water pressure in confined seams of sand and silt, artesian pressure in abutments, or hydrostatic excess developed due to consideration of clay seams embedded between sands or silts, etc. may reduce the shear strength of the soil, until it becomes incapable of resisting the induced shear stresses, leading to the failure of the dam foundation whithout warning. Loose sand foundations may fail by the liquefaction or flow slides.

(*b*) *Slide in Embankments.* When the embankment slopes are too steep for the strength of the soil, they may slide causing dam failure.

The most critical condition of the slide of the u/s slope is the sudden draw-down of the reservoir (Fig. 20.9); and the d/s slope is most likely to slide, when the reservoir is full (Fig. 20.10). The u/s slope failures seldom lead to catastrophic failures, but the d/s slope failures are very serious. These failures, generally occur due to development of excessive unaccounted pore pressures which may reduce the shearing strength of the soils as explained in the previous article. Many embankments may fail during the process of consolidation, at the time of construction or after the construction.

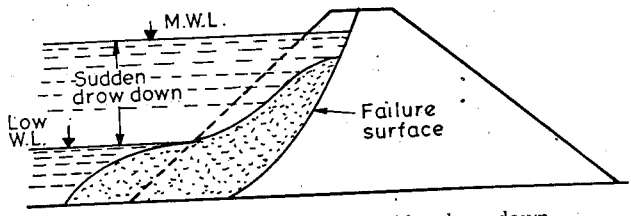


Fig. 20.9. U/S slope slide due to sudden draw-down.

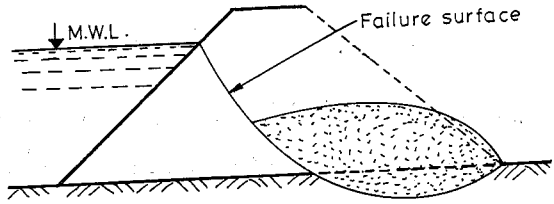


Fig. 20.10. D/S. slope slide during full reservoir condition.

20.8. Design Criteria for Earth Dams

(1) A fill of sufficiently low permeability should be developed out of the available materials, so as to best serve the intended purpose with minimum cost. Borrow pits should be as close to the dam site as possible, so as to reduce the leads.

(2) Sufficient spillway and outlets capacities should be provided so as to avoid the possibility of overtopping during design flood.

(3) Sufficient freeboard must be provided for wind set-up, wave action, frost action and earthquake motions.

(4) The seepage line (*i.e.* phreatic line) should remain well within the downstream face of the dam, so that no sloughing of the face occurs.

(5) There is little harm in seepage through a flood control dam. if the stability of foundations and embankments is not impaired, by piping, sloughing, etc. : but a conservation dam must be as watertight as possible.

(6) There should be no possibility of free flow of water from the upstream to the downstream face.

(7) The upstream face should be properly protected against wave action, and the downstream face against rains and against waves upto tail water. Provisions of horizontal berms at suitable intervals in the d/s face may be thought of, so as to reduce the erosion due to flow of rain water. Ripraps should be provided on the entire u/s slope and also on the d/s slope near the toe and up to slightly above the tail water so as to avoid erosion.

(8) The portion of the dam, downstream of the impervious core, should be properly drained by providing suitable horizontal filter drain, or toe drain, or chimney drain, etc.

(9) The upstream and downstream slopes should be so designed as to be stable under worst conditions of loading. These critical conditions occur for the u/s slope during sudden drawdown of the reservoir, and for the d/s slope during steady seepage under full reservoir.

(10) The u/s and d/s slope should be flat enough, as to provide sufficient base width at the foundation level, such that the maximum shear stress developed remains well below the corresponding maximum shear strength of the soil, so as to provide a suitable factor of safety.

(11) We know that the consolidation of the soil does not take place instantaneously when the compaction is done by external leadings. It takes place slowly as the excess pore water goes out and the load is transferred to the soil grains. In coarse gravels, the void openings are large enough so as to permit rapid escape of confined water and air, and full compaction may occur before the construction is over. But in fine grained impervious soils, the consolidation is slow. It, therefore, becomes necessary in such cases, as to provide an additional height of the fill. After consolidation, the the embankment will be of the desired height. Hence, a suitable allowance in the height of embankment (between 2 to 3% of dam height, determined by laboratory tests) must be made in fine grained soils so as to account for the consolidation that may take place upto years after construction. Dewatering the foundations may sometimes be used to accelerate the process of consolidation.

(12) Since the stability of the embankment and foundation is very critical during construction or even after construction (*i.e.* during the period of consolidation), due to development of excessive pore pressures and consequent reduction in shear strength of soil, the embankment slopes must remain safe under this critical condition also.

All the above criteria must be satisfied and accounted for, in order to obtain a safe design and construction of an earth dam.

20.9. Selecting a Suitable Preliminary Section for an Earth Dam

The preliminary design of an earth dam is done on the basis of existing dams of similar characteristics and the design is finalised by checking the adequacy of the selected section for the worst loading conditions. Empirical rules are frequently used in these designs.

A few recommendations, for selecting suitable values of top width, free board, u/s and d/s slopes, drainage arrangements, etc. are given below for preliminary designs :

Freeboard. Freeboard or minimum freeboard is the vertical distance between the maximum reservoir level and the top of the dam (*i.e.* the crown or crest of dam). The vertical distance between normal pool level or spillway crest and the top of the dam is termed as normal freeboard.

The minimum height of the freeboard for wave action is generally taken to be equal to 1.5 h_w, where h_w is given by the equations (19.11) and (19.12). Most of the hydraulic failures of earth dams have occurred due to overtopping of dams. Hence, the freeboard must be sufficient enough, as to avoid any such possibility of overtopping. Values of freeboard, for various heights, recommended by U.S.B.R. are given in table 20.1.

Table 20.1. U.S.B.R. Recommendations for Freeboard for Earth Dams

Spillway Type	*Height of Dam*	*Minimum freeboard Over MWL*
Uncontrolled (*i.e.* Free) Spillway	Any height	Between 2 m to 3 m
Controlled spillway	Height less than 60 m	2.5 m above top of gates
Controlled spillway	Height more than 60 m	3 m above top of gates

An additional freeboard upto 1.5 m should be provided for dams situated in areas of low temperatures for frost action.

Width. The top width of large earthen dams should be sufficient to keep the seepage line well within the dam, when reservoir is full. It should also be sufficient to withstand earthquake shocks and wave action. For small dams, this top width is generally governed by minimum roadway width requirements.

The top width (A) of the earth dam can be selected as per the following recommendations :

$$A = \frac{H}{5} + 3 \text{ for very low dams} \quad ...(20.11)$$

$$A = 0.55\sqrt{H} + 0.2\,H \text{ for dams lower than 30 m} \quad ...(20.12)$$

$$A = 1.65\,(H + 1.5)^{1/3} \text{ for dams higher than 30 m} \quad ...(20.13)$$

where H is the height of the dam.

Upstream and Downstream slopes. The side slopes depend upon various factors such as the type and nature of the dam, and foundation materials, height of dam, etc. etc. The recommended values of side slopes as given by Terzaghi are tabulated in Table 20.2.

Table 20.2. Terzaghi's Side Slopes for Earth Dams

Type of Material	*U/S slope (H : V)*	*D/S slope (H : V)*
Homogeneous well graded	2.5 : 1	2 : 1
Homogeneous course silt	3 : 1	2.5 : 1
Homogeneous silty clay		
(*i*) Height less than 15 m	2.5 : 1	2 : 1
(*ii*) Height more than 15 m	3 : 1	2.5 : 1
Sand or Sand and gravel with a central clay core	3 : 1	2.5 : 1
Sand or Sand and gravel with R.C. diaphragm	2.5 : 1	2 : 1

The various dimensions of low earth dams for their preliminary sections, may sometimes be selected from the recommendations of Strange, as given in Table 20.3.

Table 20.3. Preliminary Dimensions of Low Earth Dams (Strange's recommendations)

Height of dam in metres	*Maximum freeboard in metres*	*Top width (A) in metres*	*U/S slope (H : V)*	*D/S slope (H : V)*
Up to 4.5	1.2 to 1.5	1.85	2 : 1	1.5 : 1
4.5 to 7.5	1.5 to 1.8	1.85	2.5 : 1	1.75 : 1
7.5 to 15	1.85	2.5	3 : 1	2 : 1
15 to 22.5	2.1	3.0	3 : 1	2 : 1

SEEPAGE ANALYSIS

Seepage occurs through the body of all earthen dams and also through their pervious foundations. The amount of seepage has to be controlled in all conservation dams and the effects of seepage (*i.e.* position of phreatic line) has to be controlled for all dams, in order to avoid their failures.

The seepage through a pervious soil material, for two dimensional flow, is given by Laplacian equation

$$\frac{\partial^2 \phi}{\partial x^2} + \frac{\partial^2 \phi}{\partial y^2} = 0 \qquad \text{...(20.14)}$$

where $\phi = K \cdot h$ = Velocity potential

K = Permeability of the soil

h = Head causing flow.

The above equation is based on the following assumptions :

(*i*) Water is incompressible.

(*ii*) The soil is incompressible and porous. The size of the pore space do not change with time regardless of water pressure.

(*iii*) The quantity of water entering the soil in any given time is the same as the quantity flowing out of the soil.

(*iv*) Darcy's law is valid for the given soils.

(*v*) The hydraulic boundary conditions at the entry and exit are known.

A graphical solution of the above equation, (*i.e.* Eq. 20.11) suggests that the flow through the soil, following the above assumptions, can be represented by a *flow-net* ; which consists of two sets of curves, known as '*Equipotential lines*' (*i.e.* lines of equal energy) and '*stream lines*' (*i.e.* flow lines), mutually perpendicular to each other, as shown in Fig. 20.11.

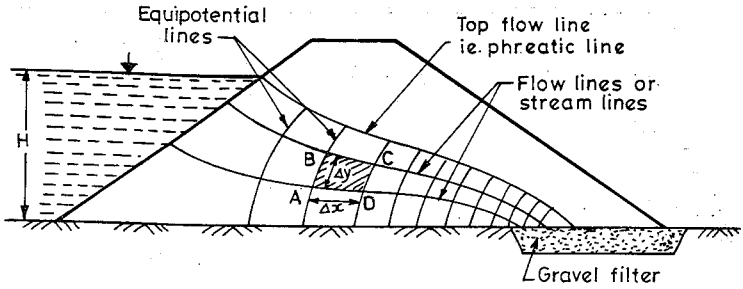


Fig. 20.11. Flow Net.

20.10. Seepage Discharge Through the Isotropic Soils

The amount of seepage can be easily computed from the flownet. Let us assume that the soil is isotropic, *i.e.* its permeability is constant in all directions, or $K_H = K_V$. (*i.e.* horizontal permeability is equal to the vertical permeability). The dam section is first of all plotted to a given scale (Same scale for horizontal and vertical directions). The flow net is drawn by free hand sketching the making suitable adjustments and corrections until the flow lines and equipotential lines intersect at right angles. It is convenient to draw only a limited number of flow lines and equipotential lines, such

that the rate of flow between each pair of flow lines (called flow channels) is equal and the energy drop between any two successive potential lines is the same. The distance between the flow lines is made equal to the distance between the potential lines, thus forming a series of squares. Where the flow lines are curved, the squares formed will be distorted, but they will be more perfect as the number of lines is increased.

The seepage rate (q) can be computed from the flow net, using Darcy's Law. Applying the principle of continuity between each pair of flow lines, it is evident that the velocity must vary inversely with the spacing. Assuming the dam cross-section of Fig. 20.11 to have a unit width, we have :

The flow through the square $ABCD$ (called field) or through the flow channel containing this square

$$= \Delta q = K \cdot iA$$

$$= K\left(\frac{\Delta H}{\Delta x}\right)(\Delta y \times 1)$$

where ΔH is the energy drop between the two equi-potential lines bounding the square $ABCD$ and Δx and Δy are defined in Fig. 20.11.

$$\therefore \qquad \Delta q = K\left(\frac{\Delta H}{\Delta x}\right)\Delta y$$

But $$\Delta H = \frac{\text{Total drop, } i.e. \text{ total head causing flow}}{\text{Number of increments into which the total drop is equally divided}}$$

or $$\Delta H = \frac{H}{N_d}$$

where $N_d =$ Total number of drops in the complete flow-net.

$$\Delta q = K\frac{H}{N_d}\left(\frac{\Delta y}{\Delta x}\right)$$

$$= \frac{K \cdot H}{N_d} \text{ (since } \Delta y = \Delta x)$$

The total flow through all the channels, *i.e.* the total flow through the unit width of the dam

$$= q = \Sigma \cdot \Delta q$$

$$= \left(\frac{K \cdot H}{N_d}\right) \times \text{number of flow channels}$$

or $$\boxed{q = \frac{K \cdot H}{N_d} N_f} \qquad \text{...(20.15)}$$

This is the required expression, representing discharge passing through a flow net and is applicable only to isotropic soils (*i.e.* soils for which $K_H = K_V$).

20.11. Seepage Discharge for Non-isotropic Soils

If the permeability of the soil is different in the horizontal direction than that in the vertical direction ; the flow net is drawn in the same manner as was explained earlier

for isotropic soils, with the only difference that the dam section shall be drawn to the same vertical scale *but to a transformed horizontal scale.* All horizontal dimensions shall be reduced by multiplying them by a factor equal to $\sqrt{\frac{K_V}{K_H}}$. Flow net and squares will be drawn in the same manner, and number of flow channesl (N_f) and number of drops (N_d) shall be counted. The discharge can then be computed by the equation

$$q = \sqrt{K_H \cdot K_V}\ \frac{H \cdot N_f}{N_d} \qquad ...(20.16)$$

20.12. Line of Seepage or Phreatic Line in Earth Dams

Line of seepage or phreatic line or saturation line is defined as the line within the dam section below which there are positive hydrostatic pressures in the dam. The hydrostatic pressure on the phreatic line is equal to the atmospheric pressure and hence, equal to zero. Above the phreatic line, there is a zone of capillary saturation called capillary fringe, in which the hydrostatic pressures are negative. The appreciable flow through the dam body below the phreatic line, reduces the effective weight of this soil, and thus reduces the shear strength of the soil due to pore pressure. But on the other hand, the insignificant flow through the capillary fringe, leads to greater shear strength, because the capillary tension in water leads to increased intergranular pressure. The effects of the capillary fringe are thus on a slightly safer side and hence neglected.

It is, therefore, absolutely essential to determine the position of the phreatic line, as its position will enable us to determine the following things :

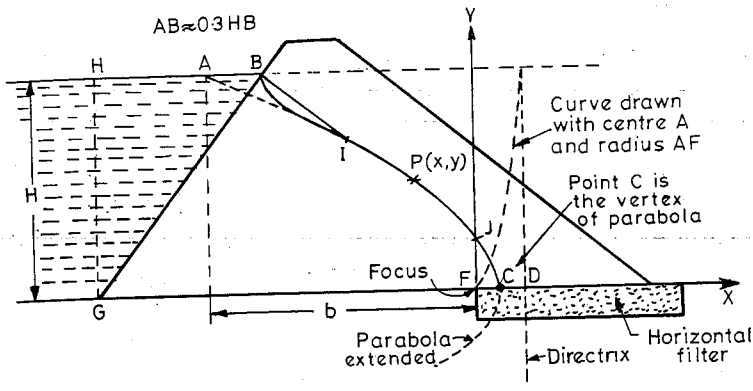
(*i*) It gives us a divide line between the dry (or moist) and submerged soil. The soil above the seepage line will be taken as dry and the soil below the seepage line shall be taken as submerged for computation of shear strength of soil.

(*ii*) It represents the top streamline and hence, helps us in drawing the flow net.

(*iii*) The seepage line determination, helps us to ensure that it does not cut the downstream face of the dam. This is extremely necessary for preventing softening or sloughing of the dam.

20.12.1. Determination of Phreatic Line when the Dam section is Homogeneous and Provided with a Horizontal Filter. It has been found by experiments that the seepage line is pushed down by the filter and it is very nearly parabolic except near its junction with the u/s face. Since the u/s face of the dam (*i.e. GB* in Fig. 20.12) becomes an equipotential line when fully covered with water (water shall rise up to the same piezometric head at every point of this line), the seepage line shall be perpendicular to this face near its junction point *B*. Let a base parabola with focus at *F* is drawn and produced so as to intersect the water surface at a point *A*. Cassagrande has shown that for dams with reasonably flat upstream slopes, $AB \approx 0.3\,HB$, where *H* is the projection of the point *G* (*i.e.* the end point of u/s slope) on the water surface. Knowing the point *A*, the base parabola *AIJC* can be drawn with its focus at *F*. It can then be corrected for the curve *BI* such that *BI* is perpendicular to *GB*, thus, *BIJC* will finally represent the seepage line.

Equation of the base parabola. The equation of the parabola can be determined from the basic property of the parabola *viz., the distance of any point P (x, y) on the parabola from its focus is the same as the distance of that point P (x, y) from a line*



Point C is the vertex of the parabola
Fig. 20.12

called directrix. Taking the focus (F) as the origin, the equation of the parabola can be written as

$$\sqrt{x^2+y^2} = x + FD$$

where the vertical line through D is the directrix.

ED is the distance of the focus from the directrix, called focal distance and is generally represented by S.

The equation of the parabola, then becomes

$$\sqrt{x^2+y^2} = x + S \qquad ...(20.17)$$

If the horizontal distance between the already determined point A and the focus (F) is taken as say b, then (b, H) represents the coordinates of the point A on the parabola. Substituting in equation (20.17), we get

$$\sqrt{b^2+H^2} = b + S$$

or

$$S = \sqrt{b^2+H^2} - b \qquad ...(20.18)$$

S can be calculated from this equation.

The focal distance S can also be measured by drawing an arc FH with centre A and radius equal to AF so as to intersect the Horizontal water surface AB produced at H. The vertical line HD through H will then represent the directrix. FD will be equal to S. The centre point (C) of FD will be the vertex of the parabola.

Also from equation (20.17) : when $x = 0$, $y_{(0)} = S$. Hence, the vertical ordinate FJ at F will be equal to S. Hence, S is many a times, represented by $y_{(0)}$. Knowing the points A, C, J and working out a few more points from the equation (20.17), the parabola can be easily drawn and corrected for the curve BI, so as to get the seepage line $BIJC$.

The discharge can also be calculated easily from the equation of the seepage line, without taking the trouble of drawing a flow net, as explained below :

Consider a unit width of the dam. Let q be the seepage discharge per unit width of the dam. Then, according to Darcy's Law, $q = KiA$. When steady conditions have

reached, the discharge crossing any vertical plane across the dam section will be the same. Hence, the values of i and A can be taken for any point on the seepage line

$$\therefore \qquad i = \frac{dy}{dx}$$

$$A = y \times 1 \text{ (i.e. saturated depth} \times \text{width)}$$

$$\therefore \qquad q = K\frac{dy}{dx} \cdot y \qquad \text{...(20.19)}$$

But the equation of the parabola is

$$\sqrt{x^2 + y^2} = x + S \qquad \text{...(20.17)}$$

or $$x^2 + y^2 = (x + S)^2$$

or $$y^2 = (x + S)^2 - x^2$$

or $$y = \sqrt{S^2 + 2xS.}$$

Equation (20.19) becomes

$$q = K\left[\frac{1}{2} \cdot (S^2 + 2xS)^{\frac{1}{2}-1} \cdot 2S\right] \cdot \left[\sqrt{S^2 + 2xS}\right]$$

$$= K \cdot S \cdot (S^2 + 2xS)^{-\frac{1}{2}} \sqrt{S^2 + 2xS}$$

$$= K \cdot S \cdot \frac{\sqrt{S^2 + 2xS}}{\sqrt{S^2 + 2xS}} = K \cdot S.$$

or $$\boxed{q = KS} \qquad \text{...(20.20)}$$

The coefficient of permeability K and the focal distance (S) are known ; the discharge q can be easily computed. This is an important equation. Strictly speaking, this equation is applicable only to dams with horizontal drainage but is used for other type of section also and gives quite close values of discharge.

20.12.2. Determination of Phreatic Line when the Dam Section is Homogeneous (without Filter). The phreatic line can be determined on the same principles as was done for dam with a filter case. The focus (F) of the parabola, in this case, will be the lowest point of the downstream slope as shown in Fig. 20.13. The base parabola $BIJC$ will cut the downstream slope at J and extend beyond the dam toe up to the point C (*i.e.* the vertex of the parabola).

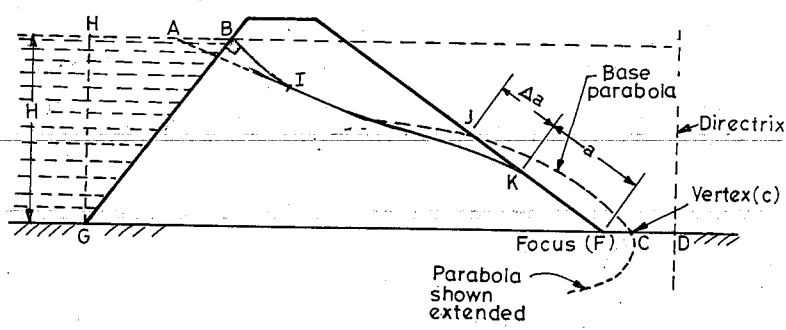


Fig. 20.13

The seepage line will, however, emerge out at K, meeting the downstream face tangentially there. *The portion KF is known as discharge face and always remains saturated.* The correction JK (say Δa) by which the parabola is to be shifted downward can be determined as follows :

(A) Graphical general solution. Cassagrande has given a general solution to evaluate Δa for various inclinations of discharge face. Let a be the angle which the discharge face makes with the horizontal. The various values of $\frac{\Delta a}{a+\Delta a}$ have been given by Cassagrande, as shown in table 20.4.

Table 20.4

α *in degrees*	$\frac{\Delta a}{a+\Delta a}$	*Remarks*
30°	0.36	*Note.* Intermediate values can be interpolated, or read out from a graph between α and $\frac{\Delta a}{a+\Delta a}$ plotted with the values given here.
60°	0.32	
90°	0.26	
120°	0.18	
135°	0.14	
150°	0.10	
180°	0.0	

$(a+\Delta a)$ is the distance FJ (*i.e.* the distance of the focus from the point where the parabola cuts the d/s face) and is known. Δa can then be evaluated. a and Δa can be connected by a general equation.

$$\Delta a = (a+\Delta a)\left[\frac{180^\circ - \alpha}{400^\circ}\right] \qquad ...(20.21)$$

The value of α will be equal to 180° for a horizontal filter case and may be equal to or more than 90° in case a rock toe is provided at the downstream end, as shown in Fig. 20.14 (*a*). α will be less then 90° when no drainage is provided.

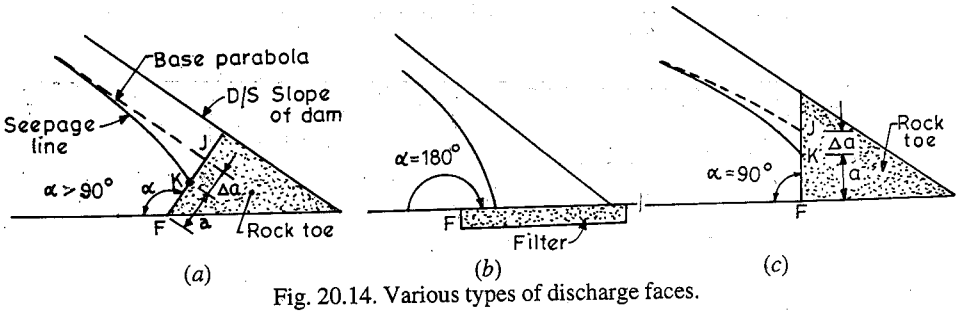


Fig. 20.14. Various types of discharge faces.

(B) Analytical solutions for determining the position of point k, *i.e.* the point at which the seepage line intersects the d/s slope.

Case (*a*) when $\alpha < 30°$

Schaffernak and Van Iterson have derived an equation for determining the value of 'a' (and thus fixing the position of point K) in terms of H, b' and α. Their final equation is

$$a = \frac{b'}{\cos\alpha} - \sqrt{\frac{b'^2}{\cos^2\alpha} - \frac{H^2}{\sin^2\alpha}} \qquad ...(20.22)$$

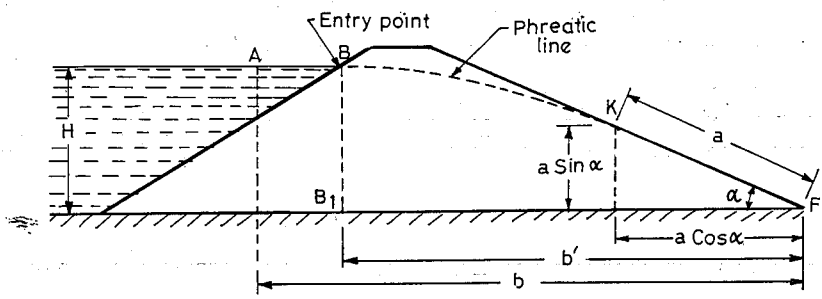


Fig. 20.15

The above equation has been obtained on the assumption that the hydraulic gradient (i) is equal to the slope of the phreatic line. This assumption is nearly true so long as the downstream slope is sufficiently flat (*i.e.* $\alpha < 30°$).

This equation can be easily derived on the basis of the above assumption as follows:

$$i = \text{Hydraulic gradient} = \frac{dy}{dx}$$

$$A = y \cdot 1$$

$$q = KiA = K \cdot \left(\frac{dy}{dx}\right) \cdot (y \cdot 1)$$

$$\text{But } \frac{dy}{dx} = \tan\alpha$$

$$\text{and} \quad y = a \sin\alpha$$

$$\therefore \quad q = K \cdot \tan\alpha \cdot a \cdot \sin\alpha$$

$$= K \cdot a \cdot \sin\alpha \cdot \tan\alpha$$

or $$K \cdot \frac{dy}{dx}(y) = K \cdot a \cdot \sin\alpha \tan a$$

or $$\frac{dy}{dx}(y) = a \sin\alpha \tan\alpha$$

or $$dy \cdot y = a \sin\alpha \tan\alpha \, dx$$

or $$y \cdot dy = a \sin\alpha \tan\alpha \, dx.$$

Integrating both sides between the limits

$$x = a\cos\alpha \quad \text{to} \quad x = b'$$

and $$y = a\sin\alpha \quad \text{to} \quad y = H,$$

we get $$\int_{y=a\sin\alpha}^{y=H} y \cdot dy = a\sin\alpha \cdot \tan\alpha \int_{x=a\cos\alpha}^{x=b'} dx$$

or $$\left|\frac{y^2}{2}\right|_{a\sin\alpha}^{H} = a\sin\alpha\tan\alpha \left| x \right|_{a\cos\alpha}^{b'}$$

or $$\frac{H^2 - a^2\sin^2\alpha}{2} = a\sin\alpha\tan\alpha\left[b' - a\cos\alpha\right]$$

or $$\frac{H^2}{2} - \frac{a^2}{2}\sin^2\alpha = ab'\sin\alpha\tan\alpha - a^2 \cdot \sin\alpha\cos\alpha \cdot \tan\alpha$$

or $$\frac{H^2}{2} - \frac{a^2}{2}\sin^2\alpha = ab'\sin\alpha\tan\alpha - a^2\cdot\sin^2\alpha$$

or $$\frac{H^2}{2} = ab'\sin\alpha\tan\alpha - \frac{a^2}{2}\sin^2\alpha$$

or $$\frac{a^2}{2}\cdot\sin^2\alpha - ab'\sin\alpha\tan\alpha + \frac{H^2}{2} = 0$$

or $$a^2\sin^2\alpha - 2ab'\sin\alpha\tan\alpha + H^2 = 0$$

or $$a^2 - \frac{2ab'}{\cos\alpha} + \frac{H^2}{\sin^2\alpha} = 0$$

or $$a = \frac{\dfrac{2b'}{\cos\alpha} \pm \sqrt{\left(\dfrac{2b'}{\cos\alpha}\right)^2 - \dfrac{4H^2}{\sin^2\alpha}}}{2}$$

Ignoring unfeasible +ve sign, we have

$$\boxed{a = \frac{b'}{\cos\alpha} - \sqrt{\frac{b^2}{\cos^2\alpha} - \frac{H^2}{\sin^2\alpha}}} \qquad ...(20.22)$$

This is the required equation.

Case (*b*). When α lies between 30° and 60°

Cassagrande has derived an equation for determining the value of '*a*' in terms of *b*, *H* and α. His final equation is

$$a = \sqrt{b^2 + H^2} - \sqrt{b^2 - H^2\cot^2\alpha} \qquad ...(20.23)$$

where *b* is defined in Fig. 20.16,
H is the head causing flow and α is the angle which the d/s face makes with the horizontal (clockwise) as defined earlier.

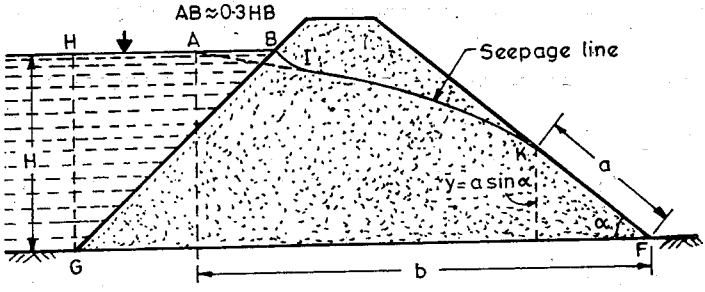


Fig. 20.16

Equation (20.23) gives satisfactory results for values of α less than 30°, but for steeper slopes, the deviations from the correct values become quite high and for such cases, Cassagrande has suggested the use of sin α in place of tan α.

In other words, the hydraulic gradient (*i*) is given by $\frac{dy}{ds}$ (*i.e.* sin α) of the seepage line and not by $\frac{dy}{dx}$ (*i.e.* tan α) as was taken in the previous case.

Therefore, $q = K\cdot i\cdot A$

$$\left(\frac{dy}{ds}\right) y \cdot 1 = K(\sin\alpha)(a\sin\alpha) \qquad \because \frac{dy}{ds} = \sin\alpha$$

$$K \cdot a \cdot \sin^2\alpha$$

$$y = a\sin^2\alpha \cdot ds.$$

tween the limits

$$y = a\sin\alpha, \qquad s = a$$
$$y = H \qquad s = S_0$$

where S_0 is the total length of parabola from the point A to the point F.

or $$\int_{y=a\sin\alpha}^{y=H} y \cdot dy = a \cdot \sin^2\alpha \int_{s=a}^{s=S_0} ds$$

or $$\left|\frac{y^2}{2}\right|_{a\sin\alpha}^{H} = a\sin^2\alpha \cdot |s|_{s=a}^{s=S_0}$$

or $$\frac{H^2 - a^2\sin^2\alpha}{2} = a\sin^2\alpha\,[S_0 - a]$$

or $$\frac{H^2}{2} - \frac{a^2}{2}\sin^2\alpha = a\sin^2\alpha \cdot S_0 - a^2\sin^2\alpha$$

or $$\frac{a^2}{2}\sin^2\alpha - S_0\sin^2\alpha \cdot a + \frac{H^2}{2} = 0$$

or $$a^2 - 2S_0 \cdot a + \frac{H^2}{\sin^2\alpha} = 0$$

or $$a = \frac{2 \cdot S_0 \pm \sqrt{4 \cdot S_0^2 - 4 \cdot \dfrac{H^2}{\sin^2\alpha}}}{2}$$

Ignoring +ve sign, we get

$$a \doteq S_0 - \sqrt{S_0^2 - \frac{H^2}{\sin^2\alpha}} \qquad \text{...(20.24)}$$

The total length of the parabola S_0 can be approximately taken to be equal to $\sqrt{b^2 + H^2}$, then

$$S_0 = \sqrt{b^2 + H^2}$$

Substituting, we get

$$a = \sqrt{b^2 + H^2} - \sqrt{b^2 + H^2 - \frac{H^2}{\sin^2\alpha}}$$

or $$a = \sqrt{b^2 + H^2} - \sqrt{b^2 - H^2\left(\frac{1}{\sin^2\alpha} - 1\right)}$$

$$\boxed{a = \sqrt{b^2 + H^2} - \sqrt{b^2 - H^2\cot^2\alpha}}$$ which is the required eqn. (20.23).

Example 20.1. *An earth dam made of a homogeneous material has the following data :*

Coefficient of permeability of dam material	$= 5 \times 10^{-4}$ *cm/sec.*
Level of top of dam	$= 200.0$ *m.*
Level of deepest river bed	$= 178.0$ *m*
H.F.L. of reservoir	$= 197.5$ *m.*
Width of the top of dam	$= 4.5$ *m.*
Upstream slope	$= 3 : 1.$
Downstream slope	$= 2 : 1.$

Determine the phreatic line for this dam section and the discharge passing through the dam.

Solution. Taking the focus (F) at the d/s toe of the dam as the origin, the equation of the base parabola is given by $\sqrt{x^2+y^2} = x + S$

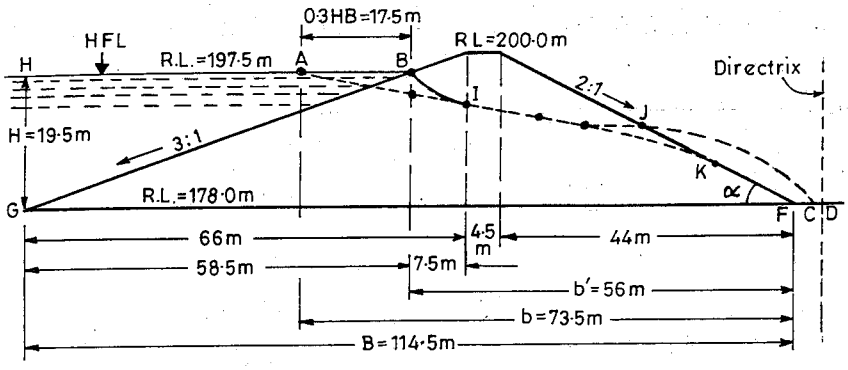


Fig. 20.17

where S is the distance of the point (x, y) from the directrix, called focal distance. Select the point A, in such a way that

$$AB \approx 0.3\, HB$$

or

$$AB \approx 0.3 \times 58.5 = 17.5 \text{ m.}$$

Plot the point A at a distance 17.5 m from B (Fig. 20.17). This is the start-point of the parabola. Now the coordinates of the point A w.r. to F as origin are (73.5 m, 19.5 m). Substituting these ordinates in the equation of the base parabola, we get

$$\sqrt{(73.5)^2 + (19.5)^2} = 73.5 + S.$$

or

$$\mathbf{S = 2.54\ m.}$$

The vertex (C) of the base parabola shall be situated at a distance equal to $\frac{S}{2}$ or 1.27 m from F, beyond the d/s toe of the dam, as shown in Fig. 20.17.

A few more coordinates of the base parabola at known distances (x) are worked out in table 20.4 using,

$$x^2 + y^2 = x^2 + S^2 + 2xS$$

or

$$y = \sqrt{S^2 + 2xS}$$

Table 20.4

x	$y^2 = 2xS + S^2 = 5.08x + 6.45$	$y = \sqrt{S^2 + 2xS}$
0	6.45	2.54
10	57.25	7.68
20	108.05	10.39
30	158.85	12.61
40	209.65	14.48
44	229.97	15.16
48.5	252.83	15.92
56	290.93	17.04
60	311.25	17.64
70	362.05	19.02
73.5	380.00	19.50

The base parabola with all these ordinates, is then drawn.

Now, this parabola has to be corrected at entry and exit as explained earlier. At entry, the phreatic line is started from the point B in such a way that it becomes at right angles to the u/s face GB of the dam. A reverse curvature BI is, therefore, given as shown in Fig. 20.17.

At exit, the point K at which the phreatic line intersects the d/s face can be easily obtained by using the Eq. (20.21) as :

$$\Delta a = (a + \Delta a)\left(\frac{180 - \alpha}{400}\right)$$

where $\tan\alpha = \frac{1}{2}$; or $\alpha = 26° \cdot 54$

$(a + \Delta a)$ = Distance FJ, *i.e.* the distance of the focus from the point at which the base parabola intersects the d/s face, and is measured from Fig. 20.17 ≈ 25.6 m.

$$\therefore \quad \Delta a = 25.6\left[\frac{180 - 26.54}{400}\right] = 25.6 \times \frac{153.46}{400} = 9.84 \text{ m ; } \textbf{say 9.8 m.}$$

$$a = 25.6 - 98 = 15.8 \text{ m.}$$

Knowing 'a', the point K is plotted and the phreatic line BIK is completed.

Discharge through the dam section can be obtained from the equation (20.20) as :

$$q = K \cdot S, \qquad \text{where } K = 5 \times 10^{-4} \text{ cm/sec} = 5 \times 10^{-6} \text{ m/sec.}$$

$$S = 2.54 \text{ m}$$

$$\therefore \quad q = 5 \times 10^{-6} \times 2.54 \text{ m}^3/\text{m run/sec.}$$

$$= 12.7 \times 10^{-6} \text{ cumes/m length of dam}$$

Hence, $q = 12.7 \times 10^{-6}$ cumecs/m length of dam. **Ans.**

Note. The value of 'a' can also be determined from Eq. (20.22), if $(a + \Delta a)$ is not to be measured, as the dam section is not to be plotted to scale.

In that case $a = \dfrac{b'}{\cos a} - \sqrt{\left(\dfrac{b'}{\cos\alpha}\right)^2 - \left(\dfrac{H}{\sin\alpha}\right)^2}$

where $b' = 56$ m

$H = 19.5$ m

$\sin 26.56° = 0.447$

$\cos 26.54° = 0.894.$

$$\therefore \quad a = \frac{56}{0.894} - \sqrt{\left(\frac{56}{0.894}\right)^2 - \left(\frac{19.5}{0.447}\right)^2}$$

$$= 62.6 - \sqrt{3{,}920 - 1{,}900} = 62.6 - 44.9 = \mathbf{17.7\ m.}$$

Hence, $a = 17.7$ m which is slightly above the value obtained from Eq. (20.21) and is thus on a safer side.

Example 20.2. *A flow net is plotted for a homogeneous earthen dam of height 22 m and freeboard 2.0 m. The results obtained are,*

Number of potential drops = *10*

Number of flow channels = *4.*

The dam has a horizontal filter of 30 m length at the downstream end and the coefficient of permeability of the dam material is 5×10^{-4} cm/sec. Calculate the discharge per m run of the dam.

Solution. The discharge through a dam section is approximately given by the Eq. (20.15) as :

$$q = K.H.\frac{N_f}{N_d}$$

where $K = 5 \times 10^{-4}$ cm/sec $= 5 \times 10^{-6}$ m/sec.

$H = 22 - 2 = 20$ m.

$N_f = 4$

$N_d = 10$

$$\therefore \quad q = 5 \times 10^{-6} \times \frac{20 \times 4}{10}$$

$$= \mathbf{4.0 \times 10^{-6}\ cumecs/m\ run\ of\ dam.\ \ Ans.}$$

Example 20.3. *For the dam section given in example 20.1, draw the seepage line if a horizontal filter of length equal to 25 m is provided inward from the downstream toe of the dam.*

Solution. The dam section is plotted from the dimensions given in example 20.1. The horizontal filter is also provided. Now, the left end of the filter will act as a focus and is designated as F and is taken as origin.

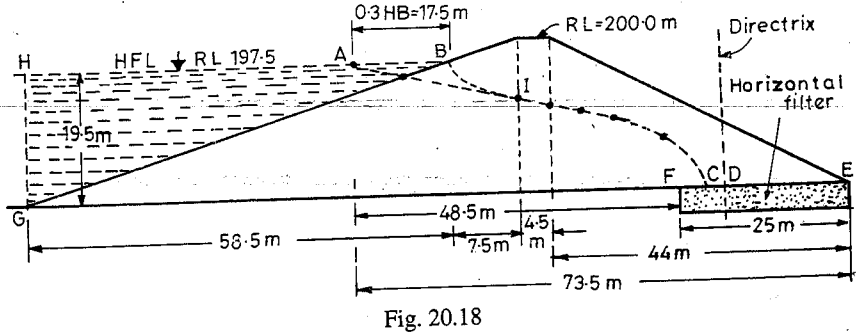


Fig. 20.18

The coordinates of any point (x, y) on a parabola of equation

$$\sqrt{x^2+y^2}=x+S$$

will now be for the point A.

The point A is determined as in example 20.1. Its coordinates w.r. to F as origin are (48.5 m, 19.5 m)

$$\therefore \quad \sqrt{(48.5)^2+(19.5)^2}=48.5+S$$

or $$\sqrt{2{,}352+380}=48.5+S$$

or $$\mathbf{S=3.77\ m.}$$

The vertex (C) of the parabola shall be situated at a distance equal to $\frac{S}{2}$, *i.e.* 1.83 m beyond the point F and directrix shall be at a distance 3.77 m from F.

At point F, $\quad x=0$

$\therefore \quad y=S=3.77$ m.

A few more coordinates of the base parabola are worked out in table 20.5, using

$$x^2+y^2=x^2+S^2+2xS$$

or $$y=\sqrt{2xS+S^2}$$

Table 20.5

x	$y^2=(2xS+S^2)=7.54x+14.21$	$y=\sqrt{2xS+S^2}$
0	14.21	3.77
10	89.61	9.47
15	127.31	11.27
19	157.47	12.54
23.5	191.40	13.85
31	247.95	15.75
40	315.81	17.78
48.5	380.00	19.5

The base parabola is drawn and correction at the entry point for the curve BI is made by free hand sketching, in such a way that the phreatic line becomes ⊥ to u/s face GB of the dam. The exit point should also be corrected such that the phreatic line meets ⊥ to base line FCD. The final phreatic line BIC is thus drawn as shown in Fig. 20.18.

20.12.3. Phreatic Line for a Zoned Section. In case of a zoned section having a central impervious core, such as shown in Fig. 20.19 ; the effects of the outer zone can be neglected altogether.

The focus of the base parabola will, therefore, be located at the d/s toe of the core, as shown in Fig. 20.19. The phreatic line can then be drawn as usual with free hand corrections required at suitable places.

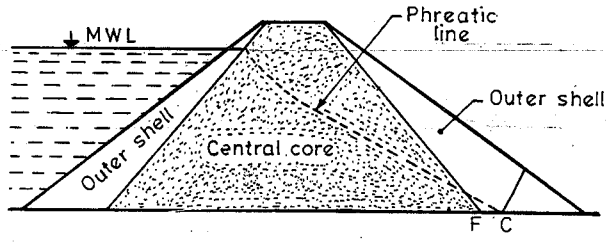


Fig. 20.19

20.13. Stability of Earthern Slopes

An earth embankment usually fails, because of the sliding of a large soil mass along a curved surface. It has been established by actual investigation of slides of railway embankments in Sweden, that the surface of slip is usually close to cylindrical, *i.e.* an arc of a circle in cross-section. The method which is described here and is generally used for examining the stability of slopes of an earthen embankment is called the *Swedish Slip Circle Method* or *The Slices Method.* The method thus assumes the condition of plane strain with failure along a cylindrical arc.

The location of the centre of the possible failure arc is assumed. The earth mass is divided into a number of vertical segments called slices. These verticals are usually equally spaced, though it is not necessary to do so. Depending upon the accuracy desired, six to twelve slices are generally sufficient.

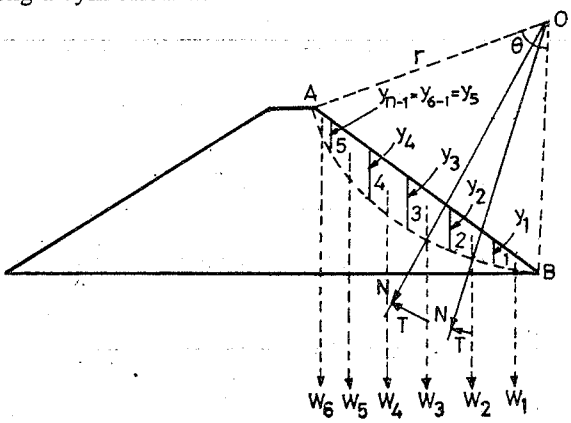


Fig. 20.20

Let O be the centre and r be the radius of the possible slip surface as shown in Fig. 20.20. Let the total arc AB be divided into slices of equal width say b metres each. The width of the last slice will be something different say let it be $m \cdot b$ metres.

Let these slices be numbered as 1, 2, 3, 4, ... and let the weight of these slices be $W_1, W_2, W_3, W_4 \ldots$

The forces between these slices are neglected and each slice is assumed to act independently as a vertical column of soil of unit thickness (⊥ to paper) and width b. The weight W of each slice is assumed to act at its centre. The weight W of each slice can be resolved into two components ; say a normal component (N) and a tangential component (T) such that

$$N = W \cos \alpha$$
$$T = W \sin \alpha$$

where α is the angle which the slope makes with the horizontal.

The normal component (N), will pass through the centre of rotation (O) and hence does not create any moment on the slice. However, the tangential component (T) causes a disturbing moment equal to ($T \cdot r$), where r is the radius of the slip circle. The tangential components of a few slices may create resisting moments; in that case T is considered as negative.

The total disturbing moment (M_d) will be equal to the algebraic sum of all the tangential moments, *i.e.*

$$M_d = \Sigma T \cdot r = r \cdot \Sigma T$$

where $\Sigma T = (T_1 + T_2 + T_3 + \ldots)$

The resisting moment is supplied by the development of shearing resistance of the soil along the arcual surface AB. The magnitude of shear strength developed in each slice will depend upon the normal component (N) of that slice. Its magnitude will be

$$= c \cdot \Delta L + N \tan \phi$$

where c is the unit cohesion, ΔL is the curved length of the slice and ϕ is the angle of internal friction of soil.

This shear resistance is acting at a distance r from O and will provide a resisting moment

$$= r\,[c \cdot \Delta L + N \tan \phi]$$

The total resisting moment over the entire arc AB

$$M_r = r\,[\Sigma C \cdot \Delta L + \Sigma N \cdot \tan \phi]$$

$$= r\,[c \Sigma\, \Delta L + (\Sigma N) \tan \phi]$$

$$= r\,[c \cdot \widehat{AB} + (\Sigma N) \tan \phi]$$

where ΣN is the sum of all the normal components.

$$= N_1 + N_2 + N_3 + \ldots$$

Length $\widehat{AB}$ of slip circle

$$= \widehat{AB} = \left[\frac{2\pi \cdot r}{360°}\right] \times \theta$$

where θ is the angle in degrees, formed by the arc AB at centre O.

Hence, the factor of safety against sliding

$$= \text{F.S.} = \frac{\text{Resisting moment}}{\text{Disturbing moment}} = \frac{M_r}{M_d}$$

or

$$= \frac{r\,[c \cdot \widehat{AB} + (\tan \phi)\, \Sigma N]}{r \cdot \Sigma T}$$

or

$$\text{F.S.} = \frac{[c \cdot \widehat{AB} + (\tan \phi)\, \Sigma N]}{(\Sigma T)} \qquad \ldots(20.24)$$

Eq. (20.24) can be worked out by working out ΣT and ΣN separately. This evaluation of ΣN and ΣT can be simplified as explained below :

If $y_1, y_2, y_3, \ldots$ are the vertical extreme ordinates (boundary ordinates) of the slices 1, 2, 3... respectively, then the respective weights can be written as

$$W_1 = \left(\frac{0 + y_1}{2}\right) \cdot b \cdot \gamma \cdot 1$$

where γ is the unit weight of soil and unit width of the slice is considered.

$$W_2 = \left(\frac{y_1 + y_2}{2}\right) b \cdot \gamma$$

$$W_3 = \left(\frac{y_2 + y_3}{2}\right) b \cdot \gamma$$

$$W_4 = \left(\frac{y_3 + y_4}{2}\right) b \cdot \gamma$$

.....

.....

$$W_n = \left(\frac{y_{n-1} + 0}{2}\right) m \cdot b \cdot \gamma$$

where n is the total number of slices.

$$\therefore \quad \Sigma W = W_1 + W_2 + W_3 + \ldots + W_n$$

$$= \left[y_1 b + y_2 b + y_3 b + \ldots \frac{y_{n-1}}{2} b + \frac{y_{n-1}}{2} m \cdot b \right] \gamma$$

$$= \left[y_1 + y_2 + y_3 + \ldots y_{n-1} \left(\frac{1+m}{2} \right) \right] \gamma \cdot b$$

Now $\quad \Sigma N = N_1 + N_2 + N_3 + \ldots$

$$= W_1 \cos\alpha + W_2 \cos\alpha + W_3 \cos\alpha + \ldots$$

$$\therefore \quad \Sigma N = \cos\alpha \, [W_1 + W_2 + W_3 + \ldots]$$

or $\quad \Sigma N = \cos\alpha \, (\Sigma W)$.

Similarly $\Sigma T = \sin\alpha \, (\Sigma W)$ if all T's are +ve.

ΣW can be evaluated by adding all the vertical boundary ordinates of all the slices. (The last ordinate should be multiplied by $\frac{m+1}{2}$ before adding) and then multiplying this summation (say Σy) by $b \cdot \gamma$.

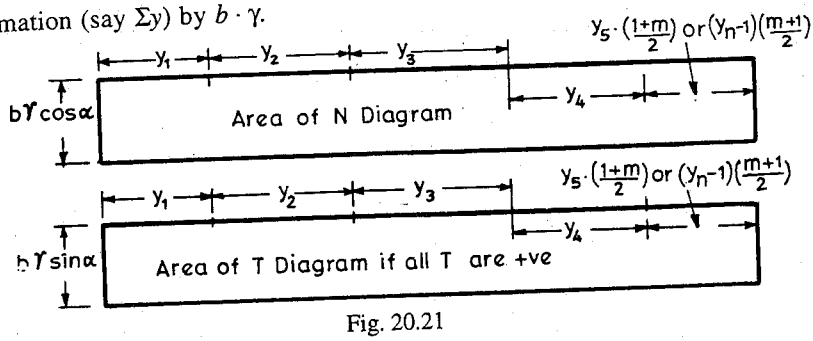


Fig. 20.21

The area of N diagram will represent ΣN and that of T diagram will represent ΣT.

As a general case, the value of ΣN and ΣT can be worked out in a tabular form as shown below in table 20.6.

Table 20.6

Slice number	*Wt. of each slice*	$N = W \cos\alpha$	$T = W \sin\alpha$	$c \cdot \Delta L$
1	W_1	N_1	T_1	
2	W_2	N_2	T_2	
3	W_3	N_3	T_3	
⋮				
n	W_n	N_n	T_n	
		ΣN	ΣT	$\Sigma c \cdot \Delta L$

The F.S. is then calculated from equation

$$\text{F.S.} = \frac{\Sigma c \cdot \Delta L + (\Sigma N - \Sigma U) \tan\phi}{\Sigma T} \qquad \ldots(20.25a)$$

20.13.1. Location of the Centre of the Critical Slip Circle. In order to find out the worst case, numerous slip circles should be assumed and factor of safety (F.S.)

calculated for each circle, as explained earlier. The minimum factor of safety will be obtained for the critical slip circle. In order to reduce the number of trials, Fellenius has suggested a method of drawing a line (PQ), representing the locus of the critical slip circle.

The determination of this line PQ for the d/s slope of an embankment is shown in Fig. 20.22(a) and similarly, the determination of this line PQ for u/s slope is shown in Fig. 20.22(b). The point Q is determined in such a way that its coordinates are ($4.5H, H$) from the toe, as shown. The point P is obtained with the help of directional angles α_1 and α_2 as shown. The value of α_1 and α_2 for various slopes, are tabulated in Table 20.7.

Table 20.7

Slope	*Directional angles*	
	α_1 *in degrees*	α_2 *in degrees*
1 : 1	27.5	37
2 : 1	25	35
3 : 1	25	35
4 : 1	25	35
5 : 1	25	35

After determining the locus of the critical slip circle, the critical slip circle can be drawn, keeping in view the following few points :

(*i*) Except for very small values of ϕ, the critical arc passes through the toe of the slope.

(*ii*) If a hard stratum exists at a shallow depth under the dam, the critical arc cannot cross this stratum, but can only be tangential to it.

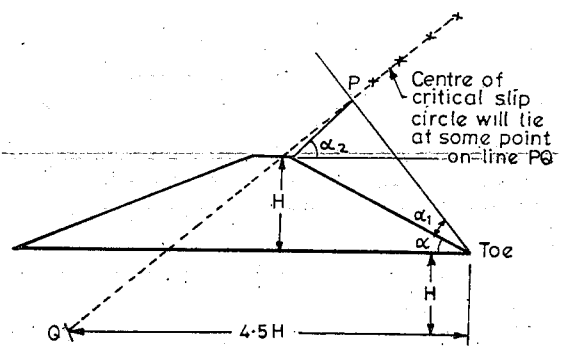


Fig. 20.22 (a). Locus of critical circle for d/s slope.

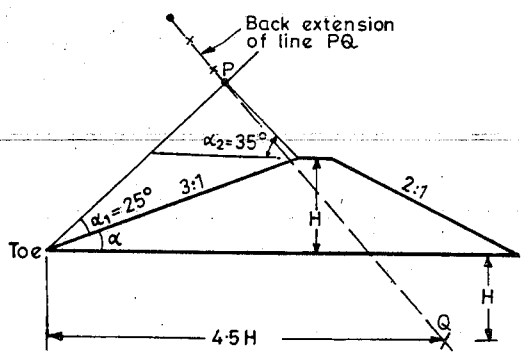


Fig. 20.22 (b) Locus of critical circle for u/s slope.

(*iii*) For very small values of ϕ (0 to 15°), the critical arc passes below the toe of the slope if the inclination of the slope is less than 53° (which is generally the case). The centre of the critical arc in such a case is likely to fall on a vertical line drawn through the centre of the slope, as shown in Fig. 20.23.

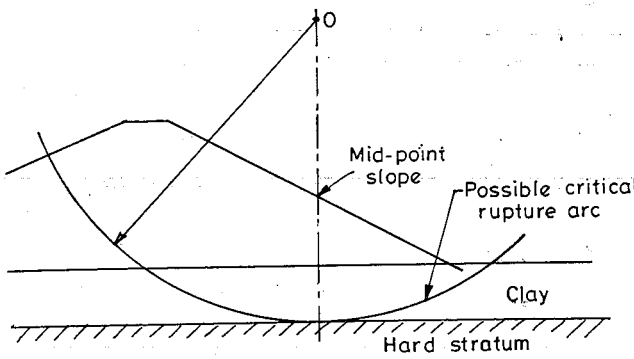


Fig. 20.23.

20.13.2. Determination of Pore Pressures from Flow Net. It was stated earlier that the soil is composed of voids which are filled with water and air. Whenever, any load is applied in the form of consolidation, it is taken up by water and gradually transferred to soil grains as the excess water drains out. The shear strength of the soil thus goes on increasing.

Immediately after construction, sizable pore pressures may be present, which may gradually dissipate. But as soon as the reservoir is filled, water enters the pores of the dams and a new pattern of pore pressure gets developed.

Under steady state seepage, the pore pressure at any point in a dam is equal to the hydrostatic head due to water in the reservoir minus the head loss in seepage through the dam up to that point. The pore pressure at any point within a dam section can easily be found from the flow net.

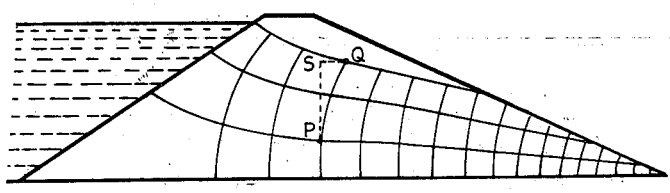


Fig. 20.24

For example, in Fig. 20.24, the pore pressure at any point say P will be equal to the difference in elevation between P and Q, where Q is the point of intersection of the phreatic line with the equipotential line through the point P.

Hence, if a piezometer is installed at F, the water shall rise up to an elevation of Q. If QS is a horizontal line through Q, PS will be the pore pressure at P.

20.13.3. Stability of Downstream Slope during Steady Seepage. The most critical condition for which the stability of the d/s slope must be examined, occurs, when the reservoir is full and the seepage is taking place at full rate.

The seeping water below the phreatic line, exerts a pore pressure on the soil mass which lies below the phreatic line. Hence, if the slices of the critical arc, happen to include this submerged soil, [Fig. 20.25 (*a*)], the shear strength developed on those slices shall be correspondingly reduced. The net shear strength on such a slice shall be $= c\,\Delta L + (N - U)\tan\phi$, where U is the pore pressure.

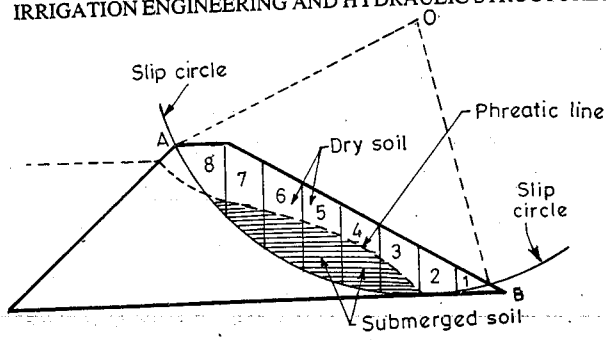


Fig. 20.25 (*a*)

The factor of safety (S.F.) for the entire slip circle is then given by the equation.

$$\text{F.S.} = \frac{c \cdot \widehat{AB}. + \tan\phi\,(\Sigma N - \Sigma U)}{\Sigma T} \qquad \text{...(20.25)}$$

where ΣU is the total pore pressure on the slip circle.

The pore pressure at a point is represented by the piezometric head at that point as explained earlier. The variation of the pore pressure along a failure arc is, therefore, obtained as explained below :

First of all, draw a flow net and thus determine the points of intersections of equipotential lines with the failure arc. At each point of intersection, measure the vertical ordinate from that intersection to the level at which that particular equipotential line cuts the phreatic line. The pore pressures represented by the vertical heights so obtained, are then plotted to a scale in a direction perpendicular to the sliding surface at the respective points of intersection.

The pore pressure distribution is thus shown in Fig. 20.25(*b*) (shaded area). The

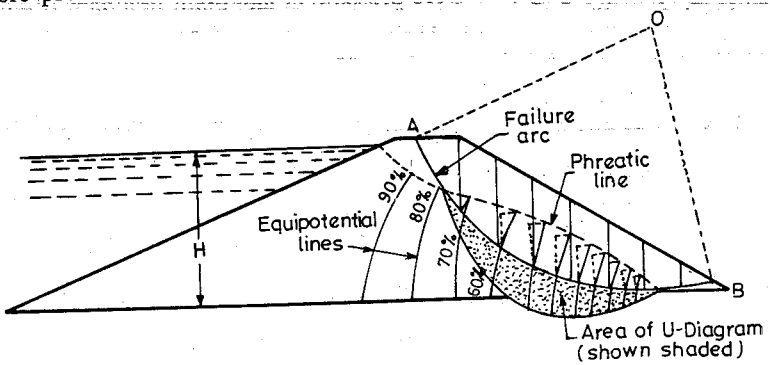


Fig. 20.25 (*b*)

area of this diagram can be measured by a planimeter. The area of this diagram can also be calculated by ordinate method as was done for N and T cases taking the unit weight of water as 9.81 kN/m^3 ($\approx$ 10 kN/m^3). Knowing ΣN, ΣU and ΣT, F.S. can be calculated easily by using equation 20.25.

Approximate method. In the absence of a flow net, the normal components, which are responsible for the shear strength of the soil, can be calculated on the basis of submerged unit weight of soil (*i.e.* $\gamma_{sub} = \gamma_{sat} - \gamma_w$). On the other hand, the values of

tangential components *i.e.* T's which are responsible for creating disturbing moments, should be calculated on the basis of saturated unit weight of soil.

In such a case, the width of N rectangle which was taken as $b \cdot \gamma \cdot \cos \alpha$ will become $b \cdot \gamma_{sub} \cos \alpha$ (Fig. 20.21) and the width of T rectangle will remain $b \cdot \gamma_{sat} \cdot \sin \alpha$. The new N diagram is a $(N - U)$ diagram assuming the entire soil to be submerged and can be called N' diagram. Equation (20.25) can be written as

$$\text{F.S.} = \frac{c \cdot \widehat{AB} + \tan \phi \cdot (\Sigma N')}{\Sigma T} \qquad ...(20.26)$$

20.13.4. Stability of Upstream Slope During Sudden Drawdown. When the reservoir is full, the critical region is near the downstream face. If no drainage arrangement is made and the d/s slope is also steep, the phreatic line may intersect the d/s slope creating serious conditions there. This can be avoided by providing drainage filter or drainage toe, etc., or by broadening the base of the dam so that the head loss is great enough to bring the line of saturation beneath the d/s toe of the dam.

For the u/s slope, the critical condition can occur, when the reservoir is suddenly emptied. In such a case, the water level within the soil will remain as it was when the soil pores were full of water. The weight of this water within the soil, now tends to slide the u/s slope along a circular arc.

The tangential component of the saturated soil lying over the arc, will create a disturbing force ; while the normal component minus the pore pressure shall supply the shear strength of the soil. High pore pressures shall be developed in this case and although a true solution can be obtained from the flow net and pressure net, an approximate solution can be easily obtained, by considering the soil resting over the failure arc as saturated, for calculating T's ; and as submerged for calculating N's.

The factor of safety (F.S.) is finally obtained from the equation

$$\text{F.S.} = \frac{c \cdot \widehat{AB} + \tan \phi \, \Sigma N'}{\Sigma T}$$

N's represent normal components on submerged density and T's represent tangential components on saturated unit weight of soil. The maximum factor of safety obtained for the critical slip circle should be 1.5, for safe designs.

20.13.5. Stability of the u/s slope portion of the dam, during sudden drawdown, from the consideration of horizontal shear developed at base under the u/s slope of the dam. It is an approximate method for checking the stability of the u/s slope against sudden drawdown. It is based on the simple principle that a horizontal shear force (say P_u) is exerted by the saturated soil (*i.e.* by the soil as well as by water contained within the soil). The resistance to this force (say R_u) is provided by the shear resistance

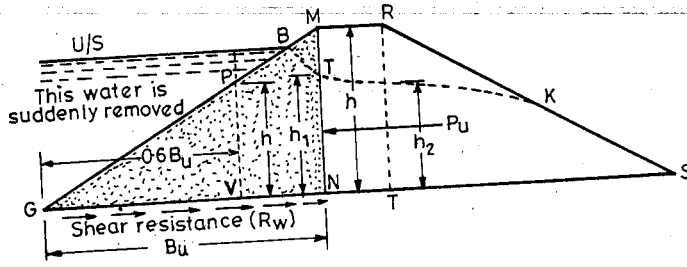


Fig. 20.26

developed at the base of the soil mass, contained within the u/s triangular shoulder *GMN* (Fig. 20.26).

Considering a unit length of the dam, the horizontal force P_u is given by the equation

$$P_u = \left[\frac{\gamma_1 h^2}{2}\tan^2\left(45 - \frac{\phi}{2}\right) + \gamma_w \cdot \frac{h_1^2}{2}\right] \text{(say in kN)} \qquad ...(20.27)$$

where γ_1 is the weighted density at the centre of triangular shoulder upstream and is given by

$$\gamma_1 = \frac{\gamma_{sub} \cdot h_1 + \gamma_{dry} \cdot (h - h_1)}{h} \qquad ...(20.28)$$

Shear resistance (R_u) of u/s slope portion of the dam developed at base GN is given by

$$R_u = C + W \tan\phi \qquad ...(20.29a)$$

where W = the weight of the u/s triangular shoulder of dam

C = The total cohesive force developed at base *GN*.

If c is the unit cohesion of the dam soil, then $C = c \times (B_u \times 1)$ where B_u = length *GN*.

The triangular profile of the u/s slope portion of dam has an area *GBTN* as the submerged soil (*i.e.* the soil below the seepage line) and an area equal to *BMT* as a dry area. The correct weight W will be equal to ($\gamma_{sub} \times$ Area *GBTN* + $\gamma_{dry} \times$ Area *BMT*). These areas can be measured by a planimeter. If the measuring of the areas is to be avoided, the entire area may be taken as submerged. By so doing, the weight W will be slightly reduced, and thus $W \tan\phi$ or R_u or F.S. will be slightly reduced. Hence, the results obtained will be on a safer side.

In such a case,

$$W = [\text{Area of } \Delta\, GMN] \cdot \gamma_{sub} = \gamma_{sub} \cdot (\tfrac{1}{2} \cdot Bu \cdot h)$$

or

$$R_u = C + W \tan\phi$$

$$= c \cdot (B_u \times 1) + (\gamma_{sub} \cdot \tfrac{1}{2} \cdot B_u \cdot h) \tan\phi \qquad ...(20.29)$$

Now P_u and R_u are known, the factor of safety against sliding can be easily calculated, using

$$\text{F.S.} = \frac{R_u}{P_u} \qquad ...(20.30)$$

It should be more than 1.5.

The factor of safety calculated above is w.r. to average shear (τ_{av}), which will be equal to

$$\tau_{av} = \left(\frac{P_u}{B_u \times 1}\right) \qquad ...(20.31)$$

It has been found by photoelastic studies, that the maximum intensity of shear stress occurs at a distance 0.6 B_u from the heel (*i.e.* 0.4 B_u from the top shoulder) and is equal to 1.4 times the average shear intensity.

∴ Maximum shear stress induced

$$= \tau_{max} = 1.4\left(\frac{P_u}{B_u}\right) \quad ...(20.32)$$

which is developed at the point V (Fig. 20.26) such that $GV = 0.6\,B_u$.

The unit shearing resistance developed at this point V is give by

$$\tau_f = c + h'\,\gamma_{sub} \cdot \tan\phi$$
$$= c + 0.6\,h \cdot \gamma_{sub} \cdot \tan\phi \quad ...(20.33)$$

F.S. at the point of the maximum shear

$$= \frac{\tau_f}{\tau_{max}} \quad ...(20.34)$$

It should be more than 1.

20.13.6. Stability of d/s slope under steady seepage from the considerations of horizontal shear at base under the d/s slope of the dam. The stability of the d/s slope under steady seepage is generally tested with Swedish slip circle method. However, the F.S. against the horizontal shear forces can be evaluated on the same principles as was done for the d/s slope in the previous article.

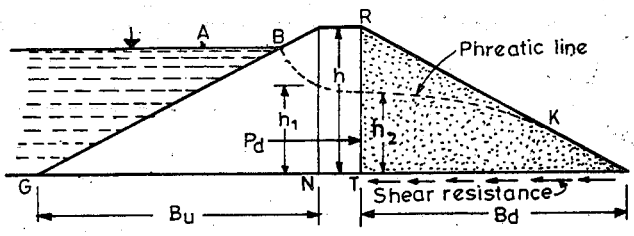


Fig. 20.27

With reference to Fig. 20.27.

The horizontal shear force P_d, in this case, is given by

$$P_d = \left[\frac{\gamma_2 \cdot h^2}{2}\tan^2\left(45 - \frac{\phi}{2}\right) + \frac{\gamma_w \cdot h_2^2}{\cdot\, 2}\right] \quad ...(20.35)$$

where γ_2 is the weighted density at the centre of the triangular shoulder downstream, given by

$$\gamma_2 = \frac{\gamma_{sub} \cdot h_2 + \gamma_{dry}\,(h - h_2)}{h} \quad ...(20.36)$$

Shear resistance R_d of d/s slope portion of dam, developed at base is given by

$$R_d = C + W\tan\phi,$$

where W = The weight of the d/s slope portion of dam. (*i.e.* $\Delta\,RTS$)

$C = c \times (B_d \times 1)$, where c is the unit cohesion.

The triangular profile RTS of the d/s slope portion of dam has an area say A_1 of dry soil above the seepage line and the area of submerged soil say A_2 below the seepage lines. These areas can be measured by a planimeter and then

$$W = [\gamma_{dry} A_1 + \gamma_{sub} \cdot A_2] \times 1$$

or

$$R_d = c \cdot B_d + \left[\gamma_{dry} A_1 + \gamma_{sub} \cdot A_2\right]\tan\phi \quad ...(20.37)$$

If the measuring of the areas is to be avoided, the entire weight W may be calculated on the basis of submerged soil, as it will be on a still safer side. In that case,

$$W = \gamma_{sub} \cdot (\tfrac{1}{2} \cdot B_d \cdot h)$$

Knowing P_d and R_d, the factor of safety against shear can be easily determined as

$$\text{F.S.} = \frac{R_d}{P_d} \qquad \text{...(20.38)}$$

The factor of safety at the point of maximum shear can also be determined in the same manner as was explained for the u/s slope portion.

20.13.7. Stability of the foundation against Shear. The generally available silt and clay foundations below the base of an earth dam, are sufficiently impervious, and there is generally no necessity of providing any treatment for under-seepage and piping for such foundations. But these foundations are weak in shear and must be investigated. In order to keep the shear stress developed at the foundations, within limits, the embankment-slopes may have to be flattened or berms on either side may be provided. If the available foundations are of plastic or unconsolidated clays, their shear strength will be very less and the matter should be seriously and thoroughly investigated.

The method given below, for determining the factor of safety against the foundation shear, is an approximate method and is based on the assumption that a soil has an equivalent liquid unit weight which would produce the same shear stress as the soil itself.

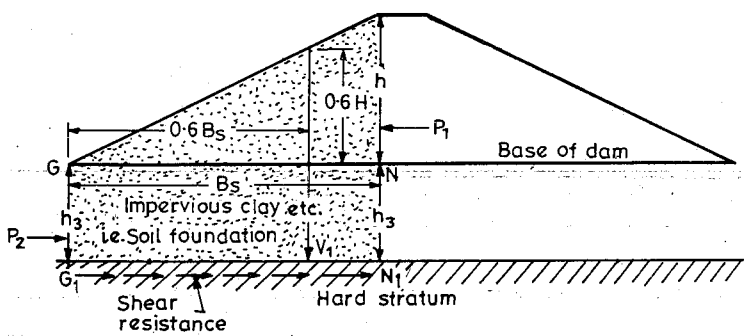


Fig. 20.28

The total horizontal shear force (P), under a slope of the dam is equal to the difference between the lateral thrust on a vertical through the top shoulder of the slope and a vertical through the toe of the slope.

or $$P = (P_1 - P_2) = \frac{\gamma_{eq}(h+h_3)^2}{2} \tan^2\left(45 - \frac{\phi_1}{2}\right) - \frac{\gamma_{eq} \cdot h_3^2}{2} \times \tan^2\left(45 - \frac{\phi_1}{2}\right)$$

or $$P = \gamma_{eq}\left[\frac{(h+h_3)^2 - h_3^2}{2}\right] \times \left[\tan^2\left(45° - \frac{\phi_1}{2}\right)\right] \text{ (kN)} \qquad \text{...(20.39)}$$

where h_3 = the depth of foundation soil below the dam base, overlying the hard stratum below it.

γ_{eq} = the equivalent unit weight of dry soil in foundation and dam. It is given by

$$\gamma_{eq} = \frac{\gamma_{dry} \text{ for dam material} \times h + \gamma_{dry} \text{ for foundation material} \times h_3}{h + h_3} \quad ...(20.40)$$

where ϕ_1 = the equivalent angle of internal friction and is given by

$$\phi_1 = \tan^{-1}\left[\frac{c_f + \gamma_{eq}(h + h_3)\tan\phi_f}{\gamma_{eq} \cdot (h + h_3)}\right] \quad ...(20.41)$$

where c_f and ϕ_f are the values of unit cohesion and angle of internal friction for the soil in the foundation.

The term, $\left[\gamma_{eq} \cdot \tan^2\left(45 - \frac{\phi_1}{2}\right)\right]$ is known as **equivalent liquid unit weight.**

Now the average shear stress at the base of the slope

$$(\tau_{av}) = \frac{P}{B_s} \quad ...(20.42)$$

where B_s is the base width below the slope.

The value of B_s will be equal to B_u for u/s slope and B_d for d/s slope. The minimum value will generate maximum stresses and hence, that particular slope should be considered which gives the minimum value of B_s, *i.e.* the slope which is less flat and is, therefore, the worst slope.

Maximum stress has been found by photoelastic studies to be 1.4 times the average stress and it occurs at a distance of 0.6 B_s from the toe of the slope.

or τ_{max} = maximum shear stress = 1.4 τ_{av}.

The unit shear resistance of the foundation soil below the toe at point G_1

$$= \tau_{f_1} = \left[c_f + \gamma_f \cdot h_3 \tan\phi_f\right] \quad ...(20.43)$$

γ_f is the unit weight of foundation soil and if the average value is given for impervious soils, that value may be used in the equation (20.43). But if there is a possibility of foundation soil getting submerged due to large scale seepage that may take place through the foundation soil, then the submerged density may be used in equation (20.43).

Similarly, the unit shear resistance of the soil vertically below the upper point of the considered slope (say at point N_1) is given by τ_{f_2}.

$$\tau_{f_2} = c_f + \gamma_3 (h + h_3) \tan\phi_f \quad ...(20.44)$$

where γ_3 = the equivalent unit weight of soil in the dam and foundation at the point N_1 and is given by

$$\gamma_3 = \frac{\gamma_f \times h_3 + \gamma_{dam} \times h}{h + h_3} \quad ...(20.45)$$

The values of γ_f and γ_{dam} may be taken as their dry densities or submerged densities depending upon the possibilities.

The average shear resistance

$$= \tau_f = \frac{\tau_{f_1} + \tau_{f_2}}{2} \quad ...(20.46)$$

Hence, overall factor of safety $= \dfrac{\tau_f}{\tau_{(av)}}$ ...(20.47)

This should be greater than 1.5.

The factor of safety at the point of maximum shear (*i.e.* the point V_1) must also be calculated as below :

The unit shear resistance at this point V_1

$$= \tau_{f(max)} = c_f + \gamma_4 (h_3 + 0.6h) \tan \phi_f$$

where γ_4 is the equivalent weight of soil in dam and foundation and is given by

$$\gamma_4 = \frac{\gamma_f \times h_3 + \gamma_{dam} \times (0.6\,h)}{(h_3 + 0.6h)} \quad ...(20.48)$$

The dry or submerged unit weights may be used as explained earlier, in the above equation.

F.S. = Factor of safety at the point of maximum shear

$$= \frac{\tau_{f\,(max)}}{\tau_{max}} \quad ...(20.49)$$

This should be greater than unity.

Example 20.4. *An earthen dam made of homogeneous material has the following data :*

Level of the top of the dam	= *200.00 m*
Level of deepest river bed	= *178.0*
H.F.L. of reservoir	= *197.5 m*
Width of top of dam	= *4.5 m*
Upstream slope	= *3 : 1*
Downstream slope	= *2 : 1*
Length of the horizontal filter from d/s toe, inwards	= *25 m*
Cohesion of soil of dam	= *24 kN/m²*
Cohesion of soil of foundation	= *54 kN/m²*
Angle of internal friction of soil in the dam	= *25°*
Angle of internal friction of soil in the foundation	= *12°*
Dry weight of the soil in the dam	= *18 kN/m³*
Submerged weight of the soil in the dam	= *12 kN/m³*
Dry unit weight of the foundation soil	= *18.3 kN/m³*
Coefficient of permeability of soil in the dam	= 5×10^{-6} *m/sec.*

The foundation soil consists of 8 m thick layer of clay, having negligible coefficient of permeability. Check the stability of the dam and its foundations.

Solution.

(1) **Overall stability of the dam section as a whole**

We will consider 1 m length of the dam. The section of the dam and the phreatic line is first of all drawn, as given in example 20.3 and shown in Fig. 20.29 (*a*). The dam section, etc. is generally drawn on a graph sheet so as to facilitate in measuring the areas above and below the seepage line, if planimeter is not available.

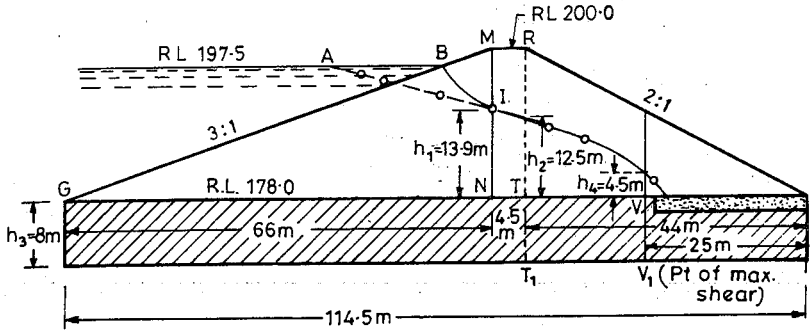


Fig. 20.29 (*a*)

The total area of dam section $= (114.5 + 4.5)\dfrac{22}{2} = 1{,}409$ sq. m

The area above the seepage line is measured and is approximately found to be 380 m^2. (In the absence of a planimeter, graph can be used).

$\therefore$ Area below the seepage line $= 1{,}409 - 380 = 1{,}029$ sq. m

Now

Weight of the dry portion of the dam section

$$= (380\ \text{m}^2 \times 1\ \text{m} \times 18\ \text{kN/m}^3 = 6830\ \text{kN}.$$

Weight of the submerged portion of the dam section

$$= 1029\ \text{m}^2 \times 1\ \text{m} \times 12\ \text{kN/m}^3 = 12{,}350\ \text{kN}$$

Total weight of dam (called average weight)

$$= 6{,}830 + 12{,}350 = 19{,}180\ \text{kN}$$

Shear resistance of the dam at the base

$$= C + W \tan \phi$$

where C = Total cohesive strength of the soil at the base

$= c \times B \times 1 = (24 \times 114.5 \times 1)$ kN

B = Total base width = 114.5 m

$W \tan \phi = 19{,}180 \tan 25°$

$\therefore$ Shear resistance at base,

$$R = 24 \times 114.5 \times 1 + 19180 \tan 25° = 11690\ \text{kN}$$

Horizontal force = Horizontal pressure of water.

$$= P = \tfrac{1}{2}\gamma_w h^2 = \tfrac{1}{2} \cdot 9.81\,(19.5)^2 = 1865\ \text{kN}$$

Factor of safety against failure due to horizontal shear at base

$$= \frac{11690}{1865} = 6.27 > 1.3\ (\therefore \text{Safe})$$

(2) **Stability of the u/s slope portion of dam (Under sudden drawdown)** *horizontal shear along the base under the u/s slope of dam*

Draw a vertical through the u/s extremity of the top width of dam [*i.e.* point *M*, Fig. 20.29 (*a*)] so as to cut the base of the dam at point *N*. This vertical *MN* cuts the seepage

line at a point, the height of which is measured as $h_1 = 13.6$ m above the base of the dam.

Horizontal force (P_u) acting on the ΔGMN is given by equation (20.27) as :

$$P_u = \left[\frac{\gamma_1 h^2}{2} \tan^2\left(45 - \frac{\phi}{2}\right) + \gamma_w \cdot \frac{h_1^2}{2}\right]$$

where γ_1 = the weighted density at the centre of triangular shoulder upstream (ΔGMN) and is given by equation (20.28) as :

$$\gamma_1 = \frac{\gamma_{sub} \cdot h_1 + \gamma_{dry}(h - h_1)}{h}$$

$$= \frac{12 \times 13.9 + 18\,(22.0 - 13.9)}{22.0}$$

$$= 14.7 \text{ kN/m}^3$$

$$\therefore \quad P_u = \frac{14.7 \times (22.0)^2}{2} \tan^2\left(45° - \frac{25°}{2}\right) + 9.81 \times \frac{(13.9)^2}{2} = 2391 \text{ kN}$$

Shear resistance R_u of the u/s slope portion of dam developed at the base GN is given by equation (20.29) as :

$$R_u = C + W \tan\phi$$

$$= c\,(B_u \times 1) + (\gamma_{sub} \tfrac{1}{2} B_u h) \tan\phi \text{ ; neglecting the small dry soil area } BMI \text{, as it is very small and this neglection is on a safer side.}$$

$$B_u = 66 \text{ m}$$

$$\therefore \quad R_u = 24 \times 66 + (12 \cdot \tfrac{1}{2} \cdot 66 \cdot 22.0) \tan 25°$$

$$= 1584 + 4062 = 5646 \text{ kN}$$

Factor of safety against horizontal shear along base under u/s slope

$$= \frac{R_u}{P_u} = \frac{5646}{2391} = 2.36 > 2.0 \qquad (\therefore \text{ safe})$$

Horizontal shear stress induced in the u/s slope portion of dam at base.

$$\tau_{av} = \frac{P_u}{B_u \times 1} = \frac{2391}{66} \text{ kN/m}^2 = 36.23 \text{ kN/m}^2$$

$$\tau_{max} = \text{Maximum shear}$$

$$= 1.4\,\tau_{av} = 1.4 \times 36.23 = 50.72 \text{ kN/m}^2$$

The maximum shear is developed at a point $0.6\,B_u$

$$= 0.6 \times 66 = 39.6 \text{ m away from point } G$$

The unit shear resistance developed at this point

$$\tau_f = c + 0.6\,\gamma_{sub} \tan\phi$$

$$= 24 + 0.6 \times 22.0 \times 12 \tan 25° = 97.9 \text{ kN/m}^2$$

$$\therefore \quad \text{F.S.} = \frac{\tau_f}{\tau_{max}} = \frac{97.9}{50.72} = 1.93 > 1 \qquad (\therefore \text{ safe}).$$

(3) **Stability of d/s portion of dam.** *Horizontal shear along base under the d/s slope of dam.*

Draw a vertical through the d/s extremity of the top width of dam (*i.e.* point *R*) to cut the base at point *T* [Fig. 20.29 (*a*)]. Let this vertical cut the seepage line in a point, the height of which from the base is measured as $h_2 = 12.5\ m$.

Horizontal force P_d acting on the portion of downstream dam (*RTS*) during steady seepage is given by equation (20.35) as :

$$P_d = \left[\frac{\gamma_2 h^2}{2}\tan^2\left(45° - \frac{\phi}{2}\right) + \gamma_w \frac{h_2^2}{2}\right]$$

where γ_2 is the weighted density at the centre of the triangular shoulder *RTS* and given by equation (20.36) as :

$$\gamma_2 = \frac{\gamma_{sub}\, h_2 + \gamma_{dry}\,(h - h_2)}{h}$$

$$= \frac{12 \times 12.5 + 18 \times (22.0 - 12.5)}{22.0}$$

$$= 14.6\ \text{kN/m}^3$$

$$P_d = \frac{14.6\,(22)^2}{2}\tan^2\left(45° - \frac{25°}{2}\right) + 9.81 \cdot \frac{(12.5)^2}{2} = 2200\ \text{kN}$$

Shear resistance R_d of the d/s slope portion of dam developed at base *TS* is given as :

$$R_d = C + W \tan\phi$$

The area A_1 of the dry soil within the $\Delta\, RTS$ above the seepage line ≈ 300 sq. m (from graph or planimeter).

The total area of the $\Delta\, RTS = \frac{1}{2} \times 44 \times 22 = 484\ \text{m}^2$

$\therefore$ Area of submerged soil

$$A_2 = 484 - 300 = 184\ \text{sq. m}$$

$$R_d = cB_d + \left[\gamma_{dry} A_1 + \gamma_{sub} A_2\right] \tan\phi$$

$$= 24 \times 44.0 + [18 \times 300 + 12 \times 184] \tan 25° = 4604\ \text{kN}.$$

F.S. against horizontal shear along base under d/s slope

$$= \frac{R_d}{P_d} = \frac{4604}{2200} = 2.09 > 2\ (\therefore\ \text{Safe})$$

Average shear induced at base

$$= \frac{P_d}{B_d} = \frac{2200}{44} = 50\ \text{kN/m}^2$$

Maximum shear stress induced

$$\tau_{max} = 1.4 \times 50 = 70\ \text{kN/m}^2$$

The maximum shear stress is developed at a point $0.6\ B_d$

$$= 0.6 \times 44 = 26.4\ \text{m away from toe}$$

This unit shear resistance developed at this point

$$\tau_f = c + 0.6h\,\gamma_{sub}\tan\phi$$

(assuming the entire height as submerged as it will give safer results)

$$= 24 + 0.6 \times 22 \times 12 \tan 25° = 97.9 \text{ kN/m}^2$$

$$\therefore \quad \text{F.S.} = \frac{\tau_f}{\tau_{max}} \frac{97.9}{70} = 1.40 > 1 \qquad (\therefore \text{ Safe})$$

(4) **Stability of the foundation soil**

Average compressive stress on foundation soil

$$= \frac{\text{Weight of dam}}{\text{Base area on which it acts}}$$

Since the compressive stress is maximum when the entire dam soil is dry, therefore, we will first calculate the dry weight of the dam.

Area of section of dam

= 1,409 sq. m (calculated earlier)

Dry weight of dam section

$$= 18 \times 1{,}409 = 25{,}362 \text{ kN}$$

Average compressive stress at base

$$= \frac{25362}{114.5} = 221.5 \text{ kN/m}^2$$

Shear stress induced at base

The total horizontal shear force (P) under the d/s slope of the dam (which is the worst case, *i.e.* the steepest slope) is given by equation (20.39) as :

$$P = \gamma_{eq}\left[\frac{(h+h_3)^2 - h_3^2}{2}\right]\left[\tan^2\left(45° - \frac{\phi_1}{2}\right)\right]$$

where γ_{eq} is the equivalent weight of dry soil in foundation and dam

$$\gamma_{eq} = \frac{18h + 18.3\,h_3}{h + h_3}$$

[$\because$ Unit wt. of foundation soil of thickness $h_3 = 18.3$ kN/m^3]

where $h = 22$ m

$h_3 = 8$ m.

$$\therefore \quad \gamma_{eq} = \frac{18 \times 22 + 18.3 \times 8}{22 + 8} = 18.1 \text{ kN/m}^3$$

ϕ_1 is given by equation (20.41) as :

$$\gamma_{eq}(h + h_3)\tan\phi_1 = c_f + \gamma_{eq}(h + h_3)\tan\phi_f$$

or $\quad 18.1\,(22 + 8)\tan\phi_1 = 54 + 18.1\,(22 + 8)\tan 12°$

or $\quad \tan\phi_1 = 0.312$

or $\quad \phi_1 = 17.3°$

$$\therefore \quad P = 18.1\left[\frac{(22+8)^2 - (8)^2}{2}\right][\tan^2(45° - 8.65°)]$$

$$= \frac{18.1}{2}[900 - 64][(0.737)^2] = 4100 \text{ kN}$$

Average shear stress induced at base of d/s slope

$$\tau_{av} = \frac{4100}{44} = 93.2 \text{ kN/m}^2$$

Maximum shear stress induced at $0.6 \times 44 = 26.4$ m away from the d/s toe inwards at point V_1 is given by

$$= \tau_{max} = 1.4 \times 93.2 = 130.4 \text{ kN/m}^2$$

Shear resistance of the foundation soil below the d/s slope portion of dam

Unit shear resistance τ_{f_1} below the toe at point S_1

$$= [c_f + \gamma_f \times h_3 \tan \phi_f]$$

$$= 54 + 18.3 \times 8 \times \tan 12°$$

$$\therefore \quad = 85.1 \text{ kN/m}^2$$

Unit shear resistance τ_{f_2} at point T_1

$$= c_f + \gamma_3 (h + h_3) \tan \phi_f$$

where $$\gamma_3 = \frac{\gamma_{sub\,for\,dam} \times h_2 + \gamma_{dry\,for\,dam} \times (h - h_2) + \gamma_f h_3}{h + h_3}$$

$$= \frac{12 \times 12.5 + 18 \times 9.5 + 18.3 \times 8}{30} = 15.6 \text{ kN/m}^3$$

$$\therefore \quad \tau_{f_2} = 54 + 15.6\,(22 + 8) \times \tan 12° = 153.5 \text{ kN/m}^2$$

The average unit shear resistance developed at foundation level in a length equal to $T_1S_1 = 44$ m, is given by

$$\tau_f = \frac{\tau_{f_1} + \tau_{f_2}}{2} = \frac{85.1 + 153.5}{2} = 119.3 \text{ kN/m}^2$$

Over all F.S. against shear

$$= \frac{\tau_f}{\tau_{av}} = \frac{119.3}{93.2} = 1.28 < 1.5 \qquad \textbf{(Hence, unsafe)}$$

The foundation soil is thus weaker to carry the load and hence the d/s slope will have to be flattened.

Shear resistance at the point of maximum shear, *i.e.* at point V_1 is given as :

$$(\tau_f)_{max} = c_f + (0.6h + h_3)\,\gamma_4 \tan \phi_f$$

$$\gamma_4 = \frac{12 \times 4.5 + 18\,(0.6 \times 22 - 4.5) + 18.3 \times 8}{0.6 \times 22 + 8} = 16.8 \text{ kN/m}^3$$

$$(\tau_f)_{max} = 54 + [0.6 \times 22 + 8]\,16.8 \tan 12° = 129.8 \text{ kN/m}^2$$

$$\text{F.S.} = \frac{[\tau_f]_{max}}{\tau_{max}} = \frac{129.8}{130.4} = 0.995 < 1.0 \qquad \text{(Hence, Unsafe)}$$

The foundation shear and F.S. can also be calculated below the u/s portion of dam soil, in the same manner as has been done for d/s slope portion, if required.

Example 20.5. *Check the stability of the d/s slope of the earthen dam section, given in example 20.4, on a possible slip circle. Determine the factor of safety available*

against such a slip. Also determine the net factor of safety that will be available, if by some how, the soil in the d/s triangular shoulder gets fully submerged. Also compare this net factor of safety with the factor of safety that can be obtained for similar conditions if the analysis was done by shear force determination at the base of the d/s slope.

Solution. The dam section and phreatic line is first of all drawn as shown in Fig. 20.29 (*b*) and as given in previous example. The points *P* and *Q* are located for Fellenius construction such as explained earlier ; the line *PQ* represents the locus of the critical slip circle. A possible slip circular arc passing through the toe of the dam is then drawn with centre at O_1. The soil mass is then divided into slices of 5 metre width. It is found

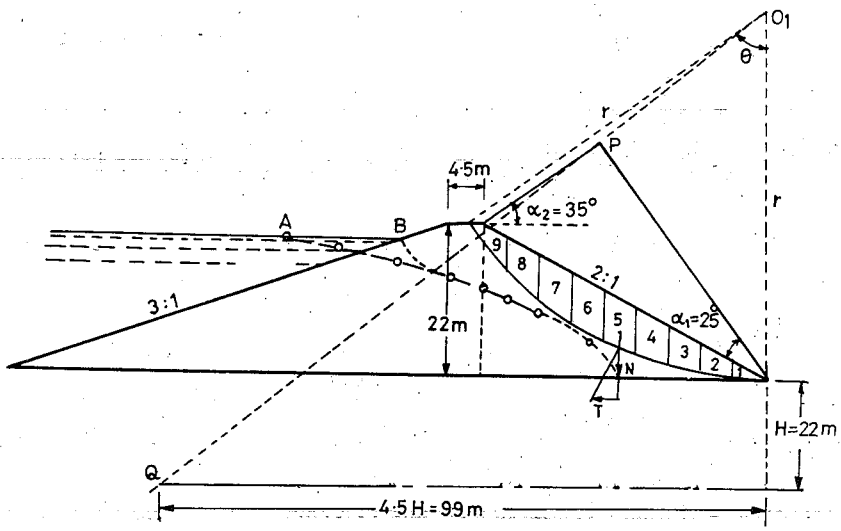


Fig. 20.29 (*b*)

by chance that full 9 slices of 5 m width are accommodated, making thereby the width of the last slice (which was *m.b*) equal to *b* or $m = 1$. By visual observations ; it is also found that the tangenital components (*T*'s) of the weights of all the slices (acting through their centres) happen to be +ve. This is because, the lines of action of all the weights are on one side (*i.e.* left side) of the centre O_1.

The end ordinates of all the slices say $y_1, y_2, y_3, \ldots, y_{n-1}$ or y_8 are measured and found to be 2.5 m, 4.5 m, 6 m, 7 m, 7.5 m, 6.0 m, and 4.5 m respectively.

The *N*-diagram is then drawn as shown in Fig. 20.30 (*a*). The dry density is used because the entire soil in the slices is dry.

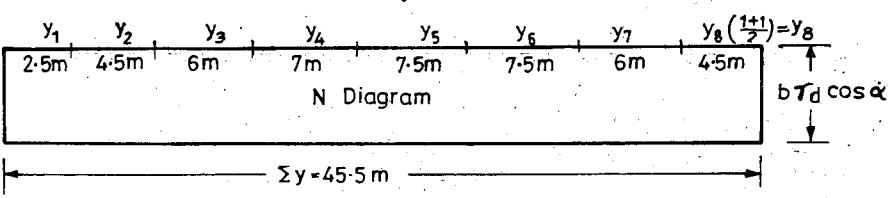


Fig. 20.30 (*a*). *N*-diagram.

Area of N-Diagram $= 45.5 \times 5 \times 18 \times \frac{2}{\sqrt{5}} = 3663$ kN

Similarly, T-Diagram is drawn as shown in Fig. 20.30 (b).

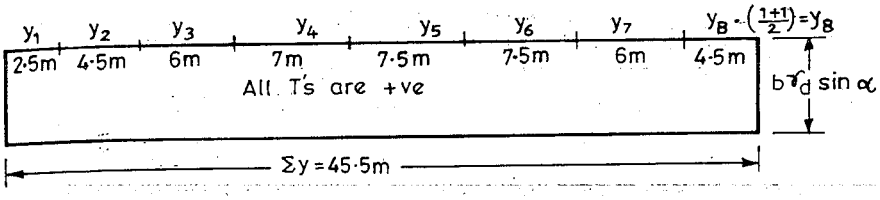


Fig. 20.30 (b). T-diagram.

Area of T-diagram $= 45.5 \times 5 \times 18 \times \frac{1}{\sqrt{5}} = 1831$ kN

Since the slip-circle-arc happens to fall completely above the phreatic line, the pore pressure area is zero ; or area of U- diagram is zero.

$\therefore \quad \Sigma N - \Sigma U = \Sigma N = 3663$ kN.

Now $\quad$ F.S. $= \dfrac{c \cdot \widehat{AB} + (\Sigma N - \Sigma U) \tan \phi}{\Sigma T}$

where $\widehat{AB} = \dfrac{2\pi \cdot r}{360°} \times \theta°$

The angle AO_1B (θ) is measured and found to be 58°. The radius r of the curve AB is also measured and found to be 53.5 m.

$\therefore \quad \widehat{AB} = \dfrac{2\pi \times 53.5}{360°} \times 58° = 54$ m

$c = 24$ kN/m^2 $\quad$ (given in previous example)

Hence $\quad$ F.S. $= \dfrac{24 \times 54 + 3663 \times \tan 25°}{1831} = 1.64 > 1.5$ $\quad$ **(Hence, Safe)**

The corresponding F.S. obtained in example 20.4 from shear analysis was 2.08. Hence, F.S. should be more than 1.5 in slip circle analysis and more than 2 in shear analysis for safe designs.

Note. The F.S. obtained above by slip circle method cannot be called as the critical F.S. or minimum F.S.; because the analysis has been carried out for only one circle. In fact, various other circles should be drawn by taking the centres on line PQ, somewhere near P or away from it. If the circle happens to pass through the foundation soil, then the values of c_f and ϕ_f (*i.e.* c and ϕ for foundation soil) should be used in evaluating W and $c\,\Delta L$ for those particular slices. A tabular form solution, as explained earlier would then be better. When the slices are crossing the phreatic line, U-diagram has to be drawn and evaluated as explained earlier.

Case (b). When soil in the slices get fully submerged, then N's will be calculated on the basis of submerged weights and T s will be calculated on the basis of saturated weights.

$\therefore \quad \Sigma N = 45.5 \times 5 \times \gamma_{sub} \times \cos \alpha$

$= 45.5 \times 5 \times 12 \times \frac{2}{\sqrt{5}} = 244$[illegible]

$$\Sigma T = 45.5 \times 5 \times \gamma_{sat} \sin \alpha$$
$$= 45.5 \times 5 \times 22 \times \sin 25° = 2115 \text{ kN}$$

Net $$\text{F.S.} = \frac{c \cdot AB + \Sigma N \tan \phi}{\Sigma T}$$
$$= \frac{24 \times 54 + 2442 \tan 25°}{2115} = 1.15$$

Net F.S. = 1.15 Ans. < 1.5 (∴ Unsafe)

Hence, if such a possibility of submergence occurs, the dam becomes unsafe.

Horizontal shear at base of d/s slope under complete submergence condition.

$$P_d = \frac{\gamma_{sub} h^2}{2} \tan^2 \left(45° - \frac{\phi}{2}\right) + \gamma_w \frac{h^2}{2}$$
$$= \frac{12 \times (22)^2}{2} \tan^2 (45° - 12.5°) + \frac{9.81 \times (22)^2}{2} = 3553 \text{ kN}$$
$$R_d = c \cdot B_d + \gamma_{sub} (\tfrac{1}{2} \cdot B_d \cdot h) \tan \phi$$
$$= 24 \times 44.0 + 12 \times \tfrac{1}{2} \times 44 \times 22 \times \tan 25° = 3764 \text{ kN}$$
$$\text{F.S.} = \frac{3764}{3553} = 1.06 < 1.5$$ (Hence, Unsafe)

F.S. with slip circle analysis = 1.09
F.S. with horizontal shear analysis = 1.06 **Ans.**

Conclusion. The dam becomes unsafe as soon as the soil in the downstream shoulder gets submerged.

SEEPAGE CONTROL IN EARTH DAMS

The water seeping through the body of the earthen dam or through the foundation of the earthen dam, may prove harmful to the stability of the dam by causing softening and sloughing of the slopes due to development of pore pressures. It may also cause piping either through the body or through the foundation, and thus resulting in the failure of the dam.

20.14. Seepage Control Through Embankments

Drainage filters called 'Drains' are generally provided in the form of (*a*) *rock toe* (*b*) *horizontal blanket* (*c*) *chimney drain*, etc. in order to control the seepage water. The provision of such filters reduces the pore pressure in the downstream portion of the dam and thus increases the stability of the dam, permitting steep slopes and thus affecting economy in construction. It also checks piping by migration of particles. These drains, consist of graded coarse material in which the seepage is collected and moved to a point where it can be safely discharged. In order to prevent movement of the fine material from the dam into the drain, the drain or filter material is graded from relatively fine on the periphery of the drain to coarse near the centre. A multi-layered filter, generally called **inverted filter** or *reverse filter* is provided as per the criteria suggested by Terzaghi for the design of such filters.

The various kinds of drains, which are commonly used are shown and described below :

20.14.1. Rock Toe or Toe Filter [Fig. 20.31 (*a*)]. The 'rock toe' consists of stones of size usually varying from 15 to 20 cm. A toe filter (graded in layers) is provided as a transition zone, between the homogeneous embankment fill and rock toe. Toe filter

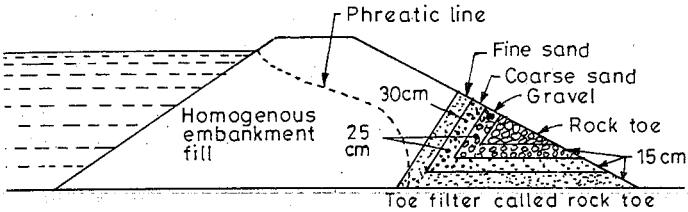


Fig. 20.31 (*a*). Rock Toe.

generally consists of three layers of fine sand, coarse sand, and gravel ; as per the filter criteria requirements. The height of the rock toe is generally kept between 25 to 35% of reservoir head. The top of the rock toe must be sufficiently higher than the tail water depth, so as to prevent the wave action of the tail water.

20.14.2. Horizontal Blanket or Horizontal Filter. [Fig. 20.31 (*b*) and (*c*)]. The horizontal filter extends from the toe (d/s end) of the dam, inwards, upto a distance varying from 25 to 100% of the distance of the toe from the centre line of the dam. Generally, a length equal to three times the height of the dam is sufficient. The blanket should be properly designed as per the filter criteria, and should be sufficiently pervious to drain off effectively.

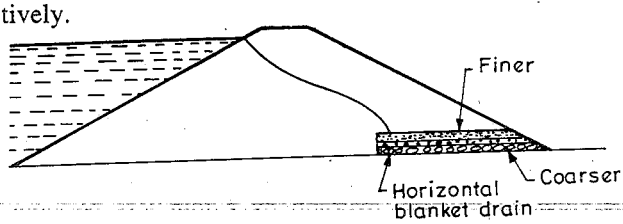


Fig. 20.31 (*b*). Horizontal Filter.

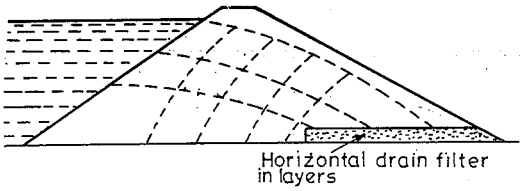


Fig. 20.31 (*c*). Inefficient 'Horizontal drain' in stratified embankments.

20.14.3. Chimney Drain. [Fig. 20.31 (*d*)]. The horizontal filter, not only helps in bringing the phreatic line down in the body of the dam but also provides drainage of the foundation and helps in rapid consolidation. But, the horizontal filter tries to make the soil more pervious in the horizontal direction and thus causes stratification. When large scale stratification occurs, such a filter becomes inefficient as shown in Fig. 20.30 (*c*). In such a possible case, a vertical filter (or inclined u/s or d/s) is placed along with the horizontal filter, so as to intercept the seep-

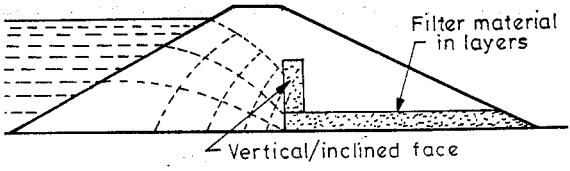


Fig. 20.31 (*d*). 'Chimney Drain' in Stratified Embankments.

ing water effectively, as shown in Fig. 20.31 (*d*). Such an arrangement is termed as *chimney drain.* Sometimes a horizontal filter is combined and placed along with a rock toe, as shown in Fig. 20.31 (*e*).

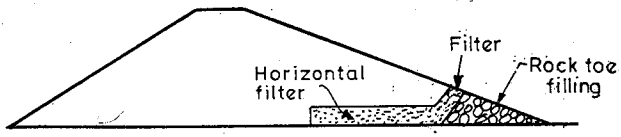


Fig. 20.31 (*e*). Horizontal filter combined with rock toe.

20.15. Seepage Control Through Foundations

The amount of water entering the pervious foundations, can be controlled by adopting the following measures :

20.15.1. Impervious Cutoffs. Vertical impervious cutoffs made of concrete or sheet piles may be provided at the upstream end (*i.e.* at heel) of the earthen dam (Fig 20.32). These cutoffs should, generally, extend through the entire depth of the pervious foundation, so as to achieve effective control on the seeping water. When the depth of the pervious foundation strata is very large, a cutoff, up to a lesser depth, called a *partial cutoff* may be provided. Such a cutoff reduces the seepage discharge by a smaller amount. So much so, that a 50% depth reduces the discharge by 25%, and 90% depth reduces the discharge by 65% or so.

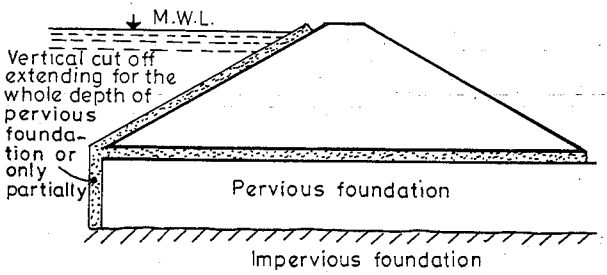


Fig. 20.32

20.15.2. Relief Wells and Drain Trenches. When large scale seepage takes place through the pervious foundation, overlain by a thin less pervious layer, there is a possibility that the water may boil up near the toe of the dam, as shown in Fig. 20.33 (*a*).

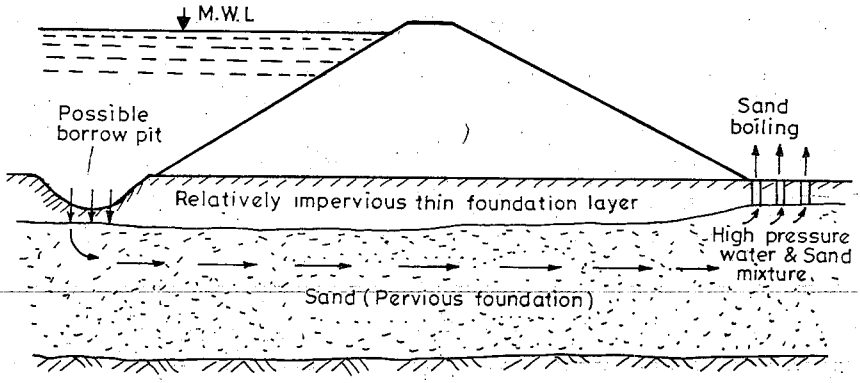


Fig. 20.33 (*a*). Sand Boiling Phenomenon.

Such a possibility, can be controlled by constructing relief wells or drain trenches through the upper impervious layer, as shown in Fig. 20.33 (*b*) and (*c*), so as to permit escape of seeping water. The possibility of sand boiling may also be controlled by

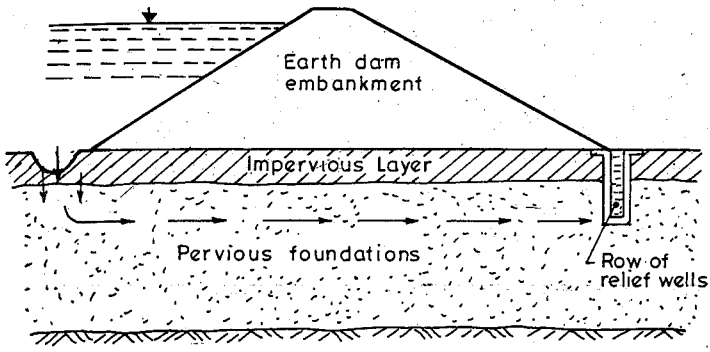


Fig. 20.33 (*b*). Provision of Relief Wells.

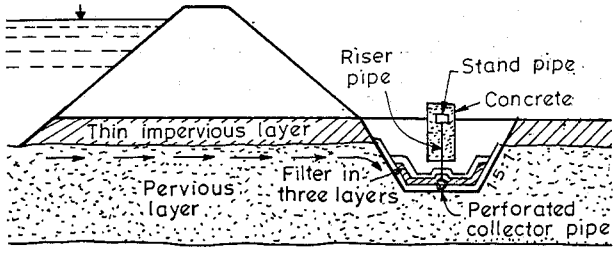


Fig. 20.33 (*c*). Enlarged View of Drain Trench.

providing d/s berms beyond the toe of the dam as shown in Fig. 20.33 (*d*). The weight of the overlying material, in such a case, is sufficient to resist the upward pressure and thus preventing the possibility of sand boiling. The provision of such berms, also protects the d/s toe from possible sloughing due to seepage.

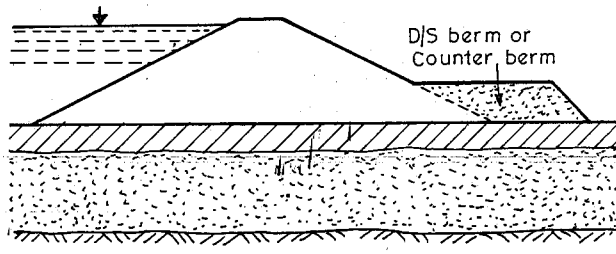


Fig. 20.33 (*d*). Provision of d/s Berms.

20.16. Design of Filters

The drainage filters must be designed in such a way that neither the embankment nor the foundation material can penetrate and clog the filters. The permeability or size of filter material should also be sufficient to carry the anticipated flow with an ample margin of safety. A rational approach to the design of filters has been provided by Terzaghi. According to him, the following filter criteria should be satisfied.

$$\frac{D_{15} \text{ of filter}}{D_{85} \text{ of base materials}} < 4 \text{ to } 5 < \frac{D_{15} \text{ of filter}}{D_{15} \text{ of base material}} \qquad ...(20.50)$$

The embankment soil or the foundation soil surrounding the filter, is known as base material.

When the ratio of D_{15} of filter to D_{85} of base material does not exceed 4 to 5, base material is prevented from passing through the pores of the filter. Similarly, when the ratio of D_{15} of filter to D_{15} of base material is more than 5 (between 5 to 40), the seepage forces within the filter are controlled up to permissible small magnitudes.

Multilayered filters (generally 3 layers) consisting of materials of increasing permeabilities from the bottom to top are, many a times, provided and are known as **inverted filters.** These filters are costly and should be avoided where possible. The minimum total thickness of filter is 1 m. However, if sufficient quantities of filter material are available at reasonable costs, thicker layers of filter may be provided. The thicker the layer, the greater the permissible deviation from the filter requirements.

20.17. Slope Protection

20.17.1. Protection of Upstream Slope. The upstream slope of the earth dam is protected against the erosive action of waves by stone pitching or by stone dumping, as shown in Fig. 20.34. The thickness of the dumped rock should be about I metre and should be placed over a gravel filter of about 0.3 m thickness. The filter prevents the washing of fines from the dam into the rip-rap. The provision of such a dumped rip-rap has been found to be most effective and has been found to fail only in 5% cases.

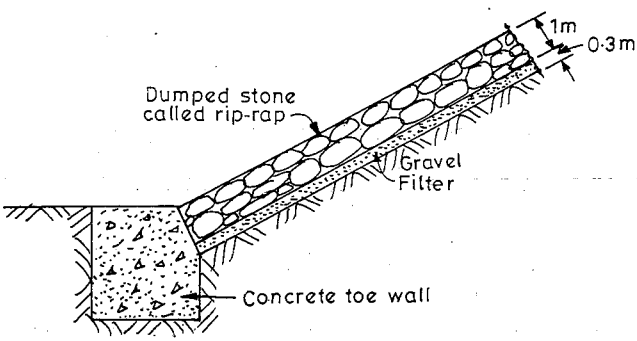


Fig. 20.34

The stone pitching, *i.e.* the hand packed rip rap requires a lesser thickness and may prove more economical if suitable rock is available only in limited quantity. However, when provided in smaller thickness (*i.e.* single layer), it is more susceptible to damage and has been founded to fail in about 30% of cases.

Concrete slabs may also be laid over the u/s slope of the earth dam. When such slabs are constructed, they must be laid over a filter and weep holes should be provided so as to permit escape of water when the reservoir is drawn down. If the filter is not provided, the fines from the embankment may get washed away from the joints creating hollows beneath the slab and causing the consequent cracking and failure of the slab under its own weight. Concrete slab protections have been found to fail in about 36% cases, mainly because of non-providing of filter below them.

20.17.2. Protection of Downstream Slope. The downstream slope of the earthen dam is protected against the erosive action of waves upto and slightly above the water depth, in a similar manner as is explained above for u/s slope.

Moreover, the d/s slope should be protected against the erosive action of rain and its run-off by providing horizontal berms at suitable intervals say about 15 m or so (Fig. 20.5) so as to intercept the rain water and discharge it safely. Attempts should also be made so as to grass and plant the d/s slopes, soon after their construction.

20.18. Rockfill Dams

Rockfill dams have characteristics lying somewhere between the characteristics of gravity dams and those of earthen dams. In other words, they are less flexible than earthen dams and more flexible, then gravity dams. Their foundation requirements are not as strict and rigid as are required for gravity dams. But the foundation requirements

are more rigid than those for earthen dams which can be constructed almost on any types of foundations. The steeper slopes are used in rockfill dams and hence, the base width is quite less. The smaller base width and the possibility of large scale seepage restricts the foundation requirements of such dams.

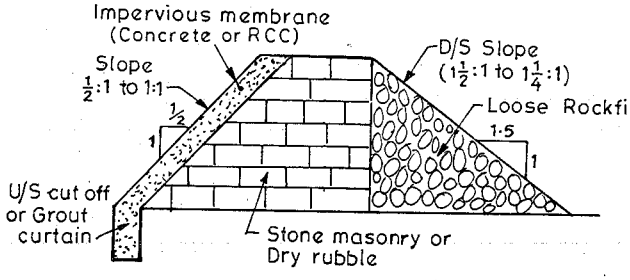


Fig. 20.35. Section of a rockfill dam.

A typical section of a rockfill dams is shown in Fig. 20.35. It essentially consists of an impervious membrane and embankment supporting the membrane. The embankment is divided into two portions. The u/s portion is made of stone masonry or dry rubble, and the d/s portion is made of loose rockfill. The u/s portion embankment supports the membrane and the water load : while the d/s portion supports the u/s embankment, membrane and waterload.

The impervious membrane is usually of concrete, with expansion joints at suitable intervals (say 10 m or so). The expansion joints may be filled with suitable bitumen filler to minimise leakage. Sometimes, R.C.C. membranes are provided with sufficient horizontal and vertical reinforcements, without any expansion joints. The slab thickness is generally less at the top and is more near the dam bottom. The usual slab thickness is between 15 to 50 cm, depending upon the design.

The upstream face containing the membrance, is sloped at a slope varying from say $\frac{1}{2}$: 1 ($\frac{1}{2}H$: 1V) in low dams up to height of about 60 m or so, to 1 : 1 or $1\frac{1}{2}$: 1 in high dams. The d/s slope of all the rockfill dams is kept at about 1.3 : 1, which represents the angle of repose of the rockfill.

Rockfill dams are generally cheaper than concrete dams and can be constructed rapidly if proper rock is available. The rock must be strong enough to withstand high intensity leadings even when wet. The size of loose rock may vary from small stones to 3 m or so. Rockfill dams are very useful in seismic regions, as they provide high resistance to seismic forces because of their flexible character. However, rockfill dams are liable to large settlements, which may lead to cracking of concrete membrane. Repairs for the membrane are, therefore, undertaken from time to time, as and when the necessity arises.

PROBLEMS

1. (*a*) What are 'earthen dams' and under what circumstances are they preferred ?

(*b*) Enumerate the different types of earthen dams, and draw neat sketches showing each type.

[**Hint** : See article 20.2]

(*c*) Enumerate the two different methods which are adopted for constructing earthen dams. Which of these methods would you prefer and why ? [**Hint** : See article 20.3, preferring rolled fill method]

2. (*a*) What is meant by 'pore water pressure' : and what is its significance in the design of earthen dams?

(*b*) What are 'rockfill dams' and what are their advantages over earthen dams. Draw a neat sketch showing the cross section of a rockfill dam.

3. (*a*) Explain in detail the various forces causing instability in a gravity dam.

(*Madras University, 1975*)

(*b*) Draw a section of an earth dam of 20 m height indicating the various parts of the dam.

(*Madras University, 1975*)

4. (*a*) What are the different types of earth dams that are usually adopted. State where each type is adopted ?

(*b*) What are the causes of failures of earth dam ? (*Madras University, 1974*)

5. What are the precautions that you would take while constructing an earth dam ?

Explain the Swedish slip circle method of analysing the stability of an earth dam slopes.

(*Madras University, 1976*)

6. (*a*) Draw a neat section of an earth dam for the following site :

(*b*) Both silty clay and coarse sand are available at site. Hard stratum is available at about 5 m below the natural ground.

(*c*) Explain briefly how the stability of earthen slopes are checked by the slip circle method.

(*Madras University, 1973*)

7. (*a*) Differentiate between 'Rigid dams' and 'Non-rigid dams' giving examples of each type.

(*b*) Explain with neat sketches how you would carry out the stability analysis of an earth dam.

(*U.P.S.C., 1974*)

8. (*a*) Differentiate between 'horizontal' and 'vertical' piping in earth dams. Suggest permanent measures to check vertical piping.

(*b*) Show with the help of simple outline sketches suitable broad designs of earth dams for different available materials and the governing geology of the site.

Note. At least four different designs should be given.

9. (*a*) Give a suitable design for a 50 m high dam for site where both clay silt and sand gravel are available in plenty and where foundation is pervious to a depth of 10 m. Assume suitable data. Give reasons favouring the suggested design.

10. (*a*) Briefly discuss the checks that are required to be made to investigate the stability of an earthen dam. (*U.P.S.C., 1975*)

(*b*) An earthen dam has to be constructed to store a maximum depth of 12 m of water over river bed consisting of coarse sand and gravel up to a depth of 3 m below river bed followed thereafter by bard and sound rock. Clayee soil is available in plenty in the vicinity of the river. Draw and detail a suitable section of the dam at the river bed.

11. Explain how the following parameters affect design of an earth dam :

(*i*) optimum moisture content ;

(*ii*) C and ϕ value of soil ; permeability of soil

(*iii*) sudden draw-down of the reservoir.

Illustrate with neat sketches the following parts of an earthen dam and state their functions briefly:

(*i*) Rock toe ;

(*ii*) horizontal drainage blanket ;

(*iii*) cut-off ;

(*iv*) Rip-rap.

12. Enumerate and explain by neat sketches the different ways by which the earthen dams may fail. Also suggest suitable precautions that should be undertaken to avoid each type of failure.

13. (*a*) Explain the meaning and importance of equipotential lines' and 'stream lines' in connection with seepage analysis of earthen dams.

(*b*) Derive an expression for calculating the seepage discharge through earthen dam bodies made of isotropic soils. What correction factor is required to be multiplied with this expression, if the dam soil is non-isotropic.

14. (*a*) Define and explain the term 'phreatic line' in earthen dams.

(*b*) How would you procced to determine the phreatic line through homogeneous earthen dams provided :

(*i*) with a horizontal filter ;

(*ii*) Without a horizontal filter.

15. (*a*) Explain and elaborate the importance of 'seepage' through earthen dams.

(*b*) What precautions and remedial measures would you undertake to control the 'seepage' through

(*i*) earthen dam body ;

(*ii*) through the dam foundation.

16. Write short notes on any five of the followings :

(*i*) Rock toe

(*ii*) Climney drain

(*iii*) Relief wells

(*iv*) Slope protection in earthen dams

(*v*) Rock fill dams

(*vi*) Consolidation of earthen dams

(*vii*) Pore pressure and its significance in relation to earthen dam construction

(*viii*) 'Seepage failures' of earthen dams

(*ix*) Rational design of drainage filters for earthen dams.

17. A section of a homogeneous earth dam is shown in Fig. 20.36. Calculate the seepage discharge per metre length, through the body of the dam. The coefficient of permeability of the dam material may be taken as 8×10^{-5} m/sec.

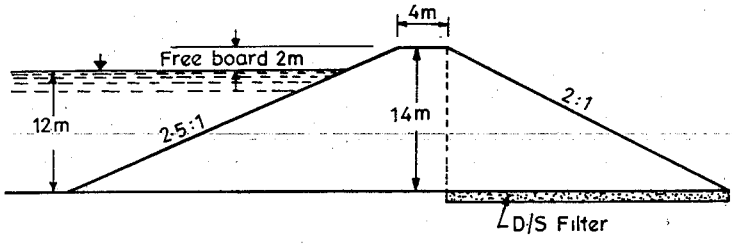


Fig. 20.36

[Ans. 2.88×10^{-1} cumecs/m]

18. An earthen dam made of homogeneous material has the following data :

(A) Hydraulic data of dam

Level of top of dam	= 210 m.
Level of deepest river bed	= 192 m.
HFL of reservoir	= 208 m.
Width of top of dam	= 10 m.
Upstream slope	= 3 : 1
Downstream slope	= 2.5 : 1
Length of horizontal filter from d/s toe inwards	= 16 m.

(B) Properties of the material of the dam

Dry density	= 18 kN/m^3
Saturation density	= 21 kN/m^3
Average angle of friction	= 30°
Average cohesion	= 16 kN/m^2.

(C) The foundation soil consists of 4 m thick layer of soil having the following properties :

Average unit weight $= 17\ kN/m^3$

Average cohesion $= 54\ kN/m^2$.

Average angle of internal friction $= 7°$

Check the dam section for the following :

(*i*) Sloughing of u/s slope during sudden drawdown.

(*ii*) Stability of foundation against shear.

[**Hint.** Follow example 20.4]

19. In order to determine the factor of safety of the d/s slope during steady seepage, the section of a homogeneous earth dam was drawn to scale of 1 cm = 10 m ; and the following results were obtained on a trial slip circle.

Area of N-diagram = 12.15 sq cm.

Area of T-diagram = 6.50 sq cm.

Area of U-diagram = 4.02 sq cm.

Length of arc = 11.60 cm.

The dam material has the following properties :

Effective angle of internal friction $= 26°$

Unit of cohesion $= 19.62\ m^2$

Unit weight of soil $= 19.62\ m^2$

Determine the factor of safety of the slope. [**Ans.** F.S. 1.21]

21

Spillways, Energy Dissipators, and Spillway Gates

21.1. Introduction

A spillway is a structure constructed at a dam site, for *effectively disposing of the surplus water from upstream to downstream.* Just after the reservoir gets filled up, up to the normal pool level, water starts flowing over the top of the spillway crest (which is generally kept at normal pool level). Depending upon the inflow rate, water will start rising above the normal pool level, and at the same time, it will be let off over the spillway. The water can rise over the spillway crest, upto the maximum reservoir level, which can be estimated from the inflow flood hydrograph and the spillway characteristics, by the process of flood routing, explained earlier. Therefore, it is only the spillway, which will dispose of the surplus water and will not let the water rise above the maximum reservoir level. Had there been no such structure, over which the water would have overflown, the water level must have exceeded maximum reservoir level, and ultimately would have crossed the freeboard and thus overtopped the dam, causing the failure of the dam. *Hence, a spillway is essentially a safety valve for a dam.* It must be properly designed and must have adequate capacity to dispose of the entire surplus water at the time of the arrival of the worst design flood.

Many dams have failed (especially the earthen dams) because of the improperly designed or inadequate spillways.

21.2. Location of a Spillway

A spillway can be located either within the body of the dam, or at one end of it or entirely away from it, independently in a saddle. If a deep narrow gorge with steep banks, separated from a flank by a hillock with its level above the top of the dam (such as shown in Fig. 21.1), is available, the spillway can be best built independently of the dam.

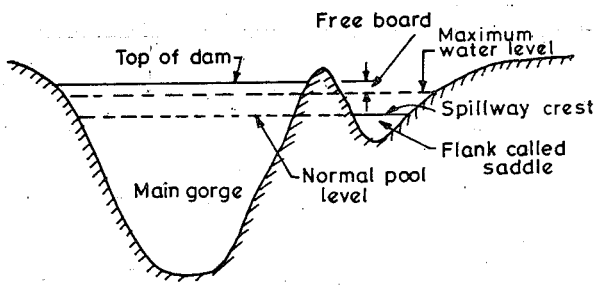


Fig. 21.1

Under such circumstances, a concrete or an earthen dam can be constructed across the main valley and a spillway can be constructed independently into the saddle. Sometimes, a concrete or a masonry dam along with its spillway can be constructed in the main valley, while the flank or flanks are closed by earthen dikes or embankments. The

top level of such an embankment is kept at maximum reservoir level. The materials and designs of these embankments are such that they fail as soon as water overtops them. Hence, if by chance, either due to excessive flood above the design flood or due to failure of gates of main spillway, etc., the water rises above the maximum reservoir level, it shall overtop such embankment, which at once fails ; providing sufficient outlet for the disposal of excessive water. This type of a secondary safety arrangement is generally provided on large dams especially on earth and rockfill dams, and is *known as Subsidiary Spillway* or *Emergency Spillway* or *Breaching Section.*

The main spillway is constructed to dispose of the designed flood above the normal pool level and upto the maximum reservoir level. It is situated either within the dam, or at one end of it, or independently in a saddle away from the main dam. A separate independent spillway is generally preferred for earthen dams, although due to non-availability of sites, a concrete spillway is sometimes constructed within or at one of the ends of an earth dam. If the main spillway is situated in a flank, a secondary emergency spillway may be situated in another flank as shown in Fig. 21.2.

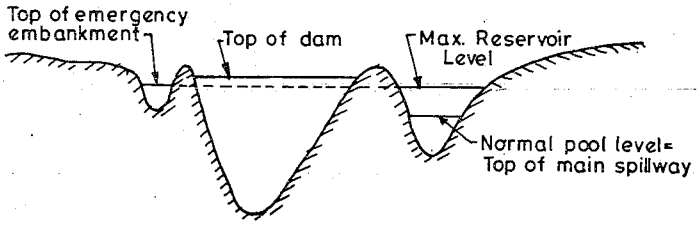


Fig. 21.2

21.3. Design Considerations for the Main Spillway

The main spillway, often called the spillway, is properly designed so as to dispose of the excess water without causing any damage to the dam, or to any of its appurtenant structures. The spillway structure should be structurally and hydraulically adequate and must not give way under worst and variable loading conditions.

The required discharging capacity of the spillway should be as closely estimated as possible. The underestimation will lead to overtopping of the main dam and its consequent damages ; while the over estimation will lead to unnecessarily costly constructions which shall never he utilised during the life of the dam, and hence, will remain a waste investment. However, on large dams, a conservative view is always preferred because the failure of a single dam due to inadequate capacity may result in the loss of numerous human lives to which no cost allocation can be made. Moreover, an emergency spillway or a breaching section is generally provided, the failure of which under necessary circumstances, may though cause serious erosion on the downstream, but shall protect the main dam from failure.

The water passing over the spillway and falling on the downstream side must not be allowed to erode the downstream soil, and hence, arrangements must be made for effectively dissipating the energy of the falling water.

21.4. Controlled and Uncontrolled Spillways

The flow of water over a spillway may be controlled by installing gates over the spillway crest. In such a case, the spillway is known as a controlled spillway. The out flow can be controlled in such spillways and hence, preferred in modern days. However, some spillways are left just by constructing their crest at normal pool level. As water

will flow over such a spillway, depending upon the reservoir level and the corresponding head over the spillway, such uncontrolled spillways are guided only by the available water head, and hence, are called uncontrolled spillways. The comparative advantages and disadvantages of controlled and uncontrolled reservoirs have been discussed in chapter 18.

VARIOUS TYPES OF SPILLWAYS

Depending upon the type of the structure constructed for disposing of the surplus water, the spillways can be of the following major types :

(1) Straight Drop Spillway.

(2) Overflow Spillway generally called *Ogee Spillway.*

(3) *Chute Spillway* often called *Trough Spillway* or Open channel Spillway.

(4) Side Channel Spillway.

(5) Shaft Spillway.

(6) Syphon Spillway.

The various types of spillways enumerated above are described below along with the design details of 'Ogee Spillway' and 'Chute Spillway'.

21.5. Straight Drop Spillway or Overfall Spillway

This is the simplest type of spillway and may be constructed on small bunds or on thin arch dams, etc. It is a low weir and simple vertical fall type structure, as shown in Fig. 21.3. The downstream face of the structure may be kept vertical or slightly inclined. The crest is sometimes extended in the form of an overhanging lip, which keeps small discharges away from the face of the overfall section. The water falls freely from the crest under the action of gravity. Since vacuum gets created in the underside portion of the falling jet, sufficient ventilation of the nappe is required in order to avoid pulsating and fluctuating effects of the jet. The design of such a spillway is done as that of a weir which was explained in the chapter on

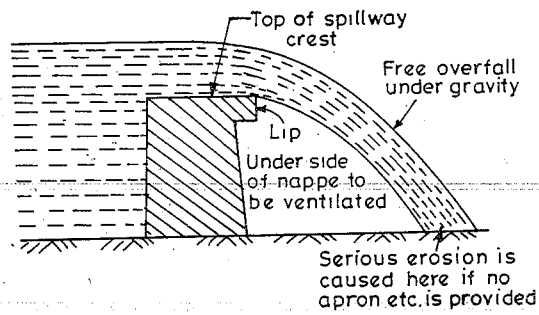


Fig. 21.3. (*a*) Straight drop spillway without d/s protection.

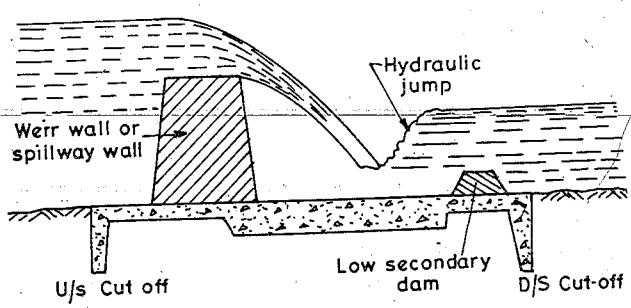


Fig. 21.3. (*b*) Straight drop spillway with d/s protection works.

weirs. Sometimes, a secondary dam of low height is constructed on the downstream side to create an artificial pool of water so as to dissipate the energy of the falling water.

21.6. Ogee Spillway or Overflow Spillway

Ogee spillway is an improvement upon the 'free overfall spillway, and is widely used with concrete, masonry, arch and buttress dams. Such a spillway can be easily used on valleys where the width of the river is sufficient to provide the required crest length

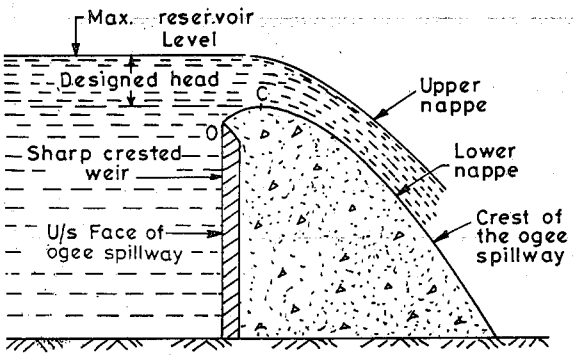


Fig. 21.4. (*a*) Section of an ogee spillway with vertical u/s face.

and the river bed below can be protected from scour at moderate costs. *The profile of this spillway is made in accordance with the shape of the lower nappe of a free falling jet, over a duely ventilated sharp crested weir,* as shown in Fig. 21.4 (*a*).

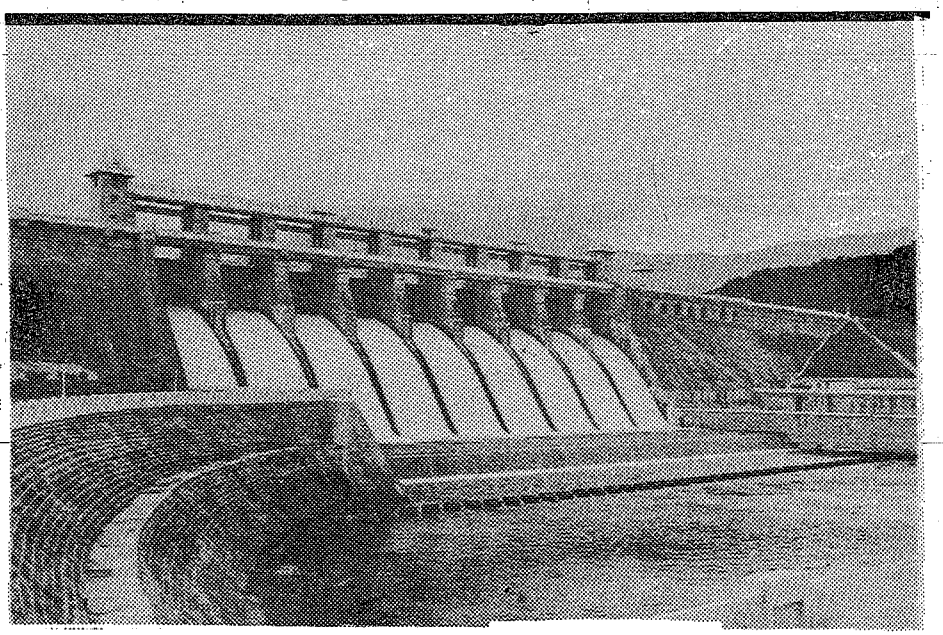


Fig. 21.4. (*b*) Photoview of an Ogee Spillway of Amaravathi dam (earthern cum masonry dam) located on river Amaravathi (a tributary of river Cauvery) in Coimbatore District in Tamil Nadu state.

The shape of the lower nappe of freely falling jet over a sharp crested weir can be determined by the principle of projectile. It generally rises slightly (to point *C*) as it originates from the crest (*O*) of a sharp crested weir and then falls to make a parabolic form. Now, if the space between the sharp crested weir and the lower nappe is filled with concrete or masonry, the weir so formed will have a profile similar to an 'ogee' (*S*-shaped curve in section), and hence called an 'ogee weir' or an 'ogee spillway'. This lower nappe, will then become the crest of the spillway. Since the lower nappe of the free falling jet will be different for different heads over the crest of the sharp crested weir, *the profile of the ogee weir is generally confined to the lower nappe that would be obtained for maximum head over the spillway (i.e. upto the maximum reservoir level).*

In a free overfall spillway, the water jet falls clearly away from the face of the spillway, and the gap between the jet and the face is kept ventilated. While in an ogee spillway, the falling water glides over the curved profile of the spillway, and there is no space between water and crest of the spillway, under normal design conditions.

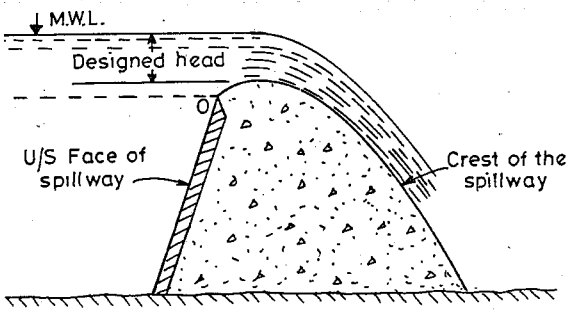


Fig. 21.5. Ogee spillway with inclined u/s face.

Normally, the upstream face of the spillway is kept vertical and the crest shape confirms to the lower nappe of a vertical sharp crested weir under maximum head. But if the upstream face of the spillway is kept sloping, the crest shape should also confirm to the lower nappe that would be obtained for an inclined sharp crested weir (Fig. 21.5).

21.6.1. Cavitation. The crest of the ogee spillway can be made to confirm only to one particular nappe that would be obtained at one particular head. This head is called the *designed head* and represented by, say, H_d. But in practice, the actual head of water on the spillway crest, called the *operating head,* may be less or more than the designed head. *If this operating head on the spillway is more than the designed head, the lower nappe of the falling jet may leave the ogee profile, thereby generating negative pressure at the point of separation.* The generation of vacuum or negative pressure (*i.e.* pressure below the atmospheric pressure) may lead to formation of bubbles or cavities in the water. These cavities or bubbles filled with air, vapour and other gases are formed in a liquid, whenever the absolute pressure (*i.e.* atmospheric pressure—vacuum pressure) of the liquid is close to its vapour pressure, so as to commence evaporation. Such a condition may arise when the head of water is more than the designed head and the consequent high velocity jet causes reduced pressures or negative pressures in the lower region of the water jet.

Such cavities, on moving downstream, may enter a region where the absolute pressure is much higher (*i.e.* more vacuum). This causes the vapour in the cavity to condense and return to liquid with a resulting implosion or collapse of the cavity. When

the cavity collapses, extremely high pressures are generated. The continuous bombardment of these implosions will thus take place near the surface of the spillway, causing fatigue failure of its material. The small particles of concrete or masonry are thus broken away, causing formation of pits on its surface and giving the surface a spongy appearance. This damaging action of cavitation is called 'pitting'. The cavitation plus the vibrations from the alternate making and breaking of contact between the water and face of the spillway, may thus result in serious structural damages to the spillway crest. *Hence, it can be concluded that if the head of water over the spillway is more than the designed head, cavitation may occur. On the other hand, if the head of water over the spillway is less than the designed head, the falling jet would adhere to the crest of the ogee spillway, creating positive hydrostatic pressures and thereby reducing the discharge coefficient of the weir.*

21.6.2. Designing the Crest of the Ogee Spillway. The ogee spillways were being designed in the earlier periods, in accordance with the theoretical profile obtained for the lower nappe of a free falling jet. The profile was known as Bazin's profile. Theoretically, the adoption of such a profile, should cause no negative pressures on the crest under designed head. But in practice, there exists a lot of friction due to roughness on the surface of the spillway. Hence, negative pressure on such a profile seems inevitable. The presence of negative pressure causes the danger of cavitation and sometimes fluctuations and pulsations of the nappe. Hence, while adopting a profile for the spillway crest, the avoidance of negative pressures must be an objective along with consideration of other factors such as practicability, hydraulic efficiency, stability and economy. Depending upon research work based on these objectives, various modified profiles have been proposed these days.

Several standard ogee shapes have been developed by U.S. Army Corps of Engineers at their Waterways Experimental Station (WES). Such shapes are known as 'WES Standard Spillway Shapes'. The d/s profile can be represented by the equation

$$x^n = K \cdot H_d^{n-1} \cdot y \qquad \text{...(21.1)}$$

where (x, y) are the co-ordinates of the points on the crest profile with the origin at the highest point C of the crest, called the apex.

H_d is the *design head including the velocity head.*

K and n are constants depending upon the slope of the upstream face. The values of K and n are tabulated in Table 21.1.

Table 21.1

Slope of the u/s face of the spillway	K	n
Vertical	2.0	1.85
1 : 3 (1H : 3V)	1.936	1.836
1 : $1\frac{1}{2}$ (1H : $1\frac{1}{2}V$)	1.939	1.810

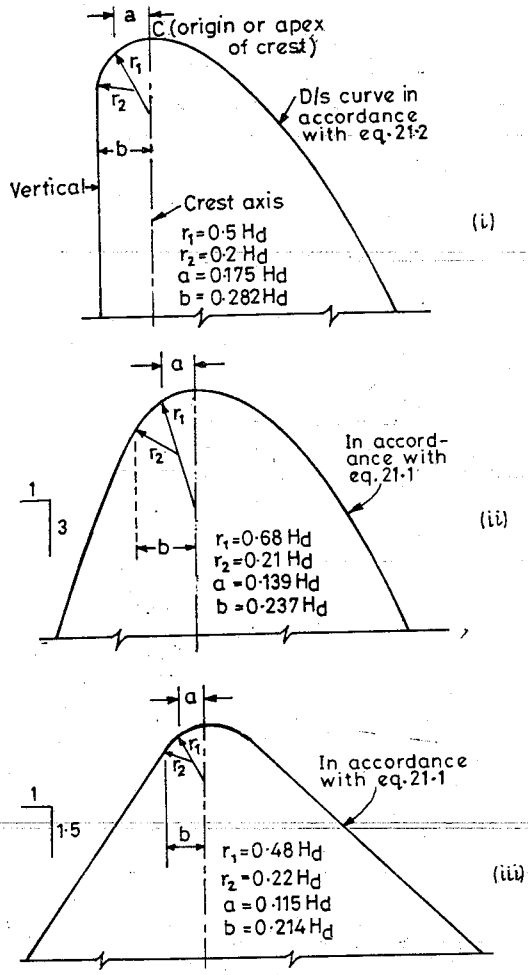


Fig. 21.6. WES profiles for Ogee Spillways for different u/s slopes.

Thus, for a spillway having a vertical u/s face, the d/s crest is given by the equation

$$x^{1.85} = 2 \cdot H_d^{0.85} \cdot y \qquad ...(21.2)$$

Different upstream curves were given by WES for different slopes, as shown in Fig. 21.6.

According to the latest studies of U.S. Army Corps, the u/s curve of the ogee spillway having a vertical u/s face, should have the following equation :

$$y = \frac{0.724\,(x + 0.27\,H_d)^{1.85}}{H_d^{0.85}} + 0.126\,H_d - 0.4315\,H_d^{0.375} \times (x + 0.27\,H_d)^{0.625} \qquad ...(21.3)$$

The u/s profile extends up to $x = -0.27\,H_d$

Co-ordinates for the upper nappe for various WES shapes of ogee spillway are also available and can be utilised in the design of training walls and spillway bridges, etc.

The profile for an ogee spillway, having a vertical upstream face, can be determined on the basis of its WES profile, or Table 21.2 may be used for making the ogee profile.

Table 21.2. Table for marking Ogee profile

$\frac{x'}{H_d}$	$\frac{y}{H_d}$	
	Lower Nappe	*Upper Nappe*
0	– 0.125	0.831
0.10	– 0.033	0.807
0.25	– 0.000	0.763
0.50	– 0.034	0.668
0.75	– 0.129	0.539
1.0	– 0.283	0.373
1.5	– 0.738	– 0.088
2.0	– 1.393	– 0.743
3.0	– 3.303	– 2.653
4.0	– 6.013	– 5.363
5.0	– 9.523	– 8.873

(x', y) are the co-ordinates of a point on the profile as shown in Fig. 21.7.

The co-ordinates of the lower nappe determine the crest profile, while the plotting of the upper nappe is useful in determining the clearance for the spillway deck bridge and the top levels of training walls on the sides of the spillway.

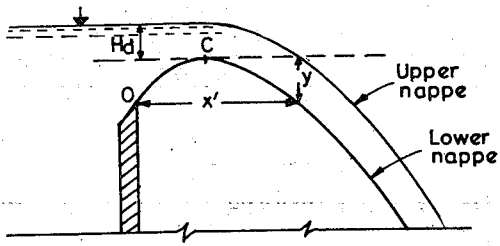


Fig. 21.7

After having plotted most of the profile of the ogee spillway, a smooth gradual reverse curvature is provided at the bottom of the downstream face, as shown in Fig. 21.8. The reverse curve turns the flow into the apron of a stilling basin or into the spillway discharge channel. A radius of about one-fourth of the spillway height is satisfactory for this reverse bottom curve.

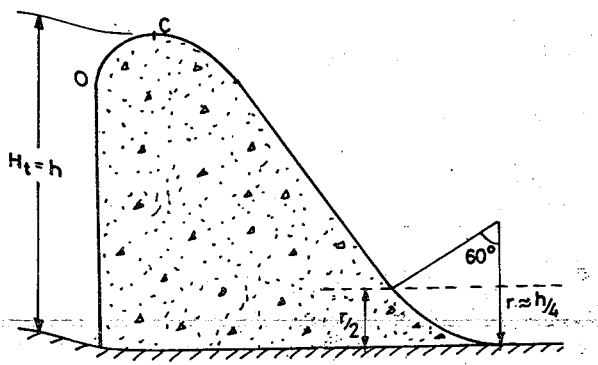


Fig. 21.8

21.6.3. Practical Profile Consistent with the Profile of Gravity Dam. When the profile for the crest of the ogee spillway is plotted over the triangular profile of the section of a gravity dam (non-overflow section), it is found that it goes beyond the downstream face of the dam, thus requiring thickening of the section for the spillway (Fig. 21.9). This extra concrete required, can be saved by shifting the curve of the nappe in a backward direction until this curve becomes tangential to the downstream face of the dam (Fig. 21.10). Hence, a saving can thus be affected, by providing a projecting corbel on the upstream face of the spillway section (Fig. 21.10). The construction of the spillway is thus carried out as if it was a non-overflow dam. Only the slight modifications are made after reaching the required height (up to O) at

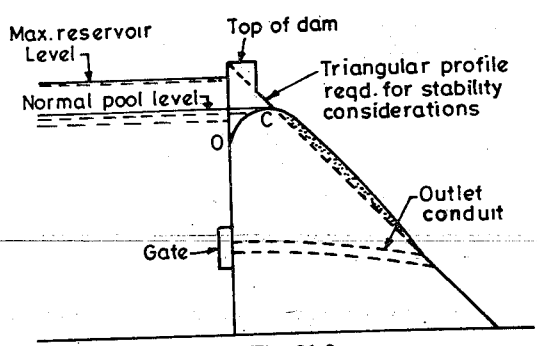


Fig. 21.9

which corbel is provided, and a smooth required curve *OCA* is given, as shown in Fig. 21.10.

If the spillway is provided with gates on the upstream face to control the flow in outlet conduits (Fig. 21.9), then the corbel will interfere with gate operation and hence the above concrete saving cannot be affected.

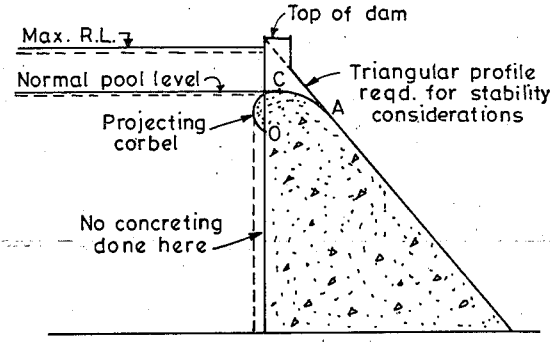


Fig. 21.10. Provision of Corbel.

The structural design and stability requirements of ogee spillway are exactly the same as that of a gravity dam. The forces acting on a gravity dam also act on the ogee spillway and remain predominant. The pressure exerted on the crest of the spillway by the flowing water and the drag force caused by the fluid-friction are usually negligible as compared to other forces which are acting on a gravity dam and also act on an ogee spillway. Hence, the design and construction of the ogee spillway is consistent with that of the gravity dam. Due to this reason, the spillway is sometimes called overflow portion of the dam, and the real dam section is termed as non-overflow section of the dam.

However, the change in momentum of the flow in the vicinity of the reverse curve may create a dynamic force on the spillway, in addition to the forces acting on the gravity dam. This force must be considered in designing bottom of the d/s face of spillway or the bucket type energy dissipator. This is dealt separately while discussing the design of energy dissipators for the spillways.

21.6.4. Discharge Formula for the Ogee Spillway. The discharge passing over the ogee spillway is given by the equation :

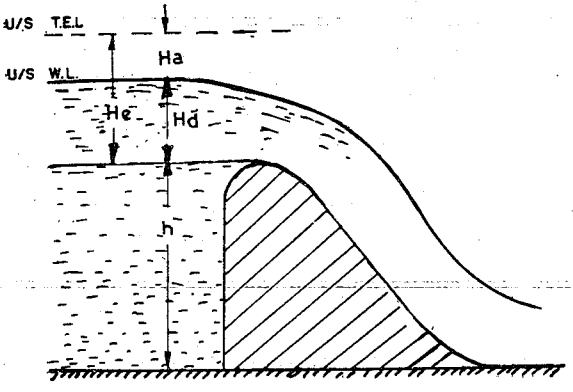


Fig. 21.11

$$Q = C \cdot L_e H_e^{3/2} \qquad ...(21.4)$$

where Q = Discharge

L_e = Effective length of the spillway crest

C = Coefficient of discharge which depends upon various factors such as relative depth of approach, [*i.e.* d/H_d ratio (Fig. 21.11), relation of actual crest shape to the ideal nappe shape, slope of upstream face, downstream apron interference, and submergence, etc.

H_e = Total head over the crest including the velocity head.

If the discharge Q is used as the design discharge in Equation (21.4), then the term H_e will be the corresponding design head (H_d) plus the velocity head (H_a). In such a case, $H_e = H_d + H_a$. For high ogee spillways, the velocity head is very small, and $H_e \approx H_d$.

21.6.5. Variation of Coefficient of Discharge with Various Factors. This is discussed below in details :

(*i*) *Depth of approach.* The coefficient of discharge firstly depends upon the depth of approach. In other words, it depends upon the height of the ogee weir (h) to the design head over the weir (H_d). If the height of the weir is more than 1.33 times the design head, the velocity of approach has been found to have a negligible effect upon discharge, and as such H_d becomes equal to H_e or $\frac{H_e}{H_d} = 1.0$. In such a case, the coefficient of discharge, say $C = C_d$, has been found to *be 2.2 in M.K.S. or S.I. Units* and 4.03 in *F.P.S. Units.*

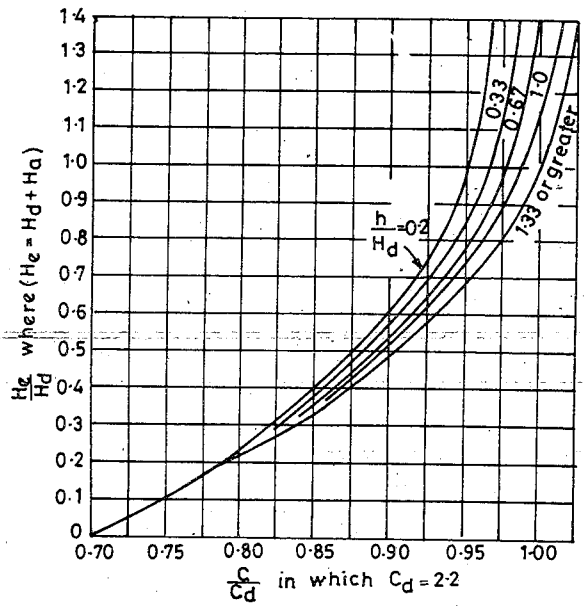


Fig. 21.12

However, in low spillways, with $\frac{h}{H_d} < 1.33$; the approach velocity is having an appreciable effect. The curves given in Fig. 21.12 can be used in such cases, to evaluate the coefficient of discharge C, using $C_d = 2.2$.

(*ii*) *U/s Slope.* The coefficient of discharge is also affected by the slope of the u/s face of the ogee weir. The values of C_d and C found up to now were for a vertical upstream face. If the u/s face is sloping, a correction factor by which the above values of C should be multiplied can be obtained from the curves given in Fig. 21.13.

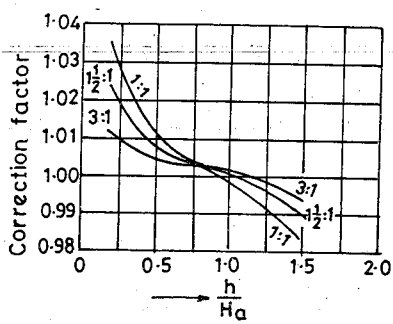


Fig. 21.13

(iii) D/s apron interference and submergence effects. The third important factor affecting the discharge coefficient of an ogee weir is the effect of downstream apron interference and downstream submergence.

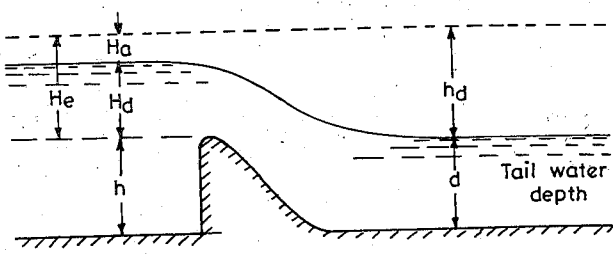


Fig. 21.14. Maximum tail water depth for a non-submerged weir.

When the tail water level is such that the top of the weir is covered by it, such that the weir cannot discharge freely ; the weir is then said to be a **submerged weir.**

Where the hydraulic jump occurs, the coefficient of discharge may decrease due to back pressure effect of the downstream apron and is independent of the submergence effect. When the value of $\frac{h_d + d}{H_e}$ (Fig. 21.15) exceeds 1.7, the downstream apron is found to have negligible effect on the coefficient of discharge. But there may be a decrease in the coefficient due to tail water submergence. The correction factor, by which the value of C should be multiplied in order to get the modified or correct value of coefficient of discharge, can be obtained from the curve of Fig. 21.15.

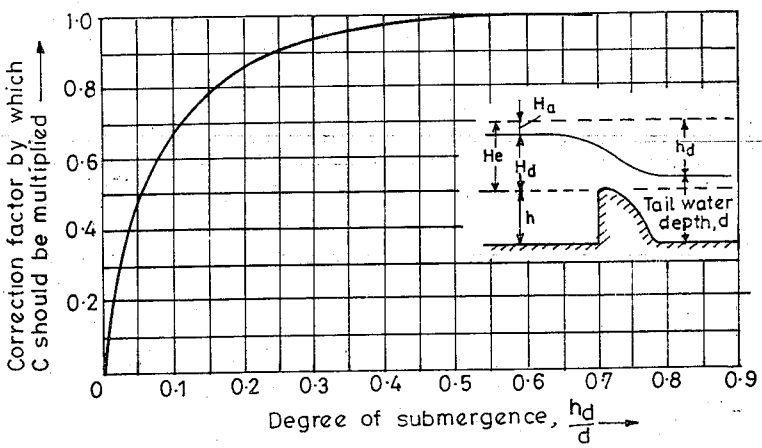


Fig. 21.15. Effect of submergence on C.

21.6.6. Effects of Actual Prevailing Head on the Discharge Capacity of a Spillway. It was pointed out earlier that when once a spillway has been designed and constructed for a design head (H_d), and for a corresponding coefficient of discharge (say C), it will not always find the same head over its crest in its actual operations. The actual operating head (H) *i/c* velocity head, may be less or more than the designed head. Since the design is done for maximum head, the possibility of a head more than the designed head is very meagre. When the actual operating head passing over the spillway is less than the designed head, the prevailing coefficient of discharge (C_d) tends to reduce, and is given by the equation

$$C_d = C \cdot \left(\frac{H}{H_e}\right)^{0.12} \qquad ...(21.5)$$

where H_e is the designed head including velocity head.

Since an overflow spillway is sufficient in height (*i.e* $h > 1.33\ H_d$) ; the coefficient of discharge C at designed head can be taken as 2.2. The prevailing coefficient of discharge at 50% head (*i/c* velocity head) will then be

$$C_d = 2.2 \times \left(\frac{0.5\ H_e}{H_e}\right)^{0.12} = 2.2 \times 0.92 = 2.02.$$

Similarly, for still lower heads, the coefficient of discharge goes on reducing and tends to become constant at about 1.7. (Because at very low heads, the velocity head becomes the governing factor, which tries to make H a constant).

The effective length (L_e) of ogee spillway.

The effective length of the spillway crest (L_e) to be used in equation (21.4), is given by the equation

$$L_e = L - 2\left[K_p \cdot N + K_a\right] H_e \qquad ...(21.6)$$

where L = the net clear length of the spillway crest

K_p = Pier constraction coefficient

K_a = Abutment contraction coefficient

N = Number of piers

H_e = Total design head on the crest including velocity head.

The values of K_p and K_a depend mainly upon the shape of the piers and that of the abutments. The greater is the divergence from streamlined flow, the greater is the constraction coefficient and lesser is the effective length of the crest. A 90° cut water nose pier is most efficient and has quite a low value of K_p, and is generally preferred. Values of K_p and K_a are given in Tables 21.3 and 21.4. Various shapes of piers are shown in Fig. 21.16.

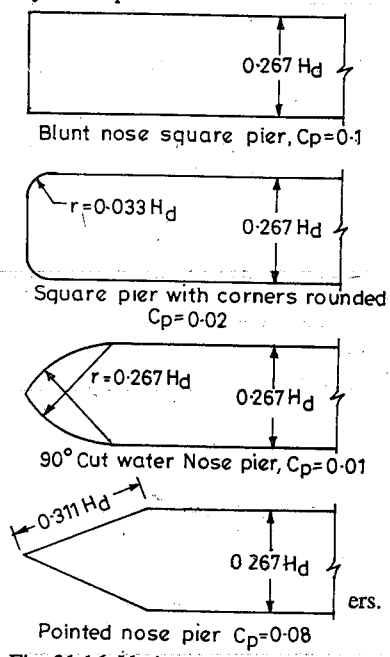


Fig. 21.16. Various shapes of piers.

Table 21.3

S. No.	Pier Shape	Contraction coefficient K_p
1.	Square nosed piers without any rounding	0.1
2.	Square nosed piers with corners rounded on radius equal to 0.1 of pier thickness	0.02
3.	Rounded nose piers and 90° cut water nosed piers	0.01
4.	Pointed nose piers	0.0

Table 21.4

S. No.	*Shape of abutment*	*Contraction coefficient* K_a
1.	Square abutment with head wall at 90° to the direction of flow	0.2
2.	Rounded abutment with head wall at 90° to the direction of flow	0.1

21.6.7. Aeration Arrangements in Gated Ogee Spillways. The entire design of an overflow spillway has been done with the assumption that upper and lower nappe are subjected to full atmospheric pressure. But in practice, due to insufficient aeration, development of negative pressures takes place beneath the nappe due to removal of air by the falling jet. The development of negative pressures causes the danger of cavitation and induces fluctuation and pulsation effects on the jet, which may be very objectionable if the spillway is to be used as a discharge measuring device.

To control the development of negative pressures, aeration pipes 25 mm dia at say 3 m centre to centre may be provided along spillway face below the gate lip. These pipes can be connected to a bigger sized header. Formulas have been developed on the basis of model studies, for evaluating the amount of air required for aeration, and may have to be used in large dam designs, but are beyond the scope of this book.

Example 21.1. *Design a suitable section for the overflow portion of a concrete gravity dam having the downstream face sloping at a slope of 0.7 H : 1 V. The design discharge for the spillway is 8,000 cumecs. The height of the spillway crest is kept at RL 204.0 m. The average river bed level at the site is 100.0 m. The spillway length consists of 6 spans having a clear width of 10 m each. Thickness of each pier may be taken to be 2.5 m.*

Solution. Since the given spillway looks like a high weir, the coefficient of discharge may be assumed to be 2.2.

Now $$Q = C \cdot L_e H_e^{3/2}$$

where $L_e = L - 2\,[N K_p + K_a]\, H_e$

Let us first work out the approximate value of H_e for a value of

$$L_e \approx L = \text{clear waterway} = 6 \times 10 = 60 \text{ m.}$$

$$\therefore \quad 8{,}000 = 2.2 \times 60\, H_e^{3/2}$$

or $$H_e^{3/2} = \frac{8{,}000}{2.2 \times 60} = 60.6$$

or $$H_e = (60.6)^{2/3} = 15.5 \text{ m.}$$

The height of the spillway above the river bed (see Fig. 21.15)

$$= h = 204 - 100 = 104.0 \text{ m}$$

Since $\dfrac{h}{H_d}$, *i.e.* $\dfrac{104}{15.5} > 1.33$,

it is a high spillway, the effect of velocity head can, therefore, be neglected.

Since $\frac{h_d + d}{H_e} = \frac{H_e + h}{H_e} = \frac{15.5 + 104}{15.5} > 1.7$;

the discharge coefficient is not affected by tail water conditions, and the spillway remains a high spillway.

U/s Slope. The upstream face of the dam and spillway is proposed to be kept vertical. However, a batter of 1 : 10 will be provided from stability considerations in the lower part. This batter is small and will not have any effect on the coefficient of discharge.

Effective length of spillway (L_e) can now be worked out as

$$L_e = L - 2\,[N.K_p + K_a]\,H_e$$

Assuming that 90° cut water nose piers and rounded abutments shall be provided, we have

$$K_p = 0.01$$

and $$K_a = 0.1$$

No. of piers $= N = 5$.

Also assuming that the actual value of H_e is slightly more than the approximate value worked out (*i.e.* 15.5 m), say, let it be 16.3 m, we have

$\therefore$ $$L_e = 60 - 2\,[5 \times 0.01 + 0.1] \times 16.3 = \mathbf{55.1\ m.}$$

Hence $$Q = 2.2 \times 55.1 \times H_e^{3/2}$$

or $$8{,}000 = 2.2 \times 55.1 \times H_e^{3/2}$$

or $$H_e^{3/2} = \frac{8{,}000}{2.2 \times 55.1} \cong 66.0$$

or $$H_e = (66.0)^{2/3} = \mathbf{16.4\ m} \cong 16.3 \text{ (assumed)}$$

Hence, the assumed H_e for calculating L_e is all right. The crest profile will be designed for $H_d = 16.4$ m (neglecting velocity head).

Note. The velocity head (H_a) can also be calculated as follows :

$$\text{Velocity of approach} = V_a = \frac{8{,}000}{(60 + 5 \times 2.5)\,(104 + 16.4)}$$

$$= \frac{8{,}000}{72.5 \times 120.4} = 0.917 \text{ m/sec.}$$

$$H_a = \text{Velocity Head} = \frac{V_a^2}{2g} = \frac{(0.917)^2}{2 \times 9.81} = 0.043 \text{ m.}$$

This is very small and was, therefore, neglected.

Downstream profile. The W.E.S. d/s profile for a vertical u/s face is given by equation (21.2) as :

$$x^{1.85} = 2 \cdot H_d^{0.35} \cdot y$$

or $$y = \frac{x^{1.85}}{2\,(H_d)^{0.85}} = \frac{x^{1.85}}{2 \times (1.64)^{0.85}}$$

or $$y = \frac{x^{1.85}}{2 \times 10.8}$$

or $$y = \frac{x^{1.85}}{21.6} \qquad ...(21.7)$$

Before we determine the various co-ordinates of the d/s profile, we shall first determine the tangent point.

The d/s slope of the dam is given to be 0.7 H : 1 V.

Hence, $$\frac{dy}{dx} = \frac{1}{0.7}$$

Differentiating the equation of the d/s profile w.r. to x, we get

$$\frac{dy}{dx} = \frac{1.85x^{1.85-1}}{21.6} = \frac{1}{0.7}$$

or $$x^{0.85} = \frac{21.6}{1.85 \times 0.7} = 16.7$$

or $$x = 22.4 \text{ m.}$$

$\therefore$ $$y = \frac{(22.4)^{1.85}}{21.6} = 14.6 \text{ m.}$$

The co-ordinates from $x = 0$ to $x = 22.4$ m are worked out in Table 21.5.

Table 21.5

x *metres*	$y = \frac{x^{1.85}}{21.6}$ *metres*
1	0.046
2	0.166
3	0.354
4	0.60
5	0.905
6	1.274
7	1.710
8	2.162
9	2.684
10	3.240
12	4.575
14	6.020
16	7.88
18	9.74
20	11.85
22	14.35
22.4	14.60

The u/s profile. The u/s profile may be designed as per equation (21.3), as :

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

Using $H_d = 16.4$ m, we get

$$y = \frac{0.724\,[x + 0.27 \times 16.4]^{1.85}}{(16.4)^{0.85}} + 0.126\,(16.4)$$

$$- 0.4315\,(16.4)^{0.375}\,(x + 0.27 \times 16.4)^{0.625}$$

or $$y = 0.07\,(x + 4.44)^{1.85} + 2.07 - 1.234\,(x + 4.432)^{0.625} \quad ...(21.8)$$

This curve should go upto $x = -0.27\,H_d$

or $$x = -0.27 \times 16.4 = -4.443 \text{ m.}$$

Various values of x such as, $x = -0.5, x = -1.0, x = -2.0, x = -3.0, x = -4.0, x = -4.443$ are substituted in equation (21.8) and corresponding values of y are worked out, as given below in Table 21.6.

Table 21.6

x in metres	*y in metres*
– 0.5	0.020
– 1.0	0.063
– 2.0	0.27
– 3.0	0.65
– 4.0	1.34
– 4.443	2.07

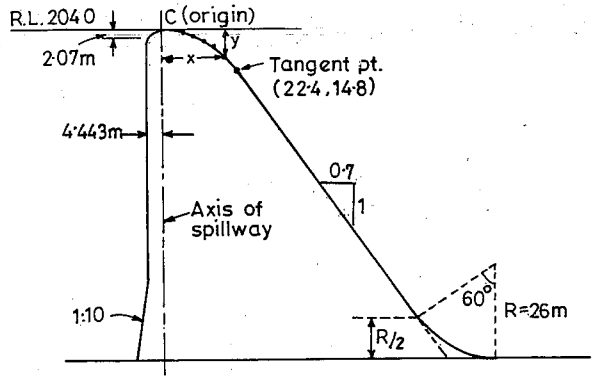


Fig. 21.17

The profile of the spillway has been determined and plotted in Fig. 21.17. A reverse curve at the toe with a radius equal to $\frac{h}{4} = \frac{104}{4} = 26$ m can be drawn at angle 60°, as shown in Fig. 21.17. Aeration pipes (say 25 mm pipes at 3 m c/c) can be installed along the spillway face below the gate lip, so as to prevent the development of negative pressures. The energy dissipation arrangements have not been shown. They should be designed depending upon the position of the jump height curve and tail water curve, as explained afterwards. A sky jump bucket or an apron may be provided as per the prevailing conditions.

21.7. Chute Spillway or the Trough Spillway

An ogee spillway is mostly suitable for concrete gravity dams especially when the spillway is located within the dam body in the same valley. But for earthen and rockfill dams, a separate spillway is generally constructed in a flank or a saddle, away from the main valley, as explained earlier. Sometimes, even for gravity dams, a separate spillway is required because of the narrowness of the main valley. In all such circumstances, a separate spillway may have to be provided. The *Trough Spillway* or *Chute Spillway* is the simplest type of a spillway which can be easily provided independently and at low costs. It is lighter and adaptable to any type of foundations ; and hence provided easily on earth and rockfill dams. A chute spillway is sometimes known as a **waste weir**. If it

is constructed in continuation to the dam at one end, it may be called a **flank weir.** If it is constructed in a natural saddle in a bank of the river separated from the main dam by a high ridge, it is called a **saddle weir.**

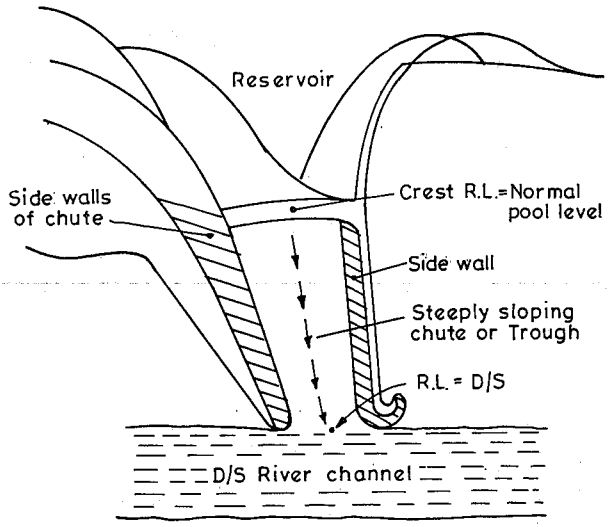


Fig. 21.18 (*a*). Simplified Line sketch of a Chute Spillway.

A chute spillway (Fig. 21.18) essentially consists of a steeply sloping open channel, placed along a dam abutment or through a flank or a saddle. It leads the water from the reservoir to the downstream channel below. The base for the channel is usually made of reinforced concrete slabs, 25 to 50 cm thick. Light reinforcement of about 0.25% of the concrete area is provided in the top of the slabs in both directions. The chute is sometimes of constant width, but is usually narrowed for economy and then widened near the end to reduce the discharging velocity. Expansion joints are usually provided in the chute at intervals of about 10 metres in either direction. The expansion joints should be made watertight so as to avoid any under-seepage and its troublesome effects. Under-drains are also provided, so as to drain the water which may seep through the trough bottom and side walls. These under-drains may be in the form of perforated steel pipes, clay tiles or rock-filled trenches.

If the slope of the chute can confirm to available topography, the excavations shall be minimum. But the slope of the chute must be high enough, and should atleast be able to maintain super critical flow, to avoid unstable flow conditions.

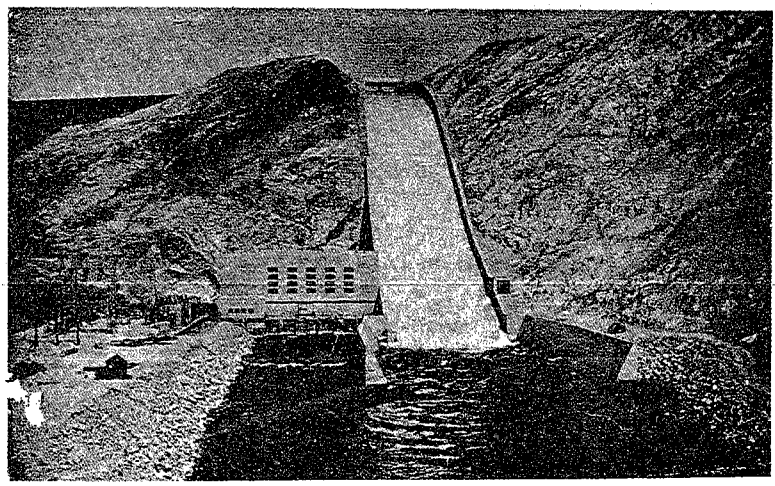


Fig. 21.8. (*b*) Photoview of the Chute Spillway of Anderson Ranch Dam, Idaho (USA).

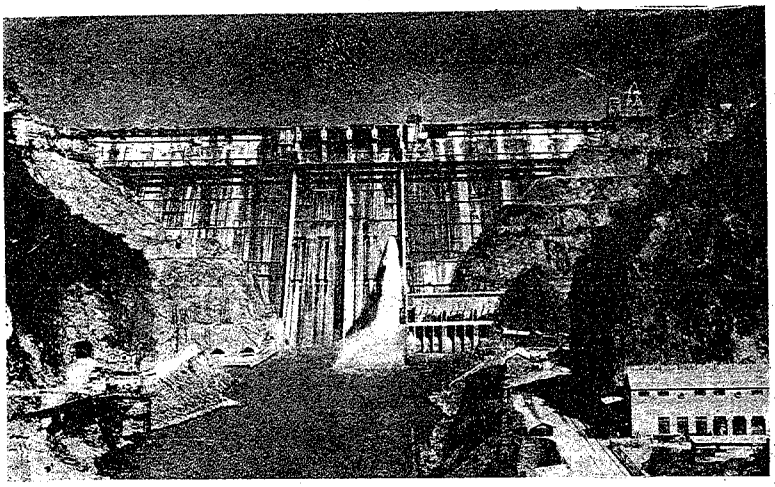


Fig. 21.18. (*c*) Photoview of two chutes provided at Bhakra Dam (India).

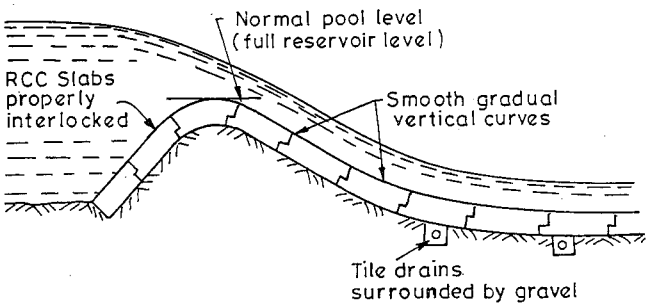


Fig. 21.18. (*d*) Section through a Chute Spillway.

When a vertical curve is provided at a point where the chute slope changes, it must be gradual and designed so as to avoid any separation of flow.

21.7.1. Control Structure or a Low Ogee Weir. Since the trough spillway is provided in a flank or a saddle, the height of spillway or ogee weir required to be constructed in that flank, will be small ; sometimes almost flat low weir shall be required depending on the natural levels of the bottom of the flank. If the flank bottom is at a level lower than the natural pool level, an ogee weir shall have to be constructed upto that level. If the flank bottom is at higher level than the normal pool level, excavations will have to be done upto that level. In such a case, the weir crest is normally left flat as it shall seldom be economical to excavate the rock just for the sake of constructing on 'ogee shape' for obtaining high coefficient of discharge.

21.7.2. Chute Slope. The water spilling over the control structure (*i.e.* ogee weir), then flows through the chute channel. The minimum slope of the chute is governed by the condition that supercritical flow must be maintained. The slope of the chute is kept just sufficient to meet this flow requirement from the crest for as long a distance as possible without any filling. After that, the slope is made as steep as possible without endangering the stability or without getting into heavy excavations.

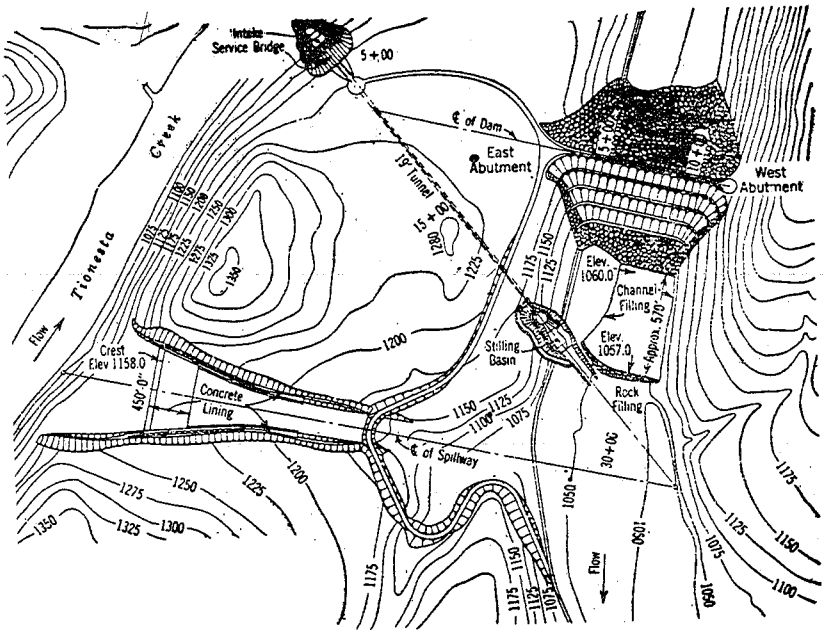


Fig. 21.18. (*e*) Layout Plan of Tionesta Chute Spillway.

21.7.3. Side Walls. The side walls (called training walls) of the chute should be of such a height that water does not spill over them. A sufficient freeboard must, therefore, be provided above the top water nappe, for obtaining the top levels of the side walls. The freeboard is generally given by equation,

$$\text{Freeboard} = 0.61 + 0.04\, V_m \cdot d_m^{1/3} \text{ (M.K.S. or S.I. units)} \qquad \text{...(21.9)}$$

where V_m = mean velocity of water in the chute reach

d_m = mean depth of water in the chute reach under consideration.

The side walls of the chute may be kept vertical or sloping. But in the vicinity of gated ogee weirs, they will have to be vertical. Generally, a rectangular chute channel is designed.

21.7.4. Design of the Small Ogee Weir Required as a Control Structure for the Chute Spillway. The profile for low ogee weir as recommended by W.E.S. confirms to the following equations for different ratios of H_a/H_e, where H_a is the head due to velocity of approach and H_e is the design head over the crest including velocity head, *i.e.* ($H_e = H_d + H_a$).

Table 21.7. Table showing equations for d/s profile of a low ogee weir

Values of $\frac{H_a}{H_e}$	$\frac{h}{H_e}$ *range*	*Equation for the d/s profile*
0.0	≥ 1.0	$x^{1.78} = 1.852\, H_e^{0.78} \cdot y$
0.08	1.0 – 0.58	$x^{1.75} = 1.869\, H_e^{0.75} \cdot y$
0.12	0.58 – 0.30	$x^{1.747} = 1.905\, H_e^{0.747} \cdot y$

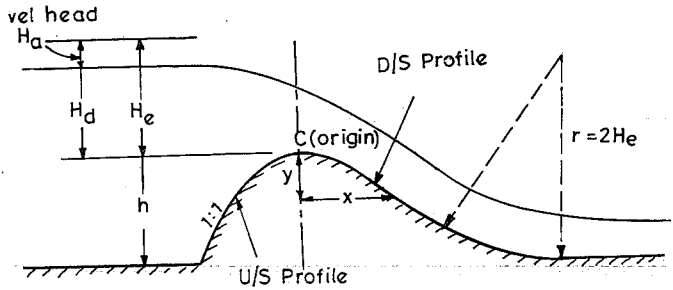


Fig. 21.19. Profile of the small ogee weir to be used as control structure for a chute spillway.

The co-ordinates of the u/s profile, which should merge in a slope of 45° (*i.e.* 1 : 1), as shown in Fig. 21.19, are given in Table 21.8.

Table 21.8. Co-ordinates of u/s profile for low ogee weir

$\frac{x}{H_e}$	$\frac{y}{H_e}$ *for different values of* $\frac{H_a}{H_e}$		
	$\frac{H_a}{H_e}=0.00$	$\frac{H_a}{H_e}=0.08$	$\frac{H_a}{H_e}=0.12$
– 0.000	0.0000	0.0000	0.0000
– 0.020	0.0004	0.0004	0.0004
– 0.060	0.0036	0.0035	0.0035
– 0.100	0.0103	0.0101	0.0099
– 0.120	0.0150	0.0150	0.0147
– 0.140	0.0207	0.0208	0.0199
– 0.150	0.0239	0.0235	0.0231
– 0.160	0.0275	0.0270	0.0265
– 0.175	0.0333	0.0328	0.0325
– 0.190	0.0399	0.0395	0.0390
– 0.195	0.0424	0.0420	—
– 0.200	0.0450	—	—

The reverse curve at the toe may have a radius equal to $2H_e$.

21.7.5. Design of Vertical Curves of the Chute. This is discussed below :

Concave Curve. Whenever the slope of chute changes from steeper to milder [Fig. 21.20 (*a*)], a concave vertical curve shall be provided. In no case, the radius of this curve should be less than 10 *d*, where *d* is the depth of water in metres.

Convex curve. Whenever the slope of the chute changes from milder to sleeper [Fig. 21.20 (*b*)], convex vertical curve shall have to be provided. The curvature should approximate to a parabolic shape given by the equation :

$$q = -x \tan\theta - \frac{x^2}{K\,[4\,(d+h_v)\cos^2\theta]} \qquad ...(21.10)$$

where θ is the slope angle of the floor upstream ;

$(d+h_v)$ is the specific energy of flow at the junction point ;

K is constant which is ≥ 1.5.

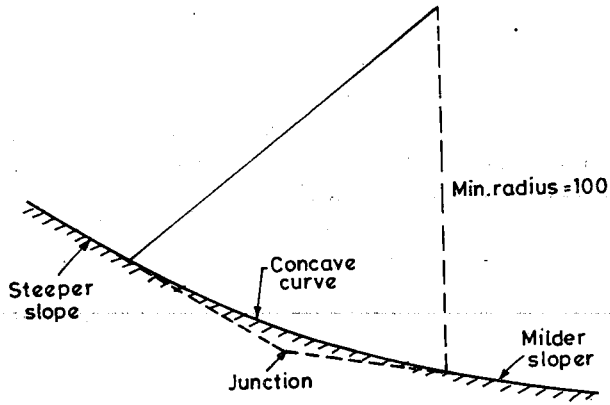


Fig. 21.20 (*a*). Concave Vertical Curve.

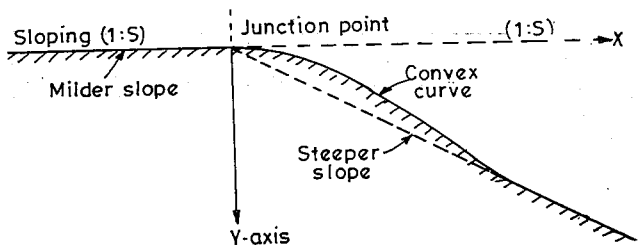


Fig. 21.20 (*b*). Convex Vertical Curve.

21.7.6. Approach Channel of Chute Spillway. An entrance channel called an approach channel, trapezoidal in shape with side slopes 1 : 1, may be constructed so as to lead the reservoir water up to the control structure (low ogee weir). If any curvature (in plan) is required, it is generally confined to the entrance channel, because the velocity of flow is low in this channel. The chute channel called the discharge channel or *discharge carrier* is generally kept straight in plan. If, however, any curvature is required to be provided, its floor should be super elevated to guide the high velocity flow around the bend, thus avoiding piling up of flow towards the outside of the chute. The friction head lost in the entrance channel upto the spillway crest can be calculated by Manning's formula, given as :

$$h_f = S_f \times L = \frac{n^2 \cdot V^2 \cdot L}{R^{4/3}} \qquad ...(21.11)$$

where n = Manning's coefficient of roughness
V = Velocity in channel
R = Hydraulic mean depth
L = Length of channel
S_f = Mean energy slope between two points.

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

(*i*) Entrance channel
(*ii*) Control structure (Low Ogee weir)
(*iii*) Chute channel or Discharge carrier
(*iv*) Energy dissipation arrangements at the bottom in the form of a stilling basin.

The design principles for the various components have already been explained except those for energy dissipation. The energy dissipation arrangements required at the bottom of the spillway shall be explained a little later.

Example 21.2. *Design a suitable profile for a chute spillway with the following data :*

Spillway crest level = *200.0 m*

Level of bottom of flank at which the low Ogee weir is to be constructed = *192.0 m*

Design discharge = *5,000 cumecs.*

Downstream tail water level corresponding to 5,000 cumecs = 103.0 m.

The spillway length consists of 5 spans of 10.0 m clear width each. The thickness of each spillway pier may be assumed to be 3 m. Assume any other data required.

Solution.

Design of approach channel :

$$Q = C \cdot L_e H_e^{3/2}.$$

Assume the coefficient of discharge as 2.18, and taking L_e as the clear width (approximately) $= 10 \times 5 = 50$ m :

and assuming $H_e \approx H$

we get $5{,}000 = 2.18\,(50)\,H^{3/2}$

or $H^{3/2} = \left(\dfrac{100}{2.18}\right) = 45.8$ m.

or $H = (45.8)^{0.667} = 12.8$ m.

Upstream water level = Crest level + H = 200 + 12.8 = 212.8 m.

Bed level of river in flank = 192.0

∴ Water depth = 212.8 − 192.0 = **20.8 m.**

Assuming the trapezoidal approach channel with 1 : 1 side slopes,

The width of the channel = B

= Total length of spillway = $50 + 4 \times 3 = 62$ m.

Area of channel $= A = (B + y)\,y = (62 + 20.8)\,20.8 = 1{,}720$ sq. m.

Velocity of approach $= V_a = \dfrac{5{,}000}{1{,}720} = 2.9$ m/sec

Velocity Head $= H_a = \dfrac{V_a^2}{2g} = \dfrac{(2.9)^2}{2 \times 9.81} =$ **0.43 m.**

The wetted perimeter P of the channel

$= [B + 2\sqrt{2}\,(\text{water depth})]$

$= [62 + 2\sqrt{2} \times 20.8] = (62 + 58.8) = 120.8$ m.

$$R = \frac{A}{P} = \frac{1{,}720}{120.8} = \mathbf{14.2\ m.}$$

Assuming the length of the channel to be 160 m ; the head loss due to friction upto the spillway crest, is given by Manning's formula, as :

$$h_f = \frac{n^2 V^2 L}{R^{4/3}} \quad \text{(assume } n = \text{Manning's rugosity Coefficient} = 0.019)$$

$$\therefore \quad h_f = \frac{(0.019)^2 \times (2.9)^2 \times 160}{(14.2)^{1.33}} = \mathbf{0.016\ m.}$$

Level of u/s *TEL* = u/s water level + Velocity Head – Head lost up to spillway crest

$$= 212.8 + 0.43 - 0.016 = 213.214 \text{; say } \mathbf{213.21\ m.}$$

Hence, head over the crest including velocity of approach

$$= H_e = 213.21 - 200.0 = \mathbf{13.21\ m.}$$

and $$H_d = 13.21 - 0.43 = \mathbf{12.78\ m.}$$

Correct Coefficient of Discharge

(*i*) *Correction due to height of weir,*

$$\frac{h}{H_d} = \frac{8}{12.78} = 0.63$$

$$\frac{H_e}{H_d} = \frac{13.21}{12.78} = 1.03$$

From Fig. 21.12 ; $\frac{C}{C_d} = 0.98.$

(*ii*) *Correction due to u/s slope of 45° (i.e. 1 : 1 slope)*

From Fig. 21.13 ; Correction factor = 1.008.

(*iii*) Assuming that the downstream apron elevation is maintained such that it does not affect C_d, we have,

The correct value of coefficient of discharge

$$= C = 2.2 \times 0.98 \times 1.008 = 2.17$$

Effective length $= L_e = 50 - 2\,[N \cdot K_p + K_a]\,H_e$

assuming $K_p = 0.01$ and $K_a = 0.1$

$$L_e = 50 - 2\,[4 \times 0.01 + 0.1] \times 13.21 = 50 - 3.7 = \mathbf{46.3\ m.}$$

The correct head H_e will then be given by

$$5{,}000 = 2.17 \times 46.4 \times H_e^{3/2}$$

or $$H_e^{3/2} = \frac{5{,}000}{2.17 \times 46.3} = 49.7$$

or $$H_e = (49.7)^{0.667} = \mathbf{13.6\ m.}$$

Hence, corrected $H_d = 13.6 - 0.43 = 13.17$ m ; say 13.2 m.

Hence, the design shall be done for a design head equal to 13.2 m.

Design of Crest Profile

$$\frac{H_a}{H_e} = \frac{0.43}{13.2 + 0.43} = \frac{0.43}{13.63} = 0.0315$$

$$\frac{h}{H_e} = \frac{8}{13.6} = 0.58$$

Since $\frac{H_a}{H_e}$ lies between 0 and 0.08 and $\frac{h}{H_e} \approx 0.58$, we can use the equation $x^{1.75} = 1.869\, H_e^{0.75} \cdot y$ (from Table 21.7).

Hence, the d/s profile is given by the equation

$$x^{1.75} = 1.869\,(13.63)^{0.75} \cdot y$$

or $$x^{1.75} = 13.2\,y$$

or $$y = \frac{x^{1.75}}{13.2} \qquad \text{...(21.12)}$$

Position of the d/s apron of spillway

The apron or toe of spillway should be at such an elevation that it does not affect the coefficient of discharge.

or $$\frac{h_a + d}{H_e} \geq 1.7$$

or $$h_d + d \geq 1.7\,(13.63) = 23.2 \text{ m.}$$

Hence, maximum apron elevation

$$= \text{T.E.L.} - (h_d + d)$$

where T.E.L. $= 212.63 \times 0.016$

$= 213.614$ m ; say **213.61 m**

$\therefore$ Max. apron elevation $= 213.61 - 23.2 = 190.41$ m.

Hence, provide the toe of the spillway (*i.e.* apron level) at *RL* **190.4 m.**

Discharge intensity downstream of spillway piers

$$= q = \frac{5{,}000}{50 + 12} = \frac{5{,}000}{62} = 80.7 \text{ m}^2/\text{sec.}$$

If d is the width, then the velocity $= \dfrac{80.7}{d}$ m/s

$$\text{Specific energy} = d + \frac{V^2}{2g} = g + \frac{\left(\dfrac{80.7}{d}\right)^2}{2 \times 9.81}$$

But specific energy $= 213.614 - 190.4 = 23.214$

$$\therefore \quad d + \frac{\left(\dfrac{80.7}{d}\right)^2}{2 \times 9.81} = 23.214$$

or $$d + \frac{(80.7)^2}{19.62\, d^2} = 23.214$$

or $$d^3 + 331 = 23.21\, d^2$$

$$d^3 - 23.21\, d^2 + 331 = 0$$

Solving by hit and trial, we get

$$\boxed{d = 4.2 \text{ m}}$$

The d/s profile is designed between *RL* 200.0 m (crest level) and *RL* = 190.4 (apron level)

Hence, maximum ordinate $y = 200.0 - 190.4 =$ **9.6 m.**

Corresponding value of x is obtained from eq. (21.12), as :

$$9.6 = \frac{x^{1.75}}{13.2}$$

or $$x^{1.75} = 126.7$$

or $$x = (126.7)^{0.57} = 15.08$$

The remaining co-ordinates of d/s profile between $x = 0$ and $x = 15.08$ m are worked out, as given in Table 21.9.

Table 21.9. Coordinates of d/s Profile

x in metres	$y = \frac{x^{1.75}}{13.2}$ *in metres*
0	0
0.5	0.023
1.0	0.077
2.0	0.25
4.0	0.86
6.0	1.74
8.0	2.88
10.0	4.03
12.0	5.84
14.0	7.9
15.08	9.6

The co-ordinates of the u/s profile are calculated in Table 21.10.

Table 21.10. Co-ordinates of u/s Profile
$H_e = 13.61$ m

$\frac{x}{H_e}$	$\frac{y}{H_e}$	x	y
– 0.000	0.0000	0	0
– 0.020	0.0004	– 0.272	0.0054
– 0.060	0.0035	– 0.817	0.0476
– 0.100	0.0101	– 1.361	0.1375
– 0.120	0.0150	– 1.633	0.2042
– 0.140	0.0208	– 1.905	0.2831
– 0.150	0.0235	– 2.042	0.3198
– 0.160	0.0270	– 2.178	0.3675
– 0.175	0.0328	– 2.382	0.4464
– 0.190	0.0395	– 2.586	0.5375
– 0.200	0.0420	– 2.722	0.5716

The u/s crest is joined at an angle of 45° to the bottom, as shown in Fig. 21.21.

Design of the Chute or Discharge Carrier

The critical depth $y_c = \sqrt[3]{\frac{q^2}{g}} = \sqrt[3]{\frac{(80.7)^2}{9.81}} = 8.77$ m.

The depth at the top of spillway (d) was calculated to be 4.2 m, which is less than y_c. Hence, the flow at the top of the spillway is supercritical.

The chute channel or the discharge carrier should now be given a milder slope for a little distance from toe, but in no case less than the critical slope, so that the flow remains super-critical.

Critical velocity $= V_c = \frac{q}{y_c} = \frac{80.7}{8.77} = 9.18$ m/sec.

But $$V = \frac{1}{n} \cdot R^{2/3} \cdot S^{1/2}$$

A rectangular channel with bottom width 62 m should be provided as the chute channel (discharge carrier) for the chute spillway.

At critical flow, $A = 62 \times 8.77$

$$P = 62 + 8.77 \times 2$$

$$\therefore \quad R = \frac{A}{P} = \frac{62 \times 8.77}{(62 + 8.77 \times 2)} = 6.82$$

Critical slope S_c is, therefore, given as :

$$9.18 = \frac{1}{0.019} (6.82)^{\frac{2}{3}} \cdot S_c^{1/2}$$

or $$S_c^{1/2} = \frac{9.18 \times 0.019}{(6.82)^{2/3}} = \frac{9.18 \times 0.019}{3.61} = \frac{1}{20.7}$$

or $$S_c = \frac{1}{428}$$

A slope of say 1/250 is provided in 25 m distance from the top of the spillway.

Bed level at the end of $\frac{1}{250}$ slope $= 190.4 - \frac{25}{250} = 190.3$ m.

The reverse curve at toe may have a radius equal to

$$2H_e = 2 \times 13.63 = 27.22 \text{ m ; say } 27.5 \text{ m.}$$

From this point onwards, *i.e.* $RL = 190.3$ m and upto RL 103.0 m, the slope of the discharge carrier may be steepened depending upon the site contours. A slope such as 8 : 1 to 4 : 1 may be given in the initial stage and then a steeper slope of say 2 : 1 or 3 : 1 may be given. The entire design is now like that of a rectangular channel. Wherever, the slope is changed, transition curve (convex or concave) may be designed as explained earlier.

In the given question, since the site contours are not known, we shall assume that a slope of 6 : 1 is given for the first 21 m fall (*i.e.* up a RL 169.3) and then a slope of say 2 : 1 is given for the rest of the reach. The depth, velocity, etc. at the end point of 1 in 250 slope may be taken to be the same, as they were at the toe of the spillway, because the small length of 25 m shall not produce much difference. With this assumption, the TEL at the starting point of new slope 6 : 1 ($RL = 190.3$) is equal to $213.21 - 0.1 = 213.11$ m.

The calculations of water depth, velocity, etc. can now be carried out for the entire reach ($RL = 190.3$ to $RL = 103.0$ m) of the discharge carrier by dividing the channel length into small reaches ; say of length 42 m each, as shown in Table 21.11. The depth (d) at the point is assumed in Column (6). Specific energy is calculated in Col. (9). The TEL at the end is calculated in Col. (10). The drop in energy line (h_f) in the interval is calculated in Col. (17). The drop (h_f) at the end of the interval is subtracted from the final calculated TEL at the beginning of interval in Col. (18). The Col. (18), is then compared with Col. (10). They should be almost equal. If difference is large, the assumed depth is changed till equivalence is obtained. The table is otherwise self explanatory.

Table 21.11. Calculations of water depth on chute channel (*i.e.* Discharge Carrier)

S.N.	Distance from start of 6 : 1 slope	Length L	Drop in bed	Bed level	Depth (d) assumed)	Velocity $v = q/d = 80.7/d$	Vel. head $= v^2/2g$	Sp. energy $= d + v^2/2g$	Bed level + Sp. energy = TEL calculated at the end	$A = Bd = 62d$ (Area)	$P = B + 2d = 62 + 2d$ wetted perimeter	$R = A/P$	$R^{4/3}$	$S_f = (n^2 . v^2)/R^{4/3}$; $n = 0.019$	Average S_f	$L . S_f = h_f$	Actual TEL	Froude number $F = V/\sqrt{gd}$
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)
1	0	0		190.3	4.2	19.2	18.9	23.1	213.4	260	70.4	3.7	5.72	0.0232			213.53	2.97
2	42	42	7	183.3	3.7 3.7 3.62	21.4 22.4 22.3	24.2 25.7 25.3	27.9 29.3 28.92	211.2 212.6 212.22	229 223 224	69.4 69.2 69.24	3.31 3.23 3.24	4.94 4.78 4.8	0.0378 0.0378 0.0374	0.029 0.0305 0.0303	1.22 1.28 1.27	212.18 212.12 212.13	3.74
3	84	42	7	176.3	3.30 3.28	24.45 24.60	30.4 30.8	33.7 34.08	210.0 210.38	205 205	68.6 68.56	2.98 2.98	4.28 4.28	0.0504 0.0505	0.0439 0.0439	1.84 1.84	210.38 210.38	4.33
4	126	42	7	169.3	3.05	26.4	35.7	38.75	208.05	188	68.08	2.77	3.88	0.0652	0.0578	2.43	207.95	4.95
Start of Slope 2 : 1																		
5	168	45	21	148.3	2.5	32.3	53.2	55.7	204.0	155	67	2.32	3.07	0.1224	0.0938	3.94	204.11	6.46
6	210	42	21	127.3	2.21	36.5	67.9	70.11	197.41	136	66.4	2.05	2.56	0.1895	0.1559	6.54	197.46	7.84
7	258.6	48.6	24.3	103.0	1.98 2.015	40.7 40.0	84.7 81.85	86.68 83.865	189.98 186.865	122.6 125.0	66 66.04	1.86 1.895	2.29 2.35	0.259 0.245	0.2242 0.2172	10.9 10.54	186.51 186.87	9.00

Design of Curve No. 1. (At the junction of 250 : 1 and 6 : 1 slopes, a convex curve shall be provided).

The convex curve No. 1 can be designed as per Equation (21.10), as :

$$y = -x \tan\theta - \frac{x^2}{1.5\,[4\,(d+h_v)\cos^2\theta]}$$

$(d+h_v)$ is the sp. energy at the junction point

$$= 4.2 + \frac{\left(\frac{80.7}{4.2}\right)^2}{2\times 9.81} = 23.21 \text{ m.}$$

$\tan\theta$ = slope of the angle of the floor u/s of the junction point $= \frac{1}{250}$

Since $\tan\theta$ is small, $\cos^2\theta$ will be approximately unity.

$$\therefore \qquad y = -\frac{x}{250} - \frac{x}{6\times 23.21} = \frac{-x}{250} - \frac{x^2}{139.3}$$

Differentiating w.r. to x, we get

$$\frac{dy}{dx} = \frac{1}{250} - \frac{x}{69.7}$$

The curve meets the downstream slope where

$$\frac{dy}{dx} = -\frac{1}{6}$$ (–ve sign shows that as x increases, y decreases).

Equating, we get $\frac{1}{250} + \frac{x}{69.7} = \frac{1}{6}$

or $$x = 69.7\left(\frac{1}{6} - \frac{1}{250}\right) = 69.7\left[\frac{244}{1{,}500}\right]$$

or **x = 11.3 m.**

Other co-ordinates of this curve for values of x between $x = 0$ and $x = 11.3$ m, determined from the equation $y = \frac{x}{250} + \frac{x^2}{139.3}$ (y is negative of course) are given in table 21.12.

Table 21.12

x in metres	$y = \frac{x}{250} + \frac{x^2}{139.3}$ *in metres*
0	0
1	0.011
3	0.077
5	0.20
7	0.371
9	0.619
10.0	0.78
11.3	0.97

Design of curve No. II (convex). *At the junction of 6 : 1 slope and 2 : 1 slope.*

From Table 21.11, the specific energy at this point is found to be

$$= d + h_v = 3.05 + 35.7 = 38.75 \text{ m}$$

$$\therefore \quad y = -\left\{x \tan\theta + \frac{x^2}{1.5[4(d+h_v)\cos^2\theta]}\right\}$$

$$\tan\theta = \frac{1}{6}\,;\ \cos\theta \approx 1$$

$$\therefore \quad y = -\left[\frac{x}{6} + \frac{x^2}{6 \times 38.75 \times 1}\right]$$

or $$y = -\left[\frac{x}{6} + \frac{x^2}{232.5}\right]$$

$$\therefore \quad \frac{dy}{dx} = -\left[\frac{1}{6} + \frac{x}{116.25}\right]$$

It meets the downstream slope of 2 : 1 ; $\therefore \frac{dy}{dx} = -\frac{1}{2}$ (–ve sign shows that as x increases, y decreases.)

$$\therefore \quad \frac{1}{6} + \frac{x}{116.25} = \frac{1}{2}$$

or $$\frac{x}{116.25} = \frac{1}{2} - \frac{1}{6} = \frac{1}{3}$$

or $$x = \frac{116.25}{3} = 38.75 \text{ m}$$

are $$x = \mathbf{38.75\ m.}$$

It means that this convex curve shall become tangential to the slope of 2 : 1 after traversing a distance of 38.75 m. The co- ordinates of this curve can be found (between $x = 0$ and $x = 38.75$ m), using the equation $y = -\left[\frac{x}{6} + \frac{x^2}{232.5}\right]$ as shown in table 21.13.

Table 21.13

x in metres	$y = \frac{x}{6} + \frac{x^2}{232.5}$ *in metres (y is downward)*
0	0
1	0.17
5	0.95
10	2.10
15	3.47
20	5.05
25	6.85
30	8.87
35	11.10
38.75	12.93

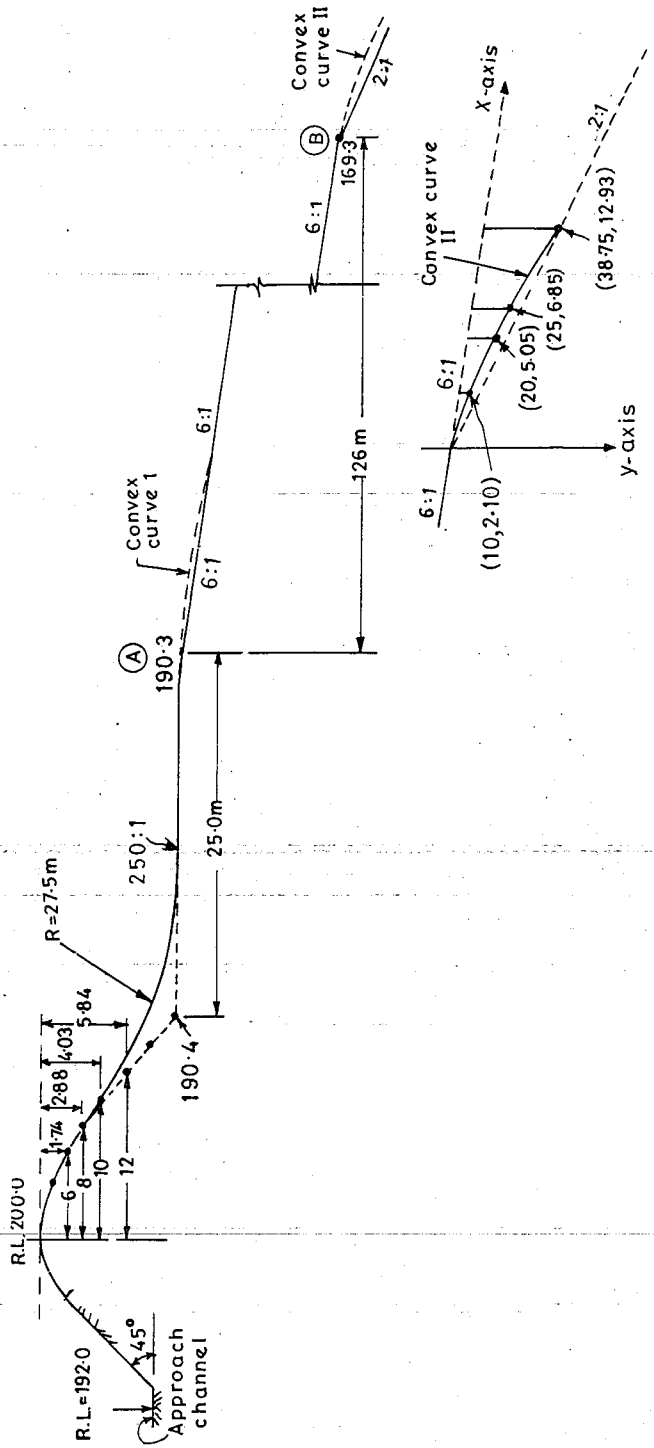


Fig. 21.21. Section of Chute Spillway.

Position of Stilling Basin for Energy Dissipation by Hydraulic Jump (Explained later)*

From Table 21.11, the depth at tail water level

$$d = 2.015 \text{ m}$$
$$F = 9.00$$
$$V = 40 \text{ m/sec.}$$
$$\therefore \quad y_1 = 2.015 \text{ m}$$
$$V_1 = 40 \text{ m/sec.}$$
$$F_1 = 9.0.$$

Hence
$$y_2 = \frac{y_1}{2}\left[\sqrt{1+8F_1^2} - 1\right] = \frac{2.015}{2}\left[\sqrt{1+648} - 1\right]$$
$$= 1.0075\,[\sqrt{649} - 1] = \mathbf{24.7\ m.}$$

Provide the basin tail water depth, 50% more than the conjugate depth, as the Froude number is large, or at a depth

$$= 1.05 \times y_2 = 1.05 \times 24.7$$
$$= 25.9 \text{ m say ; } \mathbf{26\ m.}$$

Hence, floor level of jump basin

$$= 103 - 26 = \mathbf{77.0\ m.}$$

U.S.B.R. stilling basin II can be *provided*. This is described in this chapter in article 21.15.

21.8. Side Channel Spillway

The side channel spillway (Fig. 21.22) differs from the chute spillway in the sense that while in a chute spillway, the water flows at right angles to the weir crest after spilling over it, whereas in a side channel spillway the flow of water after spilling over the crest, is turned by 90° such that it flows parallel to the weir crest (*AB*), as shown in Fig. 21.22 (*a*).

This type of spillway is provided in narrow valleys where no side flanks of sufficient width to accommodate a chute spillway are available. If a crest length equal to *AB* is provided along *AC* (*i.e.*, along axis of a chute spillway), heavy cutting shall be required. In such topographies, a chute spillway may be replaced by a side channel spillway.

The design of side channel, required for diverting the flow, is beyond the scope of this book. However, it may be mentioned that the analysis of flow in the side channel, is made by the application of the momentum principle in the direction of flow. The water entering the side channel has no momentum in the direction in which it has to move. The slope of the side channel should, therefore, be sufficient to overcome friction losses as well as to provide acceleration in the direction of flow against the mass of incoming water.

After the end of the crest *A*, the water is taken away as in an ordinary chute channel, till it joins the river downstream.

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* Readers are advised to go through this article, only after going through its theory as given in article 21.11.

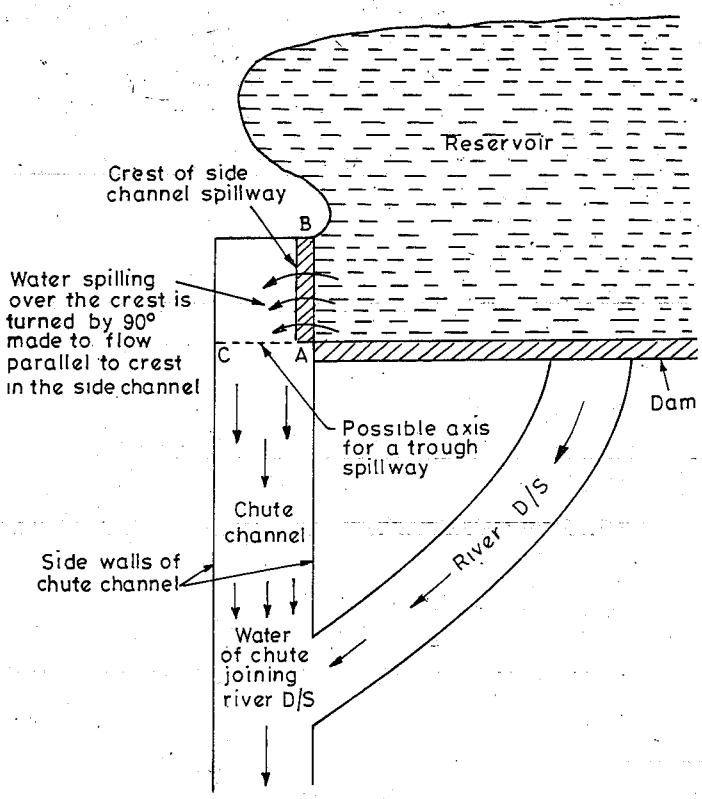


Fig. 21.22. (*a*) Simplified line sketch of a Side Channel Spillway.

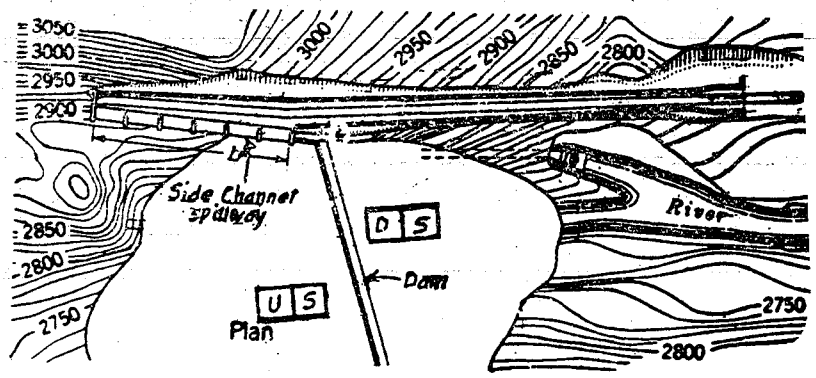


Fig. 21.22. (*b*) Layout plan of a Side Channel Spillway.

Many other spillways may be constructed somewhere in between the chute spillway and the side channel spillway. In such cases, the direction of water after passing over the crest is changed somewhere between 0° and 90°.

21.9. Shaft Spillway

In a shaft spillway (Fig. 21.23), the water from the reservoir enters into a vertical shaft which conveys this water into a horizontal tunnel which finally discharges the water into the river downstream. Sometimes, the vertical shaft may be excavated through

some natural rocky island or rocky spur existing on the u/s of the river near the dam. Sometimes, artificial shafts may be constructed. For small heights, the shafts may be constructed entirely of metal or concrete, or clay tiles. But for larger heights, reinforced cement concrete may be used. For smaller heights, no special inlet design is necessary, but on large projects, a flared inlet called **morning glory** is often used.

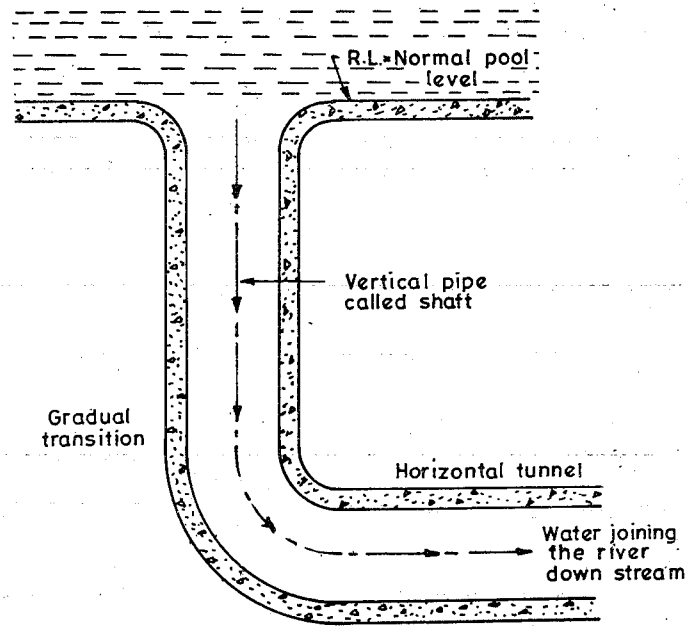


Fig. 21.23. Shaft Spillway.

The horizontal tunnel or the conduit may be taken either through the body of the dam (as may be done in concrete gravity dams) or below the foundations (as may be done in earthen dams). The diversion tunnels constructed for diversion of the river, may sometimes be planned and used for shaft spillways, as shown in Fig. 21.24.

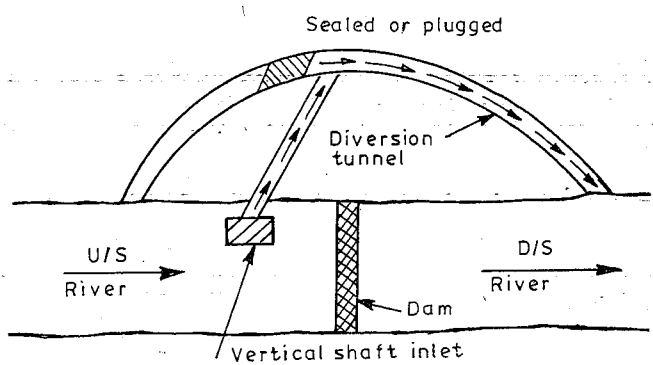


Fig. 21.24. Diversion Tunnel being used as a conduit for the shaft spillway.

A shaft spillway may be adopted when the possibility of an over-flow spillway and a trough spillway has been ruled out because of non-availability of space due to topography. The choice may then lie between a side channel spillway and a shaft spillway. If some suitable rock spur, etc. is available near the reservoir on the upstream, naturally, shaft spillway may become economical and consequently the first choice.

The discharge through the shaft spillway does not increase at such a high rate as it increases in weir type spills. Hence, if the unestimated high floods occur, the shaft spillway may not prove as useful as a weir type spillway (such as overflow, chute or side channel) would have been. Hence, a shaft spillway design should be much more conservative than that of other weir type spillways.

A gradual transition must be provided between the vertical shaft and the horizontal conduit, in order to avoid danger of cavitation. Hydraulic analysis of shaft spillways is difficult and their functioning is generally tested in models. The entry of debris and other materials floating in the reservoir must be prevented from entering the shaft, otherwise, they will clog the shaft or the conduit, especially at the junction point. Properly designed or tested trash racks or floating booms may be provided for this purpose. Since, a shaft spillway is surrounded by water on all sides, it must be connected to the dam or the hill side by a bridge.

21.10. Syphon Spillway

A siphon spillway essentially consists of a siphon pipe, one end of which is kept on the upstream side and is in contact with the reservoir, while the other end discharges water on the downstream side. Two typical installations of siphon pipes are shown in Figs. 21.25 and 21.26.

21.10.1. Tilted Outlet Type of a Syphon Spillway. The siphon pipe in Fig. 21.25 has been installed within the body of the dam. When the valley is very narrow and no space is available for constructing a separate spillway, the siphon pipes can be installed within the dam body, as shown in Fig. 21.25. An air vent may be connected with the siphon pipe. The

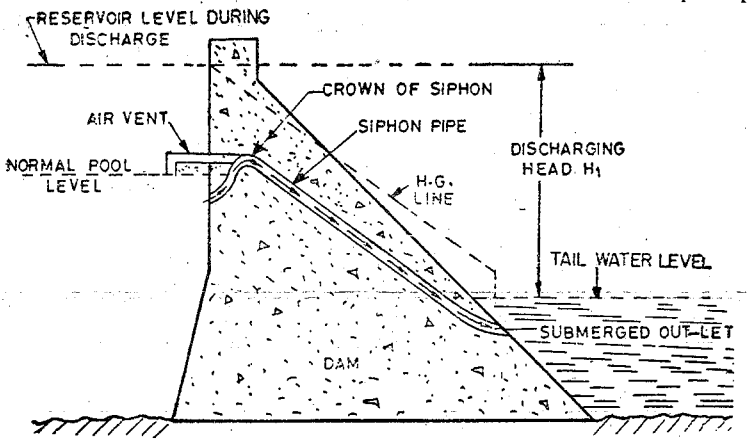


Fig. 21.25. Siphon pipe installed within the gravity dam.

level of the air vent may be kept at normal pool level, while the entry point of the siphon pipe may be kept still lower so as to prevent the entry of debris, etc. in the siphon. The outlet of the siphon may be submerged so as to prevent the entry of the air in the siphon from its d/s end.

When the water in the reservoir is upto or below the normal pool level, air enters the siphon through the vent and siphonic action cannot take place. When once the water level in the reservoir goes above the normal pool level, and if once the siphon is filled with water (*i.e.*, it is primed) ; the water will start flowing through the siphon by siphonic action. The outflow will continue till the water level in the reservoir falls back to normal pool level. As soon as it happens, the air will enter the siphon through the new exposed air vent and the flow will stop.

Discharge formula. The velocity of flow at the outlet of the siphon can be obtained by equating the effective head H_1 (*i.e.*, the difference of water level in the reservoir and the tail water level, for submerged outlet) to the velocity head.

$$\therefore \quad \frac{V^2}{2g} = H_1$$

or $$V = \sqrt{2gH_1}$$

$$\therefore \quad Q = \text{Discharge} = C_d \cdot A\sqrt{2gH_1}.$$

Hence, the discharge through the siphon is given by the equation,

$$Q = C_d A \cdot \sqrt{2gH_1} \qquad \text{...(21.13)}$$

where C_d is the co-efficient of discharge, which is usually about 0.9.

The above equation clearly shows that the discharge, through a siphon spillway is sufficiently independent of the water surface elevation of the reservoir. If the water surface in the reservoir rises, the discharge through the spillway is affected to a less extent because change in H_1 is small as compared to the corresponding change in head over an ogee type spillway. Hence, the discharge through the siphon is nearly always at capacity, when once the water level has risen above the normal pool level. This makes the siphon spillway particularly advantageous in disposing of sudden surges of water, such as may occur in canals and forebays when the outlet gates are closed rapidly. As the rise of water above the crest level is smaller (because of higher discharging capacity even at the start of inflow flood above normal pool level), the height of the non-overflow section of the dam can be kept smaller for the same height of overflow section.

Sometimes, it may not be possible to keep the outlet submerged in tail water because of limitations of negative pressure at the crown. In such a case, the outlet can be left open above the water level, but air entry through the outlet will have to be controlled by priming devices. The effective head H_1, in such a case, will be the difference of reservoir water level and outlet level of siphon.

21.10.2. Hooded Type of a Syphon Spillway. The construction of a Hooded type of siphon spillway [Fig. 21.26 (*a*)] is more commonly adopted. In this case, a reinforced concrete hood is constructed over an ordinary overflow section of a gravity dam. The inlet of this hood is kept submerged so as to prevent the entry of debris, ice, etc. A small depriming hood is kept above the main hood and both these hoods are connected by an

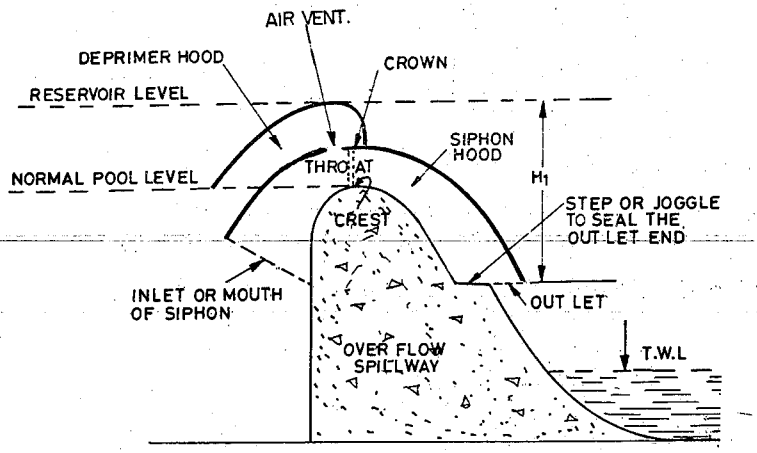


Fig. 21.26. (*a*) Siphon installed over the overflow spillway to increase its effectiveness and discharging capacity.

air vent. The inlet of the deprimer hood is kept at normal pool level. The principle of functioning of both types of installations is essentially the same, except for the initial filling up of the siphon or 'priming'. At normal pool level (*i.e.* full reservoir level), the water stands up to the crest of the spillway. If now a flood enters the reservoir, the water level would start rising and a sheet of water would start flowing over the spillway crest. Since the water level is above the deprimer hood, the air entry at the inlet is sealed. The air entry at the outlet is also sealed by tail water, etc. Hence, the water spilling over the crest, sucks all the remaining air from the hood within minutes. Siphonic action gets established after the air in the bend over the crest have been completely exhausted. A photoview of this type of siphon spillway, called Sarala Sagar spillway, flanked by earth dam constructed on China Vagu stream (a tributary of Krishna river) is shown in Fig. 21.26 (*b*).

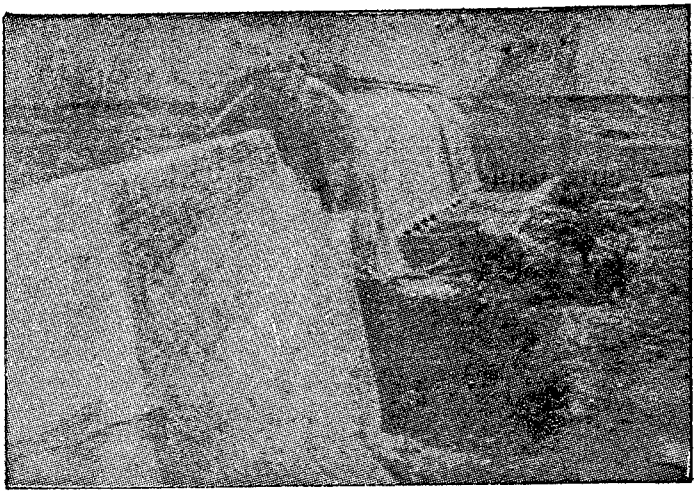


Fig. 21.26 (*b*). Photoview of a Siphon Spillway (Hooded Type).

21.10.3. Priming Devices. When the outlet cannot be sealed by tail water, some other devices, called *priming devices*, are used which lead to an automatic priming of the siphon at a certain rise of water level above the crest. The maximum rise of water level is called *priming depth*. In Fig. 21.26 (*a*), the priming is accomplished by means of a step called *Joggle* which deflects the sheet of water to strike against the lower end of the cover or hood, thus sealing the lower end from the atmosphere. Sometimes, an additional *baby siphon* may be installed as a priming device, as shown in Fig. 21.27.

When the water level reaches slightly above the crest, the baby siphon, which is an additional siphon, starts running full. The sheet of water issuing from it, is arranged to shoot across the lower end of the main siphon, so as to seal it from the atmosphere.

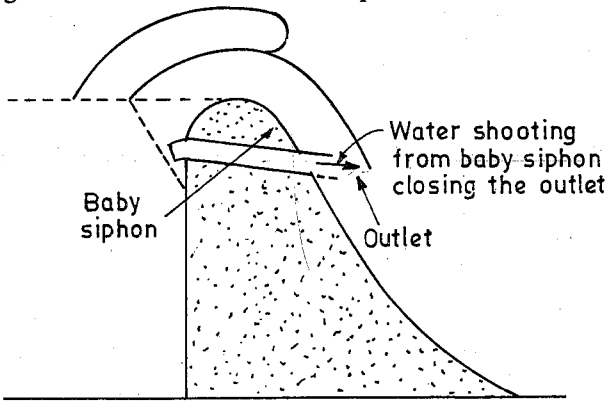


Fig. 21.27. Baby Siphon installed as a priming device.

21.10.4. Negative Pressures. As soon as the siphon is primed, a vacuum is formed at the throat. The negative pressure developed should be limited to such a magnitude that the absolute pressure of water does not exceed the vapour pressure of water at the temperature. This is necessary to avoid cavitation (explained earlier) and its ill effects. Hence, on an average, a maximum negative head equal to 7.5 m or so can be allowed. In other words, the vertical distance from the crown of the siphon (Top point) down to the discharging point (more precisely it is down to the hydraulic gradient line) should not exceed a value of about 7.5 m or so, under average conditions [∵ atmospheric pressure – vapour pressure = limiting vacuum pressure ; or $10 - 2.5 = 7.5$ m of water'. At high altitudes or in hotter regions, this limit may still go down. This fixes a limit on H_1 and on the discharging capacity of a siphon spillway.

Both types of syphon spillways described above are known as **Saddle siphon spillways**. A special type of siphon spillway called *Volute Siphon Spillway* has been designed in India by Ganesh Iyer and is shown in Fig. 21.28.

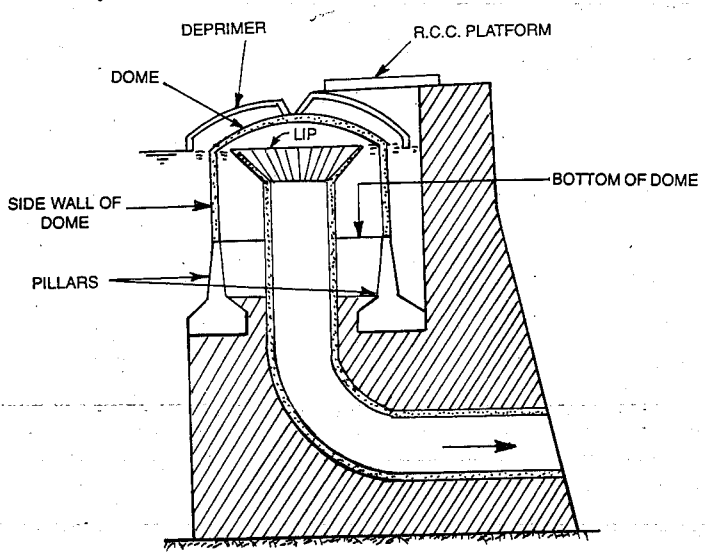


Fig. 21.28. Volute Siphon Spillway.

ENERGY DISSIPATORS

21.11. Energy Dissipation below Overflow Spillways

The water flowing over the spillway acquires a lot of kinetic energy by the time it reaches near the toe of the spillway (because of conversion of potential energy into kinetic energy). If arrangements are not made to dissipate this huge kinetic energy of water, and if the velocity of water is not reduced, large scale scour can take place on the downstream side near the toe of the dam and away from it. These arrangements are known as energy dissipation arrangements or *energy dissipators.*

In general, the kinetic energy of this super-critical flow can be dissipated in two ways :

(*i*) By converting the super critical flow into sub-critical flow by *hydraulic jump.*

(*ii*) By directing the flow of water into air and then making it fall away from the toe of the structure. The energy is dissipated by the *aeration of jet and impact* of water on the river bed. Though some scour will take place, but it is too

small or too far away from the dam to endanger it. Bucket type energy dissipators work on this principle.

21.11.1. Hydraulic Jump Formation. The phenomenon of hydraulic jump has already been explained in details in Chapter 10. It was mentioned therein, that a hydraulic jump can form in a horizontal rectangular channel, when the following relation is satisfied between the pre-jump depth (y_1) and post-jump depth (y_2).

$$y_2 = -\frac{y_1}{2} + \sqrt{\frac{y_1^2}{4} + \frac{2q^2}{gy_1}} \quad \textit{i.e.}\ \text{Eq. (10.4)}$$

where q is the discharge intensity.

For a given discharge intensity over a spillway, the depth y_1 is equal to q/V_1 ; and V_1 is determined by the drop H_1, being equal to $\sqrt{2gH_1}$.

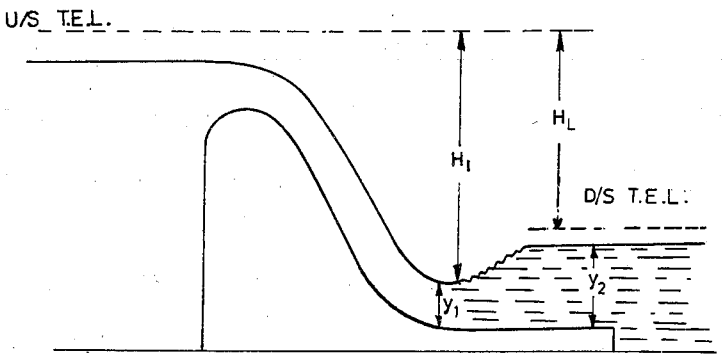
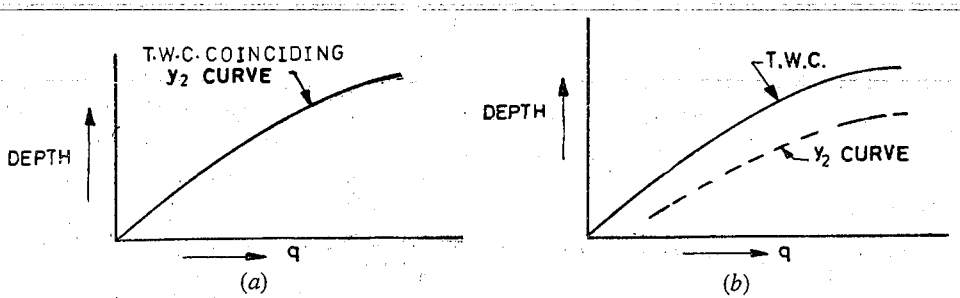


Fig. 21.29

Hence, for a given discharge intensity and given height of spillway, y_1 is fixed and thus y_2 (*i.e.* the depth required for the formation of hydraulic jump) is also fixed. But the availability of a depth equal to y_2 in the channel on the d/s cannot be guaranteed as it depends upon the tail water level, which depends upon the hydraulic dimensions and slope of the river channel below. The problem should, therefore, be analysed before any solution can be found. Hence, for different discharges, the tail water depth is found by actual gauge discharge observations and by hydraulic computations. The post jump depths (y_2) for all those discharges, are also computed from equation (10.4). If a graph is now plotted between q and tail water depth, the curve obtained is known as the *Tail Water Curve (T.W.C.)*. Similarly, if a curve is plotted on the same graph, between q and y_2, the curve obtained is known as the *Jump Height Curve (J.H.C.)* or y_2 *curve.*

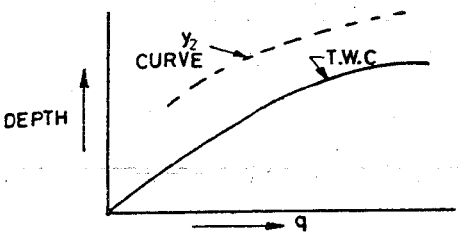
Now there are five possibilities

(*a*) T.W.C. coinciding with y_2 curve at all discharges [Fig. 21.30 (*a*)].

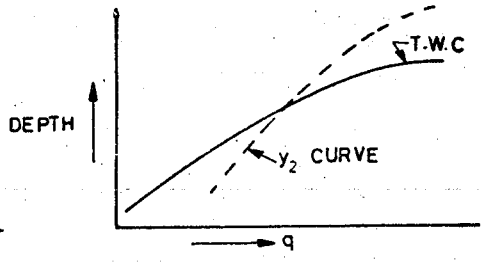


(*b*) T.W.C. lying above the y_2 curve at all discharges [Fig. 21.30 (*b*)].

(*c*) T.W.C. lying below the y_2 curve at all discharges [Fig. 21.30 (*c*)].



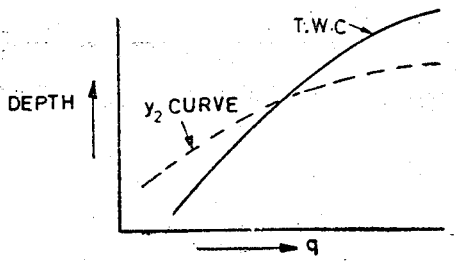
(*c*)



(*d*)

(*d*) T.W.C. lying above the y_2 curve at smaller discharges and lying below the y_2 curve at larger discharges [Fig. 21.30 (*d*)].

(*e*) T.W.C. lying below the y_2 curve at smaller discharges and lying above the y_2 curve at larger discharges [Fig. 21.30 (*e*)].



(*e*)
Fig. 21.30

Depending upon the relative positions of T.W.C. and y_2 curve, the energy dissipation arrangements can be provided below the spillway, as explained below for all these five cases.

21.11.1.1. Energy dissipators for case (*a*) : *When T.W.C. coincides with y_2 curve at all discharges.* This is the most ideal condition for jump formation. The hydraulic jump will form at the toe of the spillway at all discharges. In such a case, a simple concrete apron of length $5\,(y_2 - y_1)$ is generally sufficient to provide protection in the region of hydraulic jump, as shown in Fig. 21.31 (*a*).

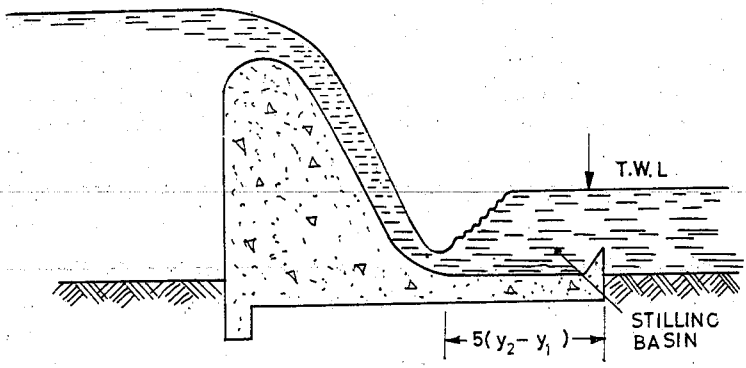


Fig. 21.31. (*a*) Simple horizontal apron.

21.11.1.2. Energy dissipators for case (*b*) : *When T.W.C. is lying above the* y_2 *curve at all discharges.* In this case, when y_2 is always below the tail water, the jump forming at toe will be drowned out by the tail water, and little energy will be dissipated. Water may continue to flow at high velocity along the channel bottom for a considerable distance.

The problem can be solved :

(*i*) by constructing a **sloping apron above the river bed** level as shown in Fig. 21.31 (b_1). The jump will form on the sloping apron where depth equal to y_2 (lesser than the

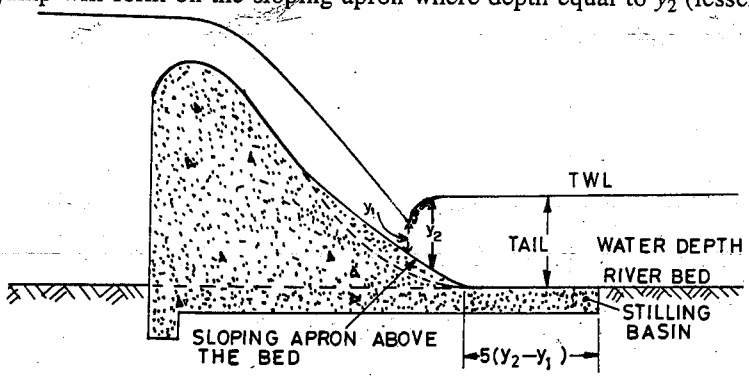


Fig. 21.31. (b_1) Sloping apron above the bed.

tail water depth at toe) is available. The slope of the apron is made in such a way that proper conditions for a jump will occur somewhere on the apron at all discharges. A lot of extra concreting is required to be done, as shown.

(*ii*) A second solution of this problem can be in the form of providing a **roller bucket** type of energy dissipator. It consists of an apron, which is upturned sharply at ends, as shown as in Fig. 21.31 (b_2).

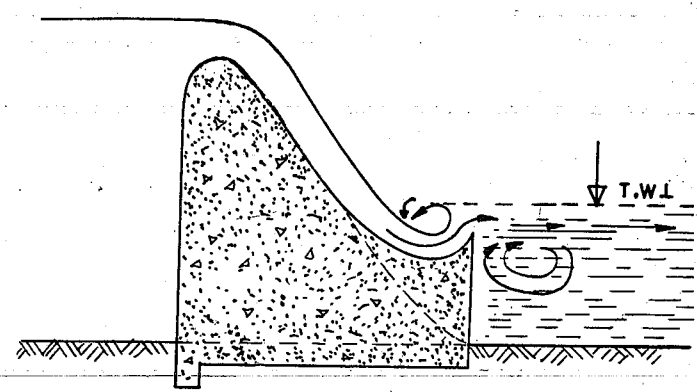


Fig. 21.31. (b_2) Roller Bucket.

Two main rollers are formed which dissipate the energy due to internal turbulence.

The roller which is formed downstream of the bucket, tends to move the scoured bed material towards the dam, thus, preventing serious scour at toe of the dam. Sometimes, the scoured material may enter the bucket under the action of u/s roller, and may cause severe abrasion. A *dentated bucket lip* may, therefore, have to be provided, so as to permit removal of material caught in the bucket.

21.11.1.3. Energy dissipators for case (*c*) : *When T.W.C. lies below the* y_2 *curve at all dischages.* (*i*) If the tail water is very low, the water may shoot up out of the above bucket, and fall harmlessly into the river at some, distance downstream of the bucket. *This bucket is then known as* **ski jump bucket** and can be used for energy dissipation in case (*c*) : *i.e.* when the tail water depth is insufficient or low at all discharges. The ski jump bucket type of an energy dissipator requires sound and rocky river bed, because a part of the energy dissipation takes place by impact, although some of the energy is dissipated in air by diffusion and aeration.

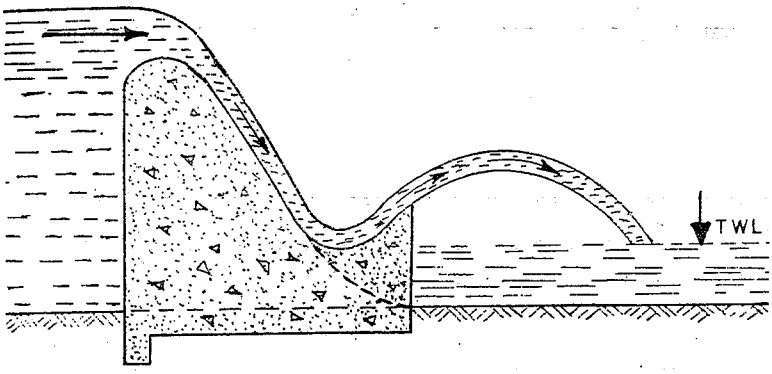


Fig. 21.31. (c_1) Ski jump bucket.

(*ii*) The second solution to the problem can be the *provision of a* **sloping apron** as *in case (b) but* **below the river bed,** as shown in Fig. 21.31 (c_2). The required depth y_2

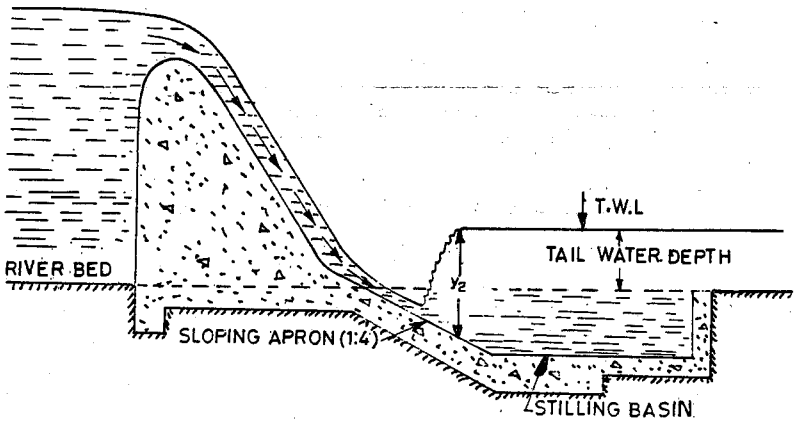


Fig. 21.31. (c_2) Sloping apron below the bed.

which is greater than T.W. depth, can thus be made available by letting the jump form on this sloping apron as shown. This sloping apron and the horizontal cistern of length $5(y_2 - y_1)$ shall be entirely in cutting and may be expensive, though otherwise quite satiactory.

(*iii*) The third solution to this problem may be the construction of a **subsidiary dam** below the main dam, so as to increase the tail water depth and cause a jump to form at the toe of the main dam, as shown in Fig. 21.31 (c_3).

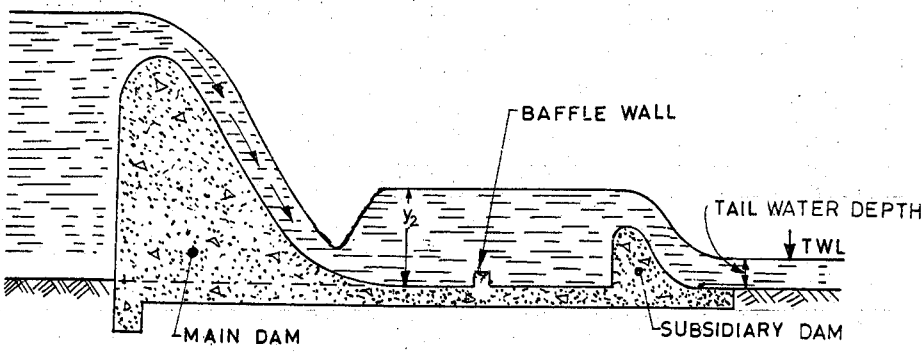


Fig. 21.31. (c_3) Subsidiary dam construction.

If the tail water defficiency is small, a **baffle wall** or a **row of friction blocks** may be provided so as to dissipate the residual energy. The baffles, generally give way, under high velocity jets due to their cavitation effects, and hence, are suitable only for low spillways or weirs. They should be sufficiently strong to withstand impact from ice and floating debris. The location, shape, size and spacing of these baffles can be best determined by model studies. Their use for weirs and canal falls have already been explained in the earlier chapters.

21.11.1.4. Energy dissipators for case (*d*). *When T.W.C. lies above the y_2 curve at low discharges and lies below the y_2 curve at high discharges.* In this case, at low discharges, the jump will be drowned and at high discharges, tail water depth is insufficient. The solution to the problem lies in providing a **sloping apron partly above and partly below the river bed** as shown in Fig. 21.31 (*d*). The horizontal apron and end sill should also be provided.

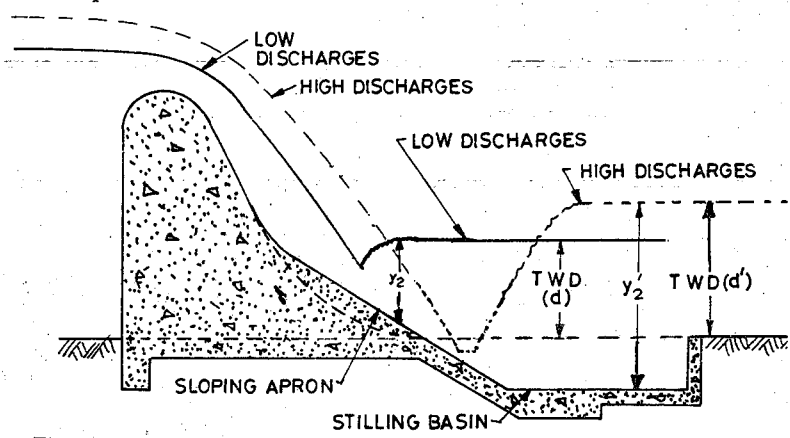


Fig. 21.31. (*d*) Sloping apron partly above and partly below the ground level.

At low discharges, the jump will form on the apron above the river bed, where the available depth is equal to the required depth and less than the T.W. depth. Similarly, at high discharges, the jump will form on the apron below the river bed, where the available depth is more than the T.W. depth and equal to the depth required for jump formation.

21.11.1.5. Energy dissipators for case (*e*). *When tail water depth is insufficient at low discharges and is greater at high discharges.* This case is just the reverse of case (*d*)

and the same arrangement which was made in case (*d*) will serve the purpose. The only difference will be that at low discharges, the jump will form on the apron below the bed; and at high discharges, the jump will form on the apron at a point above the bed.

21.12. Energy Dissipation Below Other Types of Spillways

The five hydraulic conditions between tail water depth and post jump depth (y_2) discussed in the previous article, can occur below any type of spillway or even below the sluiceways or dam outlets, and erosion control can be obtained by the methods described. A chute or a shaft or a side channel spillway generally discharges water at a point far away from the dam. Hence, the protection is required only for the spillway, as the danger to the main dam is not there. Due to this reason, a hydraulic stilling basin is generally sufficient, and may be provided at the discharging point of the spillway. If sound rocks are available, a Ski jump bucket may be provided at low cost.

21.13. Energy Dissipation Below Sluiceways or Dam Outlets

Discharge through the sluiceways is generally small, and less protection is required below them. The water from the sluiceway is often allowed to fall directly into the stilling basin. Sometimes, the jet may be spread by a *jet deflector* at the outlet. Sometimes the energy dissipation arrangement of the main spillway may be utilised for sluiceways also.

21.14. Use of Hydraulic Jump as Energy Dissipator and Design of Stilling Basins

It was stated in chapter 10, that the hydraulic jump formation depends considerably upon the Froude number of the incoming flow (F_1). The pre-jump depth (y_1) and post-jump depth (y_2) are also governed by the equation (10.6) [derived in chapter 10] as:

$$\frac{y_2}{y_1} = \frac{1}{2}\left[\sqrt{1 + 8F_1^2} - 1\right] \qquad \text{...(10.6)}$$

$$\text{where } F_1 = \frac{V_1}{\sqrt{g \cdot y_1}}$$

It was also stated in the same chapter, that the energy dissipation in the jump, depends upon the Froude number. Different types of jump were described for different values of Froude numbers. If this incoming Froude number F_1 is higher, the greater energy dissipation can take place. The approximate percentage loss of energy for various values of F_1 are given in Table 21.14.

Table 21.14

F_1	*% loss of energy*
2.5	17
4.5	45
9.0	70
14.0	80
20.0	85

However, the real problem in the design of stilling basins, is not the absolute dissipation of energy, but is the dissipation of this energy in as short a length as possible.

Hence, the effectiveness of a hydraulic jump has to be viewed in this light. The types of jumps for different ranges of Froude numbers, given in chapter 10, are again summarised below :

(*i*) When $F_1 < 2.5$, the jump is weak and energy loss is low. No blocks or other devices are provided in this range.

(*ii*) When F_1 lies between 2.5 to 4.5, as in case of weirs and barrages, the jump is trouble-some and oscillating, which gives rise to heavy waves on the surface. *Wave Suppressors* may be needed in this range. The length of jump may be taken as $5\,(y_2 - y_1)$.

(*iii*) *When F_1 lies between 4.5 and 9.0 ; as is generally the case for dam spillways,* the jump performance is at its best. The jump is then called a steady jump. The length of the jump is almost constant and equal to $6y_2$ in this range. Hence for large spillways, where y_2 may be quite high, very long and expensive stilling basins may be required. Some auxiliary devices may also be introduced for further stabilising the flow and to reduce the length of the basin.

(*iv*) When $F_1 = 9.0$ and larger ; it is a strong jump. But the jump is likely to be rough and choppy. Hence, a bucket type of energy dissipator is preferred to a stilling basin of hydraulic jump type.

21.15. Standard Stilling Basins

Various types of stilling basins have been generalised for use on different types of works, by various agencies. The designs of these basins have been developed on the basis of long experience and on model studies, keeping in view the protection obtained consistent with economy. These basins are not simple concrete aprons but are generally provided with **auxiliary devices** such as chute blocks, sills, baffle walls, etc. These devices can help in dissipating the energy of flow by offering resistance to flow and may stabilise the flow in a shorter length of the basin, thus affecting economy.

In general, a stilling basin may be defined, as a structure in which the energy dissipating action is confined. If the phenomenon of hydraulic jump is basically used for dissipating this energy, it may be called a hydraulic jump type of stilling basin. The auxiliary devices may be used as additional measures for controlling the jump, etc.

Before we reproduce a few standard stilling basins, let us first describe, in brief, the effects produced by auxiliary devices.

Chute Blocks. Chute blocks are a kind of serrated device (*i.e.* row of small projections like teeth of saw) and provided at the entrance of the stilling basin. The incoming jet of water is furrowed and partly lifted from the floor, producing a shorter length of jump than what would have been without them. They also help in stabilising the flow and thus improve the jump performance (Fig. 21.32).

Sills and Dentated Sills. Sill or more preferably dentated sill is generally provided at the end of the stilling basin. The dentated sill diffuses the residual portion of high velocity jet reaching the end of the basin. They, therefore, help in dissipating residual energy and to reduce the length of the jump or the basin (Refer Fig. 21.32).

Baffle Piers. They are the blocks placed within the basin, across the basin floor. They help in breaking the flow and dissipate energy mostly by impact. These baffle piers, sometimes called **friction blocks,** are very useful in small structures, such as low

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spillways and weirs, etc. They, however, give way due to cavitation, under the influence of high velocity jets, and hence are unsuitable for large works.

21.15.1. U.S.B.R. Basins. U.S.B.R. has standardised stilling basins for different ranges of Froude numbers. The important of these basins, are :

(1) **U.S.B.R. stilling basin II.** This is recommended for use on large structures, such as dam spillways, large canal structures, etc., when the incoming *Froude number* (F_1) *is more than 4.5.* The dimensions of the chute blocks, dentated sill, etc. are shown in Fig. 21.32. The length of the basin is related to the Froude number (F_1) as given in Table 21.15.

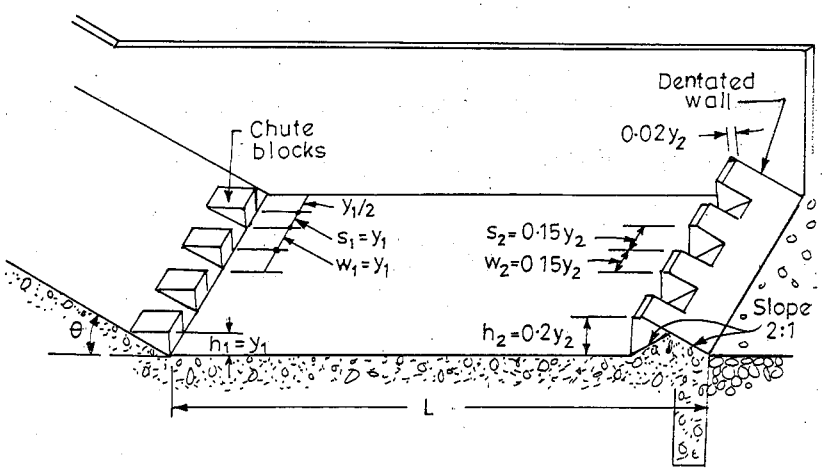


Fig. 21.32. U.S.B.R. Stilling basin II ($F_1 > 4.5$).

Table 21.15

F_1	*Length of the basin*
4	3.6 y_2
6	4 y_2
8	4.2 y_2
10 or more	4.3 y_2

An economy in the length of the basin up to about 35% (4.3 y_2 in place of 6y_2) is thus obtained with auxiliary devices. The floor of the basin should be set at such a level as to provide 5% more water depth than y_2.

(2) **U.B.S.R. stilling basin IV.** This type of stilling basin is shown in Fig. 21.33. It is used for Froude number varying between 2.5 and 4.5, which generally occurs in canal weirs, canal falls, diversion dams, etc. This basin is applicable only to rectangular cross sections. Since oscillating waves are generated in this range of Froude number, they are tried to be controlled at source by providing large chute blocks. The spacing between the blocks may be further reduced than what is shown in Fig. 21.33 (up to say 0.75 W) for better hydraulic performance. The floor of the basin may be set at such a level as to

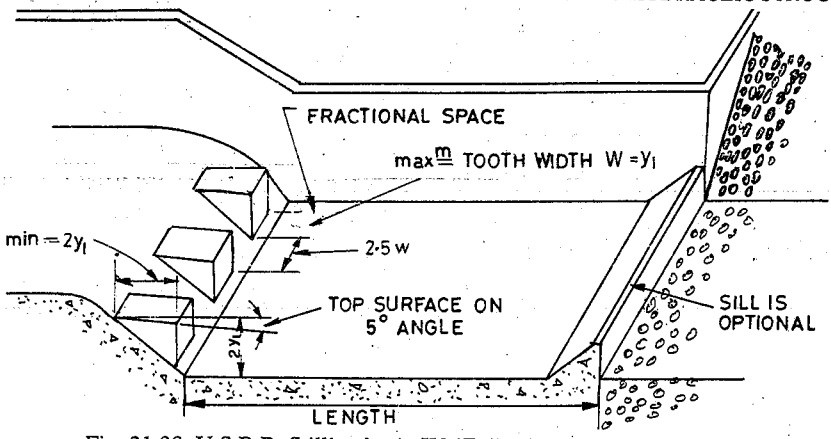


Fig. 21.33. U.S.B.R. Stilling basin IV (F_1 lies between 2.5 and 4.5).

provide 5 to 10% more water depth than the theoretical y_1. The length of the basin is generally kept equal to 5 $(y_2 - y_1)$.

21.15.2. I.S.I. Standardised Basins. Indian Standard Institution has also standardised certain stilling basins for uses under different conditions. A full description of such basins is available in IS : 4997 – 1968. Stilling basin I and II (for $F_1 < 4.5$ and $F_1 > 4.5$ respectively) with horizontal aprons and stilling basins III and IV with sloping aprons, have been standardised and described in the above reference. Students or field staff may refer to this I.S.I. publication in their particular needs.

21.16. Dynamic Force on Spillway Bucket and Spillway Bottom

When water flows over the curved bottom portion of an ogee spillway, there is a continuous change of velocity. Due to this change in velocity, there is a change in momentum (momentum = mass × velocity). Hence, a force is exerted on the spillway in accordance with Newton's second law of motion, *i.e.* rate of change of momentum is equal to this impressed force. This force is known as *dynamic force.* The spillway bottom where this curvature is provided is subjected to this force. The evaluation of this force is, therefore, necessary in the design of the bottom portion, or the design of the bucket, if it is provided as energy dissipator in an overflow spillway.

The magnitude of this force can be easily determined by using Newton's second law and by resolving all the forces acting on the curved element of water *AB* [Fig. 21.34 (*a*)] in horizontal and vertical directions. Mass of water when multiplied by net change in velocity in one direction will give the net component of this dynamic force in that direction. If *F* is this dynamic force, then

$$F_{(H)} = [V_{2(H)} - V_{1(H)}] \times \text{mass of water in the element}$$

$$F_{(V)} = [V_{2(V)} - V_{1(V)}] \times \text{mass of water in the element}$$

$$F = \sqrt{F^2_{(H)} + F^2_{(V)}}$$

where V_1 and V_2 are the velocities at the entrance and exit of the curved element. $V_{1(H)}$ and $V_{2(H)}$ are their horizontal components, and $V_{1(V)}$ and $V_{2(V)}$ are their vertical components. An illustrative example has been solved below :

Example 21.3. *An overflow ogee spillway of height 15 m is discharging water with a head of 2 m over the crest. A reverse curvature of radius 4 m, subtending an angle of 60° at the centre, is provided at the spillway bottom as shown in Fig. 21.34 (a). Assuming the discharge coefficient for the spillway as 2.2, determine the magnitude of the dynamic force on the curve portion AB of the spillway.*

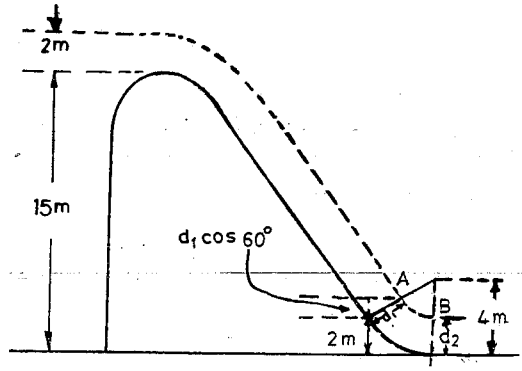


Fig. 21.34 (*a*)

Solution. The discharge over the spillway is given as :

$$Q = C \cdot L \cdot H^{3/2}$$

Discharge per unit length,

$$q = C \cdot H^{3/2}$$

$$= 2.2 \times (2)^{3/2} = 6.23 \text{ cumecs/m}$$

Let d_1 and d_2 be the water depths, V_1 and V_2 be the flow velocities at A and B respectively.

Neglecting any loss of energy and the head due to velocity of approach and applying Bernoulli's theorem between u/s water surface and at sections A and B, respectively we, get

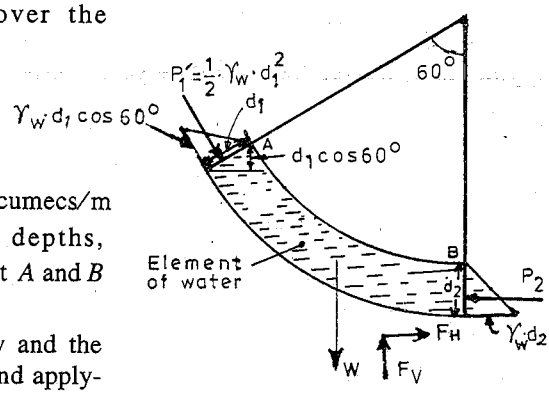


Fig. 21.34 (*b*)

$$15 + 2 = 2 + d_1 \cos 60° + \frac{V_1^2}{2g} = d_2 + \frac{V_2^2}{2g}$$

But $\quad V_1 = \frac{q}{d_1}$; and $V_2 = \frac{q}{d_2}$

$$\therefore \quad 17 = 2 + d_1 \cos 60° + \frac{q^2}{2gd_1^2} = d_2 + \frac{q^2}{2gd_2^2}$$

or
$$17 = 2 + \frac{d_1}{2} + \frac{(6.23)^2}{2 \times 9.81\, d_1^2} = d_2 + \frac{(6.23)^2}{2 \times 9.81\, d_2^2}$$

or
$$17 = 2 + \frac{d_1}{2} + \frac{1.97}{d_1^2} = d_2 + \frac{1.97}{d_2^2}$$

or
$$17 = 2 + \frac{d_1}{2} + \frac{1.97}{d_1^2} \qquad \text{...}(i)$$

and
$$17 = d_2 + \frac{1.97}{d_2^2} \qquad \text{...}(ii)$$

$\therefore$ (*i*) yields :

$$15 = \frac{d_1}{2} + \frac{1.97}{d_1^2}$$

or $$30d_1^2 = d_1^3 + 3.94$$

Solving by hit and trial, we get

$$\boxed{d_1 = 0.365 \text{ m}}$$

Similarly, solving equation (*ii*), we get

$$17 = d_2 + \frac{1.97}{d_2^2}$$

or $$17d_2^2 = d_2^3 + 1.97$$

by hit and trial, we get $\boxed{d_2 = 0.344 \text{ m}}$

Now the hydrostatic forces P_1 and P_2 per unit length are given as :

$$P_1 = \tfrac{1}{2} \cdot \rho_w \cdot d_1^2 \cos 60° = \tfrac{1}{2} \times 9.81 \times (0.365)^2 \times \tfrac{1}{2} = 0.327 \text{ kN/m run}$$

$$P_2 = \tfrac{1}{2} \cdot \rho_w \cdot d_2^2 = \tfrac{1}{2} \times 9.81 \times (0.344)^2 = 0.58 \text{ kN/m run}$$

Resolving all the Forces in the Horizontal Direction

Algebraic sum of all the forces in the horizontal direction

$$= P_1 \cos 60° + F_H - P_2 \qquad ...(iii)$$

The rate of change of momentum per unit length in the horizontal direction

$$= \frac{\rho_w \cdot q}{g} [V_2 - V_1 \cos 60°]$$

$$= \frac{\rho_w \cdot q}{g} \left[V_2 - \frac{V_1}{2} \right] \qquad ...(iv)$$

But $$V_1 = \frac{q}{d_1} = \frac{6.23}{0.365} = 17.06 \text{ m/sec.}; \text{ and } V_2 = \frac{q}{d_2} = \frac{6.23}{0.344} = 18.12 \text{ m/sec.}$$

Equating (*iii*) and (*iv*), we get

$$P_1 \cos 60° + F_H - P_2 = \frac{\rho_w q}{g} \left[V_2 - \frac{V_1}{2} \right]$$

or $$0.327 \times \tfrac{1}{2} + F_H - 0.58 = \frac{9.81 \times 6.23}{9.81} \left[18.12 - \frac{17.06}{2} \right] = 59.75$$

or $$F_H = \mathbf{60.16 \text{ kN}}$$

Absence of –ve sign indicates that the assumed direction of F_H is correct.

Resolving all the Forces in the Vertical Direction

The weight W of the water in the curved element AB

$$= \left[\frac{2\pi \cdot R}{360°} \times \theta° \right] \cdot y_{mean} (\rho_w)$$

$$= \left[\frac{2\pi \times 4}{360°} \times 60°\right] + \left[\frac{0.365 + 0.344}{2}\right] \times 9.81 = 14.57 \text{ kN}.$$

Resolving all the forces in the vertical direction, we get

$$W + P_1 \sin 60° - F_V = \frac{\gamma_w \cdot q}{g} [0 - V_1 \sin 60°]$$

or $$14.57 + 0.327 \times \frac{\sqrt{3}}{2} - F_V = \frac{9.81 \times 6.23}{9.81} \left[-17.06 \sin 60°\right]$$

$$14.85 - F_V = -92.04$$

or $$F_V = \mathbf{106.89\ kN}$$

Absence of –ve sign indicates that the assumed direction of F_V is correct and is vertically upward.

Resultant force $= \sqrt{F_H^2 + F_V^2} = \sqrt{(60.16)^2 + (106.89)^2}$

$= \mathbf{122.64\ kN.}$

Hence, the resultant dynamic force is equal to 122.64 kN and acts at an angle $= \tan^{-1}\left(\frac{106.89}{60.16}\right) = 60.6°$ with the horizontal in the upward direction, as shown in Fig. 21.34 (*c*).

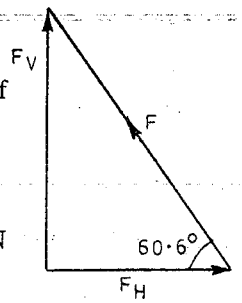


Fig. 21.34 (*c*)

SPILLWAY CREST GATES

If a temporary barrier can be installed over the permanent raised crest of a spillway, additional water can be stored between the spillway crest and the top of the barrier during the fag-end of the rainy season. The small flows in excess of the barrier top level, may be permitted to pass over the barrier. If, however, large flood occurs, the barrier may be removed and full spillway capacity made available for the outflow.

Sometimes on large dams, regular gates may be installed over the permanent crest, so as to function like a movable additional crest. In such a case, the height of the permanent raised crest can be reduced and the balance provided by the movable crest (*i.e.* gate). If there is a permanent raised crest up to the gate top, the storage, of course would be equal to that of a gated crest ; but in times of serious floods, the rise in flood level would be much more as compared to what would have been in a gated crest. This is because, the gates would be opened during serious floods so as to provide more head and hence larger discharge and consequent lesser rise in flood levels. Hence, the top level of the non-overflow section and the value of land acquisition for the reservoir which has to be determined by the maximum rise of flood above the spillway crest, can be reduced by providing gated crest or controlled crests. In other words, the dam height can be reduced for the same useful storage, or more useful storage can be obtained for the same height, provided the dam spillway is controlled by gates, etc.

This saving in the dam height and land acquisition will be more, if more height of the gates is provided. This saving is, however, counter-balanced by the cost of the gates. The cost of the gates would be more if their height is increased. (The gate cost include the principal cost and OMR, *i.e.* operational, maintenance and repair costs.) An economic balance between these two factors must be worked out and the cheapest combination found before deciding the height of the permanent crest and the height of

the temporary crest (*i.e.* gates). This is also governed by the limitations of maximum available gate heights.

Gates can be provided on all types of spillways except siphon spillways. In siphon spillways, the gates are not required as the rise in flood level is already small compared to other types of spillways. The gates for earthen dams should be provided with caution, since the faulty operation or failure of their operation may lead to serious rise in flood levels, causing overtopping and failure of dam.

21.17. Types of Spillway Gates

The various types of spillway gates are described below in brief :

21.17.1. Dropping Shutters or Permanent Flash Boards. They consist of wooden panels usually 1.0 to 1.25 m high. They are hinged at the bottom and are supported against the water pressure by struts (Fig. 21.35). The shutters fall flat on the crest when the downstream supporting struts are tripped. Hence they are not suitable for curved crests.

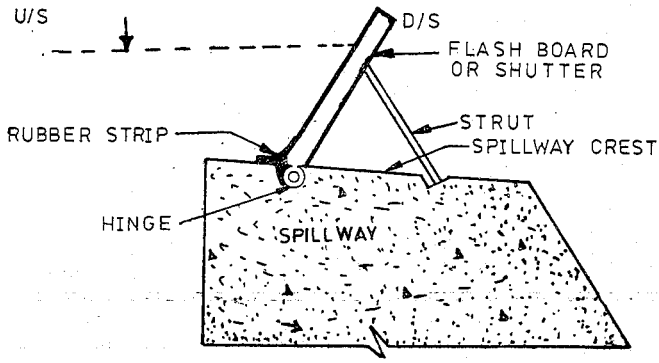


Fig. 21.35. Flash board or Dropping shutter.

These shutters can be raised or lowered from an overhead cableway or a bridge. Various types of shutters which drop and hoist themselves automatically, have been designed these days. These automatic shutters work on the principle of counter weights acting against the water pressure. Automatic shutters do not function well when interferred by floating debris, ice, etc.

Sometimes temporary flash boards, which shall fall as soon as overtopped by water, may be used for very minor works.

All kinds of flash-boards do have some disadvantages and hence used only on small spillways of minor importance.

21.17.2. Stop Logs and Needles. Stop logs consist of wooden beams or planks placed one upon the other and spanning in the grooves between the spillway piers (Fig. 21.36). They can be placed and removed either by hand or with hoisting mechanism. Considerable time may get wasted in removing them, if they become jammed in the slots. Leakage between the logs is also a big problem. They are, therefore, used on very minor works.

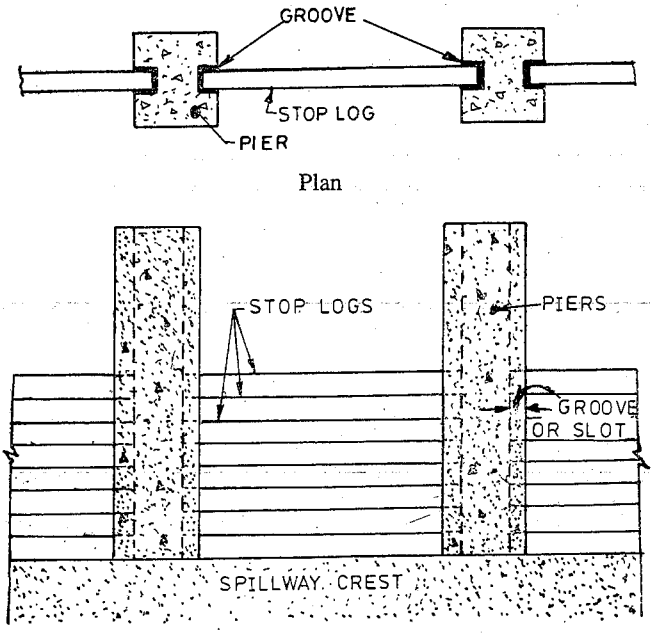


Fig. 21.36. Stop logs.

Needles. Needles are wooden logs kept side by side with their lower ends resting in a keyway on the spillway and upper ends supported by a bridge (Fig. 21.37). It is very difficult to handle these needles at the time of flow and hence they are not used on any major works. They are sometimes used for emergency bulk heads, where they need not be replaced until the flow has stopped.

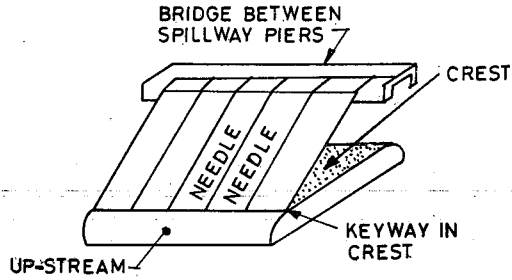
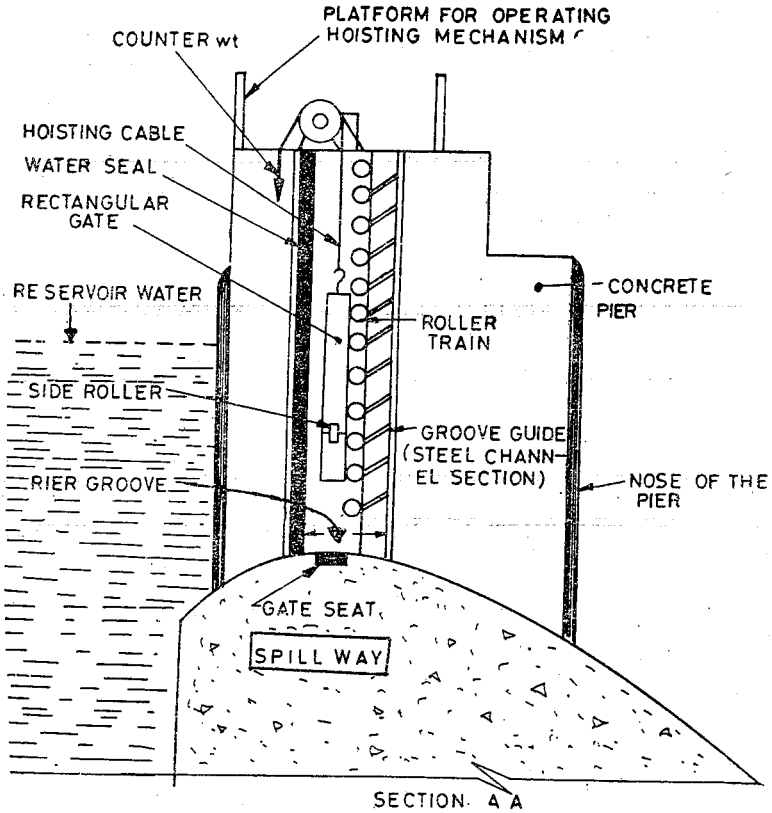


Fig. 21.37. Needles.

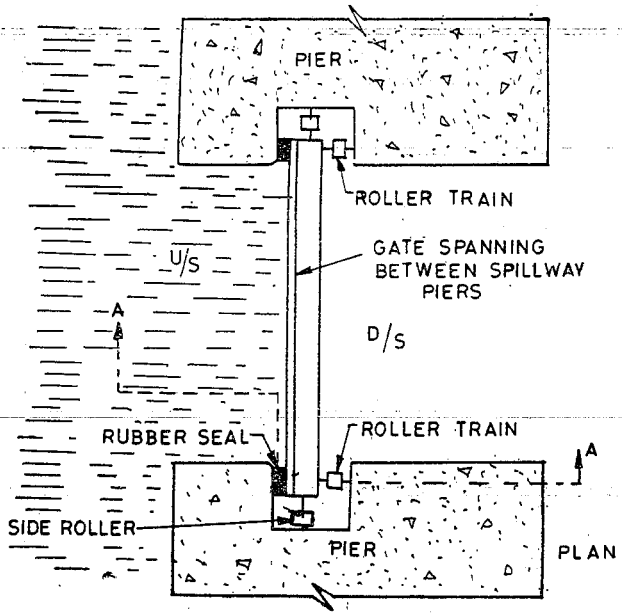
21.17.3. Vertical Lift Gates or Rectangular Gates. These are rectangular gates spanning horizontally between the grooves made in the supporting spillway piers (Fig.21.38). The grooves are generally lined with rolled steel channel sections of appropriate size, so as to provide a smooth bearing surface having sufficient bearing strength and are known as *grove guides.* These rectangular gates move between the groove guides, and can be raised or lowered by a hoisting mechanism at the top.

The gates are often made of steel, although they may be made of concrete or wood. They are generally placed vertical, although they may be kept slightly inclined downstream.

Because of the hydrostatic force caused by the upstream water standing against the gate, large friction is developed between the gate and the downstream groove guides. Hence, if the gate is in direct contact with the guides, as is there in a *sliding gate*, large friction will be developed, and it will be very difficult to move the gate. Hence, in a *sliding gate* relatively larger hoisting capacity is required to operate the gate because of the sliding friction that has to be overcome. The sliding gates are, therefore, seldom used.



(*a*)



(*b*)

Fig. 21.38. 'Vertical stoney gate' or 'Free roller gate'.

This friction problem can be solved by placing cylindrical rollers between the bearing surfaces of the gate and the guide grooves. A train of rollers or wheels is, therefore, generally placed between the gate and the d/s guide, so that the sliding friction is much smaller. These rollers may be placed independent of the gate and the guide, thus eliminating axle friction, as shown in Fig. 21.38. Such an arrangement, when the rollers are neither attached to the gate nor to the guide grooves but rolls vertically between the two when the gate is moved, is known as a *Stoney gate* or a *Free Roller gate.* The design and construction of such a gate is difficult and rollers are, therefore, generally attached to the gate. Such an arrangement in which the rollers or wheels are attached to the gate and ride in tracks on the downstream side of the groove guide, is known as a *Fixed wheels gate* or *Fixed roller type gate.* Rubber seals are used to seal the openings between the upstream leaf plate and the sides of the pier grooves, as shown.

Large vertical lift gates may be counter balanced by a counter weight beam, which is loaded to balance the self-weight of the gate. Hence, hoisting force is required only to balance the frictional resistance.

Vertical lift gates have been used in size 15 m × 15 m (height and span). If the gate height is larger, head room is required for lifting the gate clear of the maximum reservoir level ; thus increasing the height of the operating platform. To reduce the height of the operating platform, high gates may be broken up into two horizontal sections, so that the upper portion may be lifted and removed from the guides before the lower portion is moved. This also reduces the load on the hoisting mechanism.

The discharge through a partially raised vertical gate, takes place by an undershot orifice flow, and the discharge formula is given by $C_d A \sqrt{2g \cdot H_1}$, where A is the area of the opening and H_1 is the water head above the centre line of the opening.

21.17.4. Radial Gates or Tainter Gates. A radial gate has a curved water supporting face made of steel. The curved water face which is in the shape of a sector of a circle is properly braced by steel frame work which is pivoted on horizontal shafts called *trunnions* or *pins* (Fig. 21.39). The pins are anchored in the downstream portion of the spillway piers. The gate can

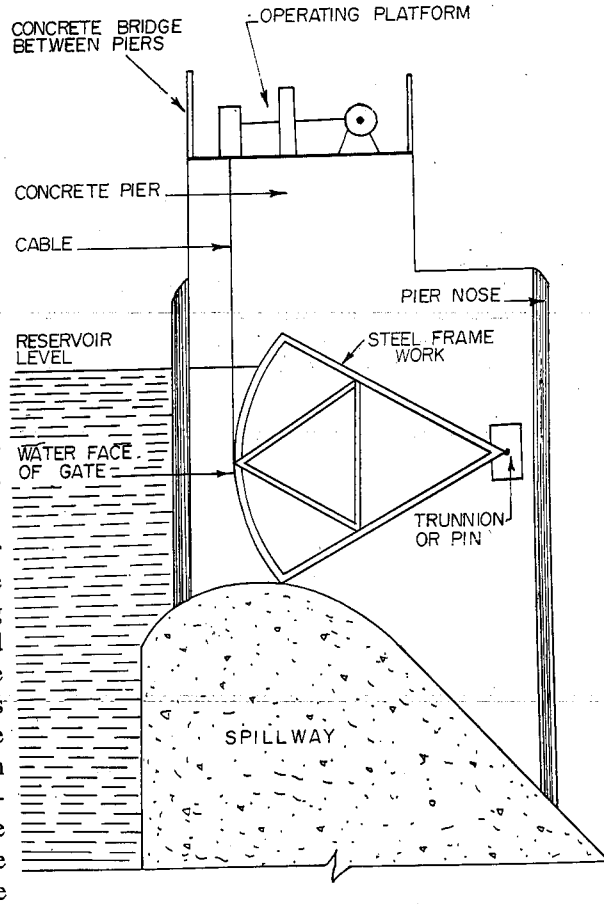


Fig. 21.39. Radial gate or Tainter gate.

thus rotate about the fixed horizontal axis. Hoisting cables are attached to the gate and lead to winches on the hoisting platform. The winches are usually motor driven, although hand driving is possible for smaller works or at times of power failures.

The water-face segment is made concentric to the supporting pins so that the entire water thrust passes through the pins, thus creating no moment against the lifting of the gate. Hence, the lifting force is required only against the weight of the gate, the friction between the seals and the piers, and the frictional resistance at the pins. Counter weights, in order to counter balance the self weight, may also be used, which further reduces the lifting force. Moreover, the hoisting load is nearly constant for all gate openings. Hence, radial gates can be used with smaller lifting force for all heads, and hand operating hoisting mechanism may suffice for smaller works ; whereas in the vertical lift gates of the same size, power mechanism might be needed.

21.17.5. Drum Gates. Drum gates are useful for longer spans of the order of 40 m or so and medium heights say 10 m or so. The drum gate consists of a segment of a cylinder which may be raised above the spillway crest or may be lowered into the *recess* made into the top of spillway.

The U.S.B.R. drum gate (Fig. 21.40) is completely enclosed and is hinged at the upstream end. The buoyant forces due to head water pressure underneath the drum, aid

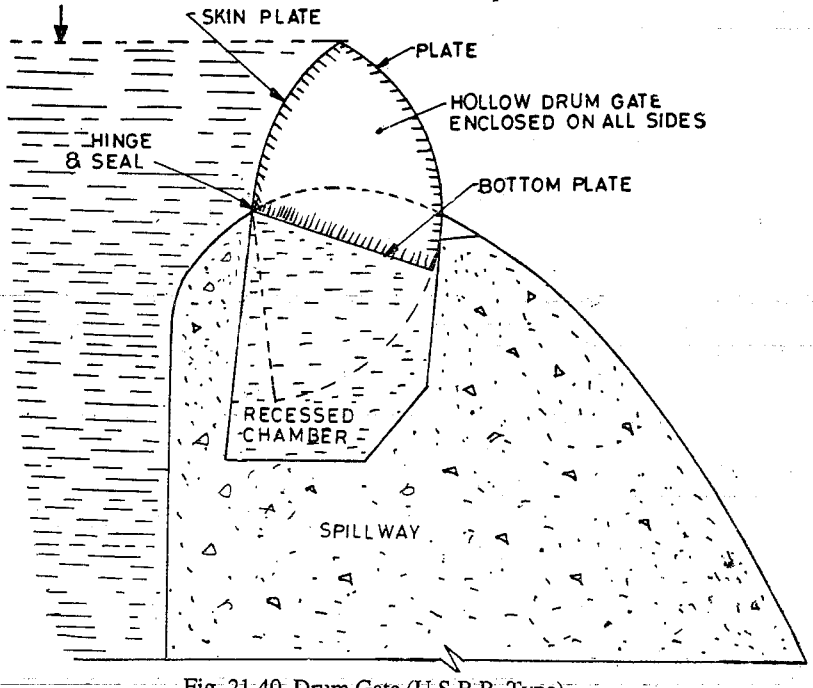


Fig. 21.40. Drum Gate (U.S.B.R. Type).

in its lifting. In this type of dam gate, the drum is enclosed on all the three sides as well as on the ends, thus forming a water tight vessel. When the drum is lowered, it fits into the recess in such a way that the surface becomes coincident with the designed ogee shape of the crest.

The other type of drum gate may have no bottom plate and shall be raised only by the buoyant action of water entering the recess, underneath the skin plate of drum.

The drum gates require large recess and hence, are not suitable for smaller spillways.

PROBLEMS

1. (*a*) "A spillway is a safety valve in a dam". Discuss the statement.

(*b*) Discuss the location of the 'main spillway' and 'subsidiary spillway' in gravity dams as well as in earthen dams.

(*c*) Briefly describe an 'ogee spillway'.

2. (*a*) What are spillways and where are they provided ?

(*b*) Enumerate the various types of spillways, and describe in details the most widely used type.

3. (*a*) What are the different kinds of spillways and how are they selected for individual conditions ? (*Madras University, 1975*)

(*b*) Sketch an ogee profile and mark in it the different zones. (*Madras University, 1976*)

4. (*a*) What are spillways and what is their necessity ?

(*b*) Enumerate the different type of spillways which are used in dam construction.

(*c*) Discuss briefly the design principles that are involved in the design of an ogee spillway ; and a chute spillway.

5. (*a*) What is an ogee spillway and how is it designed ?

(*b*) Discuss the necessity of the aeration arrangements that are required in gated ogee spillways.

6. Write down an equation for calculating the discharging capacity of an ogee spillway. How does 'coefficient of discharge' in this equation varies with the :

(*i*) Depth of approach ;

(*ii*) Slope of upstream face of the ogee spillway ;

(*iii*) Submergence of the spillway by the tail water ;

(*iv*) The ratio of design head to actual head over the spillway.

7. (*a*) How would your compute the discharge passing over an ogee spillway. Discuss the various factors affecting the coefficient of discharge in the discharge equation.

(*b*) Compute the discharge over an ogee spillway with coefficient of discharge $c = 2.3$ at a head of 3.8 m. The effective length of the spillway is 110 m. Neglect the velocity of approach.

[**Ans.** 1872 cumecs]

8. Design a suitable section for the overflow section of a concrete gravity dam having the d/s face sloping at a slope of 0.7 H : 1 V. The design discharge for the spillway is 6000 cumecs. The height of the spillway above the river bed is 60 m. The effective length of spillway may be taken as 50 m.

[**Hint.** Follow example 21.1].

9. (*a*) What is meant by a 'spillway' and what is its necessity in dam construction ?

(*b*) Enumerate the different types of spillways, and draw neat sketches for all the types, showing the different parts of each.

10. (*a*) What is a siphon spillway ? Enumerate the two types of saddle syphon spillways, and describe with a neat sketch the component parts and functioning of one of these two types :

(*b*) A siphon spillway has the following cross-section at its throat :

Height of the throat = 1.5 m

Width of the throat = 4 m.

At the design flow, the tail water elevation in 7 m below the summit of the siphon, and the head water elevation is 2 m above the summit, (*i*) Taking a coefficient of discharge as 0.6 ; determine the capacity of the syphon (*ii*) Determine the head that would be required on an ogee spillway 3.8 m long to discharge this flow, if coefficient of discharge is 2.25 ; (*iii*) What length of this ogee weir would be required to discharge the same flow with a head of 2.2 m on the crest.

[**Solution.** (*a*) Discharge through siphon spillway (*i.e.* capacity)

$$= 0.6\,(1.5 \times 4)\sqrt{2 \times 9.81 \times (7+2)}$$

$$= 47.8 \text{ cumecs. } \textbf{Ans.}$$

(*b*) $$47.8 = 2.25 \times 3.8\, H^{3/2}$$

or $$H = 3.15 \text{ m. } \textbf{Ans.}$$

(*c*) $4.78 = 2.25\, L\, (2.2)^{3/2}$

or $L = 6.5$ m. **Ans.]**

11. (*a*) Describe briefly the construction and functioning of a 'Hooded type' of siphon spillway. What is its discharge equation ?

(*b*) What is meant by priming ? Discuss the priming arrangements used in saddle siphon spillways.

12. (*a*) What is a 'chute spillway' ? Where is it preferred to ogee and other types of spillways ?

(*b*) Discuss briefly the component parts and their design for a chute spillway.

13. Discuss the design of the following components of a chute spillway :

(*i*) Approach channel

(*ii*) Control structure

(*iii*) Discharge carrier

(*iv*) Energy dissipation arrangement at the bottom.

14. Write short notes on :

(*i*) Volute siphon

(*ii*) Side channel spillway or Chute spillway or Shaft spillway

(*iii*) Ogee spillway

(*iv*) Priming devices for siphon spillways.

15. What is meant by an 'energy dissipator' ? Discuss the various methods used for energy dissipation below spillways.

16. Discuss briefly the various types of energy dissipators that are used for energy dissipation below overflow spillways, under different relative positions of T.W.C. and J.H.C.

17. (*a*) What is a 'spillway gate' and what are the merits and demerits of installing such gates ?

(*b*) Enumerate the important types of spillway gates. Describe with a neat sketch the construction and working of 'Tainter gate'.

18. (*a*) Discuss the merits and demerits of different types of spillway gates.

(*b*) Discuss with a neat sketch the construction, and functioning of a 'Stoney gate' as well as a 'Drum gate'.

19. Write detailed notes on the following :

(*i*) Energy dissipation below spillways

or

Spillway buckets and stilling basins.

(*ii*) Spillway gates.

22

Outlet Works Through Dams and River Intakes

22.1. Sluicesways or Dam Outlets

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

The outlets of most of the dams consist of one or more sluiceways with their inlets at about minimum pool level. In most of the cases, a number of outlets are generally provided at different levels ; as a single large capacity outlet may be structurally unsatisfactory or difficult to construct. Moreover, by having more number of smaller capacity outlets, greater control on discharge can be obtained, which can be varied as and when desired. Hence, when wider fluctuations in the demand are anticipated, it is always, advisable to go in for more number of small capacity sluiceways, although it may prove to be somewhat costlier as compared to a few large capacity sluiceways.

A sluiceway is a pipe or a tunnel, circular or rectangular in section, that passes through the body of the dam or through some hillside at one end of the dam and discharges into the stream below. For masonry or concrete gravity dams, these sluiceways can easily pass through the body of the dam or the spillway (Fig. 22.1) ; but for earthen dams, it is preferred to place them outside the limits of the embankments. But, if no such adjacent hill site is available and there is no alternative left but to pass the sluiceway through the dam, *projecting collars* must be provided so as to reduce seepage along the outside of the conduit as shown in Fig. 22.2. The seepage is thus, reduced by increasing the length of the seepage path by at least 25 per cent.

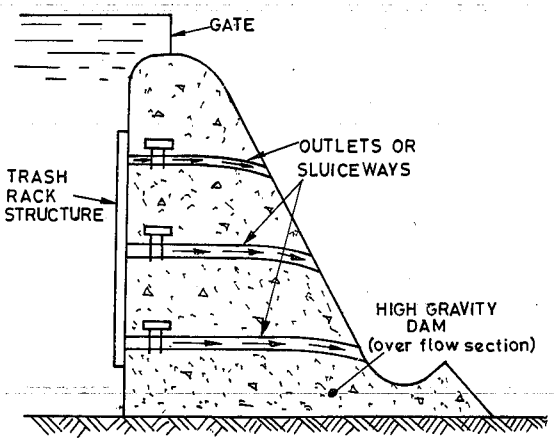


Fig. 22.1. Typical outlet arrangements through a concrete gravity dam.

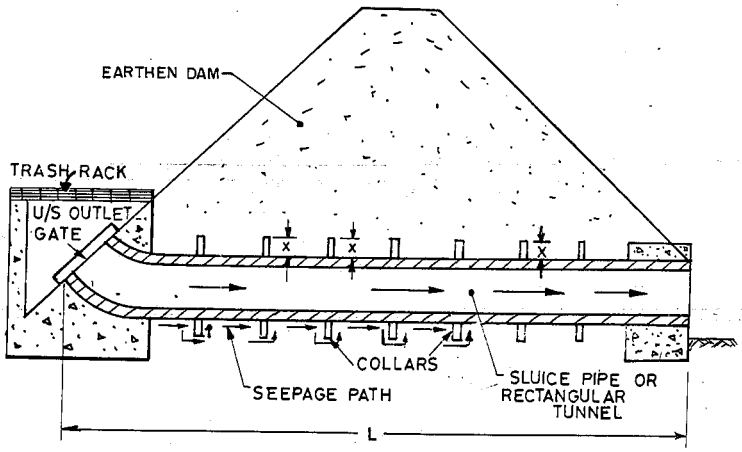


Fig. 22.2. Typical outlet arrangement through an earthen dam.

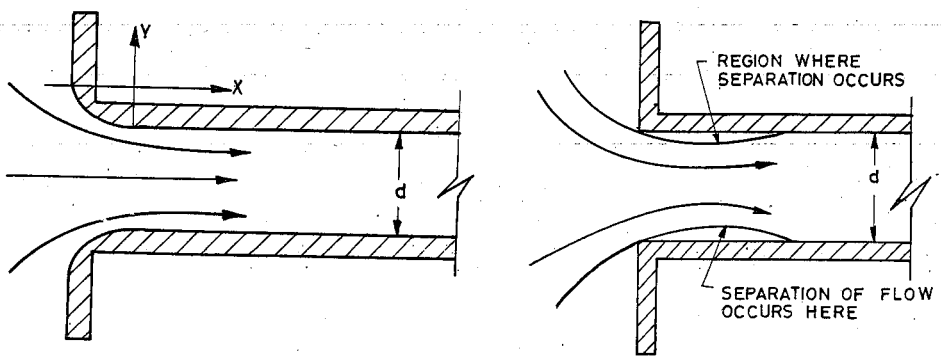
In Fig. 22.2, if L is the total length of sluiceway from upstream to downstream, the length of the seepage path will be given as :

$$= L + (2x)N$$

where x is the projection of each collar ; and
N is the number of collars.

The increase in seepage path, *i.e.* $2 \cdot N \cdot x$ must generally be greater than $L/4$.

The interior of sluiceway tunnel or pipe, must be smooth, without any projections or cavities. The projections or cavities, etc. if at all present, may cause separation of flow from the boundary of the conduit, causing development of negative pressures and the consequent danger of cavitation*. The entrance of the sluiceway is also very impor-



(*a*) Square edged entrance (*b*) Bell-mouthed entrance.

Fig. 22.3

tant, since a square edged entrance [Fig. 22.3 (*a*)] is likely to cause more separation of flow and more danger of consequent cavitation as compared to a bell mouthed or some other type of entrance. There have been several such cases where large scale damages have occurred due to cavitation in the vicinity of square-edged entrances. A bell-mouthed entrance [Fig. 22.3 (*b*)] is much better and superior to any other type, and the

* Explained earlier in Chapter 21.

extra cost involved in shaping this entrance is usually justified except for small projects under low heads.

The shape of the bell mouth is generally elliptical, and Douma has suggested the following equations for it :

(*i*) *For circular conduits.*

$$4X^2 + 44.4\,Y^2 = d^2 \qquad ...(22.1)$$

where X, Y are the co-ordinates of any point on the curve as defined in Fig. 22.2 (*b*) ; and
d is diameter of the circular conduit.

(*ii*) *For rectangular tunnels or conduits*

$$X^2 + 10.4\,Y^2 = d^2 \qquad ...(22.2)$$

where X and Y have same meaning as above ; and
d is the width or height of the conduit, depending on whether the sides or top and bottom curves are being designed.

22.2. Hydraulics of Outlet Works

The discharge passing through the dam outlet can be calculated easily, be using the equation :

$$Q = C_d \cdot A\sqrt{2gH} \qquad ...[22.3(a)]$$

where Q = Discharge
A = Area of outlet sluice
H = Differential head causing flow, *i.e.* the difference of u/s and d/s water levels.
C_d = Coefficient of discharge, whose value depends upon various factors such as the type of gate and trash rack provided, conduit friction, transitions etc.

For precise designs, the total head losses encountered in the outlet conduits may be calculated. These losses include those caused by trash racks, conduit entrance, conduit friction, gates and valves, transitions and bends. The entrance loss may be taken as $0.5\,\frac{V^2}{2g}$ for square edged entrance, and equal to $0.04\,\frac{V^2}{2g}$ for bell mouthed entrance, where V is the flow velocity through the conduit. Head losses caused by conduit friction may be calculated by standard pipe formulas, such as :

$$h_L = \frac{f' \cdot LV^2}{2gd}$$

or

$$h_L = \frac{n^2 \cdot V^2 \cdot L}{R^{4/3}}$$

The head loss through the gate depends upon the type of gate and valve provided. A loss of about $0.2\,\frac{V^2}{2g}$ may be taken for fully open gate and butterfly valves. This head loss is taken as nil for ring follower gates. The head loss through the trash rack depends upon the velocity, and may be taken as shown in Table 22.1.

Table 22.1. Head Loss through Trash Racks

Velocity through trash rack in m/sec	*Head loss in metres*
0.15	0.006
0.30	0.03
0.45	0.09
0.62	0.15

The net effective head which is responsible for flow, should then be taken as $= H_{eff} =$ Differential head (H) – Head losses. The discharge can then be calculated easily, using

$$Q = A\sqrt{2g \cdot H_{eff}} \qquad ...[22.3\ (b)]$$

22.3. River Intakes

When water is withdrawn through a conduit from a river or a reservoir independently, and as such the entrance of the conduit is not an integral part of the dam or any other related structure, then an intake structure must be constructed at the entrance of the conduit. An intake structure may vary from a simple concrete block supporting the end of the conduit pipe to huge concrete towers, depending upon various factors such as, reservoir characteristics, capacity and discharge requirements, climatic conditions, etc. The basic function of an intake structure is to help in safely withdrawing water from the reservoir over a predetermined range of pool levels, and thus to protect the conduit from being damaged or clogged by ice, trash, debris, waves, etc.

Various types of intake structures are discussed below :

(1) **Simple Submerged Intakes.** A simple submerged intake consists of a simple concrete block, or a rock filled timber crib supporting the starting end of the withdrawal pipe, as shown in Figs. 22.4 (*a*) and (*b*).

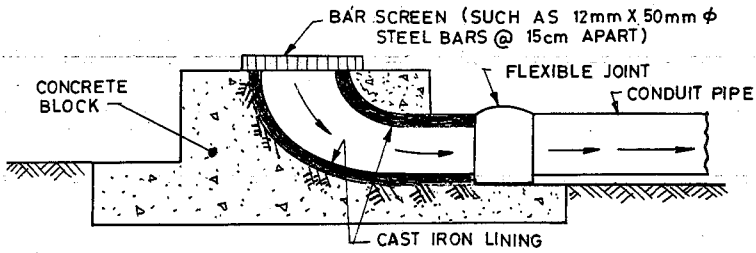


Fig. 22.4. (*a*) Simple concrete block submerged intake.

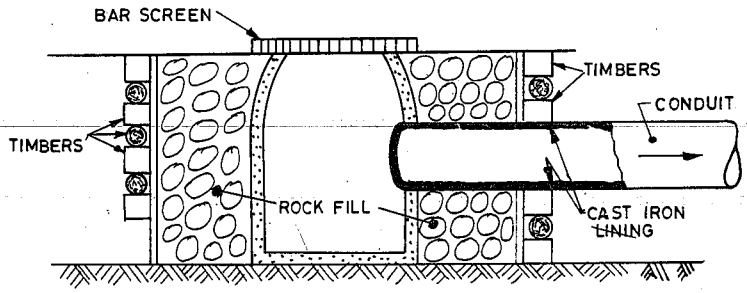


Fig. 22.4. (*b*) Rock-filled timber-crib submerged intake.

Such intake structures should be placed in the river or the reservoir at a place where they may not get buried under the sediment. These submerged intakes are very economi-

cal and donot obstruct navigation, and are, therefore, widely used on small works, and are particularly suitable as water-supply intakes from small rivers*. They are sometimes used as intakes to sluiceways of earthen dams with hydraulically operated gates for flow regulation. These intakes are not used on bigger projects as their main disadvantage is the fact that they are not easily accessible for repairing of their gates, etc.

(2) **Intake Towers.** Intake towers are generally used on large projects and where there are large fluctuations of water level. Openings at various levels, called *ports* are generally provided in these concrete towers, which may help in regulating the flow through the towers and permit some selection of the quality of water to be withdrawn. If the ports are submerged at all levels, then there is no problem of any clogging or damage by ice or debris, etc. However, the level of the lowest port should be high enough above the reservoir bed, so that the sediment is not drawn into them. There are two major types of intake towers, *viz.*

(*i*) Wet intake towers ; and

(*ii*) Dry intake towers.

(*i*) *Wet intake towers.* A typical section of a wet intake tower is shown in Fig. 22.5 (*a*). It consists of a concrete circular shell filled with water upto the reservoir level, and has

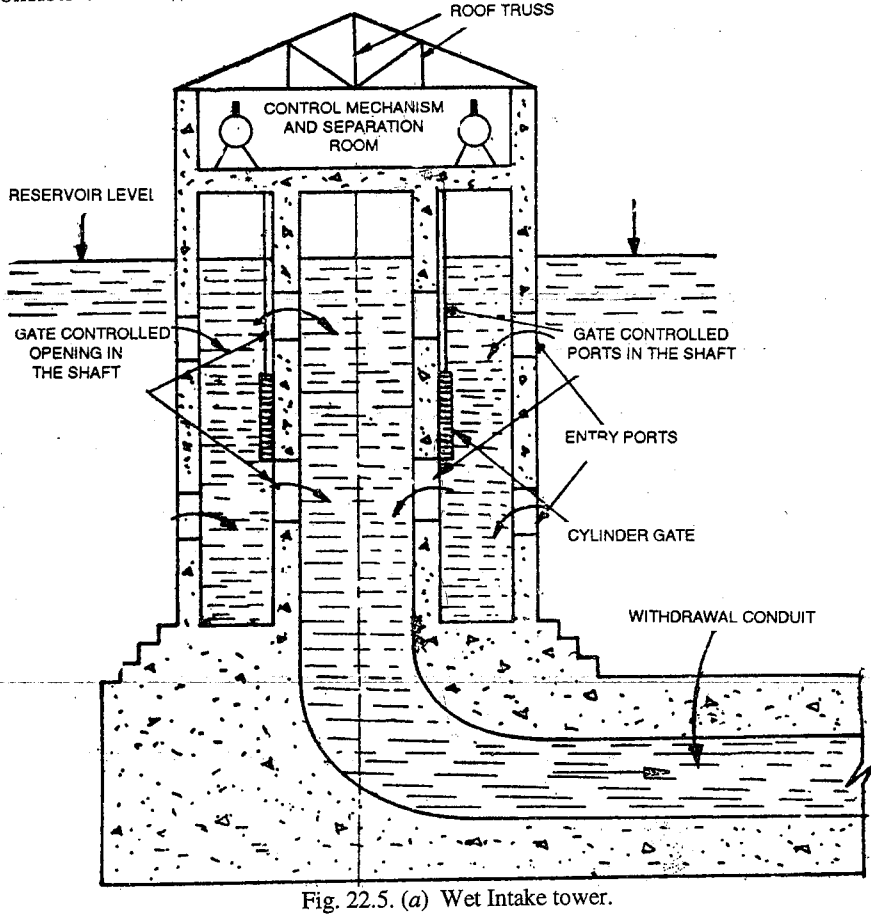


Fig. 22.5. (*a*) Wet Intake tower.

* More detailed description of river and canal intakes is given in "Water Supply Engineering" by the same author.

a vertical inside shaft which is connected to the withdrawal pipe. Openings are made into the inside shaft also. Gates are usually placed on the shaft, so as to control the flow of water into the shaft and the withdrawal conduit.

(*ii*) *Dry Intake Towers.* The essential difference between a wet intake tower and a dry intake tower is that, whereas in a wet intake tower, the water enters from the entry ports into the tower and then it enters into the conduit pipe through separate gate controlled openings ; in a dry intake tower, the water is directly drawn into the withdrawal conduit through the gated entry ports as shown in Fig. 22.5 (*b*). *A dry intake tower will, therefore, have no water inside* the tower if its gates are closed, whereas the

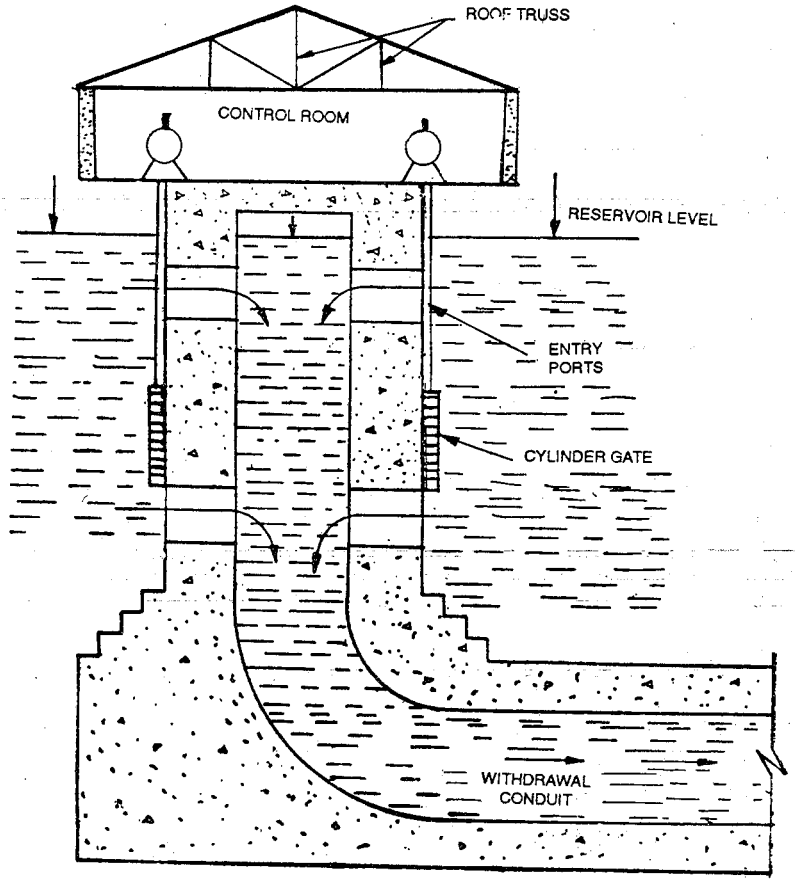


Fig. 22.5 (*b*). Dry Intake Tower.

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

Intake towers are huge structures standing in the river, and hence should be located so as not to interfere with navigation, and must be properly designed so as to withstand the worst possible combination of various forces, such as hydrostatic pressures, wind and earthquake forces, and forces caused by waves, ice and debris, etc.

22.4. Trash Racks

The entrances to intakes and dam outlets are generally covered with trash racks so as to prevent the entry of debris, ice, etc. into the conduit. These racks are generally nothing but bar screens, made from steel bars spaced at 5 to 15 cm centre to centre (in both directions) depending upon the maximum size of the debris required to be excluded from entering the conduit. The velocity of flow through the trash rack is kept low (generally less than 0.62 m/sec), so as to minimise losses. This is sometimes accomplished by constructing the trash rack in the form of a half cylinder, as shown in Fig. 22.6.

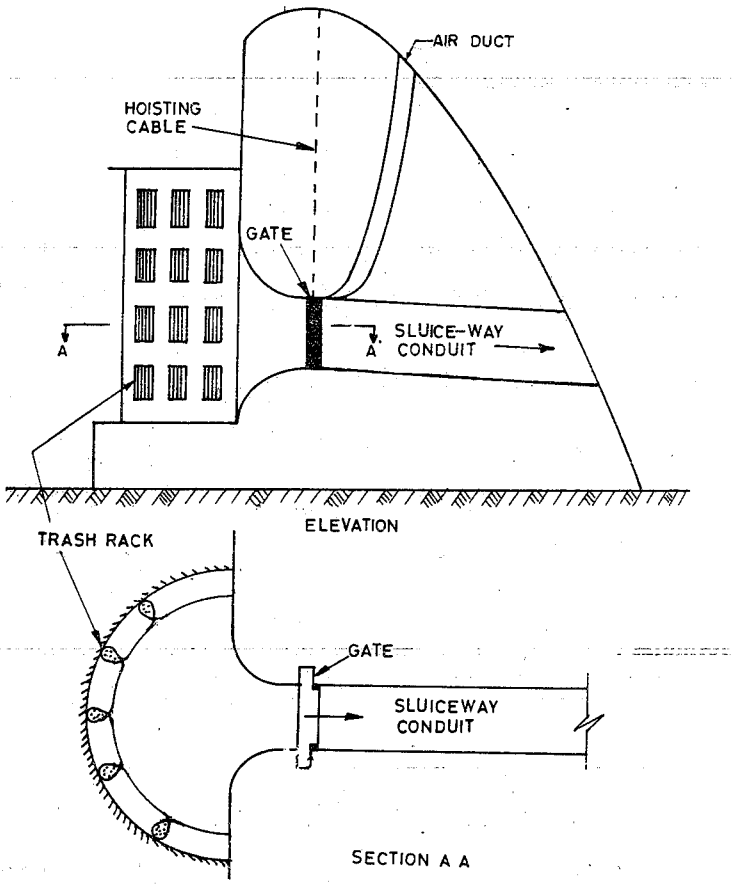


Fig. 22.6. Half Cylinder-Trash rack and Tractor gate installation.

The floating debris, ice, etc. which are stopped by the racks and thus get collected on them, are removed by manual labour when required ; but on large projects, where much debris are expected, automatic power-driven racks are used. Steam or electrically heated racks are preferred in cold countries, so as to prevent ice formation on trash racks.

PROBLEMS

1. (*a*) What is meant by a 'Dam sluice' ? Why are such sluices necessary in dam construction ?
(*b*) Show with neat sketches, the provision of sluiceways in
   (*i*) an earth dam.
   (*ii*) a concrete gravity dam.
2. Write short notes on :
   (*i*) Sluiceways in dams.
   (*ii*) River intakes.
   (*iii*) Trash Racks.

23

Pressure Conduits

23.1. Definition, Etc.

A pressure conduit (such as a penstock) is a pipe which runs under pressure and, therefore, runs full. This type of conduits prove economical than canals or flumes, because they can generally follow shorter routes. Moreover, their biggest advantage is: that the water or any other fluid flowing through them is not exposed anywhere and hence, there are no chances or very less chances of its getting polluted. Hence, these pressure conduits are preferably used for city water supplies. Since the water wasted in percolation, evaporation, etc. is also saved, when water is carried through these conduits, they are preferably used when water is scarce. The flow of water through conduit pipes is generally turbulent, and hence, it will be considered so, while dealing with the hydraulics of flow through such pipes.

23.2. Hydraulics of Flow and Discharging Capacities of Pressure Conduits

The pressure conduits are designed in such a way that the available pressure head is just lost in overcoming the frictional resistance offered to the flow of water by the pipe. The velocities generated in these pipes should neither be too small to require a large size diameter pipe, nor should be so large as to cause excessive loss of pressure head.

The head loss by pipe friction may be found by using either of the following formulas :

(*i*) **Darcy-Weisbach Equation.** It states that

$$H_L = \frac{f' \cdot L \cdot V^2}{2gd}$$

where H_L = Head loss in metres.

L = Length of the pipe in metres.

d = Diameter of the pipe in metres.

V = Velocity of flow through the pipe in m/sec.

f' = The friction factor, which depends upon Reynold number $\left(i.e.\ R_e = \frac{Vd}{\nu}\right)$ and relative roughness of the pipe.

The relative roughness $\left(\delta = \frac{2e}{d}\right)$ of a pipe depends on the absolute roughness (e) of the inside surface and the diameter of the pipe d. The approximate values of f' are given by the following empirical relations :

$$f' = 0.04\left[1 + \frac{1}{35\,d}\right] \text{ for old pipes} \qquad ...(23.2)$$

and $$f' = 0.02\left[1 + \frac{1}{35\,d}\right] \text{ for new pipes} \quad ...(23.3)$$

The accurate value of f' depends on R_e and δ, and may be given by formula, such as

$$f' = a + \frac{b}{R_e^m} \quad ...(23.4)$$

where a, b and m are constants depending upon δ.

For R_e varying between 20,000 to 20,00,000, Schiller has given

$$f' = 0.005 + \frac{0.396}{R_e^{0.3}} \quad ...(23.5)$$

and Nikuradse has given for values of R_e varying between 20,000 to 32,40,000 as

$$f' = 0.0032 + \frac{0.221}{R_e^{0.237}} \quad ...(23.6)$$

(*ii*) **Manning's formula.** Manning's formula is also applicable to turbulent flow in pressure conduits and yields good results, provided the roughness coefficient n is accurately estimated. Head loss according to Manning's equation is given as :

$$H_L = \frac{n^2 \cdot V^2 \cdot L}{R^{4/3}} \quad ...(23.7)$$

where n = Manning's rugosity coefficient.
L = Length of pipe in metres.
V = Flow velocity through pipe in m/sec.
R = Hydraulic mean depth of pipe

$$\left[i.e.,\ R = \frac{A}{P} = \frac{\frac{\pi d^2}{4}}{\pi d} = \frac{d}{4}\right] \text{in metres}$$

(*iii*) **Hazen-William's formula.** This formula is widely used for pipe flows and states

$$V = 0.85\, C_H \cdot R^{0.63}\, S^{0.54} \quad ...(23.8)$$

where C_H = Coefficient given by Table 23.1.
R = Hydraulic mean depth of pipe, $= (d/4)$ in metres.
S = Slope of the energy line.
V = Flow velocity through the pipe in m/sec.

Table 23.1. Values of C_H for Hazen William's Formula

Pipe material	*Value of C_H*
Concrete (regardless of age)	130
Cast Iron	
New	130
5 years old	120
20 years old	100
Welded steel (New)	120
Riveted steel (New)	110
Vitrified clay	110
Brick Sewers	100
Asbestos-cement	140

The carrying capacities of pipes decrease with time, because with the passage of time, their interior surfaces go on becoming more and more rough. Hence, it is customary to select higher values of e and n and lower values of C_H for old pipes than are generally taken for new pipes.

23.3. Forces Acting on Pressure Conduits

Pressure pipes must be designed to withstand the following forces :

(1) *Internal pressure of water*

(2) *Water hammer pressures.*

(3) *Pressure due to external loads* (when buried under the ground).

(4) *Temperatures stresses* (when laid above the ground).

(5) *Longitudinal stresses* due to flow around bends or change in cross-section.

(6) *Flexural stresses* (when laid over supports at intervals or on bridges).

These forces are discussed below :

(1) **Internal Pressure of Water.** The pressure exerted on the walls of the pipe by the flowing water, in the form of Hoope's tension, is the internal pressure. The circumferential tensile stress produced is given as :

$$\sigma_1 = \frac{p_1 d}{2t} \text{ in kN/m}^2 \qquad ...(23.9)$$

where p_1 = Internal static pressure in kN/m^2.

d = Diameter of the pipe in metres.

t = Thickness of the pipe shell in metres.

σ_1 = Circumferential tensile stress to be counteracted by providing Hoope's reinforcement.

(2) **Water Hammer Pressure.** When a liquid flowing in a pipe line is abruptly stopped by the closing of a valve, the velocity of the water column behind, is retarded, and its momentum is destroyed. This exerts a thrust on the valve and additional pressure on the pipe shell behind. The more rapid the closure of the valve, the more rapid is the change in momentum, and hence, greater is the additional pressure developed. The pressures so developed are known as *water-hammer pressures* and may be so high as to cause bursting of the pipe shell (due to increased circumferential tension) if not accounted for in the designs.

The maximum pressure developed in pipe lines due to water hammer is given by the formula

$$p_2 = \frac{14.762 \cdot V}{\sqrt{1 + \frac{K \cdot d}{t}}} \qquad ...(23.10)$$

where V = Velocity of water just before the closing of the valve in m/sec.

d = Diameter of pipe in metres.

t = Thickness of pipe shell in metres.

K = Constant

$$= \frac{\text{Modulus of elasticity of pipe material}}{\text{Bulk modulus of elasticity of water}}$$

The value of K for steel comes out to be 0.01, for cast iron = 0.02, and for cement concrete = 0.1.

The water hammer pressure can be calculated by using the above equation. But for design purposes, its value is generally taken as 840 kN/m^2 for small size pipes of 7.5 to 25 cm diameter, and is gradually reduced to 490 kN/m^2 for pipes above 1.2 m diameter, as given in Table 23.2.

Table 23.2. Water Hammer Pressures

Pipe diameter in cm	*Water hammer Pressure in kN/m^2*
7.5 to 25	840
30 to 40	770
50	630
60	600
75	560
90	490
105 to 135	490

The circumferential tensile stress caused by water hammer pressure may then be calculated easily by using

$$\sigma_2 = \frac{p_2 \cdot d}{2t}$$

(3) **Stress due to External Loads.** When large pipes are buried deep under the ground, the weight of the earth-fill may produce large stresses in the pipe material. The stress due to the external earth fill load is given by

$$f = 22.7 \frac{h \cdot d^2}{t} \qquad ...(23.11)$$

where h = depth of the earth-fill above the crown in metres.

d = diameter of pipe in metres.

f = stress produced in kN/m^2.

Note. In the above formula, it is assumed that the earth to the sides does not give any lateral support and weighs about 18.4 kN/m^3.

(4) **Temperatures Stresses.** When pipes are laid above the ground, they are exposed to the atmosphere and are, therefore, subjected to temperature changes. They expand during day time and contract at night. If this expansion or contraction is prevented due to fixation or friction over the supports, longitudinal stresses are produced in the pipe material. The amount of these stresses may be calculated by the formula :

$$f = E \cdot \alpha \cdot T \qquad ...(23.12)$$

where E = Modulus of elasticity of the pipe material.

α = Co-efficient of expansion of the pipe material.

T = Change in temperature in °C.

Expansion joints at suitable intervals (say 20 m to 30 m or so) must be provided to counteract these stresses.

(5) **Stresses due to Flow Around Bends and Change in Cross-Section.** Whenever the velocity of a flow (either magnitude or direction) changes, there is a change in the momentum, and therefore, by Newton's Second Law, a force is exerted, which is proportional to the rate of change of momentum. The force required to bring this change in momentum comes from the pressure variation within the fluid and from forces transmitted to the fluid from the pipe walls.

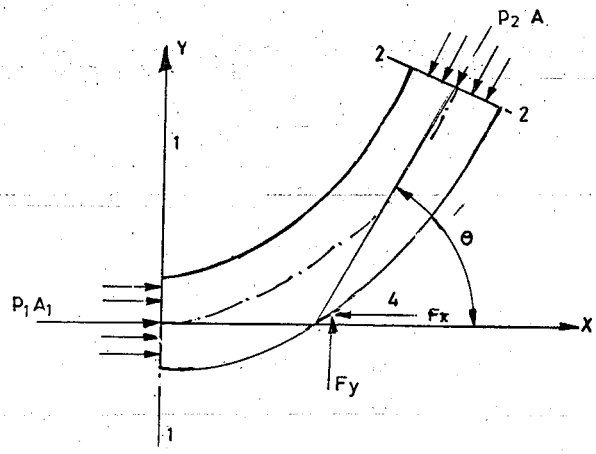


Fig. 23.1. Forces at a pipe bend.

The free-body diagram of various forces acting on the water contained in a horizontal pipe bend is shown in Fig. 23.1. Applying momentum equation, and resolving the forces in x and y direction, we get

$$p_1A_1 - F_x - p_2A_2 \cos\theta = \frac{\gamma_w Q}{g}\left[V_2 \cos\theta - V_1\right] \quad ...(23.13)$$

and

$$F_y - p_2A_2 \sin\theta = \frac{\gamma_w Q}{g}\left[V_2 \sin\theta\right] \quad ...(23.14)$$

where p_1 and p_2 are the pressures, V_1 and V_2 are the velocities at sections 1-1 and 2-2 respectively. The forces F_x and F_y are the forces which are transmitted from the pipe to the water. An equal and opposite force must, therefore, be developed in the form of stresses in the pipe wall. Similar forces will be developed when the cross-sectional area of the pipe is suddenly changed, as shown in Fig. 23.2.

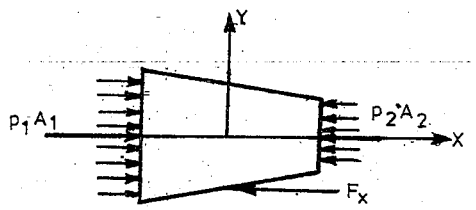


Fig. 23.2. Forces at a change in X-section of a pipe.

Due to these impressed external forces or stresses, the pipe line may be thrown out of alignment as and when such situations arise, unless held firmly by anchoring it in massive blocks of concrete or stone masonry.

(6) **Flexural Stresses.** Many a times, steel pipes are laid over concrete supports, built above the ground ; and sometimes the rain water, etc. may wash off the ground from below the pipes at intervals. Under all such circumstances, bending stresses get produced in the pipe, since the pipe then acts like a beam with loads resulting from the weight of the pipe, weight of water in the pipe and any other superimposed loads. The stresses caused by this beam action may be determined by usual methods of analysis applied to the beams. However, these stresses are generally negligible except for long spans or where there are huge superimposed loads.

23.4. Various Types of Pressure Conduits

Depending upon the construction material, the various types of pressure conduits are :

(1) Cast iron conduits.

(2) Steel conduits.

(3) Reinforced cement concrete conduits.

(4) Hume steel conduits.

(5) Vitrified clay conduits.

(6) Asbestos cement conduits.

(7) Miscellaneous types of conduits.

The selection of a particular type of material for the conduit depends mainly upon their relative economy and also upon the availability of material, labour, etc. for construction in the vicinity of site.

(1) **Cast Iron Conduits.** Cast iron pipes are widely used for city water supplies. They are sufficiently resistant to corrosion and may last as long as 100 years or so. They are generally manufactured in lengths of about 3.6 metres but may be manufactured in lengths up to 6 m or so on special orders. Cast iron pipes in different thicknesses, are generally available for withstanding different pressures upto a maximum of 2400 kN/m^2 (24.5 kg/cm^2).

In order to protect the pipes from corrosion, almost all the metal pipes are given some kind of protective coatings. Larger pipes are generally protected by bituminous linings or by cement mortar (1 : 2) linings. Smaller size pipes, such as those required for small service lanes in a water supply distribution system, are generally coated with zinc, and are known as galvanised iron pipes.

Bell and spigot joint (Fig. 23.3) is often used for connecting cast iron pipes.

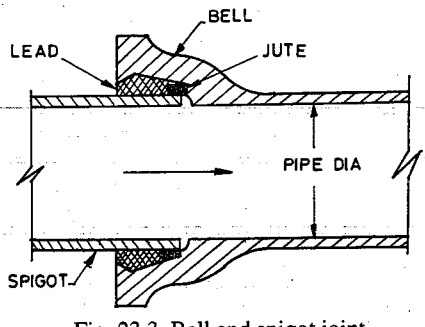


Fig. 23.3. Bell and spigot joint for cast iron pipes.

A few strands of jute are wrapped around the spigot before inserting it into the bell, and then, more jute is packed into the joint. The remaining space between the bell and the spigot is finally filled with molten lead, which gets solidified and tightly chalked into the joint after cooling, and thus making a water-tight joint. The quantity of lead required varies from 3.5 to 4 kg for 15 cm dia pipe to about 45 to 50 kg for 1.2 metres dia pipe.

(2) **Steel conduits.** Steel pipes can be used for large diameters, say upto or above 6 metres or so. Steel plates of varying thicknesses for withstanding different pressures, are generally bent and welded or sometimes riveted. Welded pipes are smoother and stronger than riveted pipes, and are, therefore, generally used these days. Since steel is strong in tension, even large size diameter pipes can be made of thin shells. They are, therefore, lighter than cast iron pipes.

Galvanised steel pipes with circumferential corrugations are much more stronger than ordinary steel pipes. They are, therefore, used where large dia pipes of smaller wall thicknesses are required. Such pipes are usually manufactured in various sizes varying

from 20 cm to 2 metres in diameter, and are lighter and can be more easily transported at distances.

Expansion joints are generally not required in steel pipes which are buried under the ground, because they are not subjected to large temperature changes. However, pipes which are exposed to the atmosphere may require expansion joints so as to minimise temperature changes.

Steel pipes get rusted quickly, which, reduces their life as well as carrying capacities. They are, therefore, protected on the inside as well as outside by protective coatings. Under ordinary conditions, steel pipes may last as long as 40 years or so. The various pipe lengths in case of steel pipes are connected by welded or riveted joints.

(3) **Reinforced Cement Concrete Conduits.** Ordinary cement concrete pipes are manufactured in small sizes (*i.e.* up to say 0.6 m diameter) ; while they are reinforced with steel for large dia pipes. They are easily available in sizes up to diameters say about 1.8 metres and may be got manufactured for larger diameters say up to about 4.5 metres, on special orders. These pipes may either be prepared at site by transporting various ingradients (*i.e.* cement, steel aggregates, water, etc.) or can be manufactured in factories and then transported to site. They are known as '*cast in situ*' pipes in the former case, and '*precast pipes*' in the latter case. Cast in situ pipes are useful when the site conditions are difficult, and where it may be difficult to carry the pipes. But since such pipes are cast at site, lesser supervision and check is possible, as compared to the case of precast pipes which are cast in the factories, and thus subjected to greater quality control and supervision.

R.C.C. pipes can be manufactured in three different ways, *viz.*

(*a*) pipes having bar and mesh reinforcement, and concrete poured by usual ordinary methods of concrete pouring, and tamped ;

(*b*) pipes made by rotating the mould or the form rapidly about the pipe axis. The mould contains concrete and fabricated reinforcement. The centrifugal force throws off the concrete which then spreads in a uniform layer over the internal surface of the mould and embed the reinforcement ; thus providing a high density watertight concrete surface. This type is known as *centrifugal type* ;

(*c*) the third type of pipes are made by lining thin cylindrical steel shells both internally and externally with rich cement concrete. These are stronger and more water-tight than the first two. They are known as *cylinder type.*

R.C.C. pipes are generally made from 1 : 2 : 4 cement concrete with maximum size of aggregates as 6 mm. They are provided with circumferential reinforcement to carry hoop tension and a nominal longitudinal reinforncement equal to 0.25% of cross-sectional area of concrete. The thickness of R.C.C. pipes generally varies from 7.5 cm to 0.6 m for pipes of diameters* varying from 0.2 m to 2.75 m.

(4) **Hume Steel Conduits.** Hume steel pipes are R.C.C. pipes patented under this name. They consist of thin steel shells coated from inside with cement mortar by centrifugal process. The thickness of the inside coating varies from 1.2 cm to 3 cm depending upon the size of the pipe. They are also coated from outside so as to protect the steel shell from external weather or soil action. The thickness of external coating is 2.5 cm for pipes upto 1 metre in diameter and is 3.75 cm for pipes of larger diameters.

* The standard mentioned diameter of pipes is always the internal diameter.

The thickness of steel shell depends upon the size of the pipe and also upon the pressure to be borne by the pipe. Like all R.C.C. pipes, they are heavy and difficult to handle.

(5) **Vitrified Clay Conduits.** They are generally not used as pressure conduits, but are extensively used for carrying sewage and drainage at partial depths. These pipes are free from corrosion and provide a smooth hydraulically efficient surface. They are not used as pressure pipes, because clay is weak in tension, and formation of water-tight joints becomes difficult in them. Clay pipes are commonly made in lengths of about 0.6 m to 1.2 m or so.

(6) **Asbestos Conduits.** Asbestos, silica and cement are converted under pressure to a dense, homogeneous material possessing high strength, called *asbestos cement.* This material is used for casting these pipes. The asbestos fibre which is thoroughly mixed with cement serves as reinforcement. These pipes are generally available in different sizes, say from 10 cm to 90 cm in diameters and in about 4 metres lengths. These pipes are generally made in 4 different grades to withstand pressures of 350 kN/m^2 to 140 kN/m^2. These pipes are joined by means of special type of flexible joint called '*simplex joint*'. The assembly consists of a pipe sleeve and two rubber rings which are compressed between the pipe and the interior of the sleeve.

Advantages. The advantages of asbestos cement pipes are :

(*i*) They are light and hence easy to transport.

(*ii*) They can be easily assembled without skilled labour.

(*iii*) They are highly resistant to corrosion.

(*iv*) They are highly flexible and may permit as much as 12° deflection in laying them around curves.

(*v*) Expansion joints are not required as the coefficient of expansion is low and the joints are also flexible.

(*vi*) They are very smooth and thus provide a hydraulically efficient pipe. Their carrying capacities do not reduce with time.

(*vii*) They are very suitable to be used as small size distribution pipes.

Disadvantages and limitations. Their disadvantages are :

(*i*) They are costly.

(*ii*) These pipes do not have much strength and are brittle and soft. They are liable to get damaged by excavating tools or during transportation transits.

(*iii*) The rubber joint seals may deteriorate if exposed to gasoline or other petroleum products, and hence cannot be used for transporting petroleum products.

(7) **Miscellaneous Types of Conduits.** Various other materials which may be used for manufacturing pipe conduits are : copper, wrought iron, plastics, etc. *Copper pipes* are very costly although they are highly resistant to acidic as well as alkaline waters, they can be bent easily and do not sag due to heat. They are, therefore, very useful for carrying hot water in the interior of the buildings. Wrought iron pipes are lighter than cast iron pipes and can be more easily cut, threaded and worked. They are more costly but neat in appearance. They are generally manufactured in small sizes and are very useful for indoor works. However, they corrode quickly and are less durable. they are,

therefore, generally protected by galvanising them with zinc coatings and they are then known as galvanised iron pipes.

Plastic pipes are lighter and free from corrosion. But they are of low strengths and less durable. Moreover, they can not withstand high temperatures exceeding 60°C or so. They may, however, be used for very minor works in house connections, etc. HDPE pipes are however, being increasingly used these days for various uses, such as in tubewell boarings, sewage mains, etc. etc. They are however, still not being made in large sizes suitable as penstock pipes.

PROBLEMS

1. (*a*) What is meant by a pressure conduit ? Discuss briefly the forces acting on a pressure conduit.

(*b*) Write a brief note on the design capacities of pressure pipes.

2. What are the different construction materials which are used for manufacturing pressure pipes ? Discuss their comparative merits and demerits.

3. Write a note on Pressure conduits'.

24

Hydro-electric Power

Faraday had shown that when a coil is rotated in a magnetic field, electricity is generated. Thus, in order to produce electrical energy, it is necessary that we should produce mechanical energy, which can be used to rotate the 'coil'. The mechanical energy is produced by running a prime mover (*known as turbine*) by the energy of fuels or flowing water. This mechanical power is converted into electrical power by electric generator which is directly coupled to the shaft of the turbine, and is thus run by the turbine. The electrical power, which is consequently obtained at the terminals of the generator, is then transmitted to the area where it is to be used for doing work.

The plant or machinery which is required to produce electricity (*i.e.* Prime mover + Electric generator) is collectively known as the *Power plant.* The building, in which the entire machinery along with other auxiliary units is installed, is known as the *Power house.*

24.1. Thermal and Hydropower

As stated earlier, the turbine blades can be made to run by the energy of fuels or flowing water. When fuel is used to produce steam for running the steam turbine, then the power generated is known as *Thermal power.* The fuel which is to be used for generating steam may either be (*i*) an ordinary fuel such as coal, fuel oil, gas, etc., or (*ii*) atomic fuel or nuclear fuel. Coal is simply burnt to produce steam from water and is the simplest and the oldest type of fuel. Diesel oil, etc. may also be used as fuels for producing steam. Atomic fuels such as uranium or thorium may also be used to produce steam. When conventional type of fuels such as coal, oil, etc. (called fossils) are used to produce steam for running the turbines, the power house is generally called an *Ordinary thermal power station* or *Thermal power station.* But when atomic fuel is used to produce steam, the power station, which is essentially a thermal power station, is called an *Atomic power station* or a *Nuclear power station.* In an ordinary thermal power station, steam is produced in a water boiler, while in the atomic power station, the boiler is replaced by a nuclear reactor and steam generator for raising steam. The electric power generated in both these cases is known as *Thermal power* and the scheme is called *Thermal power scheme.*

But, when the energy of the flowing water is used to run the turbines, then the electricity generated is called the *Hydroelectric power.* This scheme is known as *Hydel scheme*, and the power house is known as *Hydel power station* or *Hydroelectric power station.* In a hydel scheme, a certain quantity of water at a certain potential head, is essentially made to flow through the turbines. The head causing flow, runs the turbine blades, and thus producing electricity from the generator coupled to the turbine. In this chapter, we are concerned with hydel schemes only.

24.2. Classification of Hydel Plants

24.2.1. Classification of Hydroplants on the Basis of Hydraulic Characteristics. On the basis of this classification, the hydro plants may be divided into the following types:

(*i*) *Run-off river plants*

(*ii*) *Storage plants*

(*iii*) *Pumped storage plants*

(*iv*) *Tidal plants*

They are described below :

(*i*) **Run-off River Plants.** These plants are those which utilise the minimum flow in a river having no appreciable pondage on its upstream side. A weir or a barrage is sometimes constructed across a river simply to raise and maintain the water level at a pre-determined level within narrow limits of fluctuations, either solely for the power plant or for some other purpose where the power plant may be incidental. Such a scheme is essentially a low head scheme and may be suitable only on a perennial river having sufficient dry wheather flow of such a magnitude as to make the development worth while. [Pl. refer Fig. 24.2]

Run-off river plants generally have a very limited storage capacity, to supplement the normal flow. Such a small storage capacity, called **pondage,** is provided for meeting the hour to hour fluctuations of load or of streamflow over a day ; or occassionally, day to day fluctuations over a weekly cycle. When the available discharge at site is more than the demand (during off-peak hours), the excess water is temporarily stored in the pond on the upstream side of the barrage, which is then utilised during the peak hours.

The power stations constructed on diversion canals (Irrigation & Power Canals) called *Diversion canal plants*, can also be placed in this category.

The various examples of run-off river plants are : Ganguwal and Kotla power houses located on Nangal Hydel Channel, Mohammad Pur and Pathri power houses on Ganga Canal, and Sarda power house on Sarda Canal.

(*ii*) **Storage Plants.** A storage plant is essentially having an upstream storage reservoir of sufficient size, so as to permit sufficient carry-over storage from the monsoon season to the dry summer season, and thus to develop a firm flow substantially more than the minimum natural flow. In this scheme, a dam is constructed across the river, and the power house may be located at the foot of the dam such as in Bhakra, Hirakund, Rihand projects, etc. The power house may sometimes be located much away from the dam (on the down-stream side). In such a case, the power house is located at the end of tunnels which carry water from the reservoir. The tunnels are connected to the power house machines by means of pressure penstocks which may either be underground (as in Maithon and Koyna projects) or may be kept exposed (as in Kundah project).

When the power house is located near the dam, as is generally done in the low head installations ; it is known as *Concentrated fall hydroelectric development.* But when the water is carried to the power house at a considerable distance from the dam through a canal, tunnel, or penstock ; it is known as a *Divided fall development.*

(*iii*) **Pumped Storage Plants.** A pumped storage plant generates power during peak hours, but during the off-peak hours, water is pumped back from the tail water pool to the head water pool for future use. The pumps are run by some secondary power from some other plant in the system. The plant is thus primarily meant for assisting an existing thermal plant or some other hydel plant. A typical section for pumped storage plant is shown in Fig. 24.1.

During peak hours, the water flows from the reservoir to the turbine and electricity is generated. During off-peak hours, the excess power available from some other plant, is utilised for pumping water back from the tail pool to the head pool. This minor plant thus supplements the power of another major plant. In such a scheme, the same water is utilised again and again and no water is wasted.

For heads varying between 15 to 90 m, reversible pump turbines have been devised, which can function both as turbine as well as a pump. Such reversible turbines can work at relatively high efficiencies and can help in reducing the cost of such a plant. Similarly, the same electrical machine can be used both as a generator as well as a motor by reversing the poles. The provision of such a scheme helps considerably in improving the load factor of the power system.

(*iv*) **Tidal Plants.** Tidal plants for generation of electric power are the recent and modern advancements, and essentially work on the principle that there is a rise in sea water during high tide period and a fall *during the low ebb period.* The water rises and falls twice a day ; each fall cycle occupying about 12 hours and 25 minutes. The advantage of this rise and fall of water is taken in a tidal plant. In other words, the tidal range, *i.e.* the difference between high and low tide levels is utilised to generate power. This is accomplished by constructing a basin separated from the ocean by a partition wall and installing turbines in openings through this wall.

Water passes from the ocean to the basin during high tides, and thus running the turbines and generating electric power. During low tide, the water from the basin runs back to ocean, which can also be utilised to generate electric power, provided, special turbines which can generate power for either direction of flow are installed. Such plants are useful at places where tidal range is high. *Rance power station* in France is an example of this type of power station. The tidal range at this place is of the order of 11 metres. This power house contains 9 units of 38,000 k. watts.

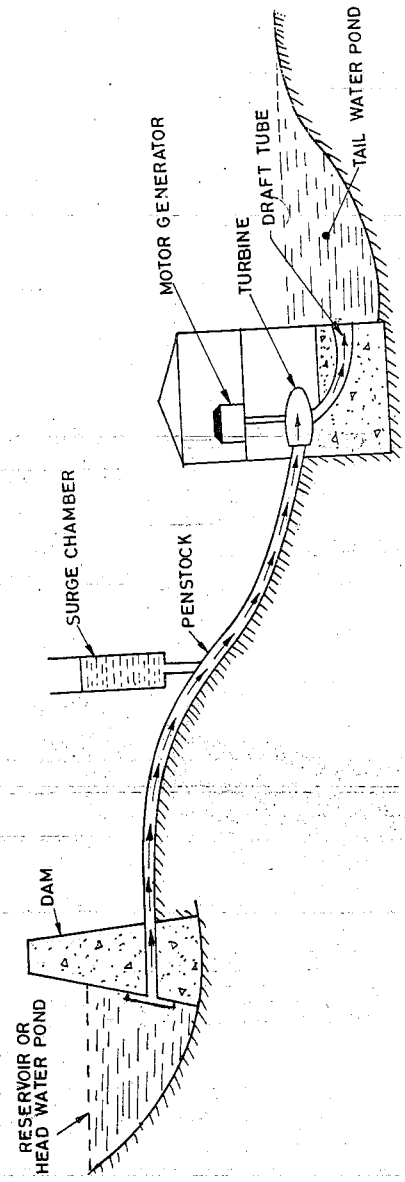


Fig. 24.1. Typical section *through a pumped storage plant.*

24.2.2. Classification of Hydroplants on the Basis of Operating Head on Turbines. On this basis, the plants may be divided into the following types :

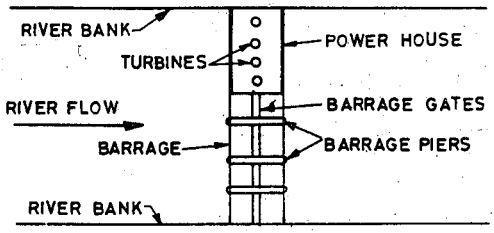
(*i*) *Low head scheme* (head < 15 m).

(*ii*) *Medium head scheme* (head varies between 15 to 60 m).

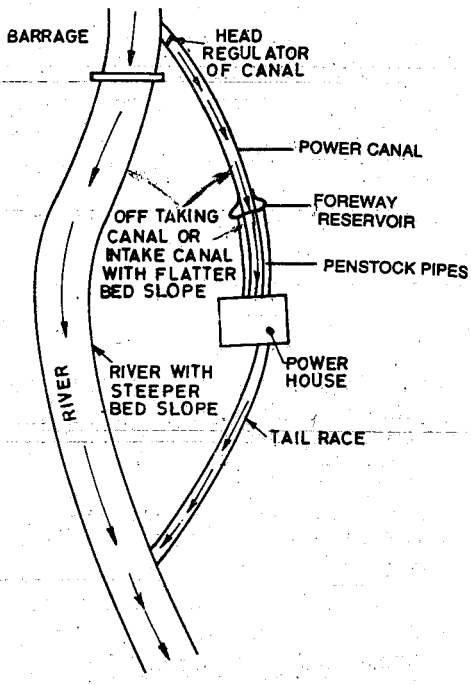
(*iii*) *High head scheme* (head > 60 m).

They are described below :

(*i*) **Low Head Scheme.** A low head scheme is one which uses water head of less than 15 metres or so. A run-off river plant is essentially a low head scheme. In this



(*a*) Run of river plant



(*b*) Diversion canal plant

Fig. 24.2. Low head schemes (in Plan)

scheme, a weir or a barrage is constructed to raise the water level, and the power house is constructed either in continuation with the barrage [Fig. 24.2 (*a*)] or at some distance downstream of the barrage, where water is taken to the power house through an intake canal [Fig. 24.3 (*b*)].

(*ii*) **Medium Head Scheme.** A medium head scheme is one which uses water head varying between 15 to 60 metres or so. This scheme is thus essentially a dam reservoir scheme, although the dam height is mediocre. This scheme is having features somewhere between low head scheme and high head scheme.

(*iii*) **High Head Scheme.** A high head scheme is the one which uses water head of more than 60 m or so. A dam of sufficient height is, therefore, required to be constructed, so as to store water on the upstream side and to utilise this water throughout the year. High head schemes up to heights of 1,800 metres have been developed. The common examples of such a scheme are : Bhakra dam in (Punjab), Rihand dam in (U.P.), Hoover

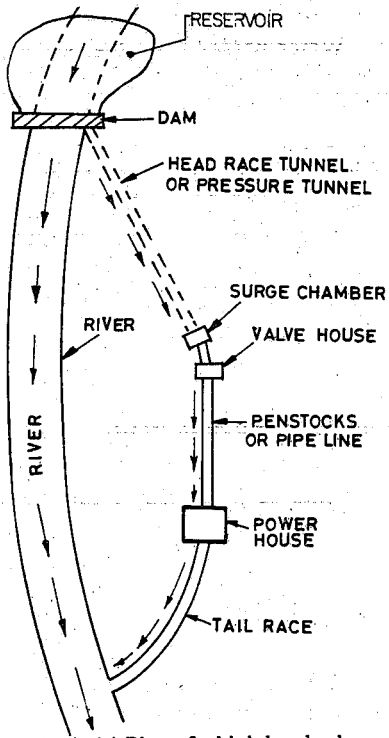


Fig. 24.3. (*a*) Plan of a high head scheme.

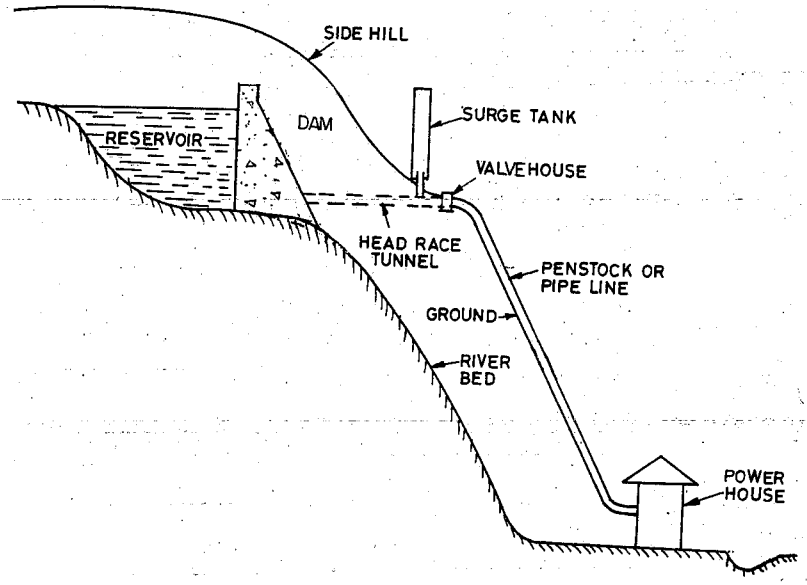


Fig. 24.3. (*b*) Section through a high head scheme.

dam in (U.S.A.), etc. A typical plan and section of such a scheme are shown in Figs. 24.3 (*a*) and (*b*).

The naturally available high falls can also be developed for generating electric power. The common examples of such power developments are : Jog Falls in India, and Niagara Falls in U.S.A.

24.3. Important Terms and Definitions Connected with Hydropower

24.3.1. Water Power Potential. The amount of power generated when Q cumecs of water is allowed to fall through a head difference of H metres is given by

$$\text{Water energy} = \gamma_w \cdot Q \cdot H \qquad \text{...(24.1)}$$

where γ_w is unit wt of water $= 9.81$ kN/m^3

$$= 9.81 \times Q \times H \text{ kN–m/sec. (}i.e.\text{ k watts)}$$

Electrical energy (E.E.) or Power in k. watts, is therefore, given as :

$$P = 9.81\,\eta \cdot Q \cdot H \text{ (k watts)} \qquad \text{...(24.2)}$$

∴ Metric H.P. generated

$$= \frac{9.81\,\eta Q \times H}{0.735} = 13.33\,\eta QH \qquad (\because 1 \text{ m H.P.} = 0.735 \text{ kW}) \qquad \text{...(24.2}a\text{)}$$

where η is the overall efficiency of turbine, generator, etc.

Using 80% efficiency, we get

$$\text{Electrical energy} = 9.81 \times 0.8 \times Q \cdot H \text{ kW}$$

$$= 7.84 \cdot Q \cdot H \approx \mathbf{8\ QH\ kW} \qquad \text{...(24.2}b\text{)}$$

where, H is the design head in metres ; Q is the design discharge in cumecs.

24.3.2. Normal Water Level (N.W.L.). The highest elevation of water level that can be maintained in the reservoir without any spillway discharge, either with a gated or a non-gated spillway, is known as Normal water level.

24.3.3. Minimum Water Level (M.W.L.). The elevation of water level which produces minimum net head on the power units (*i.e.* 65% of design head H) is known as minimum water level.

24.3.4. Weighted Average Level (W.A.L.). The level above and below which equal amounts of power are developed during an average year (*i.e.* 50% units between N.W.L. and W.A.L., and 50% units between W.A.L. and M.W.L.) is called weighted average level.

24.3.5. Design Head. It is the net head under which the turbine reaches peak efficiency at synchronous speed. Generally, the design head = W.A.L. – M.W.L. The difference of N.W.L. and M.W.L. is 125% of design head.

24.3.6. Rated Head. It is the head at which the turbine functioning at full gate opening will produce a power output, equal to that specified in the name plate of the turbine. This rated head should be equal to the design head of the turbine, so as to ensure maximum overall plant efficiency.

24.3.7. Gross Head. Gross head is the difference in the water level elevations at the point of diversion of water for the hydel scheme and the point of return of water back to the river.

24.3.8. Operating Head. Operating head is the simultaneous difference between the elevations of water surface in the *foreway* and the *tailrace*, after making due allowance for approach and exit velocity heads.

$\therefore$ Operating head = T.E.L. at forway entrance – T.E.L. at tailrace exit. ...(24.3)

24.3.9. Net Head or Effective Head. The effective head is the net head applied to the turbine, and is given by the difference of head at the point of entry and exit of turbine, and includes the respective velocity and pressure heads at both places.

24.3.10. Installed Capacity of the Power House. The total capacity in kilowatts or million killowatts of all the turbine-generator units installed in a power house, is called its Installed capacity.

24.3.11. Dependable Capacity. It is the load carrying capability of the power house with respect to the load characteristics during a specified time interval, and is decided by the power factor, capability and the load on the power house.

24.3.12. Load Factor (L.F.). Load factor is defined as the ratio of the average load over a certain period of time to the peak load during the same period. Depending upon the period chosen, we may have different load factors, such as daily, monthly or annual load factors.

$$\therefore \quad \text{Load factor} = \frac{\text{Average load over a certain period}}{\text{Peak load during that period}} \quad ...(24.4)$$

If the area under a load curve* is plotted, it would evidently represent the energy consumed in kilowatt hours (kWh). Thus, an annual load factor may also be defined as the ratio of the actual energy consumed during that year to the peak demand assumed to continue for one year.

* A graph plotted between time in hrs (on x-axis) and load in kW (on y-axis) is called a **load curve.**

∴ **Annual load factor**

$$= \frac{\text{Total yearly electrical units (kWh) produced}}{(\text{Max. power demand in kW}) \times 365 \times 24} \quad ...(24.5)$$

Note. The maximum load determines the capacity of the units, while the load factor gives an idea of the degree of utilisation of the capacity. For example, an annual load factor of 0.6 would indicate that the machines are producing 60% of their yearly rated capacity (maximum production capacity).

24.3.13. Demand Factor. A consumer is provided with the connected load of a certain rating (kilo-watts). He may or may not use the whole load at one time, as the may use only a part of it. The ratio of the maximum demand at any particular time to the connected load is known as the *demand factor.*

24.3.14. Capacity Factor or Plant Factor. It may be defined as the ratio of average output of the plant for a given period of time to the plant capacity, *i.e.*,

$$\text{Capacity factor} = \frac{\text{Average Load(over a given period of time)}}{\text{Plant capacity}} \quad ...(24.6\ a)$$

In other words, the capacity factor is the ratio of the energy actually produced by the plant in any given period to the energy it would be capable of producing at its full capacity during that period. For example, if a plant with a capacity of 10,000 kW were to produce 40,000 kWh when operating for 100 hours, then

$$\text{Capacity factor} = \frac{\frac{40,000}{100}}{10,000} = 0.4 \text{ or } 40\%$$

Hence, capacity factor

$$= \frac{\text{Energy actually produced in a given time}}{\text{Max. energy that can be produced by the plant during that given time}} \quad ...(24.6\ b)$$

The capacity factor and load factor would become identical, if the peak load is equal to the plant capacity ; *i.e.* when plant is allowed to be used up to its max. capacity, or there is no *reserve capacity.* However, load factor would be different, if the plant is not used to full capacity.

Hence, in the above example, if the peak load is equal to 8000 kW (as against the station capacity of 10,000 kW), then

$$\text{Load factor} = \frac{\frac{40,000}{100}}{8,000} = 50\% \text{ as against the capacity factor of } 40\%.$$

Also, the **reserve capacity** would then be

$$\text{Peak load} = \frac{\text{Plant capacity}}{1 + R} \quad ...(24.7)$$

where R is Reserve capacity expressed as fraction of Plant capacity,

$$\therefore \quad 8000 = \frac{10,000}{1 + R}$$

or $$1 + R = \frac{10,000}{8,000}$$

or $$R = 0.25, \textit{i.e.}\ 25\%.$$

(*i.e.* at 25% R.C. ; Peak load × 1.25 = Installed capacity

24.3.15. Utilisation Factor or Plant Use Factor. It is defined as :

$$\text{Utilisation factor (U.F.)} = \frac{\text{Water actually utilised for power production}}{\text{Water available in the river}} \quad ...(24.8)$$

It the water head is assumed to be constant, then the utilisation factor would be equivalent to :

$$\text{U.F.} = \frac{\text{Max. power utilised}}{\text{Max. power available}} \quad ...(24.9)$$

The value of utilisation factor usually varies from 0.4 to 0.9 for a hydel plant, depending upon the plant capacity, load factor and storage.

24.3.16. Firm Power. The net amount of power which is continuously available from a plant without any break on firm or on guaranteed basis is known as *firm power*. This power should be available under the most adverse hydraulic conditions. The consumers can always be sure of getting this power.

24.3.17. Secondary Power. The excess power available over the firm power during the off peak hours or during monsoon, etc. is known as *secondary power*. There is no guarantee for secondary power, and it is supplied to the consumers on 'as and when available' basis.

24.3.18. Power Factor. It is defined as

$$\text{Power factor} = \frac{\text{Actual power in kilowatts (kW)}}{\text{Apparent power in kilo volt–amperes (KVA)}} \quad ...(24.10)$$

The power factor can never be greater than unity. Its value depends upon the relationship between the inductance and resistance in the load. A load with very little inductance such as lighting bulbs, will have a power factor close to unity. The usual system load has a power factor varying between 0.8 to 0.9, but if various induction motors are installed in the load, the power factor may be as low as 0.5. Since the electrical machines are generally rated in KVA, the actual power developed depends much upon the power factor. This is the reason, why most of the Electric Supply Undertakings, do stress upon their consumers, to reduce their inductance and improve their power factors.

Example 24.1. *The load on a hydel plant varies from a minimum of 10,000 kW to a maximum of 35,000 kW. Two turbo-generators of capacities 22,000 kW each have been installed. Calculate :*

(a) total installed capacity of the plant ;

(b) plant factor ;

(c) maximum demand ;

(d) load factor ;

(e) utilisation factor.

Solution. (*a*) Since two generators, each of capacity 22000 kW are installed, we have

the *total installed capacity* $= 2 \times 22000$ kW $= 44000$ kW. **Ans.**

From eqn. (24.6*b*), we have

(*b*) *Plant factor* (*i.e.* capacity factor)

$$= \frac{\text{Energy actually produced in time } t}{\text{Max. energy that can be produced in time } t}$$

$$= \frac{\frac{(10000+35000)}{2} \times t}{44000 \times t} = \frac{22500}{44000} = 0.511, \textit{i.e.}\ 51.1\%. \textbf{ Ans.}$$

(*c*) *Maximum demand* = 35000 kW (as given). **Ans.**

(*d*) *Load factor* $= \frac{\text{Average load over a certain period}}{\text{Peak load during that period}}$

$$= \frac{\frac{10000+35000}{2}\text{ kW}}{35000\text{ kW}} = \frac{22500}{35000} = 0.643\ \textit{i.e.}\ 64.3\%. \textbf{ Ans.}$$

(*e*) *Utilisation factor* $= \frac{\text{Max. power utilised}}{\text{Max. power available}}$

$$= \frac{35000\text{ kW}}{44000\text{ kW}} = 0.795, \textit{i.e.}\ 79.5\%. \textbf{ Ans.}$$

Example 24.2. *A common load is shared by two hydel stations ; one being a base load station with 20 MW installed capacity, and the other being a stand-by station with 25 MW capacity. The yearly output of the stand-by station is* 10×10^6 *kWh and that of the base load plant as* 110×10^6 *kWh. The peak load taken by stand-by station is 12 MW and this station works for 2500 hours during the year. The base load station takes a peak of 18 MW. Find out :*

(a) Annual load factors for both stations.

(b) Plant use factors for both stations.

(c) Capacity factors for both stations.

Solution. We will work out all the required three factors for the base load plant first, and then for the other stand-by plant.

For Base Load Plant. For this base load station, data is given as :

Installed capacity $= 20\text{ MW} = 20 \times 10^3\text{ kW}$

Yearly output $= 110 \times 10^6\text{ kWh}$

Peak load taken $= 18\text{ MW} = 18 \times 10^3\text{ kW}$.

(*a*) Now, *Annual load factor*

$$= \frac{\text{Total energy (kWh) generated per year}}{\text{Max. power demand in kW} \times 365 \times 24}$$

$$= \frac{110 \times 10^6}{(18 \times 10^3) \times 365 \times 24} = \mathbf{0.698}, \textit{i.e.}\ \mathbf{69.8\%}. \textbf{ Ans.}$$

(*b*) *Plant use factor* $= \frac{\text{Max. power utilised}}{\text{Max. power available}}$

$$= \frac{18000\text{ kW}}{20 \times 10^3\text{ kW}} = \mathbf{0.9}, \textit{i.e.}\ \mathbf{90\%}. \textbf{ Ans.}$$

(c) *Capacity factor* $= \dfrac{\text{Av. energy utilised in a period (year)}}{\text{Max. energy that can be produced in that period}}$

$$= \frac{110 \times 10^6}{(20 \times 10^3) \times (24 \times 365)} = \mathbf{0.628}, \textit{i.e.}\ \mathbf{62.8\%} \quad \textbf{Ans.}$$

For stand by plant. The data given for this plant is :

Installed capacity $= 25$ MW $= 25 \times 10^3$ kW

Yearly output in 2500 hrs $= 10 \times 10^6$ kWh

Peak load $= 12$ MW $= 12 \times 10^3$ kW.

No. of working hours during the year = 2500 hr.

(a) *Annual load factor*

$$= \frac{\text{Total kWh generated per year}}{\text{Max. power demand in kW} \times (24 \times 365)}$$

$$= \frac{10 \times 10^6 \text{ kWh}}{(12 \times 10^3 \text{ kW}) \times (24 \times 365 \text{ h})} = \mathbf{0.095}, \textit{i.e.}\ \mathbf{9.5\%}. \quad \textbf{Ans.}$$

(b) *Plant use factor* $= \dfrac{\text{Max. power utilised}}{\text{Max. available power}}$

$$= \frac{12 \times 10^3 \text{ kW}}{25 \times 10^3 \text{ kW}} = \mathbf{0.48}, \textit{i.e.}, \mathbf{48\%}. \quad \textbf{Ans.}$$

(c) *Capacity factor* $= \dfrac{\text{Av. output}}{\text{Station capacity}}$

$$= \frac{10 \times 10^6 \text{ kWh}}{(25 \times 10^3 \text{ kW}) \times 2500 \text{ h}} = \mathbf{0.16}, \textit{i.e.}\ \mathbf{16\%}. \quad \textbf{Ans.}$$

24.4. Principal Components of a Hydro-electric Scheme

A hydroelectric development scheme ordinarily includes a diversion structure, a conduit (penstock) to carry water to the turbines, turbines and governing mechanisms, generators, control and switching apparatus, housing for the equipment, transformers and transmission lines to the distribution centres. In addition to these major components, trash racks at the entrance to penstock, canal and penstock gates, a foreway, a surge tank, and other appurtenances may be required. A tailrace channel from the power house back to the river must be provided, if the power house is situated at such a place that the draft tubes cannot discharge water directly into the river.

No two power development schemes are exactly alike and each will have its own unique problems of design and construction. The choice of a particular type of a plant at a given site depends upon various factors, such as :

(*i*) general available topography of the area ;

(*ii*) available head ;

(*iii*) available flow ;

(*iv*) availability of other type of power stations in the vicinity :

(*v*) requirements of power for industries, etc.

(*vi*) Political influences of the area, etc.

The major components of a hydroelectric scheme are described below :

(1) **The Foreway.** A foreway is a storage basin or any other large body of water situated just infront of intake. Its main function is to temporarily store the water which is rejected by the plant due to reduced load during off-peak hours, and also to meet the instantaneous increased demand when the load is suddenly increased. The foreway, therefore, absorbs the short interval variations in water demand and corresponding fluctuations in power load.

In many cases, when canal leads the water to the turbines, the canal itself may be large enough to absorb these flow variations. When the canal is long, its section is sometimes enlarged at the end, so as to provide necessary temporary storage. In many other cases, when a reservoir is constructed by building a dam, and the water is taken to the power house directly from the reservoir through penstocks, the reservoir itself acts as a foreway.

(2) **Intake Structure.** The foreway is provided with some type of intake structure, to direct water into the penstocks. Various types of intakes are used depending upon the local conditions.

Intakes should be provided with **trash racks** so as to prevent the entry of debris into the penstocks, and thus to avoid the possible damage to *wicket gates* and turbine runners or to avoid the choking up of the nozzles of the impulse turbines. They are made of thin flat steel bars placed 10 to 30 cm apart. The permissible velocity of water entering the trash racks is 0.6 to 1.6 m/sec.

A floating boom is often placed in the storage reservoir so as to trap as much ice and floating debris as possible, and thus to avoid their entry into the canal. At places, where severe winters do occur, provision is made so as to minimise the ice trouble in the foreway. The trash racks may be electrically heated, so as to prevent clinging of ice with them. Sometimes, an air-bubbler system which agitates the water in the vicinity of the trash rack and brings warmer water to the surface, is also used. The problem of ice automatically gets reduced, if the foreway pond is large enough, and the intake is at considerable depth.

The foreway must be provided with a spillway or a wasteway, a bypass channel, etc., so that the water can be disposed of safely, if such a need arises at any time. Syphon spillways may often be used advantageously for this purpose.

Besides trash racks, other accessories of an intake structure are :

(*i*) *Rakes and trolley arrangements* required to clear trash racks.

(*ii*) *Ice removal equipment,* such as a *floating boom, electrical heating device* for trash racks, etc., if necessary.

(*iii*) *Penstock closing gates* with their hoisting mechanisms.

(3) **Penstocks.** Penstocks are the huge diameter pipes which carry water under pressure from the storage reservoir to the turbines. The structural design of a penstock is essentially similar to that of any other pressure pipe. But, because of the possibility of sudden load changes, design against water hammer* is essential. Short length penstocks are generally designed to take this extra pressure by providing heavier pipe

* Explained in previous chapter.

wall. Slow-closing valves are then provided, so as to reduce these extra pressures. However, in case of long penstocks, a *surge tank* is generally provided, so as to absorb water hammer pressures and to provide water to meet sudden load increases.

Penstocks are generally made of steel or reinforced concrete, though wooden-stave pipes are also sometimes used. If the distance of the power house from the foreway is small, separate penstocks are generally used for each turbine. But long penstocks are generally branched at the lower end, so as to serve several turbines.

Penstock closing gates are usually provided at the entrance in the foreway. The gate can be closed to permit repair of penstock. An air vent and vent pipe connecting the top of the penstock with the open air, is provided downstream from the gate. Such a vent permits the entry of air into the penstock, as soon as the gates are closed and water drawn off through the turbine wheels. In the absence of such an arrangement, serious negative pressures will be exerted on the penstocks and may cause the penstocks to collapse. The water in the vent pipe should not get frozen, otherwise, entry of air will be prevented. Before opening the main head gate, the penstock, if empty, should be filled through a bypass, so that surge of water may not damage the turbines.

The penstocks should be located at such a level that sufficient water depth (called **water seal**) is provided above the penstock entrance in the foreway. If this is not so, and too little water depth is available, vortices and whirlpools will tend to form, which may carry air into the penstocks and the turbine wheels, and thus lowering the turbine efficiency and causing undesirable pressure surges. The entrance of the penstocks should also be flared, so as to avoid any loss of head by contraction.

Sharp bends must be avoided in penstocks, because they cause loss of head and require special anchorages. The penstocks may either be buried under the ground or kept exposed. Exposed or uncovered penstocks are usually supported on cradles and carried across ravines on trestles. Uncovered penstocks are generally costlier because they require other appurtenances such as expansion joints, saddles, anchors, etc. ; but they are easily accessible for inspection and repairs. It is not advisable to partially bury the penstock pipes, as the region near the ground surface will be subjected to excessive deterioration due to local movements. In a few cases, the penstocks can be replaced by constructing a tunnel through a hill, if a considerable saving in length can be gained.

(4) **Surge Tank or Surge Chamber.** The simplest type of a surge chamber consists of a cylindrical chamber open to the atmosphere and connected to the penstocks, as close to the power house as possible (Fig. 24.4).

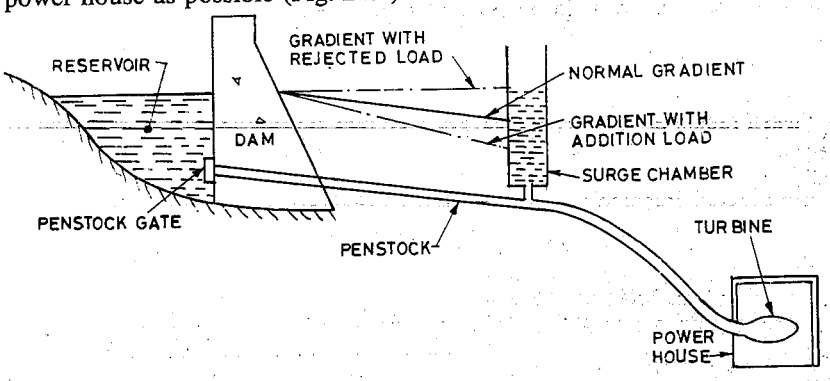


Fig. 24.4. Provisions of Surge chamber.

When the load is rejected by the power house turbines, the water level in the surge chamber rises, and decelerates the flow upstream of it. But when additional load comes, the immediate demand is met by drawing water from the surge chamber, which increases the flow gradient and thus accelerates the flow from the reservoir. When the load is steady, the water level in the surge chamber remains constant and at the level of normal hydraulic gradient line at that point, as shown in Fig. 24.4 A surge chamber, therefore, reduces the pressure fluctuations in the conduit pipe (*i.e.* penstocks) considerably, and thus prevents additional water hammer pressures from being exerted upon the walls of the conduit. The surge chamber should be high enough, to prevent any overflow of water, even at full load rejection.

Various types of surge chambers, such as (*i*) *Simple surge tank* (*ii*) *Throttled or Restricted orifice surge chamber,* (*iii*) *Differential surge chamber,* (*iv*) *Multiple surge chamber,* etc. are in use these days. The differential type of surge chamber is having a central riser pipe in addition to the main outer tank, as shown in Fig. 24.5.

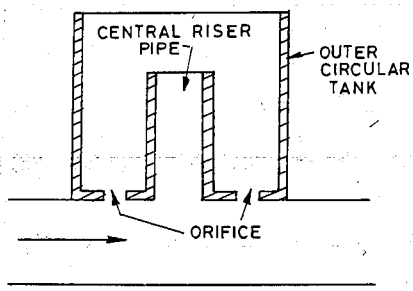


Fig. 24.5.
Differential type of surge chamber.

The main advantage of differential type of surge tank over a simple tank lies in the fact that retarding and accelerating heads are developed more promptly in the former case than in the latter case. Hence, for a given amount of stabilising effect, the capacity of a differential type of surge chamber may be less than that of a simple elementary type.

(5) **Hydraulic Turbines.** Turbines are machines which convert hydraulic energy into mechanical energy. The mechanical energy so developed by the turbine is then used to generate electric energy by directly coupling the shaft of the turbine with the generator.

In general, a turbine consists of a wheel (called runner) which is provided with specially designed blades or buckets. The water having large hydraulic energy is made to strike the runner, and thus causing it to rotate. This rotation of the turbine runner is passed on to the generator by coupling the generator and turbine together through the turbine shaft. This results in rotating the generator armature, and thus producing electrical power, called hydroelectric power.

Hydraulic turbines may be divided into two classes :

(*i*) Impulse turbines or Velocity turbines ; and

(*ii*) Reaction turbines or Pressure turbines.

These are discussed below :

(*i*) **Impulse turbines.** The important example of an impulse type of turbine is a *Pelton's wheel* or *a Pelton's turbine*. In such a turbine, all the available potential energy of water is converted into kinetic energy by passing the penstock water

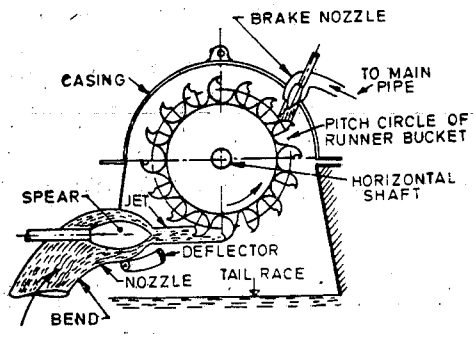


Fig. 24.6. Pelton's wheel (impulse turbine).

through a single nozzle. The water coming out of the nozzle in the form of free jet is made to strike on a series of buckets mounted on the periphery of a wheel (Fig. 24.6). This causes the wheel to revolve in open air, and water is in contact with only a part of the wheel at a time. The pressure of water is all along atmospheric, and thus there is no difference of pressure in the water at the inlet to the runner and at the outlet discharge to the tailrace. It also acts as a safeguard against accidents.

An impulse turbine is essentially a low speed turbine and is used for high heads of the order of 150 to 1,000 m (though heads of 1,700 to 1,800 m have been used in the world). Since it works under high heads, comparatively less quantity of water is required. It is, therefore used for high heads and low discharges.

(*ii*) **Reaction turbines.** The important examples of reaction turbines are : (*i*) *Francis turbine* ; and (*ii*) *Kaplan turbine.* A reaction turbine is one in which only a part of the potential energy of water is converted into velocity head (*i.e.* kinetic energy) and the balance remains as pressure head. Thus, the water entering the turbine possesses pressure as well as kinetic energy. The wheel is rotated under the action of both these forces. The water leaving the turbine also contains some pressure as well as velocity head. The pressure at the inlet is much higher than the pressure at the outlet. Since the entire flow takes place under pressure, a closed casing is absolutely necessary, so as to prevent any access of atmospheric air into the turbine. Since the water flows under pressure through such a turbine, the wheels of this turbine are submerged, and water enters all around the periphery of the wheel.

Difference between Pelton's (*i.e.* Impulse type) and **Francis** (*i.e.* Reaction type) **Turbines**

(1) In a Pelton's wheel, the total potential head is changed into kinetic head for affecting the motion of the runner ; while in a Francis wheel, only a part of it is converted.

(2) Water strikes only a few buckets at a time in Pelton's wheel ; while in Francis wheel, the water flows like that in a closed conduit. The runner is always full of water, and thus all the blades are simultaneously striken by water.

(3) In Pelton's wheel, the water falls freely to the atmosphere ; while in Francis wheel, the water is taken upto the tailrace by means of a closed draft tube, and thus, the whole passage of water is totally enclosed.

Classification of turbines according to specific speed

The specific speed (N_s) of a turbine is the speed of geometrically similar turbine that would develop one metric horse power under a head of one metre. Specific speed is given by the equation

$$N_s = \frac{N\sqrt{P_t}}{H^{5/4}} \qquad ...(24.1)$$

where N = Normal working speed of the turbine in RPM

P_t = The turbine output in metric horse power

H = Net or effective head in metres.

Table 24.1 shows the specific speeds of various turbines.

Table 24.1. Specific Speeds of Various Turbines

S. No.	*Type of turbine*	*Limit of head in metres*	*Specific Speed in Revolution per minute [i.e. R.P.M.]*
1.	Pelton turbine	Over 150 (High heads)	10 to 35
2.	Francis turbine	25 to 450 (Medium heads)	60 to 300
3.	Kaplan and Propeller turbines*	1 to 70 (Low heads)	300 to 1,000

Choice of a particular type of turbine

The selection of the type of a turbine primarily depends on head and load conditions. Different types of turbines can work satisfactorily only within a certain range of specific speeds (Table 24.1). At specific speeds greater than this usual range, there is a possibility of cavitation on the runner blades.

The variability of load also dictates the choice of a particular type of turbine. Say for example, for a head of 150 to 450 m or so ; from table 24.1, both Pelton's as well as Francis turbines can be used. But here the variability of load will influence the choice of a particular type. For higher range of heads, Pelton's wheel is preferred for part load operation in comparison to Francis turbine, although the Pelton's turbine involves higher initial cost. The quality of water also sometimes dictates the type of turbine to be chosen. For example, in the head range of 150 to 450 m, the choice of a Francis turbine is automatically ruled out, if sand is present in the water, because its runner will not be able to withstand its erosive action.

(6) **The Power House.** A power house is a building consisting of a *substructure* to support the hydraulic and electrical equipment and a *superstructure* to house and protect this equipment. For most of the plants which are equipped with reaction turbines, the substructure usually consists of a concrete block extending from the foundation to the generator floor with waterways (*i.e.* the scroll casings and draft tubes) formed within it. They are cast integrally while pouring concrete. The elevation of the turbine with respect to tail water level should be determined by the necessity of avoiding cavitation.

The superstructure is a building which generally accommodates the generators and excitors on the ground floor, and the switch board and control room on the mezzanine floor. Vertical turbines are placed just below the floor level beneath the generators, while horizontal turbines are placed on the ground floor along side the generators. The generating units are almost always arranged in a straight line across the direction of flow. A travelling crane spanning the width of the power house, is generally provided in every power house, so as to facilitate the lifting of heavy machines.

(7) **The Draft Tube.** The draft tube is a conduit which connects the outlet of a reaction turbine runner to the tailrace. Water, as it emerges out of the runner, flows through this pipe of gradually increasing diameter and comes to the tailrace level. The advantages of having a draft tube instead of letting the water fall directly into the atmosphere are :

(*i*) Effective pressure head is increased by an amount

$$= H_s + \frac{V_2^2 - V_3^2}{2g}$$

as explained below :

* Both Kaplan and Propeller turbines look alike and behave hydraulically in a similar fashion except that the Kaplan turbines have an additional arrangement of adjusting the inclination of the runner blades or vanes on the hub. This causes the Kaplan turbines to give higher efficiency even under part load conditions. *Kaplon turbines, cmpared to Propeller turbines, are therefore found to be more versatile, although costlier in their initial cost.*

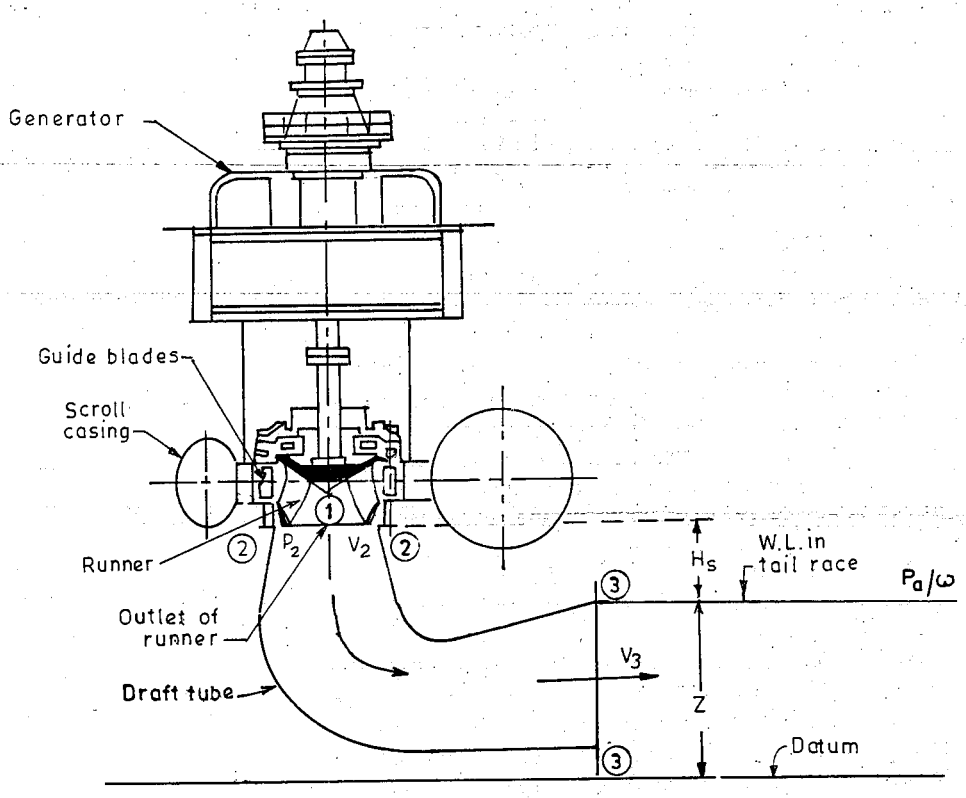


Fig. 24.7

Considering Fig. 24.7, we have

$$\frac{p_2}{\gamma_w}+Z+H_s+\frac{V_2^2}{2g}=\frac{p_a}{\gamma_w}+Z+\frac{V_2^2}{2g}$$

or

$$\frac{p_2}{\gamma_w}=\frac{p_a}{\gamma_w}+\frac{V_3^2}{2g}-\frac{V_2^2}{2g}-H_s=\frac{P_a}{\gamma_w}-\left(H_s+\frac{V_2^2}{2g}-\frac{V_3^2}{2g}\right)$$

Hence, the negative pressure created at the outlet of the runner

$$=H_s+\frac{V_2^2-V_3^2}{2g};$$

this should not exceed the vacuum pressure of water.

So if by gradually increasing the diameter of this pipe, we are able to reduce the velocity $V_3=0$, then the net increase in head

$$=H_s+\frac{V_2^2}{2g}.$$

(*ii*) The draft tube permits the turbines to be installed at a higher level than the tailwater level, which facilitates the maintenance, etc. of the turbine.

Outlet Gates. The outlet of the draft tube should be provided with gates, so that the draft tube can be dewatered for repairs. If the span between tailrace piers is not too high, timber logs may be used for this purpose. However, for larger spans, steel stop logs or

steel gates are commonly used. A hoisting mechanism is also required for lifting the gates or stop logs.

(8) **The Tailrace.** The channel into which the water is discharged after passing through the turbines is known as the *Tailrace*. If the power house is close to the stream, the outflow may be discharged directly into the stream. But when the stream is far off from the power house, one may have to construct a channel of considerable length between the power house and the stream. The tailrace must be designed properly and should not be neglected. In many low-head plants, more of the gross head may be utilised at very low cost if full attention is paid to the tailrace design.

Proper design and maintenance of the tailrace is necessary, so as to avoid excessive aggradation (*i.e.* silting) or degradation (*i.e.* scouring) of its bed. Aggradation will raise the tail water level and reduce the gross available head at the plant. The degradation will lower the tail water level, and the process may proceed up to a point where the water level falls below the top of the draft tube. This will lead to unsatisfactory flow conditions, increased losses, reduced turbine efficiency, and possibility of cavitation and consequent damage to the turbine blades.

Example 24.3. *A run-off river plant is to be constructed across a river at a site where a net head of 22 m is available on the turbines. The river carries a sustained minimum flow of 26 cumecs as dry weather flow. Behind the power station, sufficient water pondage has been provided to supply daily peak load of demand with a load factor of 70%. Assuming the plant efficiency of 58%, determine :*

(*i*) *The maximum generating capacity of the generators to be installed at the power house.*

(*ii*) *The volume of pondage to be provided to supply the daily demand, assuming that the daily load pattern consists of average load for 21 hours and of peak load for 3 hours.*

Solution. The power produced (at dry weather flow) is given by eqn. (24.2), as

$$P = 9.81\,\eta \cdot Q \cdot H \text{ (in kW)}$$

$$\text{Here } \eta = 0.58$$

$$Q = 26 \text{ cumecs}$$

$$H = 22 \text{ m}$$

$$\therefore \quad P = 9.81 \times 0.58 \times 26 \times 22 \text{ kW} = 3251 \text{ kW}$$

$$\text{Load factor} = \frac{\text{Av. load (Av. power)}}{\text{Peak load (Peak power)}}$$

$$\therefore \quad 0.70 = \frac{3251}{\text{Peak load}}$$

$$\therefore \quad \text{Peak load} = \frac{3251}{0.7} = 4645 \text{ kW.}$$

(*i*) Assuming there is no reserve capacity, we have the *Maximum capacity of the generators to be installed* = 4645 kW. **Ans.**

(*ii*) Excess water from pondage is drawn in order to meet the excess demand (demand in excess of average) for 3 hours.

Excess power reqd. to be developed during 3 hours

$$= \text{Peak demand} - \text{Av. demand}$$

$$= 4645 - 3251 = 1394 \text{ kW}$$

Excess discharge required for developing this excess power is given as

$$P = 9.81\,\eta \cdot Q \cdot H$$

$$\therefore \quad 1394 = 9.81 \times 0.58 \times Q \times 22$$

$$\therefore \quad Q = \frac{1394}{9.81 \times 0.58 \times 22} = 11.15 \text{ m}^3/\text{sec.}$$

This excess discharge is required for 3 hours each day.

∴ Required pondays per day

$$= 11.15 \times 3 \times 60 \times 60 = 12.04 \times 10^4 \text{ m}^3 \quad \textbf{Ans.}$$

Example 24.4. *A run-off river plant with an installed capacity of 15,000 kW operates at 28% load factor when it serves as a peak load station :*

(a) What should be the minimum discharge in the stream, so that it may serve as a base load station ? The plant efficiency may be assumed to be 80% when working under a head of 20 m.

(b) Also calculate the maximum load factor of the plant when the discharge in the stream is 35 cumecs.

Solution. Installed capacity = 15,000 kW

$$\text{Load factor} = 28\%$$

$$\text{Load factor} = \frac{\text{Av. Load}}{\text{Peak Load}}$$

The installed capacity plant serving as peak load plant means that the

Installed capacity = Peak load.

$$\therefore \quad \text{Peak load} = \text{Installed capacity} = 15000 \text{ kW}$$

$$\therefore \quad 0.28 = \frac{\text{Av. load}}{15000}$$

$$\therefore \quad \text{Av. load} = 15000 \times 0.28 = \mathbf{4200\ kW.}$$

(*a*) Now, when this plant has to serve as base load plant, it should develop average load, *i.e.* 4200 kW.

Hence, Q reqd. to develop 4200 kW is given by :

$$P = 9.81\,\eta \cdot Q \cdot H$$

or

$$4200 = 9.81 \times 0.8 \times Q \times 20$$

$$\therefore \quad Q = \frac{4200}{9.81 \times 0.8 \times 20} = \mathbf{26.78\ cumecs.} \quad \textbf{Ans.}$$

(*b*) When discharge in the stream is 35 cumecs, then power developed

$$= 9.81\,\eta \cdot QH$$

$$= 9.81 \times 0.8 \times 35 \times 20 \text{ kW} = \mathbf{5488\ kW.}$$

$$\text{Load factor} = \frac{\text{Av. power}}{\text{Peak power}} = \frac{5488 \text{ kW}}{15000 \text{ kW}} = 0.366, \textit{ i.e. } 36.6\%.$$

Hence, maximum load factor at 35 cumecs flow in the stream = **36.6 %.** **Ans.**

Example 24.5. *A run-off river plant is installed on a river having a minimum flow of 15 m³/sec. If the plant is used as a peak load plant operating only for 6 hours daily,* compute the firm capacity of the plant :*

(a) without pondage ;

* Also called, *6 hours peaking plant.*

(b) with pondage but allowing 8% water to be lost in evaporation and other losses. Head at the plant is 16 m, and the plant efficiency may be assumed as 80%.

Solution. (*a*) Power developed by 15 m^3/sec discharge is given by

$$P = 9.81\,\eta \cdot Q \cdot H$$
$$Q = 9.81 \times 0.8 \times 15 \times 16 = \mathbf{1882\ kW.}$$

This represents the firm capacity of the plant without pondage. **Ans.**

(*b*) With pondage, the total volume of water stored during 18 hours when the plant is not operating is

$$= 15 \times (18 \times 60 \times 60)\ m^3 = 9.72 \times 10^5\ m^3$$

The loss of water due to evaporation, etc., per day

$$= 10\% \times 9.72 \times 10^5\ m^3 = 0.972 \times 10^5\ m^3$$

Net amount of available pondage per day

$$= (9.72 - 0.972)\ 10^5\ m^3 = 8.75 \times 10^5\ m^3$$

This daily pondage can supply a uniform Q, (reqd. for 6 hrs. only)

$$= \frac{8.75 \times 10^5}{6 \times 60 \times 60}\ m^3/\text{sec} = 40.51\ m^3/\text{sec.}$$

∴ Total flow available for power generation (reqd. only for 6 hrs)

$$= (15 + 40.51)\ m^3/\text{sec} = 55.51\ m^3/\text{sec}$$

∴ Power developed due to this discharge (*i.e.* firm power, because discharge is firm)

$$= 9.81 \cdot \eta \cdot Q \cdot H = 9.81 \times 0.8 \times 55.51 \times 16 = 6963\ \text{kW.}$$

Hence, the firm power developed by the plant (if pondage during rest hours is allowed) = **6963 kW.** **Ans.**

Example 24.6. *During a low water week, a river stream gets an average daily flow of 35 m^3/sec with daily fluctuations requiring a pondage capacity of about 16% of the daily flow.*

A hydroelectric plant is to be located on this river, which will operate for 5 days a week, 24 hours a day, but will supply power at varying rates, such that the daily load factor is of 60%, corresponding to which the pondage required is 0.2 times the meanflow to the turbines. On saturdays and sundays, all the flow is ponded for use on rest of the days.

If the effective head on the turbine, when the pond is full is to be 26 m, and the maximum allowable fluctuation in pond level is 1.2 m, determine :

(i) the surface area of the pond to satisfy all the operating conditions ;

(ii) the weekly output at the switchboards in kWh.

Assume the turbine efficiency as 82% and generator efficiency as 90%.

Solution. $Q = 35\ m^3/\text{sec.}$

Now, Daily flow $= 35 \times 24 \times 60 \times 60 = 3.024 \times 10^6\ m^3$

Pondage reqd. for discharge fluctuations

$$= 16\% \text{ of daily flow} = \frac{16}{100} \times 3.024 \times 10^6 \text{ m}^3$$

$$= \mathbf{0.484 \times 10^6 \text{ m}^3} \qquad ...(i)$$

The discharge at the turbines is caused by the daily flow of 35 cumecs plus the discharge caused by the water stored during the two weekly rest days, *i.e.* saturday and sunday.

Now, *total water stored in 2 rest days*

$$= 2 \times \text{Daily flow} = 2 \times 3.024 \times 10^6 \text{ m}^3$$

$$= \mathbf{6.048 \times 10^6 \text{ m}^3} \qquad ...(ii)$$

This stored water is utilised during the 5 working days in the week. Hence, the discharge obtained per day from this storage

$$= \frac{6.048 \times 10^6}{5} \text{ m}^3 = 1.2096 \times 10^6 \text{ m}^3.$$

The streamflow caused by this storage

$$= \frac{1.2096 \times 10^6}{24 \times 60 \times 60} \text{ m}^3/\text{sec} = 14 \text{ m}^3/\text{sec}.$$

∴ Total average flow to the turbines

$$= 35 + 14 = \mathbf{49 \text{ m}^3/\text{sec}}.$$

Hence, pondage required for load fluctuations

$$= 0.2 \times \text{Mean daily flow to turbines}$$

$$= 0.2 \times 49 \times (24 \times 60 \times 60) = \mathbf{0.847 \times 10^6 \text{ m}^3} \qquad ...(iii)$$

Hence, the total ponding required

$$= (i) + (ii) + (iii)$$

$$= [0.484 \times 10^6 + 6.048 \times 10^6 + 0.847 \times 10^6] \text{ m}^3 = 7.379 \times 10^6 \text{ m}^3.$$

(*i*) With maximum variation of 1.2 m in pond level, we have the min. surface area of pond reqd. to accommodate 7.379×10^6 m^3 of water

$$= \frac{7.379 \times 10^6}{1.2} \text{ m}^2 = 6.15 \times 10^6 \text{ m}^2$$

$$= \frac{6.15 \times 10^6}{10^4} \text{ hectares} = \mathbf{615 \text{ hectares.}} \quad \textbf{Ans.}$$

Since the allowable max. fluctuation in pond level is 1.2 m, and the head at full level is 26 m, we have the average head on turbines

$$= \frac{26 + (26 - 1.2)}{2} = \frac{26 + 24.8}{2} = \frac{50.8}{2} = 25.4 \text{ m}.$$

(*ii*) ∴ Max. Power generated $= 9.81\, \eta \cdot Q \cdot H$.

where η is the combined efficiency of turbines and generators

$$= 0.82 \times 0.90 = 0.738$$

$$Q = 49 \text{ m}^3/\text{sec}$$

$$H = 25.4 \text{ m}$$

$$\therefore \quad P_{max} = (9.81 \times 0.738 \times 49 \times 25.4) \text{ kW} = 9001.4 \text{ kW}.$$

With a daily load factor of 0.60, we have the average power developed = $0.6 \times$ Max. power

$$= 0.6 \times 9001.4 = 5400.8 \text{ kW.}$$

$\therefore$ Weekly power developed at the switchboard

$$= (5400.8 \times 5 \text{ days}) \text{ kW day}$$
$$= (5400.8 \times 5 \times 24) \text{ kWh} = 6{,}48{,}100 \text{ kWh}$$

Hence, weekly output at switchboard

$$= \mathbf{6.481 \times 10^5 \text{ kWh.}} \quad \textbf{Ans.}$$

Example 24.7. *A run-of-stream station with an installed capacity of 15000 kW operates at 15% load factor when it serves as a peak load station. What should be the lowest discharge in the stream so that the station may serve as the base load station ? It is given that the plant efficiency is 75% when working under a head of 20 m.*

(b) Also calculate the maximum load factor of the plant when the discharge in the stream rises to 20 cumecs.

Solution. Installed capacity = 15000 kW

Load factor = 15%

Since the plant acts as peak load station, the installed capacity would be equal to the peak load.

Now, $\text{Load factor} = \dfrac{\text{Average Load}}{\text{Peak Load}}$

$$\therefore \quad 0.15 = \frac{\text{Average Load}}{15000}$$

$\therefore$ Average load = $15000 \times 0.15 = \mathbf{2250 \text{ kW.}}$

In order that the plant can act as a base load station, it must supply 2250 kW average power. Using

$P = 9.81\,\eta \cdot Q \cdot H$, we have

$P = 2250$ kW

$\eta = 0.75$ (given)

$Q = ?$

$H = 20$ m.

$$\therefore \quad 2250 = 9.81 \times 0.75 \times Q \times 20$$

$$\therefore \quad Q = \frac{2250}{9.81 \times 0.75 \times 20} = \mathbf{15.31.\ m^3/sec.}$$

Hence, the stream must carry a minimum discharge of 15.31 cumecs, in order to make the plant work as base load station. **Ans.**

(*b*) When $Q = 20$ cumecs, then power developed by the plant is given as :

$$P = 9.81\,\eta \cdot QH$$
$$= 9.81 \times 0.75 \times 20 \times 20 = 2940 \text{ kW}$$

$$\text{Load factor} = \frac{\text{Av. Load}}{\text{Peak Load}} = \frac{2940 \text{ kW}}{15000 \text{ kW}} = 0.196, \textit{ i.e. } \mathbf{19.6\%.} \quad \textbf{Ans.}$$

Example 24.8. *The water turbines at a hydel storage plant produces 10,000 H.P.; when working under a net head of 30 m and with an overall efficiency of 80%. The inflow in the reservoir during a year is given below :*

Table 24.1

Month	*Inflow in million cu. m (M. cu. m)*
Jan.	90
Feb.	80
March	73
April	80
May	70
June	98
July	120
Aug.	80
Sep.	96
Oct.	105
Nov.	100
Dec.	75

Find (a) the minimum reservoir capacity reqd. to satisfy the uniform demand of water ; (b) the total quantity of water wasted during the year. Assume the reservoir to be full at the beginning of November. Use analytical method.

Solution. The H.P. produced by turbines = 10,000

Since 1 H.P. = 0.736 kW, we have the power produced by turbines in kW units

$$= 10{,}000 \times 0.736 = 7360 \text{ kW.}$$

Discharge reqd. to produce 7360 kW, Q, is given by

$$P = 9.81 \cdot \eta \cdot QH$$

where $\eta = 0.80$

$Q = ?$

$H = 30$ m

$$\therefore \quad 7360 = 9.81 \times 0.80 \times Q \times 30$$

or
$$Q = \frac{7360}{9.81 \times 0.80 \times 30} = 31.29 \text{ m}^3/\text{sec.}$$

∴ Flow reqd. during each month

$$= 31.29 \times (30 \times 24 \times 60 \times 60) = 81 \times 10^6 \text{ m}^3 = \mathbf{81 \text{ M. cum.}}$$

It shows that 81 M. cum of flow is required to be compulsarily released in order to produce the requisite electric power. The inflow and this reqd. uniform outflow data is now analysed in Table 24.2 starting from Nov. onward (when reservoir is full) to

Table 24.2

Month	*Inflow in M. cum*	*Outflow in M. cum*	*Draft used from storage in M. cum.*	*Storage refill in M. cum.*	*Reservior depletion in M. cum*	*Water wasted M. cum*
Nov.	100	81	0	—	—	19
Dec.	75	81	6	—	6	—
Jan.	90	81	—	6	0	3
Feb.	80	81	1	—	1	—
March	73	81	8	—	9	—
April	80	81	1	—	10	—
May	70	81	11	—	21	—
June	98	81	—	17	4	—
July	120	81	—	4	0	35
Aug.	80	81	1	—	1	—
Sept.	96	81	—	1	—	14
Oct.	105	81	—	—	—	24
Σ						95

workout the storage used, refill, net reservoir depletion, and the water wasted, as shown in this table. The table is otherwise self explanatory.

(*a*) From this table, it can be seen that the max. reservoir depletion is **21 M. cum**, which evidently represents the minimum reservoir capacity required to satisfy the uniform demand. **Ans.**

(*b*) Total quantity of water wasted = **95 M. cum. Ans.**

24.5. Comparison of Hydro-power with Thermal Power with Reference to Indian Conditions

Water is available in abundance in India, and most of our important cities and other industrial or populated areas are situated within a maximum distance of say 500 km. from the main concentrations of hydro-resources. Hydro-power can, therefore, be easily developed in India along with other utilisations of our water resources, such as irrigation, navigation, flood control, city water supply, etc. The hydro-power has ultimately been found to be cheaper than the thermal power, although its exploitation entails large initial expenditure. It has been estimated, that on an average, the initial investment on a hydro project is about 25% more than that on a thermal project. But, the recurring cost on a hydro project is much less than that on a thermal project, and hence, the hydropower finally comes out to be more economical. Besides these economic considerations, hydropower has been mainly produced in India as a side product during the development of river valleys for irrigation and flood control.

Hydropower is much more important than thermal power in India, mainly because we do have almost inexhaustible water resources in our country, and the same is available in the vicinity of most of our needy areas. In sharp contrast to this, the coal, which is the only cheap and conventional source of thermal power, is confined only to a few regions, and has to be transported over long distances. Hence, in areas which are far away from coal bearing regions, hydropower is the only choice. Moreover, the amount of coal available in our country, is limited in quantity and, therefore, cannot be used infinitely without proper planning for the future. It can be stated that due to various inherent advantages which the hydropower possesses over thermal power, the use of hydropower has been found justifiable on overall economic considerations even in areas close to the collieries in Andhra Pradesh, Madhya Pradesh, Orissa, etc. But in areas, where hydropower has also been fully tapped, we have to think of nuclear or other thermal resources. *Pumped storage plants* may also be thought of in such areas, and their possibilities should also be investigated.

In addition to these advantages, the hydro power is considered to be environmental friendly as compared to thermal power, since thermal power generation causes tremendous air pollution, directly affecting the health of the public.

Besides all these advantages of hydropower over thermal power, there is one serious limitation of hydropower. The limitation is that the hydro power production is totally dependent upon rainfall. Though hydro-projects are designed on a very conservative basis of planning on minimum water that would be available for 90 out of 100 years from a catchment, still there may be some periods when reservoirs do not fill up. If this happens throughout the country, the power position will become difficult. This has

happened in 1972 monsoon, when practically all the reservoirs were only half full, resulting in a sudden and drastic power shortage.

Taking such contingencies into account, it can finally be concluded that we must develop a combined system of power production. Hydropower should be exploited to as much extent as possible, and sufficient thermal power reserve should be kept to supplement the hydropower during the times of water crisis.

24.6. Hydropower Potentials of India

India has a vast potential for hydropower generation, particularly in the northern and north-eastern regions. As per an estimate made by the Central Electricity Authority, the potential of the country is assessed at 84 G.W.* at 60% load factor** (*i.e.* 140 GW of installed capacity), which is equivalent to about 440 billion units+ (kWh) of annual energy generation at a power factor of 0.6. The basin wise distribution of hydropower potential and developed hydropower is shown in Fig. 24.8.

The development of hydropower began in India, sometimes in the year 1897, when a 300 kW power station was installed at Darjeeling. It was followed by a power station at Sivasamundram in Mysore State, in 1902. It was then followed by a 50 MW Tata Hydro Electric project of Bombay City in 1914. The process of installation of hydro power projects was slow before independence, but it gathered a lot of momentum after India became free. So much so, that the hydropower production which was hardly 0.5 GW* before independence has gone to as high a value as 13.2 GW (at 60% load factor) by the end of 8th five year plan (*i.e.* March 1997). Another 6 GW shall be available from the projects which are under construction, and 3 GW from the projects which have been cleared for execution.

As stated above, the 1997 figure of developed power of 13.2 GW and the total potential of 84 GW are the values at 60% load factor**, which means that the presently installed capacity (1997) equals 13.2/0.6 = 22 GW, as against the potential (installedl capacity) of 84/0.6 = 140 GW, respectively, as shown in Fig. 24.8.

This estimated economically exploitable hydropower potential of 84 GW (at 60% L.F.) for India is quite low in comparison to the corresponding World figure of 4200 GW. Against our potential of 84 GW or 440 bkWh+, we have so for installed only 13.2 GW i.e. 69 bkWh++, which is only about 15.7% of the potential of 84 GW (440 bkWh). It evidently shows that a lot of work is still left to be done in this direction. The power potentials and power exploited on different river basins of India are shown in Fig. 24.8. It can be seen from this figure that Brahmaputra river basin has the maximum potential (35 GW), though least (0.5 GW) has been exploited. A lot of work is also left to be done on Indus and Ganga basins.

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* 1 GW (giga watt) = 10^9 watts = 1 million kW = 1 MkW
1 TW (trillion watt) = 10^{12} watts. = 10^9 kW = 1 billion kW = 1 b kW

** 60% load factor means that the installed machines are run, on an average, to give output capacity equal to 60% of the station capacity. This is so, because machines will not always be run at the installed station capacity but will be run, on an average, at 60% of this capacity.

\+ 84 GW is equivalent to = 84 × 365 × 24 (0.6 *i.e.* P.F) GWh = 441504 GWh : **say 440 TWh. *i.e.* 440 b kWh.**

++ 13 GW is equivalent to 13 × 365 × 24 × 0.6 (P.F) GWh = 68328 GWh : say ≈ 68 b kWh

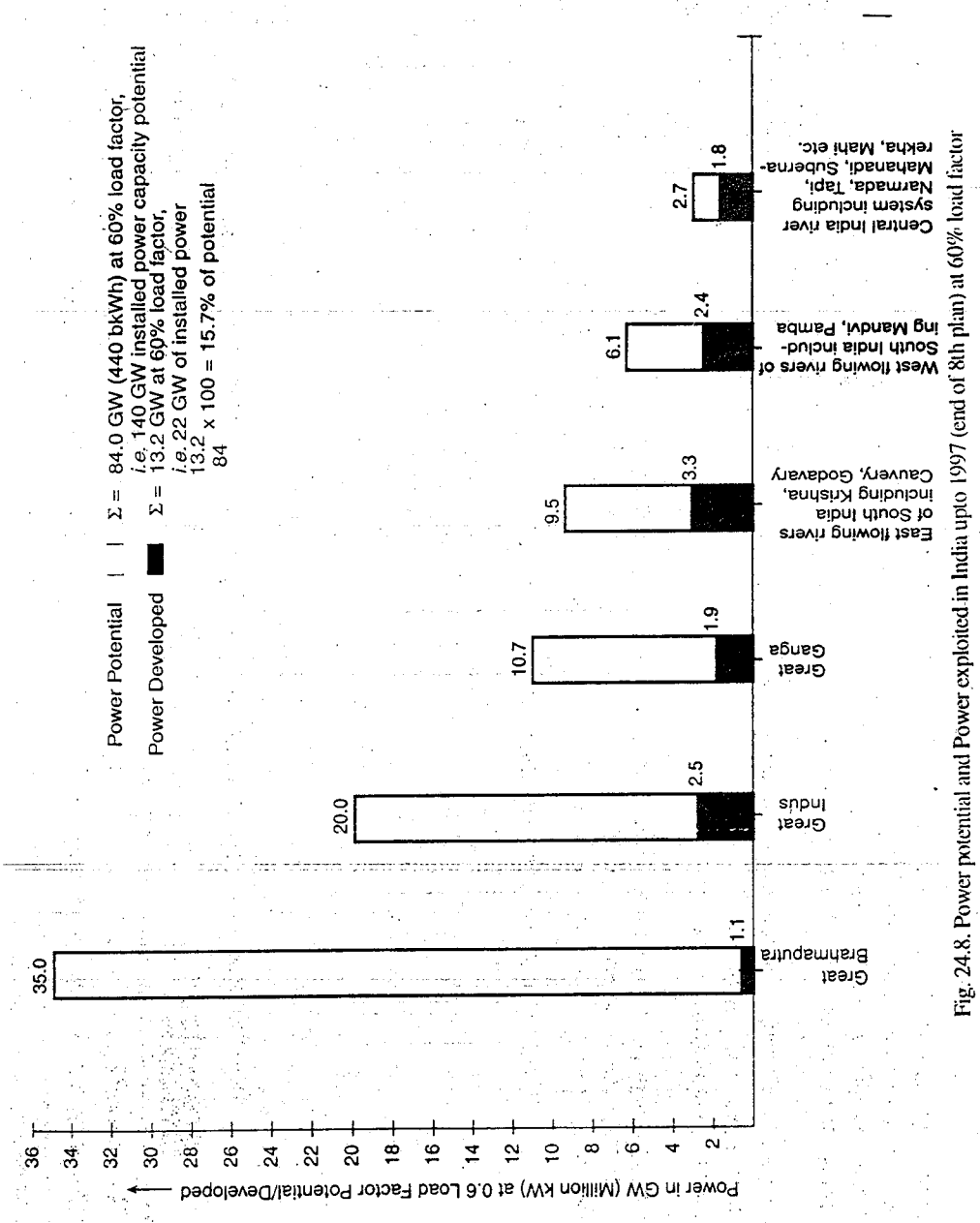

Fig. 24.8. Power potential and Power exploited in India upto 1997 (end of 8th plan) at 60% load factor

The developed hydropower capacities in India at the end of different plan periods in comparison to total power (*i.e.* hydro + thermal + nuclear) are indicated in table 24.3.

Table 24.3. Developed Hydro Power and Total Power in India at 60% Load Factor

Year end	*Installed capacity in GW (MkW)*		*%age of hydropower w.r. to total power*
	Hydropower	*Total power (Hydro + Thermal + Nuclear + Others)*	
1950	0.34	1.71	19.6
1956	0.64	2.89	22.02
1960-61	1.15	4.65	24.7
1965-66	2.47	9.03	27.4
1968-69	3.54	12.96	27.4
1974-75	4.52	18.32	24.7
1983-84	8.34	39.30	21.9
1985-86	9.30	46.80	19.9
1989-90	11.14	64.73	17.3
1996-97 (end of 8th plan)	13.2* (69 bkWh)	83.0** (435 bkWh)	15.9

*Against the potential target of 140 GW.
**Contains only about 2.5 MkW of nuclear power.

In India, the nuclear power generation is only about 3% of the total power generation, (including hydropower, thermal power, nuclear power and power from non-convention energy sources like wind power & solar power). The nuclear power is highly cost intensive*, Whereas thermal power involves huge fuel and *R* of *M* (Running of Maintenance) cost particularly for plants located far away from coal mines, or petro gas locations. The disposal of waste coal ash and air pollution caused by thermal power is also a costlier exercise, as compared to the present day cost being incurred for disposal of radioactive nuclear wastes, since such wastes can, through an expensive reprocessing, may lead to recovery of very costly plutonium, which may offset the cost of reprocessing and solve the problem of disposal of such nuclear radioactive wastes.

The Indian *Department of Atomic Energy* (DAE) is, therefore, planning to step up production of nuclear power as to boost its generation from 3% to 8% of the total electricity generation in the next 15 years. Whereas, the Deptt of Atomic Energy has calculated the nuclear power to be cheaper than the thermal power on an annual discounting of capital cost at about 8%, yet these calculations, based on various assumptions, have not been accepted by many, such as from Mr. Ramana, a fellow at *the centre for Interdisciplinary Studies in Environment and Development Bangalore,* and Mr. Amulya K.N. from the *International Energy Initiative*. Be that as it may, we need a mixed bag of various types of powers, since coal is an exhaustible source and cannot continue for ever. Hence, the aim of DAE to increase nuclear power production should not be opposed by saying that it is costlier than the thermal power or because of its high initial cost.

* The capital cost incurred on recent installation of two units of nuclear reactors of 220 MW each at the *Kaiga atomic power station (Kaiga I & II)* was about Rs. 2900 crores with additional initial fuel loading cost of Rs. 1045 crores for heavy water and Rs. 84 crores for Uranium, as compared to Rs. 612 crores spent on installing *Raichur Thermal Power Station VII*, both plants being of similar size, of recent vintage, and far away from coal supplies.

24.7. Electricity Generation from Various Sources in the World

The electricity generation from different sources in bkWh in various countries of the world are shown in table 24.4.

Table 24.4.Electricity Generation from Different Sources in Verious Countries of the World

Name of Country	*Total Electricity production in Billion kWh 1996*	*% age of production of electricity from different sources*				
		Hydro power % 1996	*Coal % 1996*	*Oil % 1996*	*Gas % 1996*	*Nuclear power % 1996*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*
Algeria	20.7	0.7	—	3.6	**95.7**	—
Argentina	69.8	32.9	2.2	5.4	48.3	10.7
Australia	177.0	8.7	78.9	1.6	9.0	—
Austria	53.5	64.0	11.5	3.7	17.5	—
Bangladesh	11.5	6.4	—	6.5	87.1	—
Belarus	23.7	0.1	—	29.1	70.8	—
Belgium	75.2	0.3	24.2	1.7	14.6	57.6
Bolivia	3.2	63.0	—	6.2	29.6	—
Brazil	289.8	91.7	1.6	3.1	0.2	0.8
Bulgaria	41.5	4.0	41.9	3.2	7.2	43.6
Canada	570.6	62.4	16.2	1.6	2.9	16.3
Chile	30.8	54.8	35.1	8.4	1.0	—
China	1080.0	17.4	75.0	6.0	0.2	1.3
Hong Kong, China	28.4	—	**98.4**	1.6	—	—
Colombia	44.6	79.1	6.9	0.5	12.8	—
Cuba	13.2	0.7	—	92.2	0.1	—
Czech Republic	63.8	3.1	73.3	1.1	1.2	20.1
Denmark	53.6	0.0	74.0	10.8	10.7	—
Egypt, Arab Republic	57.6	18.8	—	37.1	44.1	—
Finland	69.4	17.1	31.8	1.9	12.3	28.1
France	508.1	12.8	6.1	1.5	0.8	**78.2**
Germany	550.6	4.0	55.0	1.4	8.7	29.1
Ghana	6.0	99.9	—	0.1	—	—
Greece	42.3	10.3	69.3	20.2	0.2	—
Guatemala	4.3	74.3	—	20.6	—	—
Hungary	35.1	0.6	28.3	12.6	18.1	40.4
India	**435.1**	**15.9**	**73.2**	**2.8**	**6.2**	**1.9**
Indonesia	67.1	13.3	25.9	25.4	32.0	—
Iran, Islamic Republic	90.9	8.1	—	37.1	54.7	—
Iraq	29.0	2.0	—	**98.0**	—	—
Ireland	18.9	3.8	48.4	14.2	33.2	—
Israel	32.5	0.2	68.9	30.9	—	—
Italy	239.5	17.6	10.6	48.9	21.0	—

(1)	(2)	(3)	(4)	(5)	(6)	(7)
Jamaica	6.0	2.1	—	93.3	—	—
Japan	1003.2	8.0	18.2	21.0	20.2	30.1
Jordan	6.1	0.4	—	87.6	12.0	—
Kenya	4.0	81.6	—	8.8	—	—
Korea Dem Republic	35.0	64.3	35.7	—	—	—
Kuwait	25.5	—	—	21.7	78.3	—
Malaysia	51.4	10.1	6.4	12.3	71.2	—
Mauritius	—	—	—	—	—	—
Mexico	162.5	19.3	10.9	50.1	11.3	4.8
Morocco	12.4	15.7	44.9	39.4	—	—
Myanmar	4.3	38.3	0.1	15.9	45.7	—
Nepal	1.2	90.2	—	9.8	—	—
Netherlands	85.0	0.1	31.6	4.6	55.6	4.9
New Zealand	36.2	71.2	3.0	0.0	17.8	—
Niger	—	—	—	—	—	—
Nigeria	15.0	36.7	—	26.1	37.2	—
Norway	104.5	**99.2**	0.2	—	0.3	—
Pakistan	57.0	40.7	0.8	30.8	26.8	0.8
Peru	17.3	77.1	—	20.9	1.3	—
Philippines	36.7	19.3	13.2	49.6	0.1	—
Poland	141.2	1.4	96.8	1.2	0.3	—
Portugal	34.4	42.9	36.6	17.5	—	—
Romania	61.4	25.7	33.9	10.9	27.2	2.3
Russian Federation	846.3	18.2	18.5	9.4	40.4	12.9
Saudi Arabia	97.8	—	—	57.1	42.9	—
Singapore	24.1	—	—	78.7	18.6	—
South Africa	198.3	0.7	93.2	—	—	5.9
Spain	173.4	23.0	31.5	8.0	3.9	32.5
Sri Lanka	4.5	71.8	—	28.2	—	—
Sweden	139.6	36.9	3.0	5.2	0.3	52.5
Switzerland	55.6	51.0	0.0	0.5	1.2	45.2
Syrian Arab Republic	17.0	40.8	—	29.1	30.0	—
Thailand	87.5	8.4	20.0	39.3	42.0	—
Turkey	94.9	42.7	32.1	6.9	18.1	—
United Arab Emirates	19.7	—	—	17.1	82.9	—
United Kingdom	346.3	1.0	42.4	4.0	23.6	27.3
United States	**3652.0**	9.6	52.7	2.6	13.2	19.6
Vietnam	16.9	0.0	14.0	8.6	77.3	—

Note. Totals of Col. (2) to (7) infront of a particular country may not total to 100%, since other sources generating electricity, such as geothermal, solar & wind, are not shown here.

PROBLEMS

1. (*a*) What is meant by Hydro-power ? Compare hydro-power with thermal power w.r. to Indian conditions.

(*b*) Enumerate the different types of hydel plants, and describe the 'storage plant'.

2. (*a*) How do you classify a hydro-electric scheme on the basis of its operating head.

(*b*) Define and differentiate between the following in connection with hydropower :

(*i*) Firm and secondary power.

(*ii*) Load factor, utilisation factor and plant factor.

(*iii*) Installed and dependable capacity of a power house.

(*iv*) Design head, rated head, gross head, operating head and effective head.

3. What are the principal components of a hydro-electric scheme ? Discuss the utility of each component.

4. (*a*) Write a brief note on the use and types of 'turbines' in a hydroelectric scheme.

(*b*) What is a 'surge tank', and what are its types and uses ?

(*c*) What is a 'draft tube', and what are its uses ?

5. Write short notes on :

(*i*) Hydropower potential of India.

(*ii*) Hydropower *vs.* thermal power.

(*iii*) Surge tanks and their types.

(*iv*) Hydraulic turbines and their types.

(*v*) Trash racks.

25

River Navigation

25.1. Introduction

There shall be no exaggeration, if we define water as something which is a necessity, a comfort and a luxury too. There cannot be two opinions about water being a necessity, as there can be no life without water. Men, animals and even plants can never survive if there is no water. Besides the basic necessities for existence, water provides us various comforts and luxuries, and makes our lives easier. Besides the various uses of water, water provides us cheap* means of transport. Before the inventions of rail-road transport, water was the only means of communication between distant places. Even today, cargo – too heavy to be packed on wagon trains or trucks, is lifted and transported through big ships. In the modern affluent living, towing through water is not only required for transporting purposes, but is also required for recreational boating. However, boating and floating of ships through the natural rivers is not always safe. Rapids and sandbars may create problems, and may require considerable time to be passed. Isolated rocks, fallen trees, debris and other obstructions may create constant hazards, and may damage or even wreck the boats, steamers, or ships being towed through such waterways. It is, therefore, absolutely essential that all the waterways through which the boats or ships are to be towed, must be made completely safe.

The chief requirement for navigating through a waterway is the availability of sufficient water depth in the waterway. A minimum water depth of about 2.7 metres is generally required for navigating safely and economically ; although a depth of about 3.7 metres is generally aspired in the final development of a navigable waterway. Availability of lesser depth in the rivers may completely eliminate the possibility of towing the ships through such rivers or may cause increased unit cost of transport.

The cost of water transport is generally expressed in Rs. per tonne per kilometre. The total cost per kilometre of travelled distance divided by the transported tonneage, gives us the cost per tonne per kilometre. The total cost consists of fuel charges, wages of crews, etc., and other operating and incidental charges. The wages and other operating charges depend only upon the time of transit ; while the fuel consumption charges depend on one more factor *i.e.* the available draft. *The consumption of fuel decreases as the depth of water between the bottom of the ship and the bottom of the river increases.* This is because, when the clear depth between the barge and the river reduces, an increased drag force is developed which increases the consumption of fuel. So much so, that the fuel consumption can be reduced by about 25%, if this clear space is increased from 0.6 m to 1.5 m or so. In order to maintain this minimum depth available, the ship will have to be loaded only to such an extent that a certain minimum clear space remains available between the bottom of the ship and the bottom of the river. Hence,

* Water transport is cheaper because on water, the coefficient of friction being small, one horse power can pull 4 tonnes ; while on road and rail, it can move approximately 0.15 tonne and 0.5 tonne, respectively.

the amount of loading of the barge is dependent and limited by the availability of sufficient water depth at the shallowest point in the route. From this discussion, it can be easily concluded that a certain minimum depth depending upon the type and extent of traffic is always required to be maintained in the river, so as to keep it fit for shipping or towing of barges. When this available depth or draft is not sufficient or if the channel width, channel alignment, locking time, current velocity, terminal facilities, etc. are not adequate, engineering measures are undertaken so as to increase the available draft and to improve other requirements. By undertaking more and more measures, and thereby spending more and more money on improving a waterway for navigation, we can make heavy and heavier transport feasible through that waterway. A compromise between the money invested on these technical works and corresponding advantages likely to be obtained by way of better and cheaper water transport, must be weighed against each other while working out a financial justification for such works. Besides the direct tangible benefits such as increased and cheaper communications for commerce and trade, etc. ; the intangible benefits such as recreational boating and better defence facilities for carrying military equipments, during war times, should also be given due importance, while working out the economic justification for such improvement works.

25.2. Various Requirements of Navigable Waterways

There is no rigidity about the requirements of a good navigable waterway, since it all depends upon the extent and type of traffic likely to pass through it. However, the various general requirements are enumerated below :

(1) Sufficient water depth is available so as to pass the more heavily loaded barges cheaply and economically.

(2) The width of the waterway is sufficiently more than the width of the tow itself.

(3) The radii of the bends should not be sharp and should be high enough to allow the maximum length of the ship to pass through them.

(4) The alignment of the waterway should be as straight as possible, because a highly irregular alignment increases the circuitry or length in excess of air-line distance which the barge tow must travel. The existing channels generally have a length about 50 per cent greater than the air-line distances.

(5) The flow velocities should not be high, as they may cause substantial reduction in the true speed for tows moving upstream and thereby increasing the time of transit and the cost of transport per kilometre. The speed of most of the barge tows in still water is of the order of 2.8 m/sec. The flow velocities of the order of 1 m/sec may, therefore, cause sufficient reduction in true speed (*i.e.* $2.8 - 1.0 = 1.8$ m/sec) and hence, should not exceed such a value.

(6) In order to minimise the transit time, the time required for the tow to pass through locks should be minimum. In certain cases, where the lock is not large enough to accept the entire tow, the tow is generally broken and taken through the lock in portions. This increases the time lost in locking, and thereby increasing the transit time and the cost of transport. Hence, sufficient sized locks should be ensured for economic and better transport.

(7) Efficient and adequate terminal facilities for unloading the barges for transferring the cargo effectively, must be ensured for economic and better navigation.

25.3. Various Measures Adopted for Achieving Navigability

There are three basic methods which are generally adopted for improving a river for navigation. They are :

(1) *Open channel methods*

(2) *Lock and dam arrangements* ; and

(3) *Canalisation.*

They are described below :

25.3.1. Open Channel Methods. In the open channel methods, the existing waterway is improved to such an extent as to make navigation possible. This improvement in natural waterway is possible only if the following conditions are satisfied :

(*i*) Sufficient discharge is available in the river throughout the year or at least for a reasonable portion of the year.

(*ii*) The existing river is having a satisfactory alignment without excessively sharp bends.

(*iii*) The river bed slope is reasonably flat so that the flow velocities are not excessive (*i.e.* they are within 1 m/sec or so).

(*iv*) The river width is not too small and is such that it can be improved economically for the modern barge tows.

(*v*) The material of the river bed and banks should permit satisfactory treatment by one or more of the open channel methods.

If the above requirements are approximately satisfied, the channel can be economically improved and made fit for navigation. But if the available conditions are far too short of requirements, open channel methods may prove to be highly uneconomical, and, therefore, should not be considered. However, these requirements and factors may be controlled to some extent by some suitable measures. Say for example, if the discharge in the river during lean periods is very low, while the average annual flow is adequate, reservoirs may be constructed so as to store water and augment the supplies during lean weather flows. Similarly, very sharp bends may be eliminated by cut off channels, provided the resulting channel slopes remain within limits.

The **various works and techniques** that may be involved in improving the channel by the Open channel methods are :

(*a*) *Constructing and Regulating the flow through Storage reservoirs.*

(*b*) *Excavation and Dredging.*

(*c*) *Contraction works.*

(*d*) *Bank stabilisation.*

(*e*) *Straightening the waterway by artificial cut offs.*

(*f*) *Removal of snag, debris and other obstructions.*

These techniques are generally required together as one of them may rarely provide the necessary required improvement. These techniques are described below :

(*a*) **Storage reservoirs.** The storage reservoirs generally store water during high flows and can release the required amount of water during lean-flows, so as to make downstream navigation possible even during periods of low weather flows. However, the construction and planning of storage reservoirs for navigation alone is not generally justified economically. Hence, reservoirs are mostly planned under Multipurpose

projects, where navigation may be one purpose of that project. Moreover, the storage reservoirs can augment low supplies for navigation, only if the reservoir is situated at the head of a relatively short navigable reach. This is because ; *as the distance from the reservoir to the navigable river-reach increases, reservoir-releases have to be increased so as to allow for transit losses due to seepage, evaporation, etc. The releases must also be made much in advance so as to allow for travel time to the navigable reach and their quantity has to be sufficient even after reduction due to channel storage.*

(*b*) **Excavation and dredging.** Huge amounts of excavations are generally required for clearing sand bars and filled channel sections in order to make it fit for navigation. Besides the basic initial excavations, continuous desilting and proper maintenance is required in order to keep the waterway fit for navigation. These excavations from the bed and banks of the waterway are generally carried out by *dredging* by means of dredgers. Three types of **dredgers** are generally used. They are :

(*i*) *Dipper dredgers.* They are merely floating power shovels and are used on small projects.

(*ii*) *Ladder dredgers.* They have an endless chain of buckets for bringing the excavated material upto the surface. The cuttings carried by buckets are discharged on a *belt conveyer* which are disposed of through a *stacker conveyor* at the rear of the dredger. Since the stacker conveyors (generally called *spoil stackers*) are limited in length to about 100 metres or so, ladder dredgers cannot be used when the excavated materials (*i.e.* spoils) are to be discharged at a considerable distance from the dredge.

(*iii*) *Suction dredgers.* In these dredgers, the cuttings and water are collected in suction pipes, and the mixture is then discharged by pumping through a pipe supported by floats (called spoil pipe) into the desired spoil area. A line diagram of this operational process is shown in Fig. 25.1.

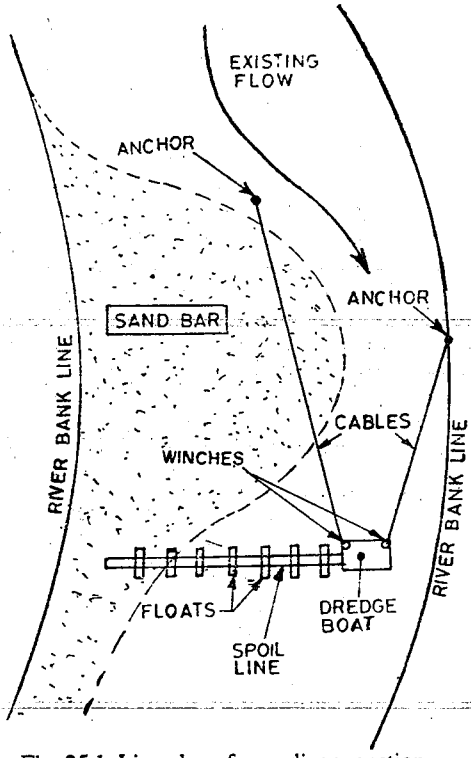

Fig. 25.1. Line plan of an ordinary suction dredge called Dust pan dredge.

A suction dredge cannot operate in rocky or boulder river reaches. The suction head of these dredges is provided with jets or rotating blades so as to cut or loosen the bed material, and also with suction openings through which the soil and water mixture enters into the suction pipe. These dredges can make cuts of about 10 m width through sand bars, and various such parallel cuts can be made in order to achieve a wider channel.

(*c*) **Contraction works.** Contraction works are those engineering works which are constructed in order to change a wide shallow river into a narrow deep river ; or to close off the river creeks (small branches) and thus to divert the entire water into the main

the material of bed and banks of a river is coarse grained with little shallow wide channel, or a number of coarse channels at low waters, will develop. ch situations may be corrected with the help of spurs or groynes. Under the process, rivers carrying huge sediment loads can be corrected with the help of properly placed permeable spurs called sal balli spurs made of sal ballies, driven at some suitable distance centre to centre in rows across the river current and braced at top. The function of permeable sal balli spurs is to slow the current and thus promote silting in the dyked area. The concentration of flow in the narrower section also encourages deepening of the channel. Several years are allowed for the effect of the structures to develop.

Similarly, the rivers carrying a little sediment load can be corrected by properly placed impermeable spurs or jetties which shall divert the flow, thereby confining the entire water in a smaller width and thus deepening the same.

(*d*) **Bank stabilisation.** A good navigable channel must have stable banks. When the river banks are not stable and start caving, the river starts meandering, creating bends, which may obstruct the path of longer barge tows. Moreover, scouring at concave banks and silting at convex banks take place due to meandering (as explained in chapter 8). Hence at bends, sufficient depth will prevail at least near the concave side. But the tangents, *i.e.* the crossings joining the two successive bends, will definitely develop shallower channels with cross bars by the deposition of sediment scoured from the upstream bend. *It is in these crossings that the controlling depths for navigation occur.*

Spurs or groynes, when suitably and intelligently placed, may prove to be useful in bank stabilisation ; because a spur placed along the concave bank shall promote silting. Banks may be protected more easily by *pitching* or by *rivetments*. The entire concave bank is generally protected by pitching. The loosely dumped stone called apron or riprap is generally used, and it is extended from top of bank to beyond the toe of the underwater slope. This extension of rivetment in the bed is essential so as to avoid the failure of rivetment due to scour and consequent undermining of the underwater edge of the rivetment. The rivetment must be flexible so as to adept itself to the surface on which it is placed. Moreover, the rivetment must be relatively impervious so as to avoid, the washing of fines through it. It must also be strong enough to resist the flow currents. Various types of rivetments are used. Concrete mattresses in the form of concrete blocks placed in wire meshes may sometimes be used, when ordinary stone dumping over a graded filter is not provided due to non-availability of stone in the nearby areas. Uncompacted asphalt paving is also finding a use in developed countries, and is under serious investigations. Compacted asphalt paving and monolithic concrete paving are not generally used, as they are liable to be cracked and damaged by uplift pressures.

(*e*) **Straightening by artificial cut-off.** It was explained in chapter 8, as to how and when a cut off develops in a meandering river. Since the development of a cut-off eliminates sharp bends which are undesirable for navigation, artificial cut-offs may sometimes be used advantageously. A pilot cut is made and allowed to develop (Fig. 25.2). These cut offs have been used with

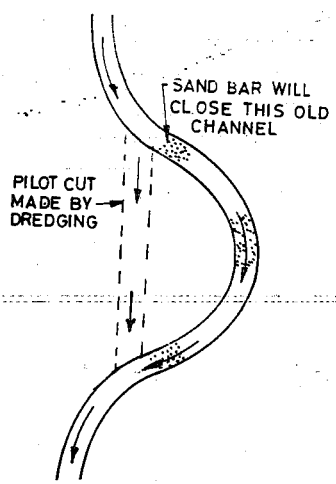

Fig. 25.2.
Development of a cut off.

success at various places, provided the banks are stabilised to avoid future caving and meandering.

(*f*) **Removal of snag, debris and other obstructions.** Presence of debris, trees, isolated rocks, and other obstructions, not only pose a direct hazard to the barge tows, but also promote the formation of sand bars. They must, therefore, be removed effectively in order to ensure safe and economical navigation. Different methods and equipments may be used in different cases, depending upon the circumstances of each case. Tractors, winches, derrick barges, explosives, etc. may be required in the process of clearing the waterway of these obstructions.

25.3.2. Lock and Dam Arrangements. The arrangement consists of dams which create a series of slack water pools through which the traffic can move with locks to lift the vessels from one pool to the next. Lock and dam construction may be adopted where existing site conditions are not favourable for adopting open channel methods described earlier. *This arrangement is a second choice to open channel methods.* In this arrangement, water is required for lockages, sanitary releases, evaporation, percolation, etc. This requirement of water is much less than that required for open channel procedures. Hence, when the available water is less, these arrangements may have to be adopted.

The slack water pools behind the dams, will submerge the rapids and channel bends, and thus overcoming those problems. Further, because of their relatively large areas of cross-sections, the velocities in these pools shall be low enough as to cause lesser reduction in true speed of the barge tow moving upstream.

Lock and dam arrangements are suitable only on rivers bringing only a little sediment load. Because, highly silt laden river-waters shall fill up the pools rapidly. Moreover, suitable sites for construction of small dams must be available for providing such arrangements. Further detailed descriptions of these arrangements are beyond the scope of this book.

25.3.3. Canalisation. A totally new channel-cut is provided artificially around an otherwise impassable obstruction or between two navigable rivers. Such a cut is generally economical only when a short length of new channel opens a large length of existing waterways. Construction of a new channel connection between two existing waterways is also sometimes adopted, so as to ensure a continuous traffic way. However, canalisation is a costly process, as the per kilometre cost of canal, capable of passing modern barge tows, is normally very high, and are adopted when very short lengths are required.

25.4. India's Navigable Waterways

Unfortunately, there is no river in India running from North to South or *vice versa.* Most of the Indian rivers run from West to East or from East to West. This is the major handicap in the development of navigation in this country. Although there are about 15,000 kilometres length of navigable waterways in India, but most of them are in very bad conditions due to silting and poor maintenance. Most of these waterways are fit only for boating except for about 2,500 kilometres length, in which the steamers can ply. Out of the total navigable lengths of about 15,000 km ; about 8000 km is on rivers, and 7000 km is on canals and backwaters. Most of these navigable waterways exist in Ganga and Brahmaputra rivers and their tributaries. On account of its inherent advantages, the water transport has been widely developed in various countries (especially in U.S.A.), but a lot of work remains to be done in India. Let us hope that as soon as our Government finds some free investment, the same will be invested in improving our navigable

water-ways. Although the initial investment may be high, but the advantages obtained shall be tremendous, as it will not only provide us cheap transport, but would relieve the burden upon our rail and road transport.

PROBLEMS

1. (*a*) What do you mean by 'River navigation'? What is the importance of river navigation in modern system of transport ?

(*b*) What are the requirements of navigable waterways, and what measures are adopted to satisfy these requirements ?

2. (*a*) Discuss the importance and necessity of improving India's waterways.

(*b*) Enumerate the different methods which are adopted for achieving navigability in a river. Discuss one of these methods in details. Also discuss the pros and cons of different methods and their use for a particular project.

3. Write a descriptive note on 'River navigation'.

26

Tank Irrigation

26.1. Definition and General Introduction

Tank irrigation may be defined as the *storage irrigation scheme*, which utilises the water stored on the upstream side of a smaller* earth dam, called a *bund*. These earthen bund *reservoirs* are, thus, infact, called as *tanks*, specifically in South India, where such works are very common. This terminology is, therefore, limited to India only.

There is, thus, no technical difference between a 'reservoir' and a 'tank', except that a large-sized tank will be termed as a reservoir. Moreover, a reservoir will generally be formed by dams of any material (such as masonry dam, concrete dam, or earth dam, etc.) ; whereas, a tank is generally said to be formed by earth dams only (or strictly speaking by earthen bunds). These earthen bunds, spanning across the streams, are called *tank bunds* or *tank banks*.

Most of the existing tanks of south India possess a *maximum* depth of 4.5 m, while a few are as deep as 7.5 to 9 m, and only a few exceptional ones exceed 11 m in depth. When the depth of the tank exceeds 12 m or so, the tank is generally referred to as a *reservoir*.

Like all earth dams, tank bunds are generally provided with *sluices* or *outlets* for discharging water from the tank for irrigation or other purposes. These *tank sluices* may be pipes or rectangular or arched openings passing near the base of the bund and through the body of the bund, and carrying the water into the downstream channel below the bund or transporting at distances where required, through pipes or canals. Sometimes, the *supply sluices* may not be passed through the body of the bund, and may be carried adjacent to it through some hill side at one end of the bund.

Similarly, as in the case of all dam reservoir projects, tanks are provided with arrangements for spilling away the excess surplus water that may enter into the tank, so as to avoid over-topping of the tank bund. These *surplus escape arrangements* may be in the form of a *surplus escape weir*, provided in the body or at one end of the tank bund, or some other arrangement like a syphon spillway may be provided as is done in the case of earth dam projects. The surplus escape weir is a masonry weir (compared to an ogee spillway in an earth dam) with its top *i.e.* crest level at equal to *Full tank level* (F.T.L.). When tank is full upto FTL, and extra water comes in, it is discharged over the surplus escape weir. The length or capacity of this surplus escape weir will be so designed that water level in the tank does never exceed the *maximum water level* (M.W.L.). The top of the tank bund will be kept at a level, so as to provide a suitable free-board above this M.W.L.

Since the surplus escape weir is a masonry weir, it will have to be properly connected to the earthen bund by suitably designed bank connections.

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* Generally less than 12 m or so in height.

26.2. Isolated Tanks and Tanks in Series

Most of the existing small sized tanks of south India form part of *groups of tanks*, which are connected together in series, such that any tank either receives the surplus water of the upper tank(s), or sends its own surplus into some lower tank(s), or do both. However, when a tank neither receives water from an upper tank nor discharges its own surplus into a lower tank, it is called an *isolated tank*. There do exist some isolated tanks also in South India.

Figure 26.1. shows a typical group of 15 tanks numbered 1 to 15, and an isolated tank A. It is evident that *considerable economy of water can be obtained from the system of grouping*, because the surplus water of the each tank and also the drainage of its wet cultivation are caught up by the next lower tank. Each tank of the group takes a share in the yield of the whole catchment above it, which can be classified as follows :

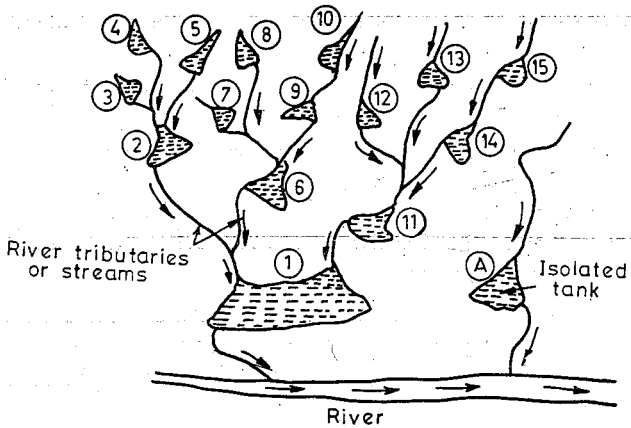


Fig. 26.1. Fig. showing isolated tanks and tanks grouped in series.

(*i*) the *free catchment*, which is the catchment area which only drains into the tank under consideration ; and

(*ii*) *the combined catchment*, which is the area of the whole catchment above the tank under consideration.

The difference between the combined and the free catchment, thus, gives the area of catchment intercepted by upper tanks. The whole catchment of the highest tank, on each drainage, shall be its free catchment. Moreover, each tank will receive the whole run- off from its free catchment ; but from the remainder of its catchment it will receive only the balance runoff which remains after the upper tanks have been filled.

One of the disadvantage of grouping of tanks, however, is that, if a breach occurs in an upper tank, it exposes all the tanks in the series below, to the risk of similar failures. Hence, while making repairs or doing restoration works in such groups of tanks, it is necessary to consider each system of tanks collectively as a whole, and to carry out repairs from the upper end of each group. The extent of danger to lower tanks due to breach of an upper tank, however, depends upon their relative capacities. For example, if a small upper tank breaches and passes its storage into lower large-capacity tank, it may pose little danger even if the breach may completely empty the small tank. However, the danger will be imminent if the case is opposite, and the large sized upper tank breaches and passes its storage into the lower small sized tank.

The provision of suitable *breaching sections** in tanks forming part of a group is, therefore, considered as a very desirable precaution against their failure by breaching. This is because, a breach occurring in a tank bund at a well selected point (at the place of breaching section) will not empty the tank, and indeed frequently does little more than supplement its surplusing power to a reasonable degree to the greater security both of the tank itself and those below it in the group.

26.3. Capacity of Water Spread of a Tank

The *gross capacity of a tank* may be defined as the cubic content of water stored in the tank upto F.T.L. The *effective capacity of a tank*, will, however, be the cubic content of water stored between F.T.L. and the bottom or sill level of the lowest supply sluice.

These storage capacities can be computed easily by using the contour plan of the area of the water-spread ; the total capacity being the sum of the capacities between successive contours. The smaller the contour interval (Δh), the more accurate the capacity computation will be. This is because, if A_1 and A_2 represent the areas enclosed between two successive contours, then the cubic content between these contours is roughly taken as $\frac{A_1 + A_2}{2}(\Delta h)$.

The summation of all cubic contents between the successive contours will be the required storage capacity of the tank.

When the contour plan is not available, and only the area of the tank at F.T.L. is known, then the effective cubic content of the tank may be roughly computed as follows

The area of tank at F.T.L. is multiplied by one- third of the depth from this level (F.T.L.) to the deep bed of the tank, or the level of the sill of the lowest sluice, whichever is higher of the two.

If the area (A_1) of the tank bed at the level of the sill of the lowest sluice, and the area (A_2) of the tank at F.T.L. are known even roughly, then the effective cubic content of the tank may be computed as $\frac{1}{3}[A_1 + A_2 + A_1A_2] \times h$, where h is the height or difference in elevations between F.T.L. and sill level of the lowest sluice. This formula is based on the assumption that the water stored is in the shape of a frustum of a cone.

These principles on which the storage capacity of a reservoir is computed, were also discussed earlier under article 18.2.

26.4. Designing the Section of the Tank Bund

The tank bund, as you know, is nothing but a small-sized earth dam ; and hence, strictly speaking, its design and construction should be carried out in accordance with the principles applicable to earth dams (already discussed in details in chapter 20).

In accordance with these very principles, the tank bunds may be of three types ; *i.e.*

Type A : *Homogeneous embankment type* (Fig. 20.1) ;

Type B : *Zoned embankment type* (Fig. 20.2) ; and

Type C : *Diaphragm type* (Fig. 20.3).

Most of the tank bunds of South India belong to type A, and they have been constructed with the soils excavated from pits in the immediate vicinity of each bund, and carried by head load to the bund.

* A 'breaching section' is a length kept lower and weaker than the remainder of the bound, so as to localise a breach in that length only.

As discussed in chapter 20, the large modern earth dams are designed after doing a lot of mathematical calculations for seepage and stability analysis, and keeping into consideration the latest advancements made in the field of soil mechanics. However, for small earthen bunds, generally no such calculations are carried out, and the sections are designed on the basis of practical considerations and experience derived from success and failure of similar works and of their working conditions.

The commonly adopted standards used for fixing the dimensions of tank bunds, in South India, are given in table 26.1.

Table 26.1. Common Dimensions of Tank Bunds

S. No.	*Depth of deep bed below F.T.L. i.e. maximum water depth in metres*	*Free board (in metres)*	*Width of top of bund (in metres)*
1.	1.5 to 3.0	0.9	1.2
2.	3.0 to 4.5	1.2	1.5
3.	4.5 to 6.0	1.5	1.8
4.	over 6.0	1.8	2.7

In favourable soils, such as red and white gravel, red and black loams, etc., the side slopes of the bund may be kept as $\frac{1}{2} : 1$ ($\frac{1}{2} H : 1\ V$) for smaller tanks with water depths not exceeding 2.5 m, and 2 : 1 for larger ones up to about 5 m in depth. In light sandy, or black cotton, or clayey soils, however, the slopes may be kept between 2 : 1 to $2\frac{1}{2} : 1$.

The upstream face of the tank bund is generally rivetted with stone apron or riprap (Fig. 26.2) so as to protect it against erosion, and if this is done, then the upstream slope

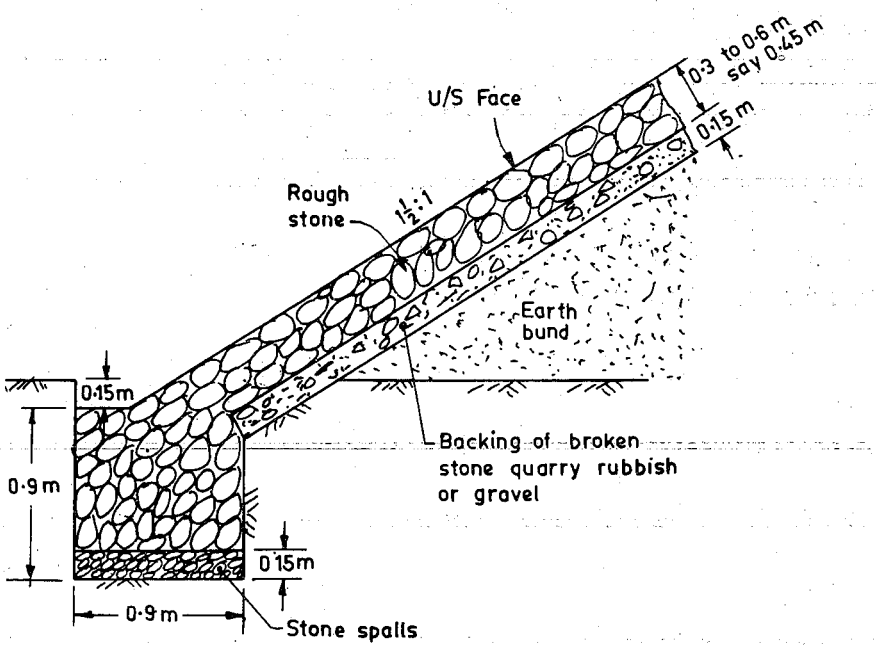


Fig. 26.2. Upstream Revetment of Tank Bunds.

is generally adopted is $1\frac{1}{2}:1$ even upto 6 m depth. For inferior soils, or greater depths, however, the rivetted slope may be made flatter, say 2 : 1.

In this way, for average cases, a $1\frac{1}{2}:1$ slope will generally be adopted for upstream face, and 2 : 1 slope for downstream face.

This practice is *contrary* to the standard recommendations adopted in various countries, where the upstream slope, even when revetted, is kept flatter than the downstream slope because of the soil being saturated. There are, however, thousands of tanks in Tamilnadu with water slopes of $1\frac{1}{2}:1$, and failure by slipping of this slope is rare ; and hence, in normal cases, the local practice in this respect can be easily adopted. In very small tanks, and in case the upstream slope is heavily revetted, upstream faces have been given 1 : 1 or even steeper slopes in actual practice, but such steeper slopes are not recommended.

26.5. The Tank Weirs or Surplus Escape Weirs and Their Design Principles.

26.5.1. Definition. As stated earlier, the excess surplus water is spilled from a tank, into the downstream channel, so as to avoid the rise of water in the tank above the M.W.L. In fact, the water will generally start spilling over the crest of this escape weir, as and when it rises above F.T.L. ; and the discharging capacity of this weir will be designed so as to pass the full maximum flood discharge (likely to enter the tank) with a depth over the weir equal to the difference between F.T.L. and M.W.L.

Although the effective storage capacity of a tank is limited by F.T.L., but the area submerged by the tank bund and revetment are all dependent on M.W.L. And hence, in order to restrict the dimensions of these, it is desirable to keep the difference between F.T.L. and M.W.L. to a smaller value. But, on the other hand, the smaller is this difference, the longer will be the surplus escape required in order to enable it to pass the given discharge. Hence, the difference (H) between F.T.L. and M.W.L. is fixed on a compromise basis, in each particular project, so as to obtain overall economy and efficiency. In small and medium sized tanks, the usual difference between F.T.L. and M.W.L. is kept from 0.3 to 0.6 metre, and it is rarely allowed to exceed 0.9 metre.

26.5.2. Types of Tank Weirs and Their Cross-sections. Tank escape weirs are similar to river-weirs (*i.e.* diversion weirs or anicuts), and hence, they may be classified into the following three general types :

Type A : *Masonry weirs with a vertical drop ;*

Type B : *Rockfill weirs with a sloping apron* ; and

Type C : *Masonry weirs with a sloping masonary apron (glacis).*

These three types of river weirs have already been discussed in article 9.3.1.

Besides these three important types of river weirs, which are used as tank weirs also, the fourth type of a tank weir which is a combination of type A and type C, may also be used. In such a *D type weir,* a number of vertical steps are made (as in the case of a stepped fall) instead of providing a horizontal or sloping downstream apron. Such weirs of type D are called *weirs with stepped aprons.* The A and D type of weirs are most widely adopted, although every type of weir has been used in South Indian tanks.

The A type of weir *i.e.* weir with a masonry pucca horizontal floor, may sometimes by provided with depressed floors, so as to provide sufficient water-cushion for absorbing the impact of high water falls. Typical sections of the two sub- types of A type, and

D type of weirs, and the conditions under which each is adopted are shown in Figs. 26.3 (*a*), 26.3 (*b*), and 26.4 respectively.

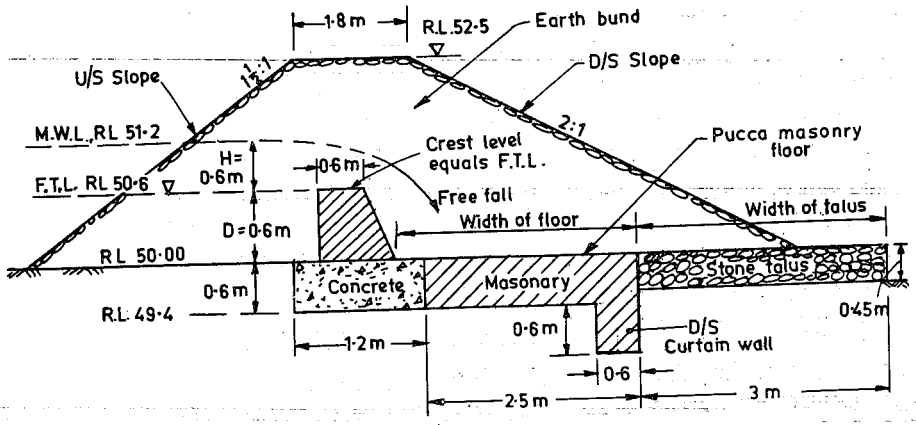


Fig. 26.3 (*a*) A typical masonry tank weir with a vertical drop, with a horizontal undepressed floor, suitable for low drops of 0.6 to 0.9 metre or for even higher drops when the soil is a rock.

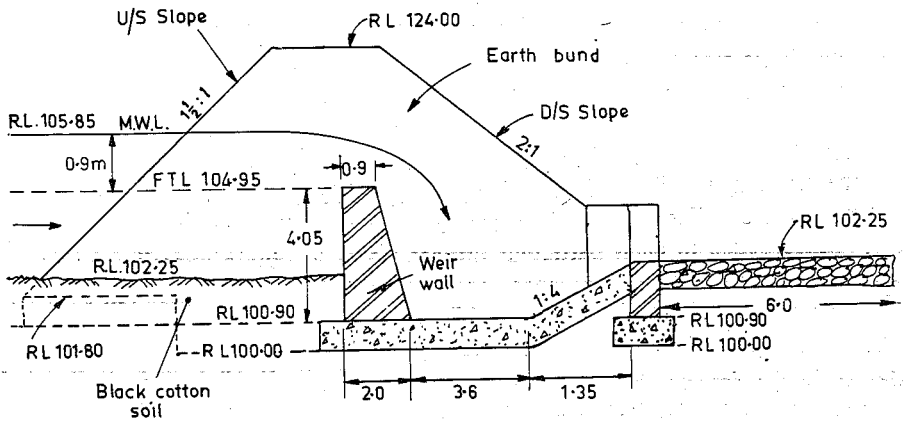


Fig. 26.3 (*b*) A typical masonry tank weir with a vertical drop with depressed floor, suitable for high drops (say 2.5 m upward) or in soft soils but only slightly pervious.

The general principles and considerations which guide the design of tank weir sections, are the same as in the case of river weirs founded on *rocks* or *loose pervious* or *impervious soils* But in case of tanks, the conditions regarding the *nature of foundation soil* and the *working conditions* are more varied than in the case of river weirs.

Say for example, the river weirs or anicuts are generally founded on soft sandy soils, whereas in tank weirs, almost every class of soil is occasionally to be built on, except this rarely met foundation sand.

Similarly, the tail water conditions in anicuts differ from those in tank weirs. This is because, the tail water backing up during floods and affording some protection to the talus and to the soil at the downstream toe of the apron, are absent in the majority of tank weirs ; and, therefore, the tank weirs built on soft soil would, other things being

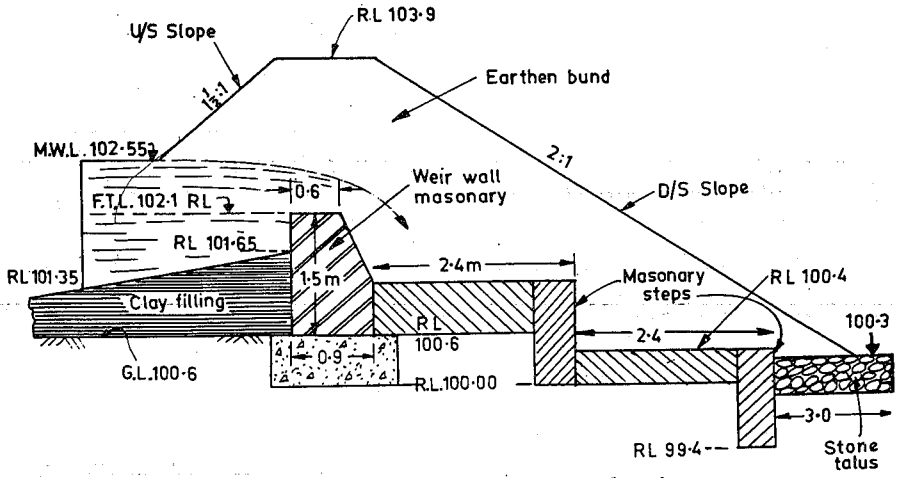


Fig. 26.4. A typical stepped-apron tank-weir, suitable for sites on soft soil when the drop exceeds 0.9 metre.

equal, be more exposed to damage by scour at the downstream toe of the apron and retrogression of levels than river weirs.

On the other hand, the depth of water passing over the crests of tank weirs is generally far less than that over anicuts, and in the case of very many tanks, it is only, on an average, for a few days each year, that any flow at all passes over the weir, and while the duration of continuous heavy flow is frequently limited to a maximum of two to three days. Tank weirs are, thus, usually worked, far less heavily and far less continuously than anicuts, and this fact may be duly considered while designing them.

Empirical Formulas for Determining Width of Floors of Tank Weirs. The width of the horizontal floors of type *A* and *D* weirs, from the foot of the drop wall to the downstream edge of the floor should never be less than 2 $(D+H)$ where D is the height of the drop wall, and H is the maximum water head over the wall. In important works, this width may be increased to 3 $(D+H)$. The rough stone apron forming a *talus* below the last curtain wall may be of varying widths depending on the nature of the soil and the velocity and annual probable quantity and intensity of runoff ; it would generally vary from 2.5 $(D+H)$ to 5 $(D+H)$ according to varying conditions.

26.5.3. Length of the Tank Weir. In order to determine the length of the escape weir, it is, first of all, necessary to determine the *maximum flood discharge* that may enter into a tank, after it is filled up to full tank level. This peak discharge that may come from the free catchment of a tank can be fairly estimated by using the empirical formulas applicable to the given region, as discussed earlier under article 7.9.2. Generally, in South India, all tank weirs have been designed on the basis of Ryve's formula (Refer equation 7.49). The formula is directly applicable for free catchments in all isolated tanks. However, for combined catchment of a tank forming a constituent unit of a group, the following modified formula is used for calculating peak discharge :

$$Q_p = C_1 \cdot A^{2/3} - c_1 a^{2/3} \qquad ...(26.1)$$

where C_1 is the coefficient in Ryve's formula (Refer Table 7.45)

A is the area of the combined catchment in sq. kilometres

c_1 is the coefficient from $\frac{1}{5}$th to $\frac{1}{3}$rd of C_1 and

a is the area in sq. kilometres of the catchment intercepted by upper tanks.

This formula is purely empirical, and should only be applied when the upper tanks are provided with adequate surplus works, and kept in good state of repairs.

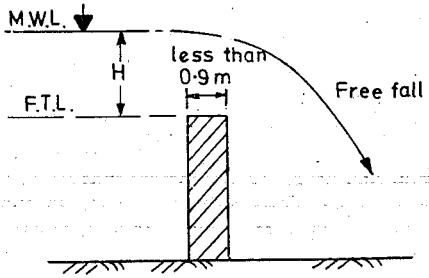
After the peak discharge is worked out and decided upon, the length of the weir can be worked out by using the appropriate discharge formula applicable to the weir type adopted in the project. In general, the discharge over a broad crested free weir and without any velocity of approach, is given as :

$$Q = C \cdot L \cdot H^{3/2} \qquad ...(26.2)$$

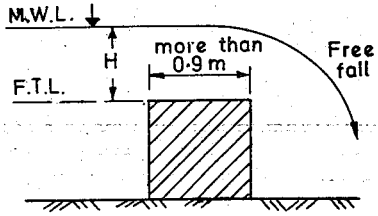
where L = the length of the weir ;

H = the head of water over the weir (*i.e.* the difference between M.W.L. and F.T.L.), and

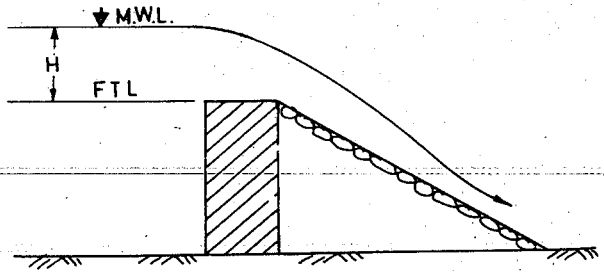
C = a constant, the common values of which for three commonly used types of weirs in South India, are shown in Fig. 26.5.



(*i*) Weirs with crests up to 0.9 m wide.



(*ii*) Weirs with crests over 0.9 m wide or weirs provided with dam-stones.*



(*iii*) Rough stone sloping escapes.

Fig. 26.5. Different types of Tank Weirs and the values of their coefficients in discharge formulas.

*A common practice which has been resorted to for centuries in South India is to fix **damstones** in the crest wall of the weir. these stones are generally about 15 cm × 15 cm in section, and 0.75 to 0.9 m

high, built into the masonry weir wall at about 0.45 to 0.6 m clear interval ; projecting above the weir crest to a height of 0.3 to 0.6 metre, their tops being generally at M.W.L.

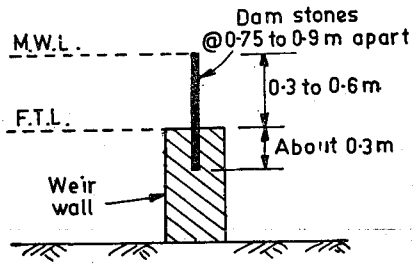


Fig. 26.6. Dam-stones.

The space between these stones are blocked up with clay and turf by cultivators, with the result that water can be held up in the tank up to M.W.L., before any surplusing takes place. When the tank receives more water, which just causes the water level in the tank to rise above M.W.L., the water over-tops the turf weir, which is rapidly washed away, and the surplus weir starts functioning with full head (H), without M.W.L. exceeding to any appreciable extent.

Whenever this occurs, all the water stored in the tank above the weir crest level, must however, be passed off before the turf dam can be rebuilt.

The dam stones cause obstruction to the discharge, and when they are installed, the length of the weir must be correspondingly increased, *i.e.* constant (C) in weir discharge formula reduced from 1.84 to 1.66.

Sometimes, instead of damstones, shutters may be provided between F.T.L. and M.W.L., so that they can be lowered and raised as and when needed, thus serving the purpose of giving full pondage up to M.W.L. before the worst flood arrives for surplus disposal. They may, however, create operational problems, and necessitate special establishment.

If no damstones or shutters are provided above F.T.L., and flood water starts spilling over the surplus work immediately after the tank water level exceeds F.T.L., then *strictly speaking*, the computed peak flood discharge should not be used to directly work out the length of the weir by the above simple and straight formulas. This is because of the fact that certain flood discharge will be absorbed between F.T.L. and M.W.L. This *flood absorption capacity* of a tank is, however, considered only in very big tanks or in large-sized tanks, and this is kept unaccounted as a further measure of safety in such works. A method of computing the allowance which may be made for the absorption capacity of big tanks has been worked out by Capt. Garret, and is given in his paper on the "General Theory of the Storage Capacity and Flood Regulation of Reservoirs" published in 1912 by the Government of India. The same may be referred to if needed along with its original tables given in the same paper.

26.5.4. Choice of a Suitable Site for Locating a Tank Weir. As pointed out earlier, a surplus weir can be located either in the body of the tank bund or at one end of it. When a saddle, disconnected with the tank bund is available, it must be preferred for locating the surplus weir, as compared to providing it within the bund body, but even in such a case, it will be necessary to locate it at one flank of the bund and connected with it.

If all the cultivation is on one side of the stream which is dammed by the bund, then it is preferable to locate the surplus weir on the other flank ; otherwise it would be necessary in many cases, to carry the distributary channels across the surplus channel,

and there may also be danger of cultivation being damaged by the surplus water when the tank surpluses freely.

Besides these considerations, the most favourable sites for surplus weirs are at places ;

(*i*) where the N.S.L. along the line of the weir approximately equals F.T.L.

(*ii*) when the foundation soil is hard both on the weir site and along the runoff channel, and where the ground contours are such as to give clear approach to the weir site and a suitable course for the surplus water.

26.6. Tank Outlets or Tank Sluices and their Design Principles

As stated earlier, a *tank sluice* is an opening in the form of a culvert or a pipe running through or under the tank bund, and supplying water from the tank to the distributary channel below, to meet the irrigation or other water requirements, as and when needed. Suitable wing walls and other bank connections are also provided as required at the head and tail end of the culvert.

In ordinary medium-sized sluices, masonry culverts with or without arch roofs are generally constructed (Refer Fig. 26.7). The size of the culvert (*i.e.* its cross-section) will depend upon the maximum quantity of water (Q)* it is required to convey, but in no case should be less than 0.6 metre wide and 0.75 metre high, so as to allow a man to enter it for examination and repairs or removal of obstructions. The size of the barrel should also be such as to limit the velocity through the sluice barrel to a maximum of 4.5 m/sec*, under the condition of plug hole being fully open and with the water at full tank level.

In case of very small sluices, earthen ware or cement or cast iron pipes may be used to take the place of masonry culverts. (Refer Fig. 26.8). Such sluices are called *pipe sluices*. They are generally not adopted in tank bunds, where the depth below F.T.L. exceeds 2.5 metre or so. This is because, in such cases, the earthen ware pipes may get fractured, or leakage through their joints may take place, resulting in a breach, as the pipes can neither be examined nor repaired easily without cutting open the bund. Moreover, these pipes and especially the cast iron pipes are rarely found to be economical compared to masonry sluices.

Besides the above types of tank sluices, one more type of sluice *i.e. a sluice with a tower head* (Refer Fig. 26.9) is also sometimes provided. This form is sometimes found more economical than the types previously referred to, owing to the saving obtained by avoiding the heavy wing walls of the previous type. Moreover, here there is lesser danger of failure by cracking and bulging which frequently occurs in the wings of the other type. But, on the other hand, with such a sluice, additional length of the barrel is required. Tower-heads or wells are generally placed in the water slope of the bund, as thereby, the expanse of a bridge or causeway leading from the bank to the regulation

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$$^{*}V = \frac{Q}{A} = \frac{\text{Discharge}}{\text{Area of barrel}} = Cd \cdot \sqrt{2g \cdot h}$$

where h is the height of the F.T.L. above the sill level of the sluice.

Cd = 0.63 for free fall.

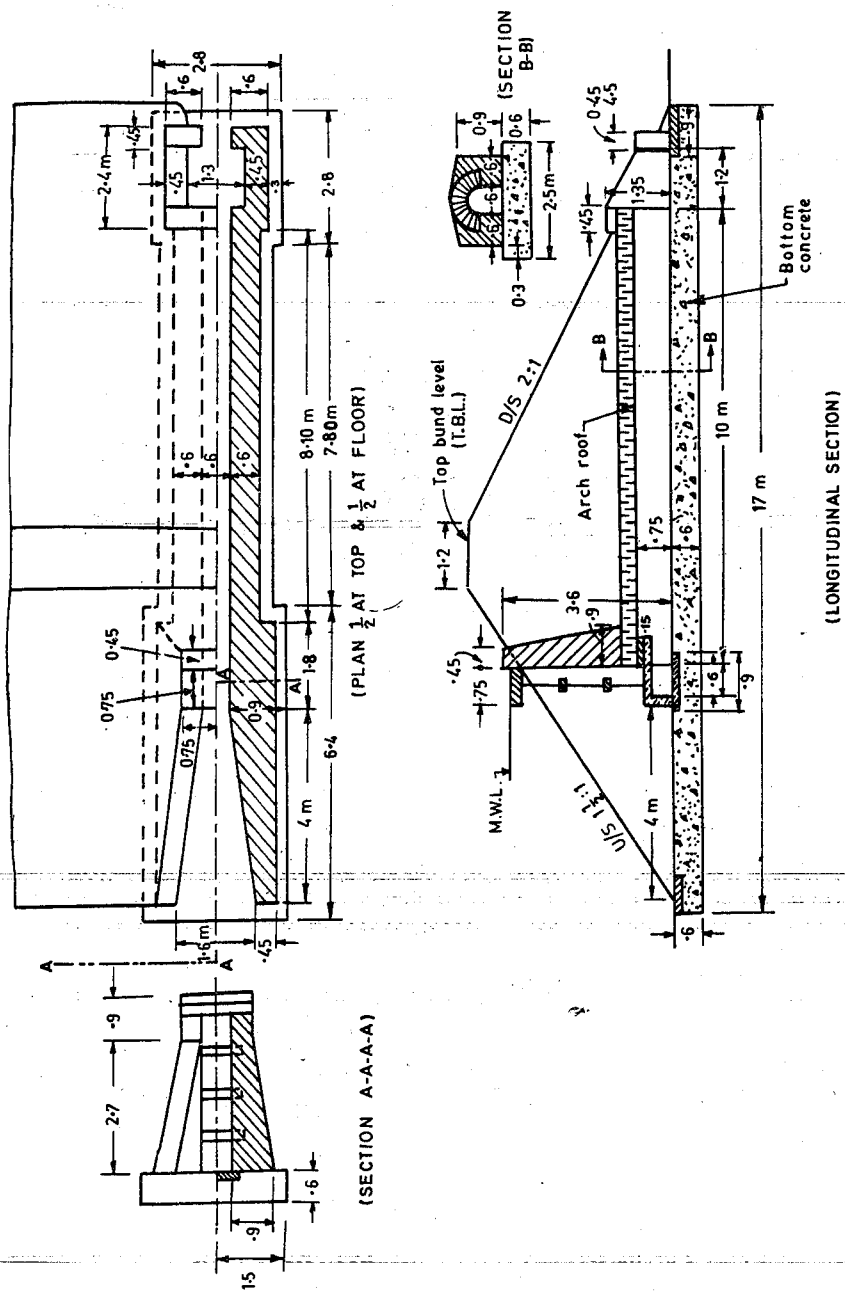

Fig. 26.7. Masonry supply sluice for a medium-sized tank.

platform is saved. But this has the disadvantage of making it impossible, while the tank has water, to have access to the portion of the barrel upstream of the well.

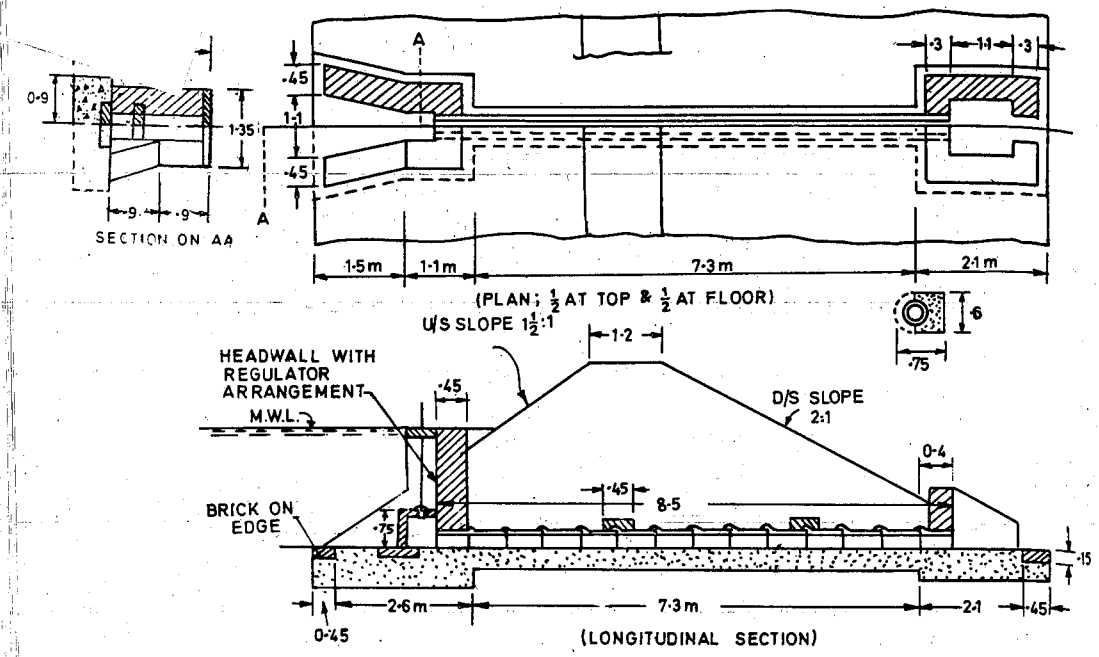

Fig. 26.8. Earthern ware pipe-sluice for a very small tank.

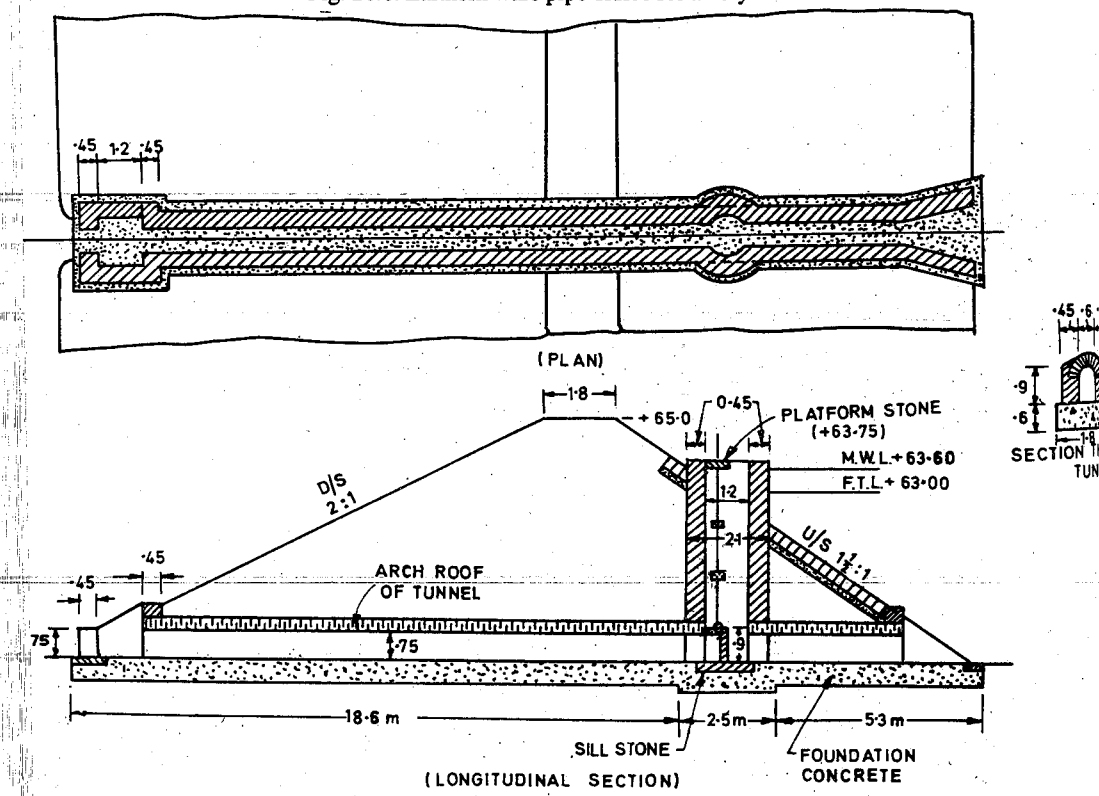

Fig. 26.9. Tank sluice with a tower-head.

26.6.1. Method of Regulation of Tank Sluices. The generally used vents for medium-sized tank sluices consist of one or more circular orifices of suitable size made in a horizontal stone, and the area of the opening is regulated by a long coned plug, the wider end of which is of the same diameter (allowing for clearance) as the vent (Refer Fig. 26.10). The vent is termed as the *plug-hole*. As the plug is raised, the hole or orifice is opened more and more, and when the plug is raised altogether clear of the plug hole by an amount not less than the diameter of the hole, the full discharge from the orifice will pass. This arrangement is both simple and economical, and affords an easily adjustable means of regulation to suit the varying conditions of water level which are inseparable from the reservoirs, and this arrangement is, thus, more suitable for tank sluices than the flat shutters which are generally used for canal sluices.

Details of plugs for tank-sluices are shown in Fig. 26.10, the standard sizes being 100, 150, 200, 250 and 300 mm dia, this being the size of the hole in the plug stone, the largest size of the cone part of the plug being about 6 mm less ; the width of the

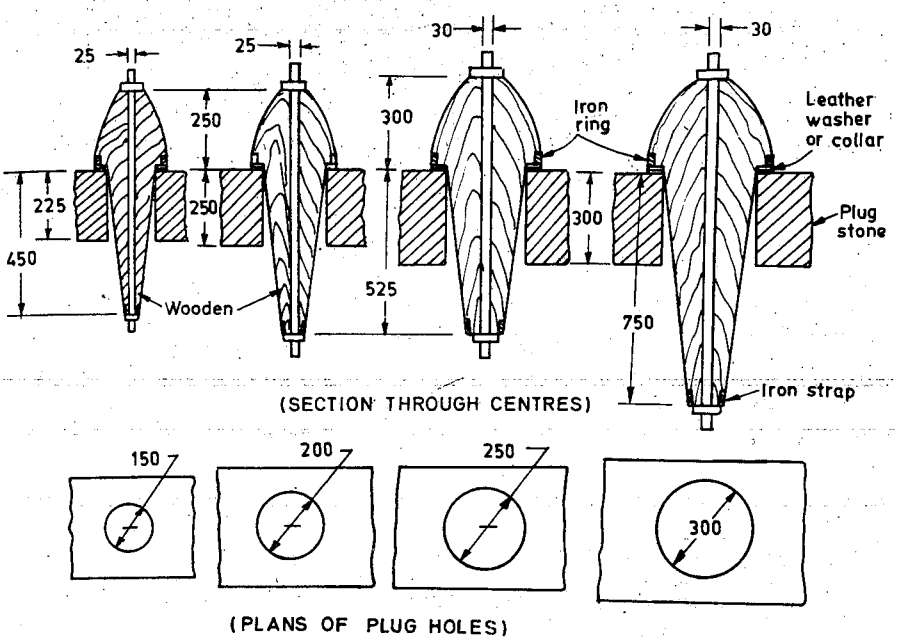

Fig. 26.10. Typical standard sized plugs for tank sluices.

seating should be from 25 mm to 30 mm. The cones should have a taper of one to four, and be made of hard wood ; the smaller end of cone and the greatest dia. of the plug should be cut cylindrical to take countersunk iron straps which should be shrunk on to prevent the plug from splitting. The lifting rod should be round iron, and secured by a split cotter pin at each end of the plug. When the rod is long, it should be passed through holes in guide beams fixed at intervals. The top end of the rod passes through a hole in the regulation platform ; it should be flat iron with holes at about 75 mm interval through which an iron pin is passed to keep the plug suspended at whatever height it gives suitable elevation.

When the computed size of a tank sluice or plug hole is more than 250 to 300 mm, in diameter, then it is generally advisable to provide two or more holes and plugs in the plug platform stone ; while for deep sluices, say over 7.5 m depth, two or more plug stones at different levels, each with separated plug holes, provide greater facility for easy regulation, and hence adopted.

The size of the orifices in the plug stone are generally calculated so as to pass the full supply with vents fully open and with 0.3 m of water standing over the plug stone platform. For regulation below this level, a rectangular vertical vent is provided which is closed by a shutter, which may be wooden but is generally a slab of stone. This vent is made of sufficient size to pass full supply with 0.15 m head. This shutter is brought into use only when the water in the tank is less than 1.2 m over the floor of the sluice; and at other times, the shutter is kept completely covering the orifice, and leakage past it, should be stopped by filling silt against its upstream face. This vent will be regulated by men standing on the plug stone.

In case of large tanks, where the quantity of water to be released is great, and where adequate establishment is maintained, flat shutters working in grooves, and regulated by screw spears, may be installed.

For heads of over 9 m or so, special balanced valves or shutters moving on rollers are generally installed.

Example 26.1. *Design a vertical drop horizontal floor tank surplus weir with the following data. Draw a neat sketch indicating the salient dimensions of the weir and bund section, and work out the length of the weir required with the following given data:*

Combined catchment	= *26 sq. km.*
Intercepted catchment	= *20 sq. km.*
Maximum water level	= *+ 7.50 (m)*
Full tank level	= *+ 6.70*
Ground level	= *+ 5.80*
Tank bund level	= *+ 1.80*
Slope on either side of bund	= *2 : 1*
Ryve's coefficient for combined catchment	= *9.0*
Ryve's coefficient intercepted	= *1.8*

Solution. Using equation (26.1), we have

$$Q_P = \text{peak flood discharge}$$

$$= C_1 A^{2/3} - c_1 a^{2/3}$$

where $C_1 = 9.0$, $c_1 = 1.8$

$A = 26$ sq. km., $a = 20$ sq. km.

$$Q = 9.0\,(26)^{2/3} - 1.8\,(20)^{2/3} = 79.2 - 13.3 = 65.9 \text{ cumecs.}$$

or $Q_p = 65.9$ cumecs.

Now, M.W.L. = 7.50 m

F.T.L. = 6.70 m

Ground level = 5.80 m

∴ Difference between MWL & FTL *i.e.* maximum

head of water over surplus weir = $H = 7.50 - 6.70 = 0.8$ m.
crest of weir is kept at F.T.L.

∴ Depth of drop *i.e.* height of weir crest

$$D = 6.7 - 5.8 = 0.9 \text{ m.}$$

Since the drop is small, we can adopt a masonry weir with a horizontal masonry floor (without depressed floor) *i.e.* type A, Fig. 26.3 (*a*), easily.

The widths of horizontal pucca floor and talus are calculated below :

The width of horizontal masonry floor $= 2\,(D + H) = 2\,(0.9 + 0.8) = 3.4$ m.

The width of rough stone talus $= 4\,(D + H) = 4\,(0.9 + 0.8) = 6.8$ m.

Use the top crest width equal to 0.6 m, and draw the sketch as shown in Fig. 26.11.

The length of the weir can be determined by using equation (26.2) as below :

$$Q_p = C \cdot LH^{3/2}$$

where $Q_p = 65.9$ cumecs (calculated above)
$L = ?$
$H = 0.8$ m.
$C = 1.84$ for weirs with crests upto 0.9 m wide.

∴ $$65.9 = 1.84 \times L \times (0.8)^{3/2}$$

or $$L = \frac{65.9}{1.84 \times 0.716} \text{ m} = \mathbf{50\ m.}$$ **Ans.**

Example 26.2. *A waste weir of a tank has the following details :*

Combined catchment	= *26 sq. km.*
Intercepted catchment	= *20 sq. km.*
F.T.L. of tank	= *+ 12.70 m.*
Crest level of tank	= *+ 12.70*
M.W.L. of tank	= *13.4*
G.L.	= *10.00*
Ryve's coefficient for combined catchment	= *9*
Ryve's coefficient intercepted	= *1.5.*

Design the length and cross-section of the body wall of the surplus work, assuming the foundation soil to be soft but only slightly pervious.

Solution. Depth of water over weir crest

$$= H = MWL - FTL = 13.4 - 12.7 = 0.7 \text{ metre}$$

The Depth of fall (D) = F.T.L. – G.L. = 12.7 – 10.0 = 2.7 m.

The choice is obviously a vertical drop fall with a water cistern.

Total length of horizontal floor

$$= 3\,(D + H) = 3\,(2.7 + 0.7) = 3 \times 3.4 = \mathbf{10.2\ m.}$$

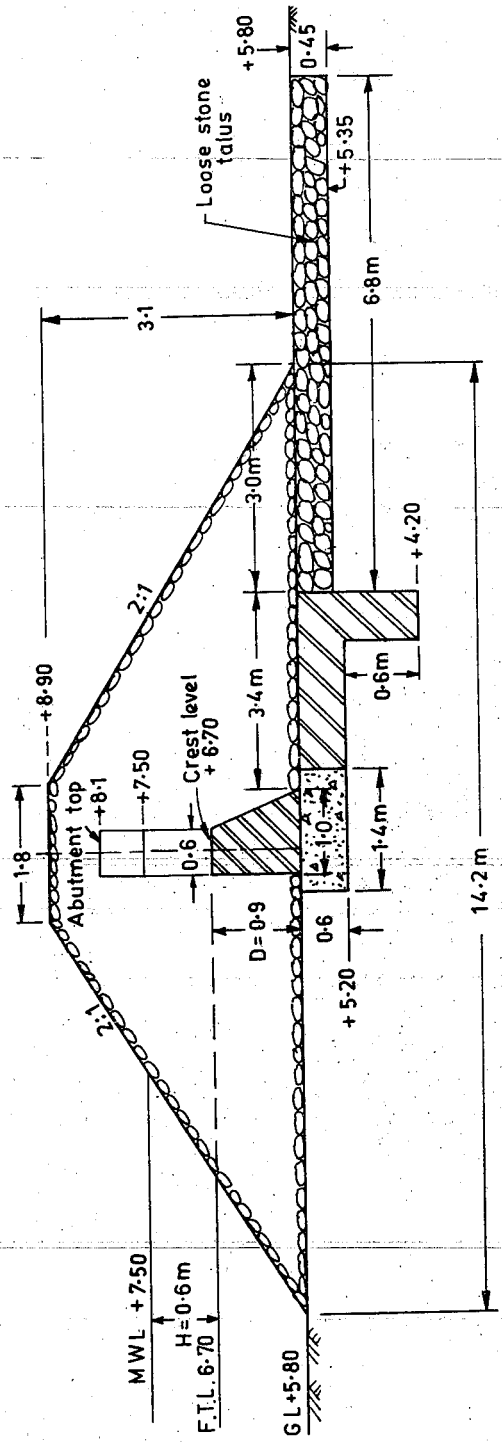

Fig. 26.11. Section of the designed Tank Surplus Weir in example 26.1.

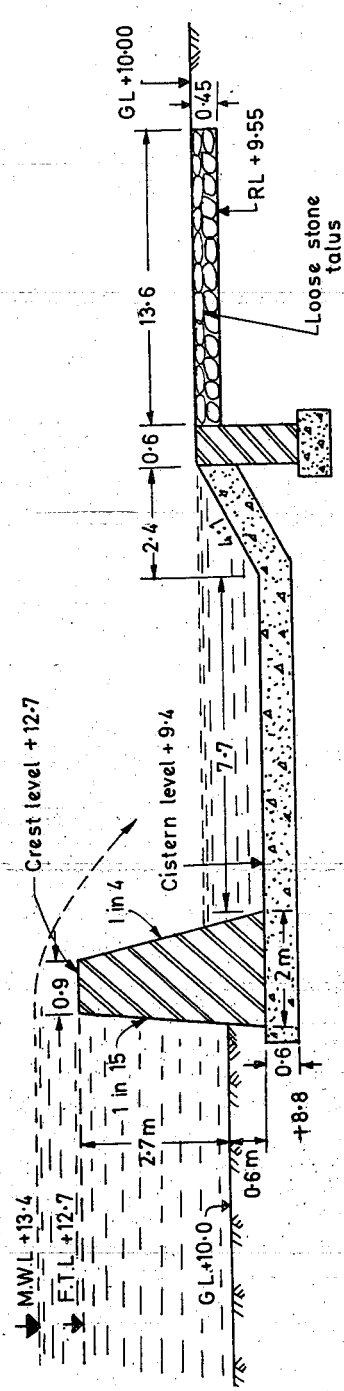

Fig. 26.12. Section of the designed Tank Weir in example 26.2.

Length of d/s talus = 4 $(D+H) = 4\,(2.7+0.7) = 4 \times 3.4 =$ **13.6 m.**

The depth to which the horizontal floor is depressed is given by equation (12.9) as $X = \frac{1}{4}(H \cdot H_L)^{2/3}$, where H_L is the difference in u/s and d/s water levels *i.e.* $(H+D)$ in this case.

or $$X = \tfrac{1}{4}(0.7 \times 3.4)^{2/3}$$

$= 0.5$; So let us use 0.6 m depth of cistern

Length of cistern is given by equation (12.8) as

$$L_C = 5 \cdot \sqrt{H \cdot H_L} = 5 \cdot \sqrt{0.7 \times 3.4} = 7.7 \text{ m.}$$

The slope length of concrete floor in a slope of 4 : 1 will be $= 4 \times 0.6 = 2.4$ m.

d/s cut-off wall thickness $= 0.6$ m

In this way, total length of floor provided

$= 7.7 + 2.4 + 0.6 = 10.7 > 10.1$; safe.

Hence, provide the section of the waste weir, as shown in Fig. 26.12.

Length of weir

$$Q_p = C_1 A^{2/3} - c_1\, a^{2/3}$$

where $C_1 = 9.0$

$c_1 = 1.5$

$A = 36$ sq. km.

$a = 20$ sq. km.

$$Q_p = 9\,(36)^{2/3} - 1.5\,(20)^{2/3}$$

$$= 9 \times 10.9 - 1.5 \times 7.4 = 98.1 - 11.1 = \mathbf{87\ cumecs}$$

Now $$Q_p = CLH^{3/2}$$

$$87 = 1.84 \times L \cdot (0.7)^{3/2}$$

$$\therefore \quad L = \frac{87}{1.84 \times 0.948} = 49.8 \text{ m ; } \textbf{say 50 m}$$

Hence, use 50 m length of weir. **Ans.**

Example 26.3. *A tank has a catchment area of 120 sq. km. ; out of which 20 sq. km. make free catchment. Average annual rainfall on catchment is 75 cm. What should be the desirable gross storage for the tank ? If rice crop, with a duty of 640 hectares per cumec at the head of main canal and a crop period of 120 days, is to be irrigated from the water of this tank, find the area under the rice crop.*

What should be the length of the waste weir to dispose of safely the high flood discharge, with a head of 0.6 m on the crest of the weir ? Use Ryve's formula, with coefficient as 7.5.

How will the live storage be affected if the tank gets filled $1\frac{1}{2}$ times in a year.

Solution. Average annual rainfall = 75 cm.

The rainfall of average bad year = $\frac{2}{3}$rd of the average annual rainfall(say).

$= \frac{2}{3} \times 75 = 50$ cm.

Runoff (as depth of water over the catchment) of this average bad year

$$= 20\% \text{ (say) of rainfall} = \frac{20}{100} \times 50 = 10 \text{ cm.}$$

The total catchment area of 120 sq. km. consists of independent catchment of 20 sq. km. and intercepted catchment of 100 sq. km.

Total volume of runoff in average bad year from free as well as intercepted catchment of the given tank

= full runoff from the free catchment + $\frac{1}{7}$th of the runoff from the intercepted catchment on the assumption that $\frac{6}{7}$ of the runoff from the intercepted catchment is impounded in the upper tanks.

$$= \left[20\,(1000 \times 1000) \times \left(\frac{10}{100}\right) + \frac{1}{7} \times 100\,(1000 \times 1000)\,\frac{10}{100}\right] \text{m}^3.$$

$$= \left[\frac{20 \times 10^6}{10} + \frac{1}{7} \times 100 \times 10^6 \times \frac{1}{10}\right] \text{m}^3$$

$$= 10^6\,[2 + 1.43] = 3.43 \times 10^6\ \text{m}^3 = 3.43 \text{ million cubic metre.}$$

$\therefore$ Live storage $= 1.1 \times 3.43 = 3.77$ M.m^3

and, Gross storage $= \left(\frac{10}{9}\right) 3.77 =$ **4.19 M.m^3** **Ans.**

Water required at the head of the main canal to irrigate 640 hectares of rice crop

$= 1$ cumec for 120 days. $= 1 \times (120 \times 24 \times 60 \times 60)$ cu.m.

$= 10.37 \times 10^6$ cu.m. $= 10.37$ M.m^3

Hence, the area of rice crop which can be irrigated from the full tank water

$$= \frac{\text{Effective storage of the tank}}{10.37} \times 640 \text{ hectares}$$

$$= \frac{0.9 \times \text{Live storage}}{10.37} \times 640 \text{ hectares}$$

(assuming the losses in the tank to be 10% of live storage).

$$= \frac{0.9 \times 3.77}{10.37} \times 640 = \textbf{209.4 hectares.} \quad \textbf{Ans.}$$

Now, the high flood discharge from the catchment into this tank (after the tank is full) is given by equation (26.2) as

$$Q_p = C_1 A^{2/3} - c_1 a^{2/3},$$

where $C_1 = 7.5$ (given)

$c_1 = \frac{1}{5} C_1$ (assumed) $= 1.5$

$A =$ Full combined catchment $= 120$ sp. km. (given)

$a =$ Intercepted catchment $= 100$ sq. km. (given)

$$\therefore \quad Q_p = 7.5\,(120)^{2/3} - 1.5\,(100)^{2/3} = 183 - 32.4 = 150.6 \text{ cumecs.}$$

Now, if L is the length of the waste weir, then

$$Q_p = C \cdot L \cdot H^{3/2} = 1.84 \cdot L \cdot H^{3/2}$$

$\therefore \quad 150.6 = 1.84 \times L \times (0.6)^{3/2}$

or $\quad L = \mathbf{176\ m.}$ **Ans.**

If the tank fills $1\frac{1}{2}$ times in a year, then the live storage

$$= \frac{3.77}{1.5} = \mathbf{2.51\ M.m^3} \quad \textbf{Ans.}$$

PROBLEMS

1. What is an irrigation tanks ? Differentiate between isolated tanks and group of tanks. how can your compute the storage capacity of an irrigation tank ?

2. (*a*) What are the causes of failure of tank bunds ?

[**Hint.** Refer article 20.7]

(*b*) Sketch a typical cross-section of a tank bund of type C. Under what circumstances such sections are chosen ?

(*c*) Explain briefly the function of a breaching section in a tank bund.

3. Design a tank surplus weir with the following data. Draw a neat sketch (not to scale) indicating the salient dimensions.

Combined catchment	= 26 sq. km
Intercepted catchment	= 20 sq. km.
Full tank level	= + 6.70 (m)
Maximum water level	= + 7.50 m
Ground level	= + 5.80 m
Foundation level	= + 4.20 m
Tank bund level	= + 8.90 m
Top width of bund	= 1.8 m
Slope on either side of bund	= 2 : 1

Downstream level is + 4.80 in a distance of 8 m.

4. A tank surplus work has the following details :

Combined catchment	= 26 sq. km.
Intercepted catchment	= 20 sq. km.
F.T.L. of tank	= 12.00 m.
Crest level of weir	= 12.00 m.
M.W.L. of tank	= 12.75 m.
G.L.	= 11.00 m.
Top of foundation concrete	= 9.60 m
Ryve's coefficient for combined catchment	= 9
Ryve's coefficient intercepted	= 1.5

Design the length and cross-section of the body wall of the surplus work.

5. What is a waste weir, and where is it installed in a project of an irrigation tank ? Discuss its importance, and principles involved in designing such a work.

6. (*a*) What is a tank weir, and what are its types ? Discuss the conditions favourable to the selection of a particular type.

(*b*) How does the foundation and working conditions differ for a tank weir as against a river weir.

(*c*) Where should a tank weir be located in a tank irrigation project ?

7. (*a*) What are dam-stones and where are they installed ? What are their advantages and disadvantages ?

8. What is a surplus weir and its types in an irrigation tank project ? Draw net sketches of the sections of A type weirs with and without water cushion provision.

9. (*a*) What do you mean by a tank sluice, and where is it installed ?

(*b*) Describe the different types of tank sluices indicating the regulation arrangements.

(*c*) For the data given below, design and sketch a tank sluice :

Discharge	= 0.25 cumec
Sill level of sluice	= + 20.00 m.
Top of bank	= + 56.00 m.
Assumed head of discharge	= 0.3 m.
Side slopes of banks in front and rear	= $1\frac{1}{2}$: 1

Assume any other data, if required.

10. Write short notes on :

(*i*) Free catchment and combined catchment (*ii*) Isolated tanks and tanks in series :

(*iii*) Tank bunds (*iv*) Tank sluices ; and

(*v*) Tank weirs.

27

Arch and Buttress Dams

ARCH DAMS

27.1. Definition and Types of Arch Dams

An arch dam may be defined as a solid wall, *curved in plan*, standing across the entire width of the river valley, in a single span*. This dam body is usually made of cement concrete. although rubble and stone masonry has also been used in the past.

This wall will structurally behave : partly as a *cantilever retaining wall standing up from its base, and partly, the load will be transferred to the two ends of the arch span by horizontal arch action*. The arch load will, thus, be transferred to the side walls of the canyon, which must be strong, stable and rocky.

The distribution of part of the load to the side walls of the canyon, reduces the load on the cantilever wall, thereby reducing its thickness, as compared to that in an ordinary gravity dam ; and that is the only benefit we derive from an arch dam in comparison to a gravity dam.

Evidently, the greater is the wall curvature (in plan), the greater will be the load that will be transferred to the sides of the canyon, and hence greater will be the economy in the dam thickness.

This economy in dam thickness can be further increased considerably by making the dam body not only curved in plan, but also curved in section. Such a non vertical dam is known as **double curvature arch dam** or a **shell-arch dam**, because such dams are designed as *shell-structures*. Such three dimensional designs are quite complex, and are taught only at M-Tech level in Structural Engineering.

Since the design and construction of an arch dam is very complicate, requiring extraordinary skill for erecting shuttering in the field, it is generally preferred in practical life to construct gravity dams. And that is why, we find only one arch dam in our country**, as against several hundreds of gravity dams. This arch dam too, is not a simple arch dam, but a shell-arch dam.

Simple arch dams, which transfer a large part of their loading by cantilever action, may also be of different types, since their faces may be either vertical or curvilinear. Depending upon the shape consideration, simple arch dams can be divided into three types, *viz :*

(*i*) *Constant radius arch dams*

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* When multiple or a number of arches are used, supported between intermediate piers, the dam is known as a *buttress dam.*

** *Idukki dam,* across Periyar river in Kerala State ; Photoview shown in Fig. 27.1.

(*ii*) *Variable radius arch dams ;* and

(*iii*) *Constant angle arch dams.*

A constant radius arch dam is the simplest in design as well as construction, but uses the maximum concrete. A constant angle arch dam on the other hand, uses about 43% of the concrete used by a *constant radius arch dam.* The variable radius arch dam is an intermediate choice, using around 58% of the concrete used by *constant radius arch dam.*

The *shell arch dams* are much more economical than even the constant angle arch dams, as their sections can be quite thin. Say for example, the famous *Vajont dam* of Italy is only 22 m thick at its base, inspite of being 261.6 m in height. Similarly, the *ldduki dam* in India is only 45 m thick at its base, even with 170.7 m height. As compared to these thin sections, the famous *Hoover dam* of USA, which is a *constant radius arch dam (with upstream face vertical) is 201 m thick at its base, with only 222 m height. The difference between the three types of arch dams are explained below :*

27.1.1. Constant Radius Arch Dams. (Fig. 27.2). A constant radius arch dam is that, in which, the radii of the *outside** curved surface are equal at all elevations, from top to the bottom. The centres of all such circular arcs, called **extrodos**, will therefore, evidently lie on one vertical line. However, the **introdos** (*i.e.* inside** curved surface of the arch) has gradually decreasing radius from top to the bottom, so as provide increased concrete thickness towards the base for accounting the proportionally increasing hydrostatic water pressure of the reservoir. The dam body will, therefore, be triangular in cross-section with upstream face vertical, and a minimum thickness at the top.

Evidently, it is only the radii of the *introdos*, which decrease with depth ; while the centres of all such circular arcs continue to lie on the same vertical line, on which lie the centres of the *extrodos.* Hence, in such a dam, the centres of extrodos, introdos, as well as the centrelines of the horizontal arch rings. at various elevations, lie on a straight vertical line that passes through the centre of the horizontal arch ring at the crest. Such a dam is, therefore, sometimes called a **constant centre arch dam**, although strictly speaking, this centre is not at one point, but lies at different heights along one vertical line.

Evidently, the central angles of the arch rings of the introdos will vary at different elevations, due to the varying width of the river valley (see Fig. 27.2); the maximum being at the top of the dam, and the minimum at the bottom of the dam.

It has further been shown that the *best* or *most economical central angle* in an arch dam is the one whose value is equal to 133° – 34′***. But in a constant radius arch dam, such an angle value can be adopted only at one place, since the angle varies with height considerably, due to narrow V-shape of the valley. It is therefore considered prudent to keep the economical angle of 133° – 34′ at about mid height. The angle at the top will, therefore, be more than this value; but due to topographical considerations, the angle at the top can not be fixed greater than about 150°. *Hence, the angle at the top should be such as to give the best average angle, but restricted to about 150°.*

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* Water side or upstream side.

** Downstream side.

*** This is derived in article 27.3.1.1, a little later.

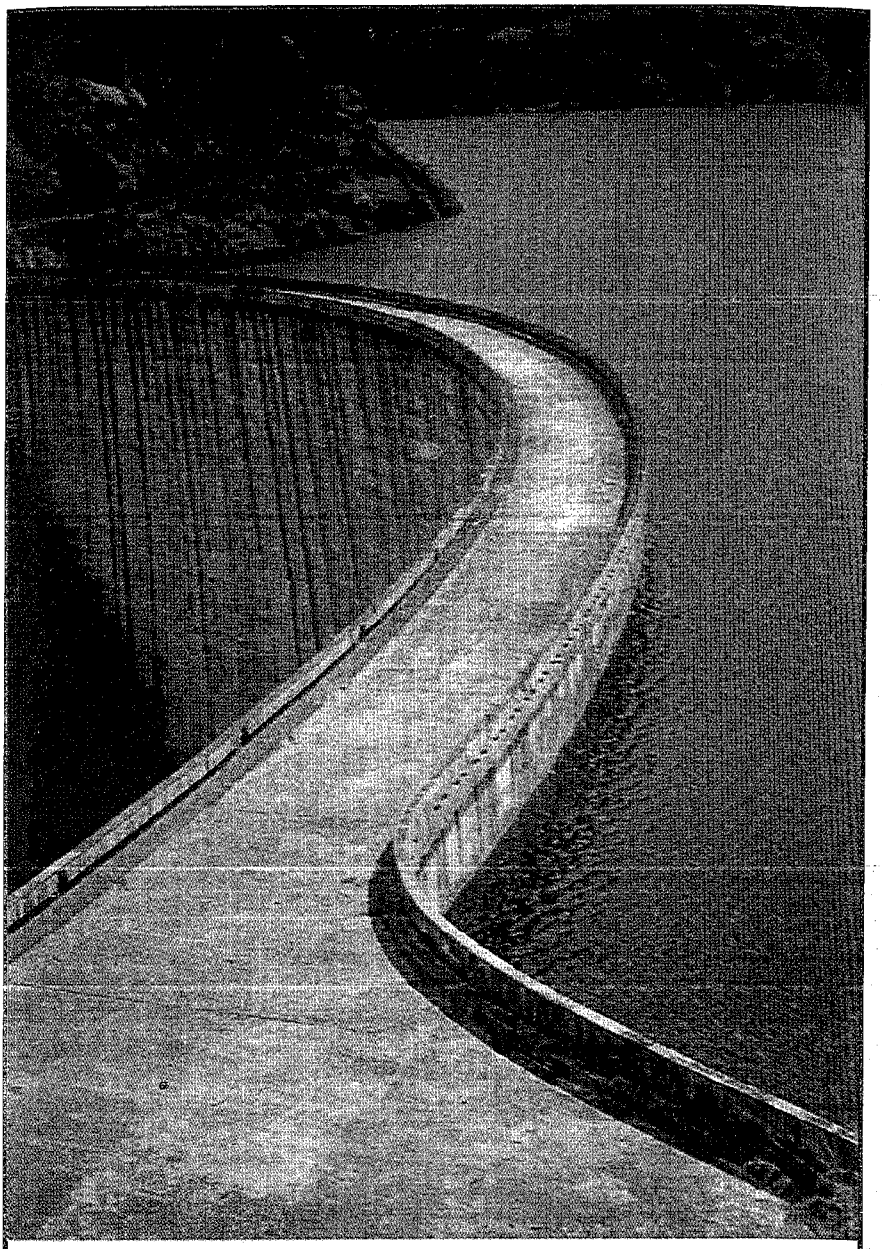


Fig. 27.1. Photoview of Idukki Dam.
A concrete arch dam, constructed across Pariyar river in Kerala State of India. The length of the dam at top is 366 m and maximum height above the lowest point of foundation is 169 m. The construction of the dam was completed during 1966-1974.

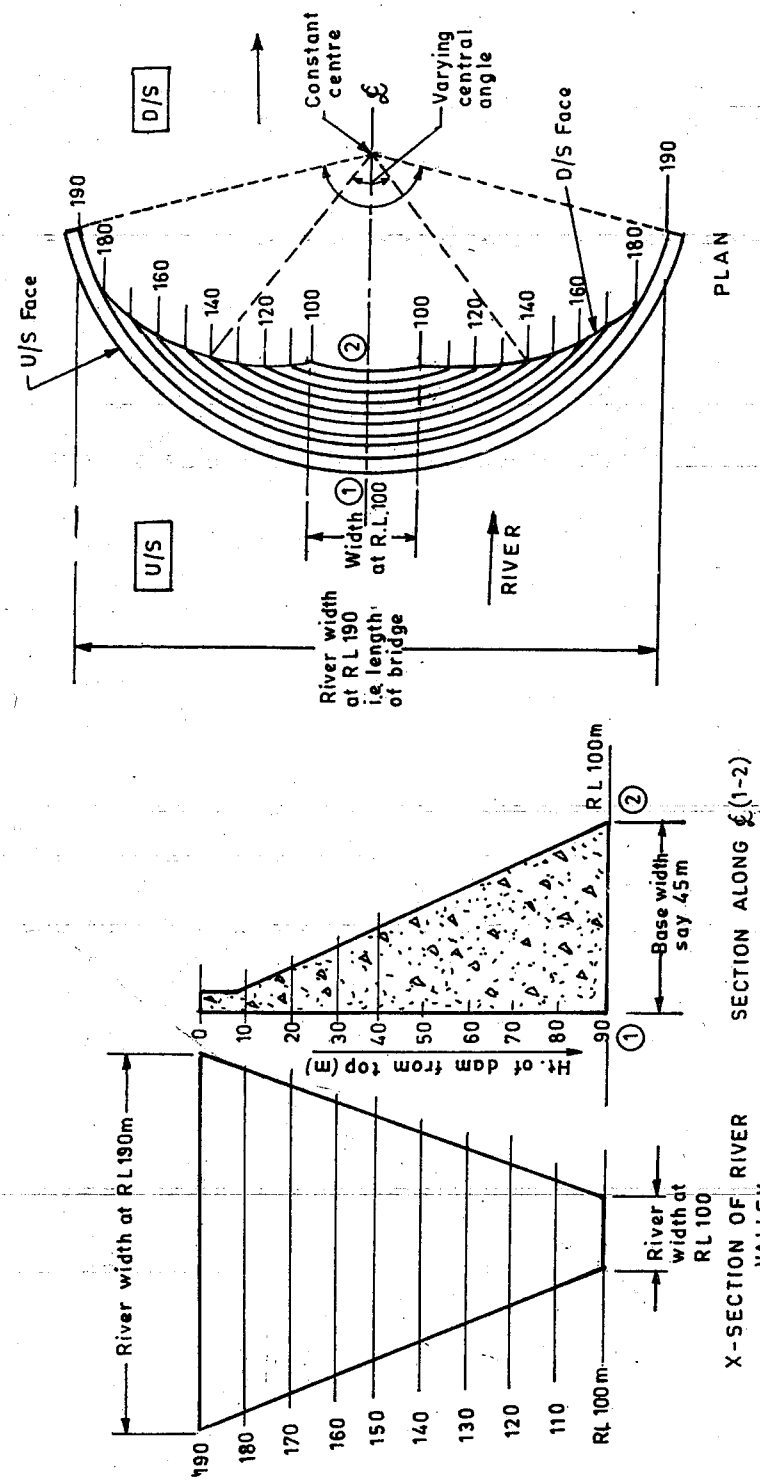


Fig. 27.2. Constant radius arch dam.

27.1.2. Variable Radius Arch Dams. (Fig. 27.3). A variable radius arch dam is the one in which the radii of the extrodos curves* and of *introdos curves*** vary at various

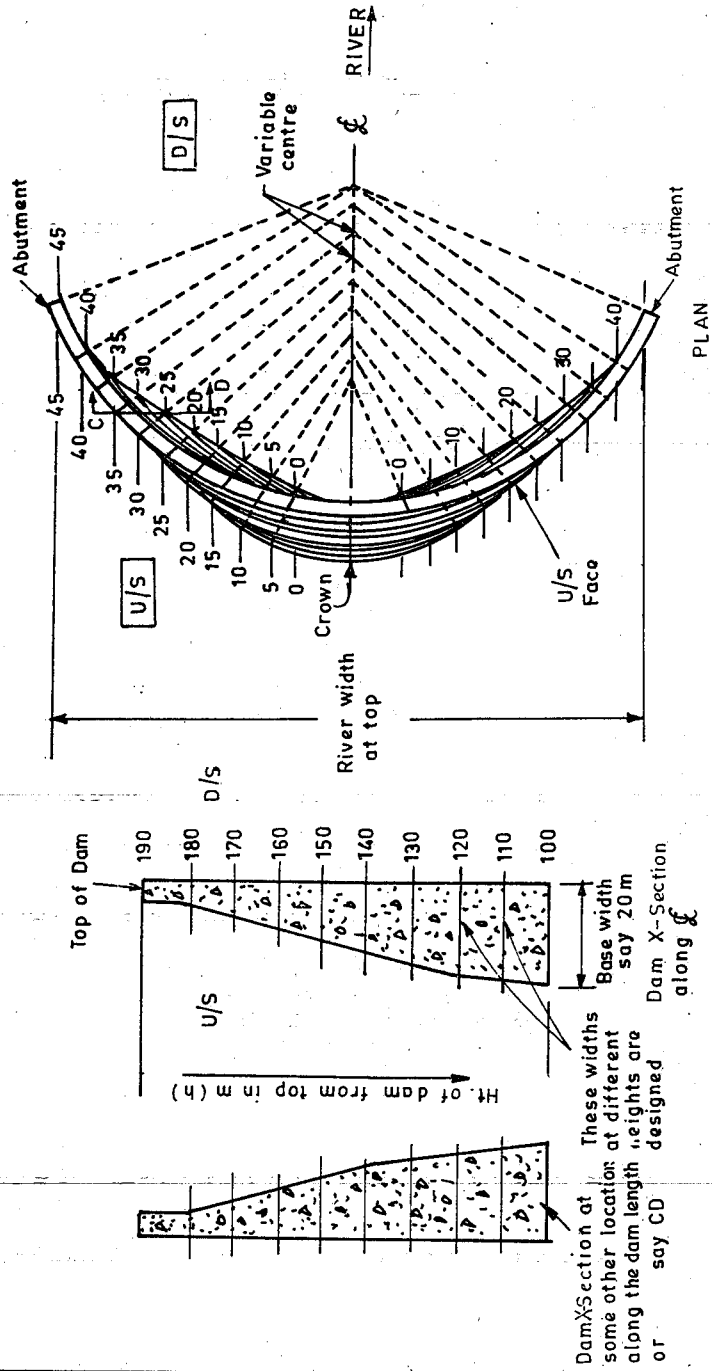


Fig. 27.3. Variable radius arch dam.

* arch rings corresponding to upsteam face.

** arch rings corresponding to downstream face.

elevations, being maximum at the top, and a certain minimum at its bottom. This makes the central angles as large as possible, so that the maximum arch efficiency may be obtained at all elevations.

In a typical design of such a dam, the downstream face of the dam at the central line (crown) is vertical ; while at all other locations, there is a batter on both the sides except at the abutments, where again, the upstream side becomes vertical. If overhangs are permitted, due to availability of stronger foundations, then the faces at the crown as well as abutments, may be provided with overhangs, affecting saving in the designed thicknesses.

Evidently, since in such an arch dam, the centres of the various arch rings at different elevations, do not lie on the same vertical line, it is also known as **variable centre arch dam.** Such dams are preferred for V-shaped valleys as compared to constant radius arch dams which may be preferred for comparatively wider U-shaped valleys.

27.1.3. Constant Angle Arch Dams. The constant angle arch dam is a special type of variable radius arch dam, in which the central angles of the horizontal arch rings are of the same magnitude at all elevations, as shown in Fig. 27.4. The design of such a dam can, thus, be made by adopting the best central angle of 133° – 34′ ; and hence such a dam proves to be the most economical, out of the three types of ordinary arch dams, as pointed out earlier also.

However, the design of such a dam usually involves providing *overhangs at abutments*, which require stronger foundations, and hence such a type cannot be used if the foundations are weak.

27.2. Forces Acting on Arch Dams

Generally, the same forces act on an arch dam, which do act on a gravity dam. These forces are : (*i*) *water pressure* ; (*ii*) *uplift pressure* ; (*iii*) *earthquake forces* ; (*iv*) *silt pressure* ; (*v*) *wave pressure* (*vi*) *ice pressure* ; as discussed in article 19.3. However, the relative importance of the forces is different in an arch dam, as compared to that in a gravity dam. Say for example, the uplift pressure in an arch dam is small and is generally neglected, because of the narrow base width of its body. On the other hand, the stresses caused by *ice, temperature changes*, and *yield of supports (i.e. abutments)*, generally become quite important in arch dams, and hence must be thoroughly examined.

Whereas, the ice pressure, applicable in cold countries, causes a continuous concentrated load along the arch element at the elevation of the ice ; the internal stresses caused by the temperature changes move the dam upstream during the summer, and downstream during the winter. Hence, the **low water temperatures** become quite important in stress analysis, since these stresses act additive to the reservoir water pressure. Moreover, even the slight **yield of abutments** due to transfer of load by arch action, may also cause high internal stresses in the arch, and should therefore, be prudently accounted for.

27.3. Design of Arch Dams

Arch dams can be designed on the basis of any one of the following three methods:

(1) *Thin cylinder theory* ;

(2) *Theory of Elastic arches ; and*

(3) *The Trial load method.*

radius r, angle subtended at the centre of the arch equal to 2α, and thickness t. Intensity of *hydrostatic pressure* p at any depth h, is given by :

$$p = \gamma_w h \qquad ...(i)$$

where γ_w = unit weight of water.

This horizontal pressure acts in the radial direction. The total downstream component of the horizontal force (P_H), acting along the axis of the river, is then given by

$$P_H = \text{Intensity of pressure} \times \text{Projected area} = \gamma_w \cdot h \times 2r \sin\alpha$$

$$= 2\gamma_w h r \sin\alpha \qquad ...(ii)$$

If R is the reaction at each abutment, its upward component is equal to $2R \sin\alpha$. Equating this to P_H for equilibrium, we get

$$2\gamma_w \cdot h \cdot r \sin\alpha = 2R \sin\alpha$$

or

$$\gamma_w \cdot h\, r = R \qquad ...(iii)$$

This equation gives an expression for the abutment reaction, which is equal to the maximum compressive force induced in the arch.

If σ is the compressive stress induced in the arch at the abutments, then

$$\sigma = \frac{R}{t} = \frac{\gamma_w \cdot h \cdot r}{t} \qquad ...(iv)$$

If f_c is the allowable compressive stress for the arch material, then

$$f_c \gg \frac{\gamma_w \cdot h \cdot r}{t}; \text{ or } t \gg \frac{\gamma_w \cdot h \cdot r}{f_c}$$

Under Limits, $$\boxed{t = \frac{\gamma_w \cdot h \cdot r}{f_c}} \qquad ...(27.1)$$

The above equation indicates that the thickness of the arch (t) should increase linearly with the depth below the water surface, and that at a given elevation, the required thickness of the arch ring is proportional to the radius of the arch (r) at that elevation.

27.3.1.1. Central angle for minimum concrete. Considering the arch ring as shown in Fig. 27.5, we have

the volume of concrete per unit height of the arch ring (V)

$$= r(2\alpha) \cdot A$$

where A is the cross-sectional area of the arch ring $= (t \times 1)$

Since $A = t \times 1$, we have

$$V = r(2\alpha) \cdot t$$

Substituting the value of t from Eq. (27.1), we get

$$V = r \cdot (2\alpha) \cdot \frac{\gamma_w \cdot h \cdot r}{f_c}$$

or

$$V = \frac{\gamma_w \cdot h}{f_c} \cdot r^2 \cdot (2\alpha) \qquad ...(i)$$

Now, if L is the span of the arch ring, then from Fig. 27.5, we observe that

$$\sin\alpha = \frac{L}{2r}$$

or $$r = \frac{L}{2\sin\alpha} \quad ...(ii)$$

Substituting this in Eq. (*i*) above, we get

$$V = \frac{\gamma_w \cdot h}{f_c}\left(\frac{L}{2\sin\alpha}\right)^2 (2\alpha)$$

$$= \frac{\gamma_w \cdot h}{f_c} \cdot \frac{L^2}{2}\left[\frac{\alpha}{\sin^2\alpha}\right] \quad ...(iii)$$

where $\frac{\gamma_w \cdot h}{f_c} \cdot \frac{L^2}{2}$ is a constant $= K$ (say)

$$\therefore \quad V = K \cdot \left[\frac{\alpha}{\sin^2\alpha}\right]$$

or $$\frac{\partial V}{\partial \alpha} = K\left[\frac{\sin^2\alpha - 2\alpha \cdot \sin\alpha\cos\alpha}{\sin^4\alpha}\right]$$

For minimum volume of concrete (V)

$$\frac{\partial V}{\partial \alpha} = 0$$

$\therefore$ $\sin^2\alpha = 2\alpha \cdot \sin\alpha \cdot \cos\alpha$

or $\sin\alpha = 2\alpha \cdot \cos\alpha$

or $\frac{\sin\alpha}{\cos\alpha} = 2\alpha$

or $\tan\alpha = 2\alpha$

or $2\alpha = \tan\alpha$

$\therefore$ $2\alpha = 133° - 34'$

This shows that the most economical central angle is $133° - 34'$, as was stated by us earlier.

27.3.1.2. Limitations of thin cylinder theory. The various limitations of this theory are :

(*i*) The arch sections are not thin cylinders. They are also not free at abutments, as assumed in this theory.

(*ii*) The theory does not consider shear and bending stresses in the arch.

(*iii*) The analysis is based on only the hydrostatic water pressure. *Temperature stresses and ice pressures, which are quite important in arch dams, get ignored in this theory.*

(*iv*) Stresses due to yielding of abutments and those due to rib shortening have not been accounted in this theory.

(*v*) plastic flow of concrete and shrinkage in concrete have not been accounted for.

27.3.2. The Elastic Theory for Design of Arch Dams. As pointed out earlier, the archs are subjected to additional forces, such as :

(*i*) *temperature stresses*, due to temperature changes ;

(*ii*) *shrinkage stresses*, due to setting of concrete ;

(*iii*) stresses due to yielding of abutments ; and

(*iv*) stresses due to rib shortening.

The stresses at (*i*) and (*ii*) are quite evident, and need no explanation. It may however, be added that the effect of rise in temperature, as in summer season, produces opposite stresses, whereas, the stresses due to fall in temperature are additive, and hence, only need be accounted. The stress at (*iii*) above is also understood, since no abutment is rigid, and yields to an extent depending upon the nature of the rock. This yield increases the span of the arch, inducing additional stresses. The stress at (*iv*) above, can be explained as below :

In case of a cylinder, the pressure acts from all sides, and the entire ring is compressed so that the diameter of the cylinder decreases by a small amount, but *there* is no deformation. While in the case of an arch, *which is a part of a cylinder*, the ends being fixed, the distance between them (*i.e.* the span) does not change. This restriction of the change in span, causes additional stresses, known as *rib- shortening stresses. In a long thin arch, these stresses are small ; but in the case of a thick arch of small angle, they are appreciable, and need to be accounted for.*

All the above stresses are taken into consideration by the theory of elastic arches.

27.3.3. General formulas. The formulas for elastic arches, usually given in the books on "Theory of Structures" or "Applied Mechanics" are reproduced here for the benefit of students, intending to solve arch dams by elastic theory.

With reference to Fig. 27.7.

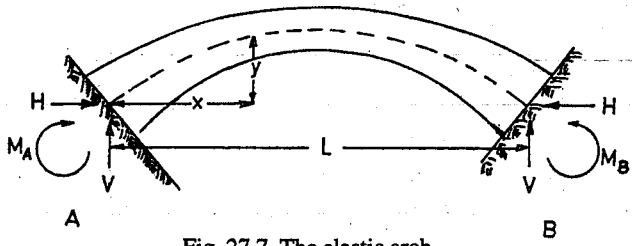


Fig. 27.7. The elastic arch.

$$M = \mu + M_A + (M_B - M_A)\frac{x}{L} + H \cdot y \qquad ...(27.2)$$

$$\int \frac{M}{E \cdot I} y ds = \int \frac{\mu \cdot y}{EI} \cdot ds + M_A \int \frac{y \cdot ds}{EI} + \frac{M_B - M_A}{L} \int \frac{xyds}{EI} + H \int \frac{y^2}{EI} ds = 0 \qquad ...(27.3)$$

$$\int \frac{Mx}{EI} ds = \int \frac{\mu \cdot x}{EI} \cdot ds + M_A \int \frac{x \cdot ds}{EI} + \frac{M_B - M_A}{L} \int \frac{x^2 \cdot ds}{EI} + H \int \frac{xy}{EI} ds = 0 \qquad ...(27.4)$$

and $$\int \frac{M}{E \cdot I} ds = \int \frac{\mu}{EI} \cdot ds + M_A \int \frac{ds}{EI} + \frac{M_B - M_A}{L} \int \frac{x \cdot ds}{EI} + H \int \frac{yds}{EI} = 0 \qquad ...(27.5)$$

where μ = the bending moment on a straight horizontal freely supported beam

M = the bending moment at any point on the arch

M_A and M_b = fixed end bending moments at both the ends of the arch

H = horizontal thrust at the abutments

For **symmetrical arch**, $M_A = M_B$; and the Eq. (27.4) vanishes ; while Eqs. (27.3) and (27.5) get simplified to :

$$\int \frac{M}{EI} \cdot y \cdot ds = \int \frac{\mu \cdot y}{EI} \cdot ds + M_A \int \frac{y ds}{EI} + H \int \frac{y^2 \cdot ds}{EI} = 0 \qquad ...[27.3\ (a)]$$

and

$$\int \frac{M}{EI} \cdot ds = \int \frac{\mu}{EI} ds + M_A \int \frac{ds}{EI} + H \int \frac{y \cdot ds}{EI} = 0 \qquad ...[27.5\ (a)]$$

As the *contraction* due to temperature effect and *rib- shortening* due to thrust, is prevented by the abutments, these effects modify the above Eqs. (27.3), (27.4) and (27.5) as

$$\alpha \cdot TL - \int \frac{M}{EI} \cdot y \cdot ds = 0 \qquad ...[27.3\ (b)]$$

where α = Coefficient of thermal expansion

T = Fall in temperature in °C

$$\int \frac{M}{EI} ds = 0 \qquad ...[27.4\ (b)]$$

$$\int \frac{M \cdot x}{EI} ds = 0 \qquad ...27.5\ (b)]$$

and

$$M = M_A - V_x + H \cdot y \qquad ...(27.6)$$

in which the additional term V stands for the vertical thrust of opposite nature at the two abutments, and is reckoned positive upwards.

For a symmetrical arch, $V = 0$; so that the Eq. (27.6) simplifies to

$$M = M_A + H \cdot y \qquad ...[27.6\ (a)]$$

and Eqs. [27.4 (*b*)] and [27.5 (*b*)] become

$$M_A \int \frac{y}{EI} \cdot ds + H \cdot \int \frac{y^2}{EI} ds - L \cdot \alpha T = 0 \qquad ...[27.4\ (c)]$$

and

$$M_A \int \frac{ds}{EI} + H \int \frac{y}{EI} ds = 0 \qquad ...[27.5\ (c)]$$

By solving these equations, the values of M_A, H and V are obtained.

Rib-shortening effect can be reckoned along with the other loads by adding its effect to the left side of Eq. (27.3).

BUTTRESS DAMS

27.4. Definition and Types of Buttress Dams

An ordinary concrete gravity dam, as we know, is a solid body of mass concrete, somewhat triangular in section, running across the entire width of the river valley. Such a solid wall requires huge amount of concrete, which partly remains *unstressed to full extent*, thereby leading to wastage of concrete. The uplift pressure also acts on the entire width of the dam body, from its bottom, which further increases its size, without giving any additional benefit as a dam.

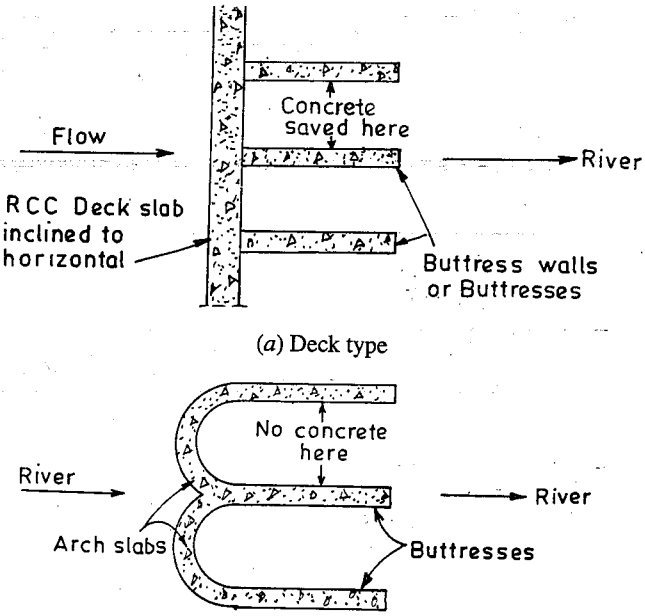
Efforts have, therefore, been made from time to time, to innovate methods for affecting economy in the use of concrete, by cutting down the concrete from those dam portions where it remains unstressed. Attempts have, therefore been made to provide *hollow gravity dams*. **Buttress dams**, are an improvement innovation over the hollow concrete gravity dams.

In such an innovated dam, therefore, *solid walls of specified thickness and section*, are constructed parallel to the flow, at some suitable intervals. These walls are called **buttresses.** *Inclined slabs* or *arch slabs* are then supported on upstream side on these buttresses, as shown in Fig. 27.8.

Although, several types of buttress dams have been devised, but the most common types, are :

1. *Deck slab type ; and*
2. *Multiple arch type.*

Both these types of dams have been shown in their simplified line diagrams, in Figs. 27.7 (*a*) and (*b*) respectively.



(*a*) Deck type

(*b*) Multiple arch type

Fig. 27.8. Simplified line sketches of buttress dams.

Slab type dams are generally preferred for smaller heights, say from 20 to 50 m, or so. The highest such dam in the world is *Rodriguez dam* in Mexico, having a height of 73.2 m. Such dams are virtually referred to as buttress dams, or **Amberson dams** (based upon the name of their inventor).

The multiple arch dams, on the other hand, are used for higher dam heights, say from 50 m onward, and the highest such dam of the world is *Manicougan-5 dam* in Canada, having a height of 210 m.

Detailed views of both these types of dams are shown in Figs. 27.9 and 27.11, respectively ; and are discussed below :

27.4.1. Simply Supported Slab Type of Buttress Dams. (Fig. 27.9). In such a type of dam, the R.C.C. deck slab is freely supported on the buttresses, which have corbels for the slab to rest on. The deck slab is inclined to horizontal by about 40° to 55°, so as to support the dead load of a portion of reservoir water, and thus, to provide the stabilising force, in addition to the 'self' weight of the dam, to avoid its sliding.

Design considerations. The loading and safety criteria for buttress dams or buttresses, is the same as that for a gravity dam section, except that the provided *buttress thickness 't'* will take the load coming from the dam length $= x + t$; where x is the clear

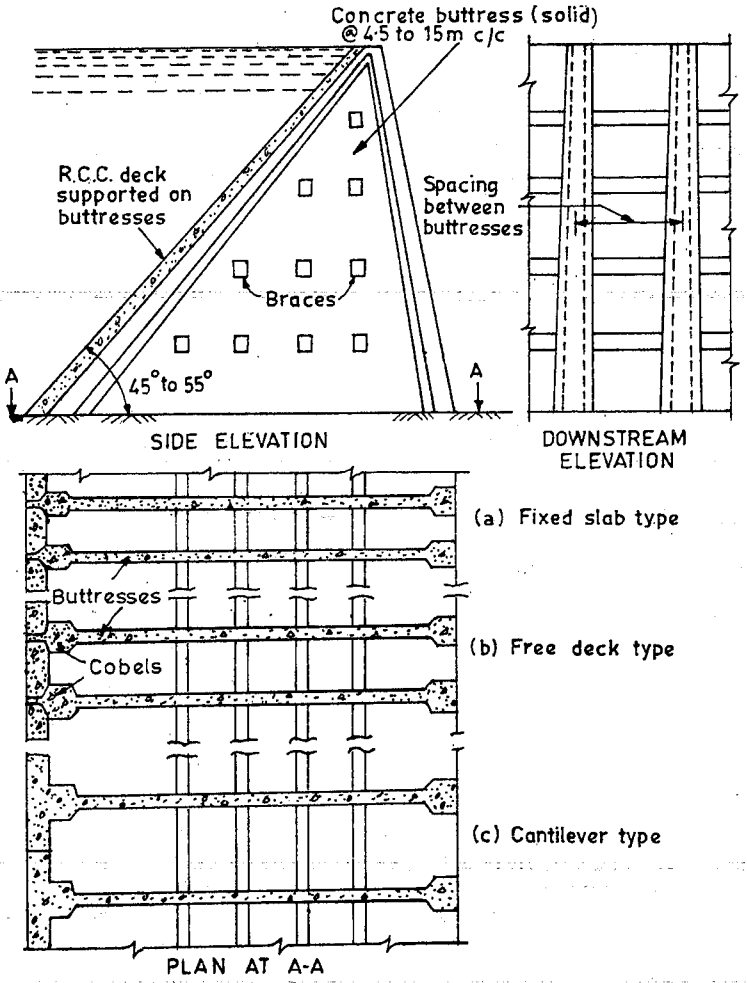


Fig. 27.9. Detailed views of a free deck type of a buttress dam.

spacing between the two consecutive buttresses. Hence, t metre length of buttress section, will take the loads coming from $(x+t)$ metre length of dam, as against the unit metre length of gravity dam section taking load from the unit metre dam length. The resultant impact can be considered by increasing the unit weight of water by multiplying its actual value by a *surcharge factor* (S), defined as :

$$S = \frac{x+t}{t} \qquad \text{...(27.7)}$$

Hence, the effective unit wight of water can be considered as $\gamma_w \left(\frac{x+t}{t}\right)$, and the section of the buttresses can be designed, exactly in the same manner, as a gravity dam section is designed, considering unit length and a continuous section.

The deck slabs can be designed as simply supported R.C.C. decks, each spanning over two adjacent buttresses, and each having a span $= x+t$.

Buttress spacing and Deck slope

(*i*) The **most economical spacing of buttresses** is the one, in which *the minimum thickness of concrete is fully utilised.* This spacing is governed to a very large extent by the values of the upstream slope of the dam (ϕ). For economy, it is also necessary that the buttress spacing be changed for dams of different heights. The suggested spacings for different dam heights, are shown in table 27.1.

Table 27.1. Suggested Economic Buttress Spacings

Mean Dam height in m	*Economic buttress spacing suggested in m, for a normal ϕ value of 40° to 50°*
Less than 15	4.5
15—30	4.5 to 7.5
30—45	7.5 to 12
above 45	12 to 15

(*ii*) The *height, thickness*, and *spacing of buttresses*, can further be controlled by :

$$\textbf{Slenderness ratio} = \frac{\text{Height of buttress}}{\text{Thickness of buttress}} = 12 \text{ to } 15 \qquad ...(27.8)$$

$$\textbf{Massiveness factor} = \frac{\text{Spacing of buttresses}}{\text{Thickness of buttress}} = 2.5 \text{ to } 3 \qquad ...(27.9)$$

(*iii*) For constant buttress spacing, variation of few degrees in the u/s slope may result in an appreciable change in the quantity of concrete. This concrete quantity required per metre length of dam (V_c) is, infact, related to *u/s* deck slope (ϕ), dam height (H), and the sliding factor $\left(F_s \; i.e. \; \frac{\Sigma H}{V}\right)$ by the equation :

$$V_c = 0.208\, H^2 \left[\frac{1}{F_s} - \cot\phi\right] \qquad ...(27.10)$$

This equation clearly shows that for a fixed dam height (H) and a given value of F_s, the concrete quantity depends solely on ϕ. Hence, for different values of ϕ, at fixed values of H and F_s, concrete quantities can be worked out. *Vertical lines*, like those shown in Fig. 27.10, are, thus, drawn for each value of the concrete quantity corresponding to each value of ϕ.

Concrete quantities are also worked out for various angles of u/s slope (ϕ) and different buttress spacings, for fixed decided dam height (H) ; and plotted to get a curve for each angle. A **master curve** is then drawn through the junctions of the curves and the vertical lines corresponding to each angle.

The master curve gives the **absolute economic value of buttress spacing** for different *u/s* slope (ϕ) for the decided dam eight.

Say for example, from the plotted curves of Fig. 27.10, we find that for a $\phi = 48°$, buttress spacing is about 8.8 m ; for a $\phi = 49°$, buttress spacing is about 10 m. *This reflects that by increasing the upstream slope, we can increase the buttress spacing.* This figure also reflects that lower values of ϕ (lesser than 47°) are not available on this particular master curve ; which shows that such lower values will not prove economical

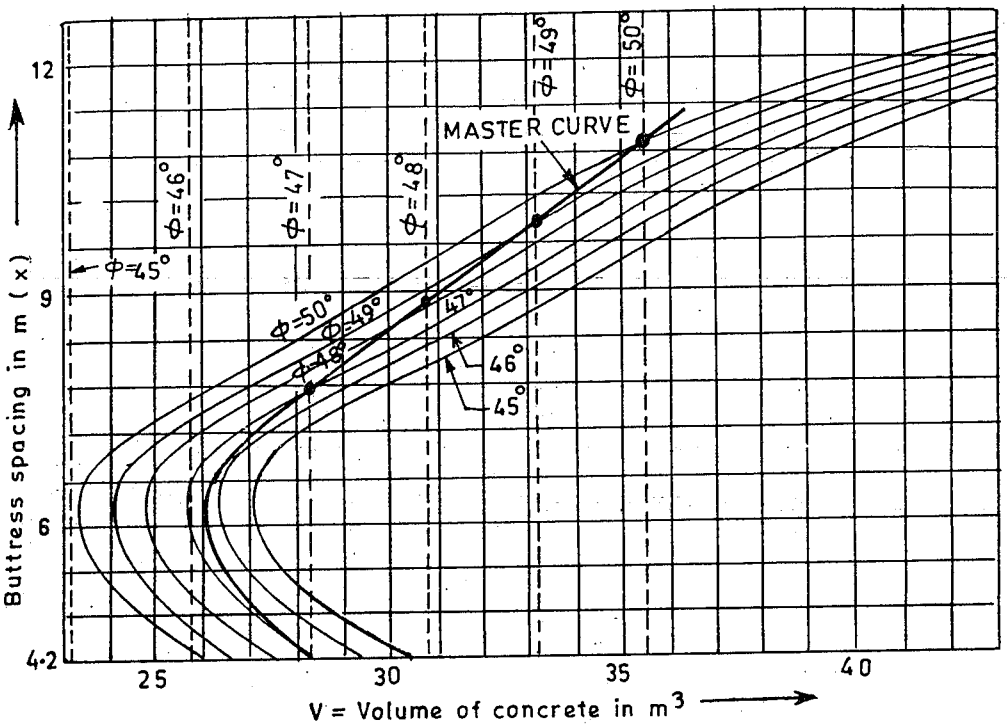


Fig. 27.10. Master curve for absolute economic value of buttress spacing.

for this project. Hence, each individual project (or dam) has, thus, to be analysed for decided dam height, to determine the permissible economical values of ϕ, and their corresponding values of buttress spacing, and thus finally to choose any one set of them.

27.4.2. Multiple Arch Type of Buttres Dams. As stated earlier, in this type of dams, arch slabs are constructed for *u/s* face of the dam, to be supported on buttresses, as shown in Fig. 27.11 and 27.12. Each buttress may be in the form of a single stiffened wall, as shown in Fig. 27.13 (*a*) ; or a double hollow wall, as shown in Fig. 27.13 (*b*).

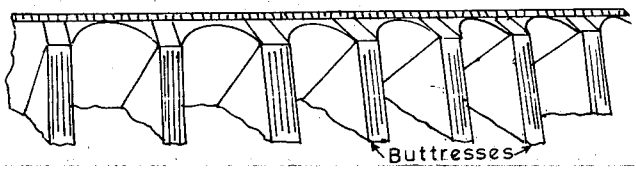


Fig. 27.11. Perspective view of a multi-arch buttress dam.

Meer Alam dam in India, built in about 1800 A.D., is the earliest recorded example of such a type of dam. As stated earlier, such dams are preferred for larger heights, as they prove more stable and flexible, as compared to the ordinary buttress dams (*i.e.* slab type).

The stability of any unit of a multiple arch dam depends on the adjacent units, so that any settlement in one would affect the others. Hence, such a dam would require

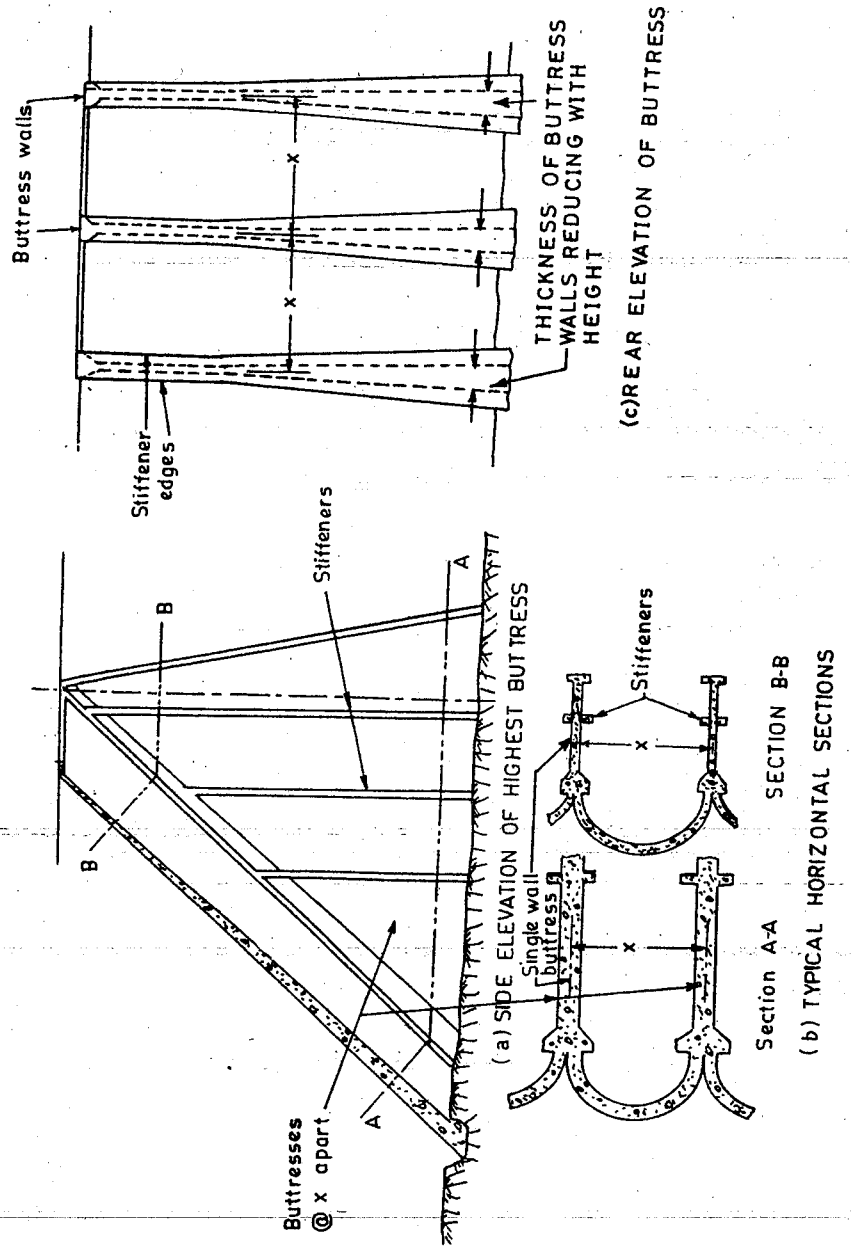
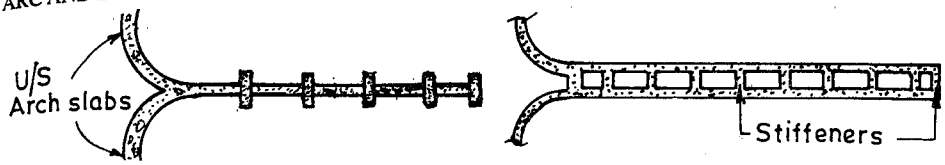


Fig. 27.12. Detailed views of a multiple arch dam.



(*a*) Single wall buttress (*b*) Double wall or hollow buttress

Fig. 27.13. Single and Double walled buttress.

better foundations, as compared to an ordinary buttress dam, where deck slabs on two adjacent buttresses, behave as independent units.

For short spans (10 to 15 m), circular arches of uniform thickness prove economical; whereas, for larger spans, arches of variable thickness are provided. If the central angle of each arch is kept between 180° to 150°, with semi circular or nearly semi circular arches, the horizontal thrust between adjoining arches gets eliminated.

For arched buttress dams, the range of buttress spacing generally adopted, is between 15 to 21 m, although as high a value as 42 m has been permitted in *Webber Greek dam* in California, USA ; a site plan of which is reflected in Fig. 27.14.

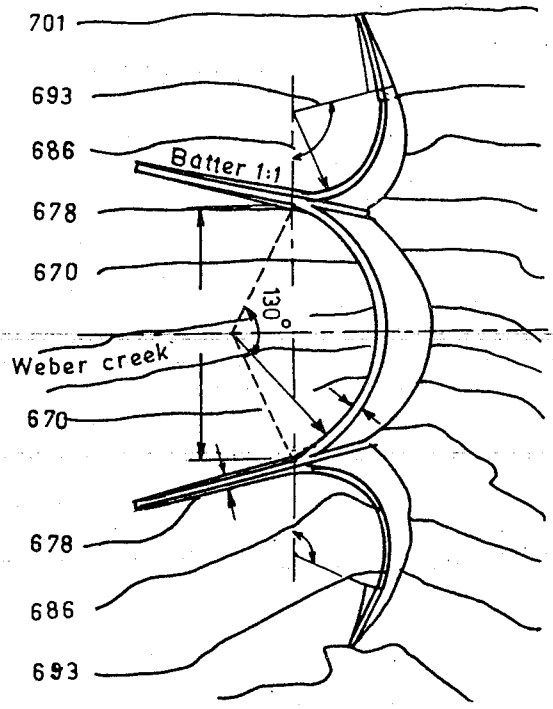


Fig. 27.14. Site plan of Webber creek dam (an example of long span-multiple arch dam).

27.4.3. Other Types of Buttress Dams. In addition to the above described two general types of buttress dams, certain other types, such as *multiple dome type ; massive head type ; columnar buttress type ;* etc., have also been adopted at some places, as discussed below :

27.4.3.1. Multiple dome type. Such a type of buttress dam is quite understandable, as it uses a number of *domes* instead of a number of arches as used in a multiple arch type of dam, and all other features remaining the same. Such a dam can help in

increasing the buttress spacing, but requires sites, where stable foundations can be had at greater depths. The *Coolidge dam on San Carlos river in Arizona is an example of this type of a dam, which is 76.2 m high with buttresses @ 54.8 m apart.*

27.4.3.2. Massive head types. This type of buttress dam is the one, which does not use slabs or archs for upstream face, but the heads of the buttresses themselves are enlarged to meet each other, and thus, to form a continuous water supporting surface, as shown in Fig. 27.15. When the enlarged heads are *cylindrical* on the outerside, it is known as *round head deck* (Fig. 27.15 *a*) ; and when they converge to a point, it is known as a *diamond head deck* (Fig. 27.15 *b*).

These *massive head buttress dams* are, infact, considered as improvement over all other types of buttress dams, as they offer, several **advantages**, such as :

(*i*) The construction work is easier, as mass concrete can be laid together in the entire dam body.

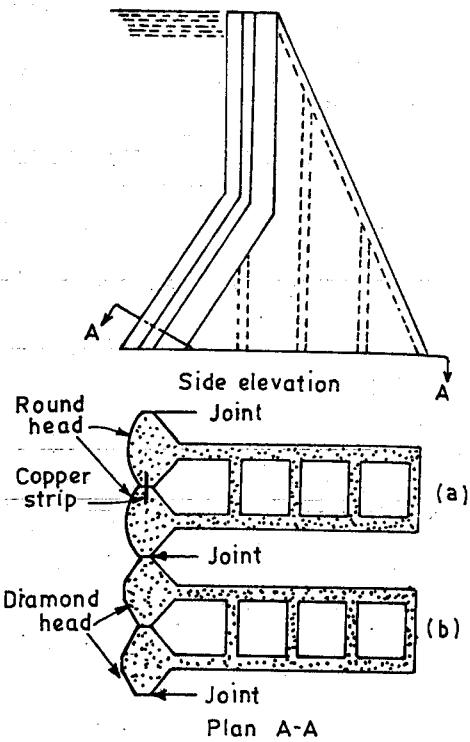


Fig. 27.15. Massive head type of buttress dam.

(*ii*) Since water pressure acts radially in a cylindrical type, and perpendicularly in a diamond type, all pressures are normally *compressive.* The bending as well as diagonal tension in the upstream part of the dam are, thus, absent in this type of dams.

(*iii*) Since the deck is not to be reinforced, there is no question of its failure by rusting of steel, as may happen in an ordinary slab type of buttress dam.

(*iv*) For smaller heights, they prove to give more economical buttress spacing, as compared to other two types of buttress dams.

(*v*) Such a dam body offers more resistance to sliding, because, it is considerably heavier, and has a greater sectional area along the horizontal planes.

27.4.3.3. Columnar buttress type. As shown in Fig. 27.16, this type is a modification of the ordinary slab type, as the deck here, is supported on columns. Very few dams have been built, since they offer the following *disadvantages*:

(*i*) they require very strong and stable foundations.

(*ii*) more skill is required in constructing the buttresses ; and even then, they cost practically as much as other conventional types.

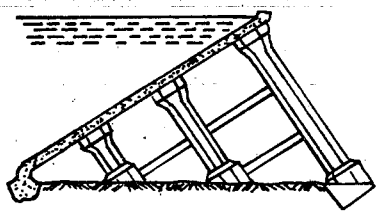


Fig. 27.16. Columnar buttress dam.

PROBLEMS

1. (*a*) Define an arch dam and a double arch dam. What are the important types of arch dams, and what are their major differences ?

(*b*) What is meant by 'the best central angle of an arch dam', and what is its value ?

2. (*a*) Contrast the design criteria for a gravity dam and that for an arch dam by the cylinder theory. What are the relative advantages of constant radius, constant angle, and variable radius dams ?

(*b*) What are the considerations for the choice of a buttress dam in preference to other types ? When do we go in for the massive head type ?

(*c*) What is meant by the following :

(*i*) Economic buttress spacing

(*ii*) Best central angle of an arch dam.

3. (*a*) Distinguish between the constant radius and constant angle layouts of an arch dam. Obtain the value of the best central angle for the latter.

(*b*) Enumerate the different types of buttress dams ; and explain as to how a slab type of buttress dam differs in its design, as compared to a concrete gravity dam.

4. (*a*) State the condition under which you would recommend the construction of an arch dam ; and explain the various methods of layout of an arch dam.

(*b*) How do you ascertain the absolute economic value of buttress spacing for an ordinary buttress dam ?

5. State the conditions under which you would recommend the construction of an 'arch dam'. Draw neat sketches of various types of arch dams and state the circumstances under which you would recommend them.

6. (*a*) What is the 'master curve' as used in connection with buttress dams ? What is its use, and how is it drawn ?

(*b*) What are multiple arch dams, and how do they differ from ordinary arch dams and double arch dams ? Draw line sketches for all these types of dams.

7. (*a*) Massive head type of buttress dams are considered to be the best of all the available types of buttress dams. Why ?

(*b*) Enumerate the design methods or theories, that are used in the design of arch dams. Which of these theories or methods, is used to compute approximate result, and which one is used for obtaining precise results. Explain any one of these in nut-shell.

8. (*a*) Differentiate between 'introdos' and 'extrodos', as are applicable in arch dams.

(*b*) What are double curvature arch dams, and how do they differ from arch dams ?

(*c*) What is the difference between a 'constant centre arch dam' and a 'constant angle arch dam' ?

(*d*) What is meant by economic spacing of buttresses ; and how is it related to dam height and upstream slope of buttress dam ?

9. Write short notes on any four of the following :

(*i*) arch dams.

(*ii*) buttress dams.

(*iii*) methods of layout of an arch dam.

(*iv*) design principles involved in arch dams.

(*v*) master curve for economic buttess spacing.

(*vi*) best central angle for an arch dam.

(*vii*) design criteria for a slab type of buttress dam, and its comparison with the design of a gravity dam.

(*viii*) massive head type of buttress dams.

(*ix*) multiple arch dams.

(*x*) double curvature arch dams.

28

Irrigation Revenue Rates

28.1. Methods of Pricing Irrigation Water

When water is supplied to the cultivators for irrigating their crops, then naturally, it has to be charged and priced at some specified rate. This rate may be based either : (*i*) on flat rate basis ; or (*ii*) on the basis of the actual quantity of water consumed by each cultivator, as discussed below :

28.1.1. Flat Rate Pricing. When pricing of irrigation water is done on flat rate or crop rate basis, then the rates are specified per hectare of each crop sown and cultivated by the cultivators. Different rates for different crops are generally specified, depending upon the water requirement of the crop.

In other words, a crop requiring the highest quantity of irrigation water, such as sugarcane or Banana, will be priced at the highest rate ; whereas the crop requiring the lowest quantity of water, such as Paddy or Groundnut, shall be rated at the lowest rate. Different crops will, thus, generally be rated at reducing rates, with their reducing irrigation requirements, such as shown in Table 28.1 (*a*).

Table 28.1. (*a*) Water Requirements, and Specified Irrigation Water Rates for Different Crops in Maharashtra State of India (1990-data)

S.No.	*Name of the crop sown*	*Water required as depth of irrigation (Δ) in cm*	*Vol. of water required per ha of crop in m^3*	*Specified Irrigation Revenue Rate in Rs/ha of each crop.*
(1)	(2)	(3)	(4)	(5)
1.	Sugar-cane (*perennial*)	270	27,000	750/–
2.	L.S. Cotton (*Kharif-Ravi*)	90	9,000	400/–
3.	Vegetables (*Ravi*)	75	7,500	—
4.	Wheat (*Ravi*)	45	4,500	75/–
5.	H.Y. Maize (*Kharif*)	45	4,500	—
6.	H.Y. Jowar (*Kharif*)	22	2,200	—
7.	Paddy (*Kharif*)	20	2,000	50/–
8.	Groundnut (*Kharif*)	15	1,500	300/–

It can be seen from Col (5) of the above table that although the specified revenue rates are generally inaccordance with the general pricing formula of specifying higher rates for higher water requirement, yet there is a deviation in the case of ground-nut crop, which is priced at six times the rate of paddy, although its water consumption is somewhat lesser than that of paddy. One of the reasons for pricing groundnut at higher rate is the *higher earning capacity* of this crop, as compared to that of paddy, as shown in Col. (7) of table 28.1 (*b*).

Table 28.1. (*b*) Earnings provided by different crops in Maharashtra State (1990-Data)

Sl. No.	*Name of crop*	*Yield per ha in Qtl (100 kg) Qtl/ha*	*Sale price in Rs./Qtl*	*Gross income per ha in Rs./ha (3)×(4)*	*Estimated cost of cultivation (without accounting for irrigation charges in) Rs/ha*	*Net income in Rs/ha (5)—(6)*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*
1.	Sugar-cane	865	30	25,950	12,450	13,500 (Max.)
2.	L.S. cotton	19	498	9462	5,002	4,460
3.	Vegetables	198	110	21,780	10,325	11,455
4.	Wheat	25	290	7,250	3,640	3,610
5.	H.Y. Maize	30	185	5,550	2,500	3,050
6.	H.Y. Jowar	27	200	5,400	2,530	2,870
7.	Paddy	37	215	7,955	4,644	3,311
8.	Ground nut	17	575	9,775	4,854	4921

Note. A comparison between Col. (5) of table 28.1 (*a*) and Col (7) of table 28.1 (*b*) will reflect that the specified irrigation revenue rates for different crops are varying between 2 to 9% of their earning capacities. It also reflects that the rates are highly beneficial to sugarcane, which is a highly earning crop.

It can, therefore, be concluded that the specified rates in a *crop rate system of pricing*, mainly depends upon two factors ; viz.

(*i*) *water requirement of each crop* ; and

(*ii*) *earning capacity of that crop.*

28.1.1.1. Demerits of the crop rate system. This method does not offer any incentive to the cultivators for economising in the use of water, since they will be charged at flat rate for the area of the crops grown. The cultivators will, therefore, never bother to avoid wastage of water, and will also try to use more and more quantity of water, under the wrong notion that more water will flourish their crops more.

This method, therefore, may lead to over-utilisation of water, particularly in command areas falling in the head-reaches of canals. *This may ultimately lead to water-logging and salinity of the irrigated lands, besides leading to the wastage of the precious and scarce irrigation water, which will ultimately cause scarcity of water at the tail reaches of the canal command, thus leading to unequitable distribution of water in the command area of the irrigation system.*

28.1.1.2. Fixed Assessment of grown crops in crop rate revenue system. The field assessment of the area of each crop grown by each cultivator is generally carried out and recorded by the area *Patwari* at the time of sowing, and also at the time of maturity and harvesting. At the end of the crop period, a *demand statement,* called **Khatauni,** for each cultivator is prepared.

The recorded measurements of crops carried out by Patwaris, are generally test checked by officers above Patwaris, such as *Tehsildars (Ziledars)*, SDM, *ADM (Revenue)*, etc, working under the Revenue Department of the Government. In States, where the engineers are also designated as canal officers, such powers are also vested in the hands of sub-divisional and divisional engineers. Checking of irrigated areas, thus, constitute an important part of the duties of engineers. In such cases, divisional engineers (*i.e.* executive engineers) will also have to raise the revenue bills against the cultivators by signing the Khatuni's on 31st June and 31st Dec. every year, for Ravi and Kharif crops, respectively.

In case of failure of crops or reduced production, due to reasons beyond the control of cultivators, full or part remissions can be granted by canal officers. Additional fines and levies may also be imposed by canal officers on cultivators, in case of unauthorised water use, or for damaging the outlets, or cutting the canal banks, or even for wasting water. The decision of the divisional canal officer is generally final and binding in all such cases, because his decisions can be appealed against only at a very high level, *i.e.* to the commissioner (Revenue division), who is normally a senior IAS officer above the Deputy Commissioner of the District.

28.1.2. Volumetric Method of Pricing. In this method, pricing of irrigation water is done on the basis of actual consumption of water by each cultivator, or by a chunk of cultivators, such as by a cooperative society of the cultivators.

This can be done only when measuring devices are installed at the outlets of irrigation minors or water-courses. The method, therefore, requires costly installations, like those of *water-meters* or *venturi-meters*, or *modular outlets.*

Due to the high installation and maintenance cost of such measuring devices as required in this system, this method of revenue assessment has generally not been adopted in our country. This method, therefore, although has its inherent advantages leading to economic use of water, does not find much practical use in India.

The method can be made use in assessing canal irrigation revenues, only if, Irrigation Departments supply water to cooperatives or Panchayats rather than dealing with the individual cultivators. If this can be achieved, then naturally, very less number of meters would be required and this system of charging can be implemented.

This method of pricing irrigation water, can of course, be used much easily in *tubewell irrigation,* where the billing can be done on the basis of number of units of electrical energy consumed in running a particular tubewell-supplying water to a particular chunk of cultivators. This method of charging by the number of electrical energy units consumed or by the number of running hours of the tubewell, is largely used where private tubewells supply water to the cultivators, although its use on Govt. irrigation supplies is still not being adopted on a large scale. The States have simply specified slightly different rates for lift irrigation supplies, and tubewell supplies are covered under those rates.

28.1.2.1. Merits and demerits of volumetric method. This pricing method offers its best merit of economical use of water, thus, avoiding the wastage of this preciously scarce commodity, and also avoiding water-logging and salinity of irrigated lands.

The *demerits* of this method are that :

(*i*) it requires costly measuring devices, atleast at each outlet head, and round-the-year-staff to maintain and look after these devices ;

(*ii*) it promotes unauthorised drawals of irrigation supplies, and also manipulation and mishandling of the measuring devices at the outlet heads ;

(*iii*) it forces the chunk of cultivators to pay equally in accordance with the water supplied at the outlet head, although the cultivators at the tail end of the water-course gets lesser supplies due to wastage of water enroute the length of the water course ; and

(*iv*) silt debris may retard the flow through the metered outlets.

28.2. Economic Water Rates Vs Prevailing Revenue Rates in India

We have stated earlier that in our country, the Govt revenue rates for irrigation waters are generally fixed by considering ; (*i*) *The water requirement of the crop* ; and (*ii*) *the earning capacity of the crop*. However, the most important factor, *i.e. the real cost of the irrigation water,* is generally being ignored.

This real cost of the water can be worked out on *Economic considerations,* by computing the cost of the irrigation project, and recovering the same during its effective life span, along with costs of interest and annual operating and maintenance charges of the irrigation system. Such a rate, when worked out in terms of Rs. per unit quantity of water obtained, is known as the *economic water rate.*

Such a rate will evidently depend upon *the cost of the irrigation project, its effective life span, the rate of interest considered on the investment, and actual O and M charges on the system.* The *annual recovery factor,* computed on Economic considerations as explained in chapter 12 of "Hydrology and Water Resources Engg"., can, thus, give us the cost of irrigation water per year from a particular project, and it can finally be used to work out the unit cost of such water.

Such economic water rates have been computed to be generally much higher than the rates being changed by the government bodies in the country.

For example, economic water rate on a typical tubewell minor irrigation project, in Gujarat State has been computed to be Rs. 0.30P per kilolitre (on 1990 rates) as against the rate being charged by the State Tubewell Corporation @Rs. 0.18P per kilo litre. Similarly, for a typical interstate minor lift irrigation scheme, executed in Gujarat and Orissa States, the economic rate has been worked out to be Rs. 1.53P per kilolitre (on 1990 rates), as against the rate of Rs. 0.18P per kilolitre being charged by the Gujarat State Corporation, and Rs. 0.02P per kilolitre being charged by the Orissa State.

It has thus, generally been seen that the rates, being charged by the Govt. Corporations or State Irrigation Departments, are *highly subsidised* and *too low* incomparison to the actual cost of water. *So much so, that the rates being charged are not even capable to repay the O* and *M charges alone, what to talk of recovering the capital investment on the project.*

Considering the urgent necessity of improving the rates, so as to reduce the heavy losses and subsidies, being incurred by the Government in the operations of irrigation systems, the newly framed *National Water Policy (1987),* has specified that :

"The irrigation water rates should be such as to convey the scarcity value of resources to the users, and to fasten the motivation for economy in water use. *The revenue rates should, therefore, be adequate to recover the annual operation and maintenance charges and a part of the fixed capital cost.* Efforts should be made to reach this ideal, over a period of time, while ensuring the assured and timely supplies

of irrigation water. The water rates for surface water and ground water should be rationalised with due regard to the interests of small and marginal farmers".

From the above, it can be seen that the built in requirement of the National policy is that the water rates should recover annual *R* and *M* charges and part of the capital cost, if not the full cost. Even if for the time being, we neglect recovering of the capital cost, we must rationalise our irrigation revenue rates in such a fashion as to fully recover our annual running and maintenance expenditure on the irrigation systems.

It has, however, been computed that the average annual irrigation revenue realisation in the country is only about 12% of the total O and M expenditure alone, with Statewise variation ranging between 32% in Delhi to 0.11% in Nagaland with Punjab not collecting any revenue at all, since it has totally abolished water charges for irrigation water supplies wef Feb. 1997.

There is, therefore, an urgent need to reconsider and revise the water rates, even at the cost of inflation and possible rise in the rates of crops.

At present, different revenue rates are prevailing in different States of the country even for the same crop ; and even in the same State, different rates are prevailing for different irrigation systems. There is no uniform method of computing the revenue rates ; and the rates are generally being decided and guided by the whims and fencies of the political powers. A few examples of the non-uniformity and irrationality prevailing in our present revenue rates, are highlighted below :

In Gujarat State, the revenue rates for lift irrigation on canals are $\frac{1}{2}$ times, and on rivers and reservoirs $\frac{1}{3}$rd times the flow irrigation rates, provided the water is lifted by the cultivators at their own cost. In Maharashtra, the water rates for all types of lift irrigations to $\frac{1}{10}$th times (when cost of lifting is met by cultivators) are hardly $\frac{1}{5}$th to $\frac{1}{10}$ times the rates prescribed for flow irrigation. In Haryana State, the rates are reduced only to $\frac{1}{2}$ times of the flow irrigation rates, when water is privately lifted from any State irrigation work. In Jammu and Kashmir State, the rates prescribed for lift irrigation (cost of lifting is met by Govt.) are about 15 times the rates prescribed for flow irrigation ; whereas, in Kerala, the prescribed rates for such Govt. lift irrigation schemes are only about $1\frac{1}{2}$ times the rates for flow irrigation. States of Andhra Pradesh, Assam, Bihar, Chhatisgarh, Jharkhand, Karnataka, Madhy Pradesh, Tamil Nadu & West Bengal have not prescribed any separate rates for lift irrigation, and flow irrigation rates remain applicable to lift irrigation rates also. The State of Punjab has completely abolished the water rates wef 14.2.1997. While Himachal Pradesh has levied extremely low rates — lowest amongst all the States. Besides Punjab, some other States like Arunachal Pradesh, Sikkim, Meghalya, Mizoram, and UT's like Andaman and Nicobar Islands, Chandigarh and Lakshadweep have also not levied any irrigation revenue rates, so far, although the States of Sikkim and of Arunachal Pradesh are in the process of introducing such rates.

While the rich-farmers State like Punjab has completely abolished the irrigation revenue rates, the progressive States of Maharashtra and Gujarat have not only revised their water rates in the year 2001, but have also made an inbuilt provision of automatic

annual increase in their notified rates. Say for example, the Maharashtra State has specified a uniform 15% annual increase over the rates notified on 1.9.2001. Similarly, the Gujarat State has provided for 25% annual increase in notified water rates of Paddy, and 15% annual increase in rates notified for all other crops wef 16.2.2001.

It can also be seen from Table 28.2 that certain States and UT's like Andhra Pradesh, Delhi, Goa, Himachal Pradesh, J & K, Karnataka, Kerala, Manipur, West Bengal, etc. have not revised their irrigation revenue rates for the last more than 10 to 50 years. NCT of Delhi is in worst condition in this respect, as it has not revised its rates since the year 1951. There is an urgent need for revision of the water rates in all such States.

Table 28.2. Table Showing the Dates on Which Irrigation Revenue Rates were Last Notified in Various Indian States

State	Date on which irrigation revenue rates were last notified	State	Last date on which irrigation revenue rates were notified
Andhra Pradesh	7.1.1196	Maharashtra	1.9.2001
Assam	30.3.2000	Manipur	Year 1977-78
Bihar	Nov. 2001	Mizoram	Not specified
Chhattisgarh	15.6.1999	Orissa	5.4.2002 for flow irrigation July 1997 for lift irrigation
Delhi	Year 1951	Punjab	Abolished wef 14.2.1997
Goa	11.2.1988	Rajasthan	24.5.1999
Gujarat	16.2.2001	Tamil Nadu	6.11.1987
Haryana	27.7.2000	Tripura	Yet to start
Himachal Pradesh	Year 1977	U.P.	18.9.1995
J & K	1.4.1976	Uttaranchal	18.9.1995
Jharkhand	14.11.1995	West Bengal	6.4.1997
Karnataka	1.7.1985	Andaman and Nicobar Island	1.11.1992
Kerala	18.9.1974	Dadar and Nagar Haveli	29.1.1996
Madhya Pradesh	15.6.1999	Daman and diu	Year 1980

The prevailing irrigation water rates in different States of the country, for flow irrigation are compared in Table 28.3; while those for lift irrigation are compored in table 28.4.

With a view to rationalise the irrigation water rates in different states, the planning commission, Govt. of India, appointed a "*Committee on Pricing of Irrigation Water*", vide notification dated 23-10-1991, under the chairmanship of Dr. A. Vaidyanathan, Professor of Madras Institute of Development studies and former Member of Planning Commission, with 16 members from Central Water Commission (CWC) and Other State Governments. This committee has submitted its report in September, 1992. But the same appears to have not yet been implemented. Some of the important recommendations of this Committee are briefly pointed out in article 28.3.

Table 28.3. Comparison of Prevailing Irrigation Revenue Rates (Maximum) in Various States of India for Flow Irrigation

Water rates for flow irrigation in Rs. per hectare of each crop grown

S.No.	Name of State	*Rice or paddy (Kharif)*	*Sugarcane*	*Wheat*	*Cotton*	*Oil seeds*							*Barley*	*Maize*
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
1.	Andhra Pradesh	494	494	—	494	494	494	—	—	—	—	—	—	—
2.	Assam	281	222	563	—	99	—	—	—	150	—	—	—	—
3.	Bihar	217	371	185	—	247	99	148	—	99	—	99	148	—
4.	Chhatisgarh	494	741	200	371	247	247	494	247	247	494	247	494	—
5.	Delhi (NCT)	99	—	45	—	30	—	89	—	—	—	—	44	30
6.	Goa	150	300	—	—	100	—	100	100	—	—	—	—	60
7.	Gujarat*	300	2750	240	275	440	130	220	440	—	330	90	—	90
8.	Haryana	148	197	124	124	124	99	148	86	—	148	99	124	86
9.	Himachal Pradesh	24	41	7	17	16	12	8	—	—	21	11	16	14
10.	Jammu and Kashmir	49	49	25	—	25	20	37	—	—	—	—	—	25
11.	Jharkhand	109	371	185	—	99	99	185	—	99	—	99	185	—
12.	Karnataka	247	988	148	148	—	—	—	148	—	87	—	—	87
13.	Kerala	99	—	—	—	—	—	—	—	—	—	—	—	—
14.	Madhya Pradesh	200	741	200	371	—	247	494	247	—	494	247	494	—

* With annual increase of 25% for Paddy and 15% for other crops since 16.2.01.

Contd....

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
15.	Maharashtra*	180	4763	360	280	548	—	270	548	—	—	—	—	—
16.	Manipur	38	—	38	—	23	23	23	—	—	—	—	—	—
17.	Orissa	—	500	170	—	170	170	280	170	84	420	—	—	140
18.	Punjab	Charges abolished wef 14.2.1997												
19.	Rajasthan	114	247	148	178	114	79	109	—	—	178	182	79	67
20.	Tamil Nadu	37	49	—	62	—	—	50	—	—	49	—	—	—
21.	Tripura (Proposed rates)	—	—	313	—	—	313	313	313	—	—	—	—	—
22.	Uttar Pradesh	287	474	287	114	—	—	287	—	—	306	—	287	—
23.	Uttaranchal	143	—	143	—	—	—	—	—	—	153	—	—	—
24.	West Bengal	37	—	49	—	—	—	—	—	—	—	—	—	—
25.	U. Ts													
	(*a*) Dadra and Nagar Haveli	140	830	—	—	—	—	110	—	—	—	—	—	—
	(*b*) Daman and Diu	200	—	—	—	—	—	—	—	—	—	—	—	—

* With annual increase of 15% wef 1.9.01

Note. States which have not levied any irrigation revenue charges so far, have not been included in this table.

Table 28.4. Comparison of Prevailing Irrigation Water Rates (Maximum) in Various States of India for Lift Irrigati

Water rates for Lift irrigation in Rs per hectare of each cnp grown

S.No.	Name of State	*Rice or paddy (Kharif)*	*Sugarcane*	*Wheat*	*Cotton*	*Oil seeds*	*Pulses*	*Vegetables*	*Groudnut*	*Jute*	*Tobacco*	*Gram*	*Barley*	*Maize*
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
1.	Andhra Pradesh*	494	494	—	494	494	494	—	—	—	—	—	—	—
2.	Assam*	281	222	563	—	99	—	—	—	150	—	—	—	—
3.	Bihar*	217	371	185	—	247	99	148	—	99	—	99	148	—
4.	Chhatisgarh*	494	741	200	371	247	247	494	247	247	494	247	494	—
5.	Delhi (NCT)	148	—	67	—	45	—	133	—	—	—	—	67	44
6.	Goa	300	600	—	—	—	120	200	200	—	—	—	—	120
7.	Gujarat**	170	420	100	90	180	55	90	90	—	—	45	113	35
8.	Haryana	74	99	62	62	—	49	74	43	—	74	49	62	43
9.	Himachal Pradesh	49	82	29	32	32	24	32	—	—	41	29	32	28
10.	Jammu & Kashmir	247	716	124	—	—	—	—	—	—	—	—	—	—
11.	Jharkhand*	109	371	185	—	99	99	185	—	99	—	99	185	—
12.	Karnataka*	247	988	148	148	—	—	—	148	—	87	—	—	87
13.	Kerala	149	—	—	—	—	—	—	—	—	—	—	—	—
14.	Madhya Pradesh*	200	741	200	371	—	247	494	247	—	494	247	494	—

Contd....

* Separate rates for Lift irrigation are not given, and hence rates shown are the same as for flow irrigation.

** Specified rates here are for lift irrigation done by cultivators at their own cost from canals & irrigation projects.

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
15.	Maharashtra	—	495	—	100	—	—	—	100	—	—	—	—	—
16.	Manipur	38	—	38	—	—	23	23	—	—	—	—	—	—
18.	Mizoram	No water rates are in Vogue												
17.	Orissa	516	1398	646	861	345	215	1292	684	430	1034	—	—	—
18.	Punjab	Charges abolished wef 14.2.1997												
19.	Rajasthan	395	573	296	356	227	158	217	—	—	356	207	158	133
20.	Tamil Nadu*	37	49	—	62	—	—	50	—	—	49	—	—	—
21.	Tripura (Proposed rates)	—	—	—	—	—	—	—	313	313	—	—	—	—
22.	Uttar Pradesh	143	237	143	57	—	—	143	—	—	153	—	143	—
23.	Uttaranchal	287	—	287	—	—	—	—	—	—	306	—	—	—
24.	West Bengal*	37	—	49	—	—	—	—	—	—	—	—	—	—
26.	U. Ts													
	(*a*) Dadra and Nagar Haveli	75	100	100	—	—	100	275	—	—	—	100	—	—
	(*b*) Daman and Diu*	200	—	—	—	—	—	—	—	—	—	—	—	—

* Separate rates for Lift irrigation are not given, and hence rates shown are the same as for flow irrigation.

nt Recommendations of the Committee on Pricing of Irrigation (1992)

s 3.11 and 3.12. A revision in the level and structure of water rates is, thus, necessary in the interest of both efficiency and equity. The revision should be such as to achieve full cost of recovery in due course and in the process promote saving, create disincentives for waste and thereby enable the service area to be expanded and more reliable service assured.

(2) **Paras 3.13 and 3.16.** Revision of water rates should go hand in hand with measures to improve the quality of service and to keep a check on costs.

(3) **Paras 3.54 and 3.56.** The objective of the *first phase* should be to rationalise and simplify the existing system of assessment (based on cropwise irrigated area on an individual basis) to a system of season-specific area rates. It is possible to estimate the relative water consumption per hectare irrigated in different seasons. The variable part of the tarrif in the case of major and medium projects and such of these minor works as are still under State management should be fixed on this basis. *We should urge that all minor systems be turned over to users immediately after completion.* Both categories of projects will pay a flat basic rate per ha. *The level of cost-recovery to be aimed at the first phase should at least cover the O and M costs and 1% interest on capital employed.*

(4) **Para 3.57.** Irrigated area under a crop which spreads over two seasons will be charged at the rates applicable to both seasons, and perennials for all three seasons ; but crops like paddy which take a lot of water for non-consumptive uses need specific treatment. Where paddy is a significant but not a dominant crop, some differentiation may have to be made. In each season, therefore, we need to distinguish at best three categories, viz. paddy, sugarcane, and, perennials and other crops.

(5) **Para 3.58.** In the *second phase* to be implemented in the course of the next decade, the aim would be to shift to a fully volumetric system. Additional investments to modify the distribution system for effective regulation of volume delivered at outlets (estimated at approximately Rs. 5,000 crores) will be needed. As system efficiency and productivity improve, the targets of cost recovery can be progressively increased.

(6) **Paras 3.59 and 3.61.** There are many important matters of detail to be decided in shifting to the volumetric system of charging. These are best decided in consultation with user's representatives. *The most crucial and also the most difficult task in this phase will be to promote the creation of sufficiently large farmer's groups.*

(7) **Para 3.62. Phase III,** which will spread over a much longer period should seek to extend and consolidate the system of farmer group management, and implement, with the involvement and participation of such groups, a programme for upgrading the system to a higher level of efficiency in water use and therefore of productivity. Besides substantial investments in conjunctive use and distribution networks, the techniques of water management will have to become tighter and more sophisticated.

(8) **Para 4.5.** There is a case for earmarking the whole or a substantial part of the receipts from each irrigation system towards the operation and maintenance of that system. *In the long run, there is a case for moving towards the conversion of each irrigation system into an independent self-financing system, whether through the formation of corporations or otherwise.*

(9) **Paras 4.9 and 4.10.** We recommend that States set up special expert groups to work out appropriate norms and a procedure for periodic monitoring and updating for different agro-climatic regions and broad categories of projects.

(10) **Para 4.26.** The States should form a high-powered autonomous Board which may be called "Irrigation and Water-Pricing Board" to review the policy, establish the norms regarding maintenance costs for various components and staff costs, assess the actual expenditure in relation to these norms, and determine the parameters and criteria for revising water rates. There should be a mandatory review of all these matters every five years with an opportunity for users to present their views.

(11) **Paras 5.1 and 5.2.** Among the various problems faced in the matter of assessment, unauthorised irrigation and the incorrect reporting of crops & irrigated areas are the major ones. *There are also delays in raising demands. In spite of low and subsidised water rates, actual revenue recoveries are substantially below the demands. Large arrears have been allowed to accumulate and these tend to be eventually written off.* The existing mechanisms for preventing unauthorised, excessive and wasteful use of water as well as for the recovery of outstanding dues have not proved very effective. Lack of coordination among different agencies involved in assessment and collection also aggravates the problem.

(12) **Paras 5.9. and 5.15.** There is considerable diversity in the mechanism for the assessment and collection of irrigation revenues. The limited data that we have seen suggest that the ratio of accumulated arrears to annual demand is generally much higher in States where the Irrigation Department is responsible for both the assessment and collection than in States where both functions are vested in the Revenue Department or where they are divided between the Irrigation and Revenue Departments. *Having considered the matter, the committee is of the view that the assessment function is best entrusted to the Irrigation Department. As for collection, States may choose one of two options — (1) entrusting both assessment and collection to the Irrigation Department, and (2) making the Irrigation Department responsible for assessment and the Revenue Department for collections — in the light of their specific circumstances and experience. Where alternative (1) is preferred, it would be necessary to empower the Irrigation Department officials to recover arrears of irrigation dues under the Revenue Recovery Laws.*

(13) **Paras 5.16 and 5.18.** The Committee would like to emphasise the need for purposive and strong measures to ensure the accurate assessment of irrigation charges and their prompt and full collection. We suggest that a regular system of independent verification of actual irrigation on a sample basis be introduced on all major and medium project commands. At the same time, we strongly recommend that a serious effort be made by the Irrigation Departments to use remote sensing as an independent source of information on irrigated area, which can be used along with sample verification to test the veracity of records maintained by the field staff. Such independent checks linked to a system of penalties for inaccurate (and rewards for accurate) recording would minimise the loss from under-assessment.

(14) **Para 5.19.** We are of the view that the practice of waiving or suspending collections of irrigation charges on account of drought is not justified in respect of areas actually irrigated.

(15) **Paras 5.20 and 5.21.** The reluctance of the governments to support the agencies concerned in enforcing the regulations has led to a situation in which these agencies have practically given up even raising demands for betterment levies. Very little is done to take cognizance of the widespread violations of rules and even less to enforce what

little penalties are levied. We need hardly emphasise that such laxity has serious consequences not just in terms of revenue but for the efficient management of the system.

(16) **Para 5.22.** With a view to improving collections, the States should consider switching from the existing system of supplying water on credit to one of supply against advance payment. The collection performance relative to demand should be an important consideration for deciding the allocation of *O* and *M* funds to individual systems. We also recommend that proceeds from the collection of accumulated arrears from a system be used for making up the cumulative effects of past neglect in the maintenance of that system.

(17) **Paras 5.23 to 5.25.** Until a system of group assessment on a volumetric basis is introduced, the State agencies will need to verify and record the area irrigated by plots in order to determine the dues from individual farmers. The proposed system of season-hectare assessment (*i.e.*, assessment on the basis of area irrigated in each season) will substantially simplify the task. In the case of minor surface works, since assessment will be at a flat rate per hectare of command, there is no need for recording cropwise area irrigated for the assessment of water rates.

(18) **Para 5.26.** The aim should be to increase user participation in management initially at the level of distributaries and minors, and in due course at the level of the system as a whole. Each system should become an autonomous entity which manages its own finances both for operation and eventually for the expansion/improvement of facilities.

(19) **Para 7.1 to 7.6.** For the purposes of illustrating the application of the suggested approach to the revision of water rates, if we use the norm suggested by the Jakhade Committee, namely, Rs. 180 per ha. gross irrigated area, with adjustments for inflation since 1987 and for departmental overheads at 25 per cent of the norm so adjusted, and add interest @ 1% on capital, *the total recovery in Phase I should average around Rs. 340 per ha.* As against this, the estimated gross receipts from major and medium projects in 1989-90 was Rs. 68 ; the actual irrigation revenue works out in 1989-90 to Rs. 50 per ha. Assuming conservatively the additional revenue on account of an increase in the rates for industrial use at Rs. 10 per ha, **the recovery from irrigation charges in Phase-I has to be Rs. 310 per ha compared to the present realisation of Rs. 50 per ha.**

(20) **Para 7.9.** A basic levy at the rate of Rs. 50 per ha is recommended for all lands in the cultivable commands of major and medium as well as minor works. This is intended as a fee for the right to get water from the system (a sort of "demand charge").

(21) **Para 7.10 to 7.12.** The rate per unit of water needs to be equalised across crops. The additional revenue through such rationalisation will be sizeable, the increase ranging from 18 per cent to 140 per cent of the revenues at current rates if the per ha. cm. rates for all crops are made equal to the irrigation rate now charged for ID crops, and from 50 to 325 per cent if they are made equal to the highest irrigation rate per ha.cm.

(22) **Para 7.13.** The level of rates will also have to be raised. The extent of increase required, depending as it does on the potential for rationalisation, cannot be quantified. It is also likely to vary from States to States. Nevertheless, on the average, the required revenue by way of irrigation charges (Rs. 310 per ha) will still be barely 6% of the gross produce per hectare of the irrigated area, and that without taking any account of likely improvements in productivity.

(23) **Para 7.15.** As a measure of inducement for farmer's groups to take over greater responsibility, we suggest that when the proposed revisions are implemented, the rates

for group delivery be fixed at substantially lower levels than for individual delivery, while keeping the basic fee of Rs. 50 per ha. common.

(24) **Para 7.22.** The implementation of the approach suggested here will require expeditious action on the part of each State to set up task forces, with adequate expert staff and authority for collecting the necessary data, to determine *O* and *M* norms by region and category of projects ; undertake sample studies in the field to determine the extent of under-assessment and under-collection at existing rates ; determine the per hectare rates applicable to paddy and other seasonal crops by season and for perennials in terms of volume of irrigation required and costs connected with carry-over between seasons ; and work out the existing and projected use by non-agricultural users and determine the rates to be charged to such users, the appropriate contractual arrangements, and other relevant details.

The committee report, however, appears to be lying in the waste paper baskets, of the States, and the States are simply not inclined to take politically inconvenient decisions of increasing the water rates. Let us appeal to the politicians of this country to forget their political benefits infront of the overall benefits of the country.

PROBLEMS

1. Discuss the two important methods, which are used to price the irrigation water supplies for collecting irrigation revenues. Compare the merits and demerits of both these methods. Also discuss as to which of these two methods is generally being used in our country and why.

2. (*a*) Explain the various points which need to be considered while fixing irrigation water rates.

(*Pune Univ. May 1995*)

(*b*) Explain the crop rate basis of assessment of water charges. State the other method.

(*Pune Univ. Nov. '95*)

3. (*a*) Enumerate the merits and demerits of volumetric assessment of water charges.

(*Pune Univ. Nov. '96*)

(*b*) How have the irrigation revenue rates been fixed in India ?

Compare the irrigation revenue rates prevailing in different States of India, explaining the urgent necessity of revision of these rates. What has the GOI done to effect this revision and/or to switch to volumetric assessment system.

4. (*a*) "The revenue rates for canal irrigation water should differ from the revenue rates for lift irrigation supplies". Critically discuss this statement in special reference to Indian States.

(*b*) Point out the major recommendations of the Expert Committee, constituted by GOI in 1992, for revision of irrigation revenue rates.

—29

Chapterwise Multichoice Objective Questions

CHAPTER -1

Q. 1. Irrigation is basically required in :
(*a*) humid regions (*b*) arid regions
(*c*) semi arid regions (*d*) all of the above.

Q. 2. Irrigation may not be needed in :
(*a*) England (*b*) Israel
(*c*) India (*d*) none of the above.

Q. 3. Pinpoint the correct statement :
(*a*) irrigation helps in adopting mixed cropping
(*b*) 'mixed cropping' means sowing of a different crop after a particular crop has been grown
(*c*) over-irrigation may lead to saving in fertilizers
(*d*) irrigation helps in avoiding mixed cropping.

Q. 4. 'Flood irrigation' method of irrigating fields, works best on :
(*a*) level or gently rolling terrain
(*b*) steeply rolling terrain
(*c*) both (*a*) and (*b*)
(*d*) none of the above.

Q. 5. In flood irrigation, the preferred method of applying irrigation water to the comparatively steeper rolling land, is :
(*a*) check flooding (*b*) boarder flooding
(*c*) wild flooding (*d*) basin flooding.

Q. 6. Fields enclosed by dikes, where rice is grown, are called :
(*a*) paddies (*b*) furrows
(*c*) basins (*d*) none of the above.

Q. 7. The method of growing crops on ridges, running on the sides of water ditches, is known as :
(*a*) flood irrigation (*b*) furrow irrigation
(*c*) check irrigation (*d*) none of them.

Q. 8. In a field under furrow irrigation, 'furrows' are referred to represent:
(*a*) ridges on which crops are grown
(*b*) narrow ditches which carry irrigation water
(*c*) both (*a*) and (*b*)
(*d*) neither (*a*) nor (*b*).

Q. 9. In a mildly water scarce area, the drip irrigation could be preferred for growing :
(*a*) wheat (*b*) fodder
(*c*) rice (*d*) fruits and vegetables.

Q. 10. The most suitable water for irrigation is :
(*a*) C 1—S 1 (*b*) C 2—S 2
(*c*) C 4—S 1 (*d*) C 1—S 4.

Q. 11. Addition of gypsum to the irrigation water is recommended to overcome difficulties posed by :
(*a*) highly saline irrigation supplies
(*b*) irrigation supplies containing high quantities of sodium
(*c*) irrigation supplies containing heavy sediment
(*d*) all of the above.

Q. 12. The electrical conductivity of medium saline water (C 2) at 25°C is of the order of :
(*a*) 50 to 100 μΩ/cm (*b*) 100—250 μΩ/cm
(*c*) 250 to 750 μΩ/cm (*d*) 750—1500 μΩ/cm.

Q. 13. The Sodium Absorption Ratio of an irrigation water is 8. This water will be called :
(*a*) low sodium water (*b*) medium sodium water
(*c*) high sodium water (*d*) none of the above.

Q. 14. High sodium content in irrigation water is :
(*a*) generally good
(*b*) generally bad
(*c*) generally good, but bad for a few crops
(*d*) generally bad, but good for a few crops.

Q. 15. Salinity in irrigation water is measured by :
(*a*) SAR value (*b*) Electrical-conductivity value
(*c*) pH-value (*d*) none of the above.

Q. 16. The time required to irrigate a strip of area 0.203 hectare by a stream discharge of 0.043 cumec, to provide an average depth of 6.35 cm to the field, is : (assume average rate of infiltration to be 5 cm/h)
(*a*) 2.75 hour (*b*) 1.35 hour
(*c*) 1.5 hour (*d*) 1.90 hour. (*Calcutta University*)

Q. 17. If the concentration of Na, Ca, and Mg in a water sample are 345, 60 and 18 mg/1, respectively, then the Sodium Absorption Ratio (SAR) of this water will be :
(*a*) 5 (*b*) 10.
(*c*) 39 (*d*) 55.

CHAPTER 2

Q. 18. With the increase in supplied irrigation water, the yield of crops:
(*a*) increases continuously
(*b*) decreases continuously
(*c*) increases up to a certain limit, and then becomes constant
(*d*) increases up to a certain limit, and then decreases.

Q. 19. The most expected crops in a hot arid district of Rajasthan State in India, in the month of September, are :
(*a*) Rice and Sugarcane (*b*) Bazra and Maize
(*c*) Wheat and Maize (*d*) Tobacco and Cotton.

Q. 20. The maximum irrigation requirement of Rice crop is exhibited by its :
(*a*) maximum delta value (*b*) maximum duty value
(*c*) minimum duty value (*d*) none of the above.

Q. 21. The crop among the following, which is expected to have the maximum duty, is
(*a*) Wheat (*b*) Rice
(*c*) Sugarcane (*d*) Cotton.

Q. 22. Kor-Watering is the irrigation water supplied to a crop :
(*a*) at the time of its sowing
(*b*) just before harvesting
(*c*) about three weeks after sowing
(*d*) about three weeks before harvesting.

Q. 23. The duty of irrigation water for a given crop is maximum :
(*a*) on the field
(*b*) at the head of the main canal
(*c*) at the head of the water-course
(*d*) none of them.

Q. 24. The important Govt. reference made to the 'duty for a crop,' is usually related to its duty :
(*a*) on the field
(*b*) at the head of the main canal
(*c*) at the head of the water course
(*d*) none of them.

Q. 25. The duty at the end point of a canal minor, where the Govt. control usually ceases, is called :
(*a*) duty on field (*b*) outlet duty
(*c*) flow duty (*d*) storage duty.

Q. 26. The first important watering of crops is usually called :
(*a*) paleo watering (*b*) kor-watering
(*c*) crop-watering (*d*) all of the above.

Q. 27. The crop sequence, which cannot serve any useful purpose in 'Crop rotation', is :
(*a*) Wheat—Juar—Gram (*b*) Rice—Gram—Rice
(*c*) Cotton—wheat—Gram (*d*) Rice—Wheat—Cotton.

Q. 28. The optimum depth of kor watering for rice is about :
(*a*) 25 cm (*b*) 19 cm
(*c*) 13.5 cm (*d*) 9 cm.

Q. 29. The kor period, within which a crop must receive its first major watering, will be :
(*a*) less for humid climates (*b*) less for dry climates
(*c*) equal for all climates (*d*) none of the above.

Q. 30. The crop, out of the following, which is not a cash crop, is :
(*a*) Jute (*b*) Tea
(*c*) Rice (*d*) Sugarcane.

Q. 31. In India, the cultivated area under Rabi season is generally x times the area under Kharif season; where x is :

(*a*) 1 (*b*) $\frac{1}{2}$

(*c*) 2 (*d*) none of them.

Q. 32. The relationship between the duty D in ha/cumecs, the water depth Δ in cm, and base period B in days, is given by :

(*a*) $D = \frac{864\,B}{\Delta}$ (*b*) $D = \frac{8.64\,B}{\Delta}$

(*c*) $D = \frac{864\,\Delta}{B}$ (*d*) $D = \frac{8.64\,\Delta}{B}$

Q. 33. Consumptive use of water for a crop represents :

(*a*) The transpiration needs of the crop

(*b*) evaporation needs of the cropped area.

(*c*) evapotranspiration needs of the cropped area plus the minor quantity required in plant metabolism

(*d*) None of the above.

Q. 34. The amount of irrigation water required to fulfil the evapo-transpiration and minor metabolic plant needs per unit of cropped area is called :

(*a*) consumptive use

(*b*) consumptive irrigation requirement

(*c*) net irrigation requirement

(*d*) none of the above.

Q. 35. When there is no percolation loss for leaching and other such soil needs, then the Net irrigation requirement (NIR) of a cropped area, will be equal to :

(*a*) the consumptive use (C_u)

(*b*) $C_u - R_e$: where R_e is effective rainfall during cropping

(*c*) $\frac{C_u - R_e}{\eta_a}$: where η_a is water application efficiency

(*d*) none of the above.

Q. 36. The Gross Irrigation Requirement (GIR) of water is equal to :

(*a*) NIR ÷ η_a (*b*) NIR ÷ $\eta_a \cdot \eta_c$

(*c*) NIR ÷ $\frac{\eta_a}{\eta_c}$ (*d*) none of above.

where η_a = water application efficiency

η_c = water conveyance efficiency.

Q. 37. The ratio of the water stored in the root zone of a crop, to the water actually delivered to the crop in the field, is known as;

(*a*) water conveyance efficiency

(*b*) water application efficiency

(*c*) water use efficiency

(*d*) none of the above.

Q. 38. The water which can be utilised by the crops from the soil is called :

(*a*) field capacity water (*b*) capillary water

(*c*) hygroscopic water (*d*) none of the above.

Q. 39. The moisture held by a well drained soil against gravity drainage, by the force of surface tension between the soil grains and water drops, is called :
(*a*) field capacity water (*b*) hygroscopic water
(*c*) capillary water (*d*) water of adhesion.

Q. 40. Permanent wilting point moisture content for a crop represents the :
(*a*) hygroscopic water (*b*) capillary water
(*c*) field capacity water (*d*) none of the above.

Q. 41. Available moisture for a crop is equal to :
(*a*) field capacity moisture content—Wilting point moisture content
(*b*) field capacity moisture content—hygroscopic moisture content.
(*c*) both (*a*) and (*b*)
(*d*) none of the above.

Q. 42. The optimum moisture content (m.c.) which is retained in the root zone of a soil, before applying irrigation water, is :
(*a*) equal to : (the field capacity m.c.—wilting point m.c.)
(*b*) less than : (the field capacity m.c.—wilting point m.c.)
(*c*) more than : (the field capacity m.c.—wilting point m.c.)
(*d*) may be more or less than : (the field capacity m.c.—wilting point m.c.) depending upon the crop grown.

Q. 43. Irrigation water is usually applied to the fields, when the available moisture in the root zone of the crop, gets depleted by :
(*a*) 0—10% (*b*) 10—25%
(*c*) 50—80% (*d*) 100%.

Q. 44. Frequency of irrigation is dependent upon the type of :
(*a*) soil and crop (*b*) soil and climate
(*c*) soil, crop, and climate (*d*) soil, crop, climate, and fertilizer.

Q. 45.* A soil has a field capacity of 25%, permanent wilting point of 15%, and specific weight of 14.7 kN/m^3. If the root zone depth of the grown crop is 90 cm, then its available moisture holding capacity is
(*a*) 10 cm (*b*) 13.5 cm
(*c*) 16.67 cm (*d*) 20 cm.

Q. 46. After how many days will you recommend supplying irrigation water to a crop having 0.8 m root zone depth, and grown in a soil having field capacity of 30%, permanent wilting = 15%, and density of soil = 1.5 gm/cc ? The consumptive use for the crop is 5 mm/day, and only 60% of available moisture is permitted to be availed.
(*a*) 9 days (*b*) 15 days
(*c*) 21 days (*d*) 40 days.

Q. 47.* If the irrigation efficiency at a 10 hectare field is 80%, and the conveyance losses from the canal outlet is 10%, then the volume of water required at the canal outlet, for supplying 10 cm water depth in the field, will be equal to;
(*a*) 10,390 kL (*b*) 7200 kL
(*c*) 13,900 kL (*d*) 1,39,000 kL.

Q. 48. The Δ (in m) for a crop, having quantity duty D (in ha/M. cum), and base period B (in days) is given by :
(*a*) $\Delta = \frac{8.64\,B}{D}$ (*b*) $\Delta = \frac{864\,B}{D}$
(*c*) $\Delta = \frac{100}{D}$ (*d*) $\Delta = \frac{10{,}000}{D}$.

*Hints are given at the end of the chapter, for solving starred questions.

Q. 49. The ratio of the water stored in the root zone during irrigation, to the water needed in the root zone prior to irrigation, is called :
(*a*) efficiency of water use
(*b*) efficiency of water storage
(*c*) efficiency of water application
(*d*) efficiency of water conveyance.

Q. 50. If the irrigation water applied to a field penetrates uniformly throughout, then the water distribution efficiency is :
(*a*) 1 (*b*) 0
(*c*) 0.5
(*d*) none of them, as more data is required to ascertain it.

Q. 51. The efficiency of water application does not depend upon :
(*a*) climatic conditions
(*b*) type of the soil
(*c*) method of application
(*d*) geometry of the conveyance system.

Q. 52. The efficiency of water conveyance does not depend upon :
(*a*) climatic conditions
(*b*) geometry of the conveyance system
(*c*) nature of the boundary of the conveyance system
(*d*) method of application of water.

Q. 53. Which of the following is not correctly matched?
(*a*) Rice—Kharif (*b*) Wheat—Rabi
(*c*) Barley—Kharif (*d*) Potatoe—Rabi.

CHAPTER 3

Q. 54. If the intensity of irrigation for Kharif is 45% and that for Rabi is 60%; then the annual intensity of irrigation, is :
(*a*) 60% (*b*) 100%
(*c*) 105% (*d*) none of them.

Q. 55.* If in a certain irrigation project, and in a given year, 72% and 56% of the culturable command remained unirrigated in Kharif and Rabi seasons, respectively; then the intensity of irrigation for that year and for that project, would be :
(*a*) 36% (*b*) 64%
(*c*) 72% (*d*) 128%.

Q. 56. The gross irrigated area cannot exceed the cultivable commanded area :
(*a*) Correct (*b*) Incorrect.

Q. 57.* The area, on which crops are grown in a particular season, is called :
(*a*) culturable commanded area (CCA)
(*b*) gross sown area (*c*) net sown area
(*d*) none of the above.

Q. 58.* The CCA for a particular State is 5 Mha; out of which, 4.5 Mha is being sown in Rabi season and 2.5 Mha in Kharif season. These areas are being irrigated to the extent of 90% and 80%, respectively. The annual intensity of irrigation for this State is :
(*a*) 80.7% (*b*) 85%
(*c*) 100% (*d*) 121%.

Q. 59. In plain areas, the irrigation canals are usually aligned along :
(*a*) ridge lines (*b*) contour lines
(*c*) valley lines (*d*) across the contours.

Q. 60. A ridge canal is also called a :
(*a*) watershed canal (*b*) contour canal
(*c*) side slope canal (*d*) none of the above.

Q. 61. The canal, which may frequently encounter cross-drainage works, will be a:
(*a*) watershed canal (*b*) contour canal
(*c*) side slope canal (*d*) none of these.

Q. 62. In a hilly district, where watershed line is very high as compared to the head-works, which type of alignment you would recommend for a proposed canal, if cross-drainage is to be avoided :
(*a*) canal along the ridge line
(*b*) canal along the contour line
(*c*) canal across the contour lines
(*d*) none of them.

Q. 63. The canal, which can irrigate only on one side, is a :
(*a*) watershed canal (*b*) contour canal
(*c*) side slope canal (*d*) none of them.

Q. 64. The capacity of an irrigation canal is usually controlled by :
(*a*) average kharif demand
(*b*) average rabi demand
(*c*) kor demand of rabi crops
(*d*) kor demand of kharif crops.

Q. 65. The optimum kor water depth for a kharif crop is 19 cm with an allowed kor water period of 3 weeks, the outlet discharge factor for this crop will be:
(*a*) 955 ha/cumec (*b*) 782 ha/cumec
(*c*) 860 ha/cumec (*d*) none of them.

Q. 66.* If the discharge required for different crops grown in a field is 0.4 cumecs, and the capacity factor and time factor are 0.8 and 0.5, the design discharge for the distributary will be :
(*a*) 0.80 cumecs (*b*) 0.16 cumecs
(*c*) 1 cumecs (*d*) 1.24 cumecs.

Q. 67.* An irrigation canal is to be designed to deliver 6.25 cumecs to meet the peak Rabi demand of a total of 5100 hectares of cropped area. The estimated canal losses are 25% of the head discharge. The duty on capacity of this canal will be :
(*a*) 816 ha/cumec (*b*) 1020 ha/cumec
(*c*) 653 ha/cumec (*d*) 612 ha/cumec.

Q. 68. 'Duty on capacity' is also called :
(*a*) outlet duty (*b*) capacity factor
(*c*) full supply coefficient (*d*) quantity duty.

Q. 69.* The 8000 hectares of gross commanded area of an irrigation project includes 20% of reserved forests, usar lands, roads, etc. The pastures and fallow lands are 10%. If the intensity of irrigation is 50%, the area to be irrigated is :
(*a*) 4000 hectares (*b*) 3200 hectares
(*c*) 2800 hectares (*d*) none of them.

Q. 70. The discharge carried by a minor distributary is usually less than :
(*a*) 0.5 cumecs (*b*) 1 cumecs
(*c*) 2.5 cumecs (*d*) none of them.

Q. 71. The discharge carried by a major distributary, is usually up to :
(*a*) 5 cumecs (*b*) 10 cumecs
(*c*) 30 cumecs (*d*) 50 cumecs.

Q. 72. Earthen irrigation canals, when aligned curvilinear in plan, must be on gentle curves. This adopted curve radius, should be :
(*a*) more for canal carrying more discharge
(*b*) less for canal carrying more discharge
(*c*) equal for all discharges.

Q. 73. The canal which is not supposed to do any irrigation is called :
(*a*) main canal (*b*) water course
(*c*) major distributary (*d*) minor distributary.

Q. 74. Unlined irrigation canals, when aligned on curvilinear routes in plan, will have to be pitched on :
(*a*) both sides (*b*) concave side only
(*c*) convex side only (*b*) none of the above.

CHAPTER 4

Q. 75. The bed of an alluvial channel along the flow will always be :
(*a*) flat (*b*) wavy
(*c*) duned and rippled (*d*) all of the above are possible.

Q. 76. The bed form, which is not expected in an alluvial channel with sediment motion, is :
(*a*) rippled (*b*) meandered
(*c*) duned (*d*) anti duned.

Q. 77. The bed of an alluvial channel carrying silted water at a high velocity is expected to be :
(*a*) rippled (*b*) duned
(*c*) flat (*d*) wavy.

Q. 78. The anti-dunes develop on beds of alluvial streams, when Froude number is:
(*a*) 0 (*b*) 0.5
(*c*) 1 (*d*) 1.2.

Q. 79.* The critical shear stress τ, at which incipient motion of sediment takes place, is proportional to :
(*a*) $\sqrt{d}$ (*b*) d
(*c*) d^2 where d is grain size (*UPSC Engg. Services*)

Q. 80.* In order to ensure that no scouring takes place in the bed of a channel of bed slope S, constructed in an incoherent alluvium of size 'd' cm, the flow velocity should be restricted to :
(*a*) $4.85 \cdot d^{1/2} \cdot S^{-1/6}$ (*b*) $4.85 \cdot d^{-1/2} \cdot S^{1/6}$
(*c*) $0.48\, d^{1/2} \cdot S^{1/6}$ (*d*) $0.48 \cdot d^{1/2} \cdot S^{-1/6}$.

Q. 81.* Manning's rugosity coefficient is proportional to :
(*a*) $\sqrt{d}$ (*b*) d
(*c*) $d^{1/6}$ (*d*) $d^{2/3}$
(*e*) $d^{3/2}$; where d is representative grain dia of bed surface.

Q. 82. The finer sediment which does not originate from the stream bed, but originates from the catchment area, is best called as the :

(*a*) bed load (*b*) suspended load
(*c*) wash load (*d*) none of these.

Q. 83. The force exerted by the flowing water on the sediment particles to cause their motion, is called :

(*a*) buoyant force (*b*) tractive force
(*c*) kinematic force (*d*) eddy force.

Q. 84. The minimum size of sediment that may remain stable in an alluvial channel, carrying discharge intensity q, with hydraulic radius R, and bottom slope S, is :

(*a*) $11\,RS$ (*b*) $1.35\left(\frac{q^2}{g}\right)^{1/2}$
(*c*) γRS (*d*) none of them.

Q. 85.* The tractive force method of designing stable channels in non-cohesive soils, is applicable only when the channels :

(*a*) carry sedimented water
(*b*) carry clear water
(*c*) carry sedimented water, but sediment concentration does not exceed 500 ppm or so
(*d*) carry any type of water.

Q. 86. In an earthen trapezoidal channel having side slopes of 1.5 H : 1 V, constructed in a non-cohesive soil, having D_{75} particle size of 'd' m, and angle $\phi = 37°$, no scouring will occur only when the hydraulic mean depth (R) of the channel is limited to :

(*a*) $\frac{d}{33\,S_0}$ (*b*) $\frac{d}{11\,S_0}$
(*c*) $\frac{d}{21\,S_0}$ (*d*) none of them.

Q. 87. If τ_c is the critical shear stress on a channel bed, then the critical shear on the side slopes (nH : IV) of this channel (τ_c') is given by :

(*a*) $\tau_c' = \tau_c \cdot \sqrt{1 - \frac{\sin^2\theta}{\sin^2\phi}}$ (*b*) $\tau_c' = \tau_c \cdot \sqrt{1 + \frac{\sin^2\theta}{\sin^2\phi}}$
(*c*) $\tau_c' = \tau_c \cdot \sqrt{1 - \frac{\sin^2\phi}{\sin^2\theta}}$ (*d*) $\tau_c' = \tau_c \cdot \sqrt{1 + \frac{\sin^2\phi}{\sin^2\theta}}$

where ϕ = angle of repose of soil

$$\theta = \tan^{-1}\left(\frac{1}{n}\right)$$

Q. 88. The Lacey's and Kennedy's empirical silt theories for designing irrigation channels in Indian non-cohesive alluvial soils, are meant to ensure :

(*a*) no scouring in the channel
(*b*) no silting from out of the sedimented water entering the channel from the headworks
(*c*) both (*a*) and (*b*)
(*d*) neither (*a*) nor (*b*).

Q. 89.* The critical velocity $V_0 = 0.55 \cdot m \cdot y^{0.64}$, as suggested by Kennedy for design of trapezoidal irrigation channels, is :
(*a*) the maximum permissible velocity
(*b*) the minimum permissible velocity
(*c*) both (*a*) and (*b*)
(*d*) none of the above.

Q. 90. Critical velocity Ratio for use in Kennedy's theory, is :
(*a*) less than 1 (*b*) more than 1
(*c*) equal to 1 (*d*) all of the above.

Q. 91. The Garret's diagrams are based on :
(*a*) Lacey's theory (*b*) Khosla's theory
(*c*) Bligh's theory (*d*) Kennedy's theory.

Q. 92. The critical velocity ratio was introduced in Kennedy's equation of critical velocity to take into account the effect of :
(*a*) channel cross-section (*b*) climatic conditions
(*c*) silt grade (*d*) roughness of bed.

Q. 93. The most important shape parameter in sediment analysis is :
(*a*) sphericity (*b*) shape factor
(*c*) roundness (*d*) form factor.

Q. 94. Lacey's silt factor is proportional to :
(*a*) $\sqrt{d}$ (*b*) d
(*c*) d^2 ; where d is the grain size.
(*U.P.S.C.; Engg. Services*)

Q. 95. In mechanics of sediment transport, the parameter $\varepsilon = \frac{\eta}{\rho}$ (where η is eddy viscosity and ρ is density) is called :
(*a*) kinematic viscosity (*b*) absolute viscosity
(*c*) eddy kinematic viscosity (*d*) none of the above.

Q. 96. In sediment analysis, the parameter which indicates the amount of wear the grain particle has undergone during relative movement of silt and water, is
(*a*) sphericity (*b*) shape factor
(*c*) roundness (*d*) form factor.

Q. 97. The wetted perimeter P of a stable channel is proportional to :
(*a*) Q (*b*) $\sqrt{Q}$
(*c*) Q^2 ; where Q is the discharge
(*U.P.S.C.; Engg. Services*)

Q. 98. The units of Shield's Entrainment function are :
(*a*) Newton/sq m (*b*) metre
(*c*) kg/m/sec (*d*) no dimensions.

Q. 99. The cross-section of natural silt transporting channels tend to have the shape of :
(*a*) trapezium (*b*) semi-ellipse
(*c*) semi-circle (*d*) none of them.

Q. 100. In Lacey's regime theory, the flow velocity is proportional to :
(*a*) $(Qf^2)^{1/2}$ (*b*) $\frac{Q}{f^2}$
(*c*) $(Qf^2)^{1/6}$ (*U.P.S.C.; Engg. Services*)

Q. 101. For all homogeneous fluids, the value of Von-Karman Universal Constant, is:

(*a*) 0 (*b*) 1
(*c*) 0.4 (*d*) 0.8.

Q. 102. The boundary shear stress, τ_0 in an open channel is given by :

(*a*) $\frac{\gamma h^2}{2}$ (*b*) γRS

(*c*) $\frac{1}{n} R^{2/3} S^{1/2}$ where h = depth

R = hydraulic radius
S = Slope
γ = Specific weight of fluid
n = Manning's roughness factor

(*U.P.S.C.; Engg. Services*)

Q. 103. Lacey's regime theory is not applicable to a canal in :

(*a*) true regime (*b*) initial regime
(*c*) final regime (*d*) none of them.

Q. 104.* When a channel is protected on the bed and sides with some protective material, and there is no possibility of change in its cross-section and longitudinal slope, then the channel is said to be in :

(*a*) initial regime (*b*) final regime
(*c*) permanent regime (*d*) true regime.

Q. 105.* Hydraulic depth is the ratio of :

(*a*) wetted area to wetted perimeter
(*b*) wetted area to top width
(*c*) wetted area to bottom width
(*d*) wetted perimeter to top width.

Q. 106. The unit of Chezy's coefficient C in Chezy's formula, is :

(*a*) $\sqrt{m/s}$ (*b*) m/s
(*c*) m (*d*) Nil.

Q. 107. For a most economical trapezoidal channel section :

(*a*) hydraulic mean radius equals the depth of flow
(*b*) hydraulic mean radius equals half the depth of flow
(*c*) bottom width is twice the depth of flow
(*d*) bottom width is half the depth of flow.

Q. 108.* The channels, whose boundaries are made of easily erodible loose soils, are called :

(*a*) mobile boundary channels
(*b*) rigid boundary channels
(*c*) non-alluvial channels
(*d*) none of the above.

Q. 109.* Lacey's scour depth for a stream, carrying a discharge of 3 cumecs per metre width and having a silt factor 1.2, is :

(*a*) 1.32 m (*b*) 2.64 m
(*c*) 3.96 m (*d*) 4.32 m. (*GATE 1992*)

Q. 110. Which one of the following is not the requirement of an ideal regime condition in Lacey's regime-theory ?

(*a*) the discharge in the channel is constant
(*b*) the channel flows through the same soil grade, as that of the sediment entering the channel from the headworks
(*c*) the sediment grade and its amount entering the channel is constant.
(*d*) the silt grade should consist of clay sized particles.
(*e*) the flow should be uniform.

Q. 111.* An irrigation canal proposed to be excavated in an incoherent alluvium of median size 0.4 mm, for carrying a discharge of 40 cumecs, should ideally have a bed slope of :

(*a*) 1 in 6000 (*b*) 1 in 5200
(*c*) 1 in 2000 (*d*) 1 in 1265.

Q. 112.* The silt factor 'f', as used by Lacey in his regime theory, was adjusted to be equal to 1 for the silt of Upper Bari-Doab alluvium. What was the median size of this alluvium ?

(*a*) 0.28 mm (*b*) 0.32 mm
(*c*) 0.40 mm (*d*) 0.5 cm.

Q. 113. Average sediment load in a regime canal of Lacey, is of the order of :

(*a*) 50 ppm (*b*) 500 ppm
(*c*) 5000 ppm (*d*) 10,000 ppm.

Q. 114.* The discharge carried by a regime canal of bed width 22 m and water depth 2.5 m, is about;

(*a*) 22 cumecs (*b*) 34 cumecs
(*c*) 84 cumecs (*d*) none of them.

Q. 115.* Two irrigation channels, A and B, are designed on Lacey's theory to carry the same discharge. The alluvium through which canal A has to pass, however, is coarser than that for canal B. In such a design, we expect :

(*a*) channel *A* to be deeper
(*b*) channel *B* to be deeper
(*c*) channel *A* to have larger wetted perimeter
(*d*) channel *B* to have larger cross-sectional area.

Q. 116. The major drawback of Laceys regime theory, as used for the design of irrigation canals, is that :

(*a*) it does not consider the quantum of sediment load, which is likely to flow into the canal
(*b*) It is applicable only to non-cohesive soils
(*c*) it is applicable only to alluvial areas of North India or Pakistan.
(*d*) none of the above.

Q. 117. "An alluvial river scours and silts its bed automatically, depending upon the variation in the flow discharge". Which theory helps us to know this important conclusion?

(*a*) Lacey's theory (*b*) Kennedy's theory
(*c*) Einstein's theory (*d*) none of them.

Q. 118. Stable channels for a given bed-load transport, can be best designed on the basis of

(*a*) Lacey's theory (*b*) Kennedy's theory
(*c*) Einstein's theory (*d*) all of the above.

Q. 119. In an irrigation canal, berms are provided at :

(*a*) NSL (*b*) FSL
(*c*) Bank level (*d*) none of the above.

Q. 120. Counter berms are provided in an irrigation canal :

(*a*) at the outer sides of the canal banks to allow movement of cattle
(*b*) at the inner sides of the canal banks at the ground level, to make bank line and bed line parallel
(*c*) at the outer sides of the canal banks to keep the saturation gradient line well within the bank section
(*d*) none of the above.

Q. 121. Berms are provided in an irrigation canal at :

(*a*) the outer edge of the canal banks to allow movement of cattle
(*b*) the inner sides of the canal bank at the ground level, to make inner bank line and bed line parallel
(*c*) the outer sides of the canal banks, to keep the phreatic line adequately covered by earth cover
(*d*) none of the above.

Q. 122.* An alluvial canal is excavated using side slopes of $n_1 : 1$ (H : V) in excavation and $n_2 : 1$ in filling. If the depth of cutting is d_1, and top of the bank is at a height of d_2 above the bed, then the berm width required to have the bed line and bank line parallel to each other, despite fluctuations in the ground level, is :

(*a*) $(n_2 - n_1)\, d_2$ (*b*) $(n_2 - n_1)\, d_1$
(*c*) $(n_2 - n_1)\,(d_2 - d_1)$ (*d*) $n_2 \cdot d_2$.

Q. 123.* An alluvial irrigation canal is constructed with 1 : 1 (H : V) side slopes in cutting and 2 : 1 in filling, and the water depth upto FSL is expected to be y. A berm width equal to y is left initially, which with the passage of time, and when channel attains regime conditions, is likely to become :

(*a*) 1.1 *y* (*b*) 1.25 *y*
(*c*) 1.5 *y* (*d*) 2*y*.

CHAPTER 5

Q. 124. A lined alluvial canal is best designed on the basis of :

(*a*) Lacey's formula (*b*) Kennedy's formula
(*c*) Manning's formula (*d*) Continuity equation.

Q. 125. Lining of irrigation channels :

(*a*) increases water logging (*b*) increases channel cross section
(*c*) increases command area (*d*) increases chances of breaching.

Q. 126.* An irrigation canal is lined along its 20 m perimeter @ Rs. 40 per m^2 of lining. The lining has resulted in net water saving @ 3 cumecs per M-m^2 of lined area, whereas each cumec of water increases the annual crop yield by Rs. 4 lakh. Assuming the life of lining as 40 years, and the saving in maintenance cost to be balanced by the interest on investment, the benefit cost ratio for the project is :

(*a*) 1.2 (*b*) 1.0
(*c*) 0.83 (*d*) none of them.

Q. 127. The transmission loss of valuable water in an ordinary alluvial irrigation canal in a North Indian State, in cumecs per million sq m of wetted area, is of the order of :

(*a*) 0.5—0.6 (*b*) 2.5—3.5
(*c*) 25—35 (*d*) 250—350.

Q. 128. A triangular lined canal section with corners rounded by a radius equal to the full supply depth of 4 m, is likely to have its hydraulic radius, as :

(*a*) 4 m
(*b*) 3 m
(*c*) 2 m
(*d*) cannot be ascertained, as side slopes are not given.

Q. 129.* A standard triangular lined channel section with 1.5 H : 1 V side slopes has a capacity of 26 cumecs and full supply depth of 2.5 m. The lining material for the channel would be safe for withstanding mean velocity of :

(*a*) 1.8 m/s (*b*) 2.0 m/s
(*c*) 2.2 m/s (*d*) none of them.

Q. 130. Safety ladders are provided in large irrigation canals, to :

(*a*) enable the fish to pass from one place to another
(*b*) enable the cattle to cross the canal
(*c*) enable the swimmers to get out of the canal
(*d*) provide safe exit to avoid accidental drowning.

Q. 131. The main advantage offered by Bentonite layer lining in an irrigation canal is that :

(*a*) seepage loss is reduced
(*b*) cross-sectional area of the channel is reduced
(*c*) cost of land acquired is reduced
(*d*) earth work excavation is reduced.

Q. 132. The free-board in lined canals is measured between :

(*a*) FSL and top of lining
(*b*) FSL and top of canal bank
(*c*) top of lining and top of canal bank
(*d*) none of them.

133. The minimum recommended free-board for lined canals carrying discharge of more than 10 cumecs is

(*a*) 0.3 m (*b*) 0.6 m
(*c*) 0.75 m (*d*) 1.2 m.

Q. 134. Under-drainage arrangements in canals are necessarily required in :

(*a*) unlined canals
(*b*) lined canals
(*c*) both (*a*) and (*b*)
(*d*) lined canals constructed on sandy soils alone.

Q. 135. When elaborate under-drainage arrangements can not be provided, and the soil strata is sandy, the preferred material for canal lining, would be :

(*a*) cement concrete lining
(*b*) soil cement lining
(*c*) brick lining
(*d*) shotcrete lining.

Q. 136. Steel reinforcement in concrete lining is justified, mainly because :
(*a*) it augments the structural strength of uncracked lining
(*b*) it enables the lining to take tensile loads
(*c*) it reduces the width of shrinkage cracks
(*d*) none of the above.

Q. 137. Which among the following is not true about Asphaltic concrete lining?
(*a*) it is fairly cheap
(*b*) it is flexible and readily confirms to subgrade
(*c*) it permits certain type of weed growth
(*d*) it decreases the rugosity coefficient to increase channel efficiency.

CHAPTER 6

Q. 138. A land is said to be water-logged, when :
(*a*) the land is necessarily submerged under standing water
(*b*) there is a flowing water over the land
(*c*) the pH value of the soil becomes as high as 8.5
(*d*) the soil pores in the root zone get saturated with water, either by the actual watertable or by its capillary fringe.

Q. 139. Water-logging of cropped land leads to reduced crop yields, due to :
(*a*) ill-aeration of root zone, causing lack of oxygen to plants
(*b*) growth of water-loving plants interfering with the sown crop
(*c*) surrounding of the root zone by the resultant saline water, which extracts the good water from plant roots by osmosis
(*d*) all of the above.

Q. 140. Pick up the incorrect statement from the following :
(*a*) intensive irrigation should be avoided in areas susceptible to water-logging
(*b*) extensive irrigation should be adopted in areas susceptible to water logging
(*c*) lift irrigation can help alleviate water-logging susceptibilities.
(*d*) none of the above.

Q. 141. Which one of the followings does not contribute to water logging?
(*a*) inadequate drainage
(*b*) seepage from unlined canals
(*c*) frequent flooding
(*d*) excessive tapping of ground water.

Q. 142. Which one of the followings, is not a remedial measure for water logging?
(*a*) good drainage for irrigated land
(*b*) conjunctive use of water in the basin
(*c*) lining of canals and water courses
(*d*) contour bunding.

Q. 143. Pick up the incorrect statement :
(*a*) moderately saline lands are represented by visible whiteish deposits of salts on the top and upper layers of soil
(*b*) $Na_2 CO_3$ is the most harmful salt for causing salinity
(*c*) Nacl is the least harmful salt for causing salinity
(*d*) a highly saline land is called an alkaline land, or a black alkali.
(*e*) none of the above.

Q. 144. Point out the incorrect statement, out of the following :
(*a*) salinity is caused by water-logging
(*b*) water logging is not caused by salinity
(*c*) salinity subsides, when once the water logging is removed
(*d*) none of the above.

Q. 145. Alkaline soils are best reclaimed by :
(*a*) leaching
(*b*) addition of gypsum to soil
(*c*) providing good drainage
(*d*) addition of gypsum to soil and leaching.

Q. 146. A recently reclaimed alkaline soil should preferably be sown with a salt resistant crop, like :
(*a*) wheat (*b*) cotton
(*c*) barseem (*d*) any of the above.

Q. 147.* A soil has an Exchangeable Sodium Percentage (ESP) of 16%, and its Electrical conductivity (EC) is 32 milli-mhos/cm. If the pH of this soil is 9.5, then the soil is classified as :
(*a*) alkaline soil (*b*) saline soil
(*c*) saline alkali soil (*d*) none of the above.

Q. 148. The soil becomes practically infertile when its pH value is about :
(*a*) 0 (*b*) 7
(*c*) 11 (*d*) none of them.

Q. 149. Which of the following crops can withstand the highest water table, which normally should be 0.7 to 2.5 m below the level of the cropped land?
(*a*) wheat (*b*) lucerine
(*c*) cotton (*d*) rice.

Q. 150. While growing crops in high water-table areas, open jointed drains, discharging into outlet drains, are sometimes laid below the cropped land to lower down the water-table and to remove the rain water effectively. These drains are called:
(*a*) tile drains (*b*) french drains
(*c*) gravity outlets (*d*) surface drains.

Q. 151. A tile drain is laid below a cropped land to remove excess irrigation water. The Drainage Coefficient of this drain, is usually expressed as;
(*a*) cm of water depth removed from the drainage area per day
(*b*) cum of water removed per second
(*c*) percentage of applied water, which is intercepted by this drain
(*d*) none of the above.

Q. 152. When a sub-surface open jointed drain is laid below a farm land to intercept excess irrigation water, then its capacity, depending upon soil type, should be of the order of :
(*a*) 5—10% of applied water
(*b*) 10—50% of applied water
(*c*) 50—80% of applied water
(*d*) none of them.

Q. 153. The method, which uses dead furrows on cropped farms for drainage of excess irrigation or rain water, is called :
(*a*) surface inlet (*b*) tile drainage
(*c*) bedding (*d*) french drain.

CHAPTER 7

250 Objective questions on this chapter are given in author's another book, titled *"Hydrology and Water Resources Engineering"*, which deals with the subject matter in greater details than given in this book. Students are advised to refer to that volume to be well versed with the subject on 'Hydrology', which in itself, is a full subject, other than from 'Irrigation'.

CHAPTER 8

Q. 154. Ghaggar river which appears and disappears in Rajasthan State of India, can be classified as a

(*a*) meandering river (*b*) deltaic river
(*c*) virgin river (*d*) none of the above.

Q. 155. The quantum of river discharge, which is large enough in magnitude as well as in frequency, so as to cause controlling effect on the river course and cross-section, is known as :

(*a*) design discharge (*b*) median discharge
(*c*) maximum flood discharge
(*d*) dominant discharge.

Q. 156. The ratio of dominant discharge for a river to the peak discharge for that river, is of the order of :

(*a*) 3/16 (*b*) 9/16
(*c*) 13/16 (*d*) 3/2.

Q. 157. The 'meander length' for an alluvial river is :

(*a*) the total channel length along its looped course
(*b*) the total channel length minus the direct straight length
(*c*) the axial length of one meander
(*d*) the looped length of one meander
(*e*) none of the above.

Q. 158. The 'meander belt' for an alluvial river is :

(*a*) the total river width between embankments
(*b*) the width between the outer edges of fully developed meander loop, measured perpendicular to river axis
(*c*) the same as meander width
(*d*) both (*b*) and (*c*).

Q. 159. For a meandering alluvial river, the ratio of its 'channel length' to 'direct axial length' is always :

(*a*) 1 (*b*) > 1
(*c*) < 1
(*d*) may be less or more than 1, depending upon the river.

Q. 160. In a meandering river, the ratio of 'arcual channel length' to 'direct axial length', is called :

(*a*) tortuosity (*b*) inverse of tortuosity
(*c*) cut off ratio (*d*) none of these.

Q. 161. Tortuosity in a meandering river, is :

(*a*) 1 (*b*) < 1
(*c*) > 1 (*d*) none of these.

Q. 162. A river reach having tortuosity of 1.2, can be said to have :
(*a*) 20% tortuosity of meander
(*b*) 80% tortuosity of meander
(*c*) 120% tortuosity of meander
(*d*) none of the above.

Q. 163. In an alluvial meandering river in flood plains, the meandering length (M_L) is x times the meandering width (M_B), where x is roughly equal to :
(*a*) 3 (*b*) $\frac{1}{3}$
(*c*) 1 (*d*) $\frac{1}{3}$ to 3.

Q. 164. For a meandering alluvial river in flood plain, the meander length is about:
(*a*) 6 W (*b*) 18 W
(*c*) 17 W (*d*) W.
where *W* is the normal river width

Q. 165. For a meandering alluvial river in flood plain, the meander width is about:
(*a*) 0.5 W (*b*) W
(*c*) 6 W (*d*) 17 W.
where *W* is the normal river width

Q. 166.* An alluvial river has a dominant discharge of 1600 m^3/s and a bed slope of 1 in 5000. The approximate value of meander belt for this river, would be :
(*a*) 0.6 km (*b*) 6 km
(*c*) 60 km (*d*) none of these.

Q. 167. An alluvial meandering river is likely to carry a peak discharge of 6000 cumecs. The extent of meandering belt in this river, estimated in km per km of meandering length, roughly equals.
(*a*) $\frac{1}{3}$ (*b*) 3
(*c*) 9 (*d*) 27.

Q. 168. Meander ratio in an alluvial meandering river is given by :
(*a*) $\frac{\text{meander length}}{\text{meander width}}$ (*b*) $\frac{\text{meander width}}{\text{meander length}}$
(*c*) $\frac{\text{meander width}}{\text{meander length}} \times 100$ (*d*) none of them.

Q. 169.* The meander belt for an alluvial stream is estimated to be 3 km. The peak discharge in this river will roughly be of the order of :
(*a*) 4000 cumecs (*b*) 700 cumecs
(*c*) 1200 cumecs (*d*) 1600 cumecs.

Q. 170. In a meandering river reach, the deepest river portions will be available at:
(*a*) the crossings
(*b*) the inner edges of meander loops
(*c*) the outer edges of meander loops
(*d*) none of them.

Q. 171. Sinuosity of a meandering river is :
(*a*) the same as tortuosity (*b*) inverse of tortuosity
(*c*) log of tortuosity (*d*) none of them.

Q. 172. Pinpoint the incorrect statement :

(*a*) guide-bunds are constructed to guide the river flow through a smaller width
(*b*) guide bunds are generally extended downstream of the hydraulic structure, in a length of about twice their upstream length
(*c*) guide banks prevent the out-flanking of the hydraulic structure by the changing river course
(*d*) aggrading rivers can be best controlled by soil conservation measures, including construction of check dams on their tributaries
(*e*) degrading rivers can be best controlled by building of cross-bars, weirs, etc., prior to other river training works.

Q. 173. Aggrading rivers are :

(*a*) silting rivers (*b*) scouring rivers
(*c*) rivers in regime (*d*) meandering rivers.

Q. 174. The river reach upstream of a newly built dam may behave, as :

(*a*) aggrading (*b*) degrading
(*c*) virgin (*d*) none of them.

Q. 175. A barrage with 600 m length is designed across an alluvial river. The U/S and D/S lengths of the proposed guide banks of this barrage, will be about:

(*a*) 750 m and 1500 m (*b*) 750 m and 750 m
(*c*) 750 m and 150 m (*d*) 750 m and 300 m

Q. 176.* A pair of guide banks are to be constructed for an alluvial river at 400 m apart at a bridge site. The design discharge of the river is 5000 cumecs at 5 m water depth of HFL above the average bed level of the river. These guide banks need to be stone pitched on water side, with apron laid from the toe in the river bed. The average estimated depth and width of this apron, respectively, will be :

(*a*) 2 m and 10 m (*b*) 1.5 m and 5 m
(*c*) 2 m and 20 m (*d*) 3 m and 20 m.

Q. 177. The repelling groynes which are largely constructed projecting from river embankments, as anti-erosion works, are :

(*a*) pointing upstream (*b*) pointing downstream
(*c*) perpendicular to the bank (*d*) none of these.

Q. 178.* If a river in an alluvial plain has a dominant discharge of 1600 cumecs, the waterway for this bridge, in the absence of model testing, can be safely adopted, as equal to :

(*a*) 190 m (*b*) 228 m
(*c*) 300 m (*d*) 480 m.

Q. 179. The upstream angle of inclination of a repelling groyne with normal to the bank line, is of the order of :

(*a*) 5 to 10° (*b*) 10 to 30°
(*c*) 30 to 50° (*d*) 70 to 90°.

Q. 180. Which one among the following is not the real cause of an alluvial river following a serpentine course?

(*a*) variation of discharge
(*b*) availability of steeper land gradient than what is required for equilibrium.
(*c*) availability of inadequate land gradient than what is required for equilibrium
(*d*) none of these.

Q. 181. An alluvial river increases its length by meandering due to;
(*a*) variation of discharge (*b*) variation in land topography
(*c*) both (*a*) and (*b*) (*d*) none of the above.

Q. 182. The factor which is not primarily responsible for meander in an alluvial river, is;
(*a*) variation of discharge (*b*) excess silt load
(*c*) deficient silt load (*d*) none of them.

Q. 183. The secondary factor, which is responsible for meandering in an alluvial river, is :
(*a*) inadequate land gradient (*b*) deficient silt load
(*c*) constant discharge (*d*) none of them.

Q. 184. Point out the incorrect statement in relation to the behaviour of a meandering alluvial river :
(*a*) Primary cause for meandering is the silting action due to excess sediment at the available discharge and surface slope
(*b*) Secondary cause for meandering is the scouring action due to deficiency of sediment at the available discharge and surface slope
(*c*) Meandering tendency is more in an alluvial river during floods, because of high variation in discharge
(*d*) if there is no variation of discharge in an alluvial river, there will be no meandering
(*e*) none of them.

Q. 185. Out of the following choices given below, choose impermeable spur(s) :
(*a*) an earthen spur protected by stone apron
(*b*) an earthen spur, unprotected by stone apron
(*c*) a balli spur (*d*) a tree groyne
(*e*) (*a*) and (*b*) both (*f*) (*c*) and (*d*) both.

Q. 186. Permeable spurs are best suitable for rivers, which :
(*a*) carry heavy suspended load
(*b*) carry large bed load, but light suspended load
(*c*) need permanent protection to dikes
(*d*) need attracting the river current, for providing deeper channel
(*e*) flow in upper hilly reaches.

Q. 187. Denehey's spur is :
(*a*) a hockey-shaped earthen spur
(*b*) a *T*-shaped stone spur, as used in Australia
(*c*) a *T*-shaped earthen spur, as used in India
(*d*) a type of a balli spur, especially developed for Indian rivers.

Q. 188. Which one of the following effects can not be attributed to have been caused by the construction of dikes along a river coarse?
(*a*) faster travel of a flood wave
(*b*) higher flood levels along the river
(*c*) increased peak discharges all along downstream points
(*d*) decrease in surface slope of the river above the leveed reach
(*e*) increased bed levels in the river
(*f*) reduction in meander belt, causing steeper hydraulic gradients.

Q. 189. Which one of the following effects produced by a Cut off in an alluvial river, is not an advantage to navigation :

(*a*) shortened route and elimination of sharp bends
(*b*) shortened travel time, particularly at low and moderate river stages
(*c*) increased water depth at low river stages
(*d*) lowering of the flood stages and flood periods.

CHAPTER 9

Q. 190. Barrages constructed across alluvial rivers help in :

(*a*) controlling floods (*b*) restoring river regime
(*c*) ensuring monsoon storage (*d*) all of them.

Q. 191. Which one among the following is a correct choice in relation to a weir?

(*a*) it is helpful in diverting excess water to a river from a canal
(*b*) it does not cause any heading up of water on its upstream side
(*c*) it stores water by raised counter-balanced gates
(*d*) it increases the chances of floods in the upstream areas
(*e*) none of them.

Q. 192. If two canals are taken off from both the flanks of a river at the site of a diversion headwork, then the number of undersluices and divide walls, will respectively, be :

(*a*) 1 and 1 (*b*) 1 and 2
(*c*) 2 and 1 (*d*) 2 and 2.

Q. 193. Pinpoint the incorrect statement :

(*a*) the old Okhla barrage near Delhi has recently been replaced by a new weir
(*b*) the cost of a barrage is usually higher than that of a weir
(*c*) a weir can pond up additional water on its upstream side, by using shutters on its top
(*d*) a barrage does not increase the susceptibility of flooding in upstream areas
(*e*) none of them.

Q. 194. In a barrage project, a divide wall is provided to :

(*a*) separate the lower crest 'undersluice side' from the higher crest 'weir side'
(*b*) separate the higher crest 'undersluice side' from the lower crest 'weir side'
(*c*) keep the cross-currents away from the barrage body
(*d*) serve none of the above purposes.

Q. 195. A canal headworks has nothing to do with a :

(*a*) weir (*b*) guide bank
(*c*) head regulator (*d*) safety ladder.

Q. 196. In a diversion headworks project, the canal head regulator is usually aligned:

(*a*) parallel to the barrage axis
(*b*) perpendicular to the divide wall
(*c*) parallel to the divide wall
(*d*) none of them.

Q. 197. The choice among the following, which does not control the discharging capacity to be provided for the undersluices, is :
(*a*) that it should be at least double the canal discharge
(*b*) that it should be about 10 to 15% of maximum flood discharge
(*c*) that it should be able to pass winter freshets and low floods.
(*d*) that it should be equal to the dominant discharge of the river
(*e*) none of them.

Q. 198. Anadromous fish often move large distances in rivers in India :
(*a*) to upstream only
(*b*) to downstream only
(*c*) to downstream in winter, and upstream in summer
(*d*) to upstream in winter, and downstream in summer.

Q. 199. A fish ladder is provided in a canal project :
(*a*) to catch the fish for commercial development
(*b*) to enable the fish to move freely in the river
(*c*) to serve the same purpose as a canal ladder.
(*d*) (*b*) and (*c*) both.

Q. 200. Head sluices are the gate controlled openings, in :
(*a*) the entire length of the barrage
(*b*) the under-sluice length of the barrage
(*c*) the regulator of the main off taking canal
(*d*) none of them.

Q. 201. Silt excluders are constructed :
(*a*) on the river bed downstream of the head regulator
(*b*) on the river bed upstream of the head regulator
(*c*) on the canal bed downstream of the canal head regulator
(*d*) none of these.

Q. 202. The silt exclusion device, constructed on the bed of the main canal, taking off from a headwork, is called :
(*a*) silt excluder (*b*) silt ejector
(*e*) both (*a*) and (*b*) (*d*) none of them.

Q. 203. The tunnel openings provided infront of a canal head regulator at a Diversion headworks :
(*a*) discharge sedimented water into the canal
(*b*) discharge sediment load into the under-sluices, from where it ejects out to the downstream river
(*c*) discharge clear water into the canal
(*d*) none of these.

Q. 204. Canal headworks in the upper rocky reaches of the rivers, are uncommon, because :
(*a*) more drops are required on the resulting canal system.
(*b*) costly head regulator is required
(*c*) more cross-drainage works are required on the resulting canal network
(*d*) all of the above.

CHAPTER 10

Q. 205. If y_1 and y_2 are water depths upstream and downstream of a hydraulic jump respectively, the loss of energy due to hydraulic jump is :

(*a*) $(y_2-y_1)^4/4y_1y_2$ (*b*) $(y_2-y_1)^3/4y_1y_2$

(*c*) $(y_2-y_1)^2/4y_1y_1$ (*d*) $(y_2-y_1)/4y_1y_2$. (GATE 1990)

Q. 206. A hydraulic jump involves;

(*a*) subcritical flow (*b*) super critical flow

(*c*) critical flow (*d*) all of the above.

Q. 207. A hydraulic jump ensures :

(*a*) change of subcritical flow to super critical flow

(*b*) change of super critical flow to subcritical flow

(*c*) change of subcritical flow to critical flow

(*d*) change of super critical flow to critical flow.

Q. 208. The phenomenon of hydraulic jump leads to :

(*a*) evolution of energy

(*b*) dissipation of energy

(*c*) sometimes (*a*) and sometimes (*b*)

(*d*) none of these.

Q. 209. If a vertical line is drawn on the specific energy curve, corresponding to a particular value of specific energy, then the two depth values are obtained which are known as :

(*a*) initial and sequent depths (*b*) alternate depths

(*c*) conjugate depths (*d*) all of these.

Q. 210. The sequent depth is always :

(*a*) less than the alternate subcritical depth

(*b*) more than the alternate subcritical depth

(*c*) equal to the alternate subcritical depth

(*d*) none of the above.

Q. 211. The formula connecting initial depth (y_1) with sequent depth (y_2) in a hydraulic jump, is given as :

(*a*) $y_2=\frac{-y_1}{2}+\sqrt{\frac{y_1^2}{4}+\frac{2q^2}{gy_1}}$ (*b*) $y_2=\frac{y_1}{2}+\sqrt{\frac{y_1^2}{4}+\frac{2q^2}{gy_1}}$

(*c*) $y_2=\frac{-y_1}{2}+\sqrt{\frac{y_1^2}{4}+\frac{q^2}{gy_1}}$ (*d*) $y_2=\frac{y_1}{2}+\sqrt{\frac{y_1^2}{4}+\frac{q^2}{gy_1}}$

where q is the discharge intensity in the channel at the section of the hydraulic jump.

Q. 212. The hydraulic jump that develops usually in barrages and canal head regulators, is of the type :

(*a*) strong jump (*b*) steady jump

(*c*) oscillating and weak jump (*d*) undular jump.

Q. 213. The formation of a hydraulic jump on a sloping glacis, as compared to that on a horizontal floor, is always :

(*a*) more definite and more efficient

(*b*) more definite and less efficient

(*c*) less definite and more efficient

(*d*) less definite and less efficient.

CHAPTER 11

Q. 214. The safety of a hydraulic structure founded on pervious foundation can be ensured :

(*a*) by providing sufficient length of its concrete floor
(*b*) by providing sufficient depth of its concrete floor
(*c*) by providing a downstream cutoff of some reasonable depth
(*d*) all of the above.

Q. 215. Point out the correct meaning of 'piping', as applied to the design of barrages:

(*a*) it refers to a network of pipes laid below the hydraulic structure to remove the seeping water
(*b*) it refers to the process of undermining of foundation and creation of hollows therein
(*c*) it refers to the uplift force caused by seeping water on the floor of the hydraulic structure
(*d*) none of the above.

Q. 216. While designing the downstream floor thickness of a weir, the ordinate of the uplift pressure at a point is 2.8 m. If the relative density of concrete is taken to be 2.4, then the minimum thickness of the floor to be provided for resisting uplift pressure, without accounting safety factor, is :

(*a*) 1.16 m (*b*) 2 m
(*c*) 0.8 m (*d*) none of them.

Q. 217.* While designing the floor thickness of a weir in its upstream length, the average head causing uplift is 2.8 m. The provided floor thickness here, with suitable factor of safety, could best be :

(*a*) 2 m (*b*) 2.5 m
(*c*) 0.8 m (*d*) 4.5 m.

Q. 218. The critical exit gradient suggested in Khosla's theory of design of weirs and barrages, is :

(*a*) less for more porous soils (*b*) more for more porous soils
(*c*) equal for all kinds of soils (*d*) none of them.

Q. 219.* Khosla's safe exit gradient for design of weirs will be the lowest for the soil type :

(*a*) fine sand (*b*) coarse sand
(*c*) shingle and gravels (*d*) none of them.

Q. 220. Point out the incorrect statement in relation to the design of weirs and barrages :

(*a*) Bligh's safe hydraulic gradient is the same as the Khosla's safe exit gradient
(*b*) the first streamline below a barrage section follows the bottom profile of the section
(*c*) equipotential lines are the lines joining the points of equal residual seepage head
(*d*) none of them.

Q. 221. Which one of the following factors, is not to be considered in the design of major hydraulic structures founded on an alluvial North Indian River?

(*a*) uplift caused by steady seepage
(*b*) piping caused by steady seepage
(*c*) uplift caused by high flood flow in the jump trough
(*d*) none of the above.

Q. 222. Bligh's theory, as applied to the design of weirs and barrages on permeable foundations, account for :

(*a*) hydrostatic forces only
(*b*) hydrodynamic forces only
(*c*) both (*a*) and (*b*)
(*d*) none of them.

Q. 223. Retrogression is :

(*a*) the back water effect of a weir
(*b*) the raising of the river bed upstream of the weir, during initial years of its construction
(*c*) the lowering of the river bed downstream of the weir, during initial years of its construction
(*d*) none of them.

Q. 224. According to Khosla's theory of independent variables for seepage below a hydraulic structure, the exit gradient, in the absence of a downstream sheet pile, is :

(*a*) 0 (*b*) 1
(*c*) ∞ (*d*) none of them.

Q. 225. The value of Khosla's critical exit gradient for usually met alluvial sandy soils of our country, is about :

(*a*) 0 (*b*) 1
(*c*) ∞ (*d*) $\frac{1}{4}$ to $\frac{1}{6}$.

Q. 226. The value of Khosla's safe exit gradient for usually met alluvial river soils of our country is :

(*a*) 0 (*b*) 1
(*c*) ∞ (*d*) $\frac{1}{4}$ to $\frac{1}{6}$.

Q. 227. Critical exit gradient, as enunciated in Khosla's theory of design of weirs and barrages on pervious foundations, is :

(*a*) the left out pressure in the seeping water at the downstream end point, where it emerges out on the river bed
(*b*) the rate of loss of pressure of the seeping water at the downstream emerging point, which is just enough to lift the soil grains at that point
(*c*) the actual pressure gradient of the seepage, at the down-stream emerging point
(*d*) none of the above.

Q. 228. Safe exit gradient, as enunciated by Khosla's theory of design of weirs and barrages, is :

(*a*) the actual seepage gradient available at the downstream exit point
(*b*) the seepage gradient at the downstream exit point, equalling the submerged weight of a unit volume of soil there
(*c*) about $\frac{1}{4}$ to $\frac{1}{6}$ times the gradient which is just able to lift the soil grain at the downstream exit point
(*d*) none of the above.

Q. 229. The critical exit gradient, as applicable to barrages, is expressed by :

(*a*) $w(1-n)(S_s-1)$,

where w = unit weight of water
n = porosity of soil
S = specific gravity of soil grains

(*d*) $\frac{H}{d} \cdot \frac{1}{\pi\sqrt{\lambda}}$

where H, d, and λ have their usual meaning

(*c*) $\frac{\text{Safe exit gradient}}{\text{Factor of safety}}$

(*d*) none of the above.

Q. 230. The minimum thickness(t) of the downstream floor, as required in the design of weirs, can be expressed by the equation :

(*a*) $\frac{h}{G+1}$ (*b*) $\frac{h}{G-1}$

(*c*) $\frac{h-t}{G-1}$ (*d*) $1.33\left(\frac{h}{G-1}\right)$

where h = ordinate of hydraulic gradient line above the top of the floor
G = Specific gravity of floor material.

Q. 231. The back water effect of a weir is best called :

(*a*) retrogression (*b*) afflux
(*c*) back water curve (*d*) none of them.

Q. 232. A breast wall is usually provided :

(*a*) in the weir section
(*b*) in the under-sluice section
(*c*) in the main canal section
(*d*) in the head regulator section.

Q. 233. Just downstream of the pucca concrete floor of a barrage section, an inverted filter covered by c.c. blocks is laid in a length of about 1.5 to 2 D, where D is :

(*a*) the Lacey's normal scour depth
(*b*) 1.5 times the Lacey's normal scour depth
(*c*) 1.5 times the Lacey's scour depth minus the downstream water depth
(*d*) none of the above.

Q. 234. Just downstream of the concrete floor of a weir section, an inverted filter covered by c.c. blocks is laid. This filter should be such, as :

(*a*) to let out the residual seepage
(*b*) not to allow the soil particles to escape
(*c*) not to get clogged in itself
(*d*) all of the above.

Q. 235. The safety against any possible scour, on upstream or downstream side of the pucca floor of a hydraulic structure, is usually ensured by laying :

(*a*) inverted filter (*b*) toewall
(*c*) rock toe (*d*) stone apron.

Q. 236. For the head regulator, the most severe condition of uplift pressure on the floor occurs when :

(*a*) the flow in the river is at flood level and canal is running at full supply depth

(*b*) the canal runs dry and the river flow is at high flood level

(*c*) the canal runs at full supply depth and the river flow is at pond level

(*d*) the canal runs dry and the river flow is at pond level. (*GATE 1992*)

CHAPTER 12

Q. 237. Canal drops are required to :

(*a*) dissipate excess energy

(*b*) dissipate inadequate land slope

(*c*) dissipate excess land slope

(*d*) none of the above.

Q. 238. A trapezoidal notch fall can maintain normal water depth in the upstream channel :

(*a*) at any one given value of the design discharge

(*b*) at all the discharges

(*c*) at any two values of the design discharges

(*d*) at no discharge at all.

Q. 239. The depth-discharge relationship of the upstream canal remains practically unaffected by the introduction of a fall of the type :

(*a*) Ogee fall

(*b*) Sarda type vertical fall

(*c*) Trapezoidal notch fall

(*d*) none of the above.

Q. 240. For low to moderate discharges of the order of 10-15 cumecs and fall height of 1 to 1.5 m, cheaper Sarda type canal falls can be recommended :

(*a*) universally, without any if and buts

(*b*) if the canal runs with highly variable discharge

(*c*) when the canal fall is not to be used as a meter and discharge is fairly constant

(*d*) when the canal fall is not to be used as a meter, irrespective of variation in discharge.

Q. 241. The type of canal fall, which can be universally used without any if and buts for moderate discharges of the order of 40-60 cumecs and low fall heights of 1 to 1.5 m, is :

(*a*) vertical drop fall (*b*) ogee fall

(*c*) glacis fall (*d*) baffle fall.

Q. 242. The type of fall, which you may recommend for very high drops and very low discharges, is :

(*a*) Sarda type fall (*b*) Siphon well drop

(*c*) Straight glacis fall (*d*) Inglis fall.

Q. 243. The canal fall, involving parabolic glacis, is called :

(*a*) Straight glacis fall (*b*) Glacis fall

(*c*) Inglis fall (*d*) Montague fall.

Q. 244. An Inglis fall, also called a Baffle fall, can be recommended for all discharges, provided :
(*a*) the fall is more than 1.5 m
(*b*) the fall is undrowned
(*c*) the fall is either flumed or unflumed
(*d*) all of the above.

Q. 245. The energy dissipation in a Sarda type canal drop is caused by :
(*a*) hydraulic jump (*b*) friction blocks
(*c*) water pool (*d*) baffle wall.

Q. 246. The best energy dissipation on the down-stream side of a canal drop, is caused in :
(*a*) Sarda type fall (*b*) Glacis fall
(*c*) Ogee fall (*d*) Montague fall.

Q. 247. A baffle wall and a water pool to dissipate energy, was introduced as an improvement, in a :
(*a*) Glacis fall (*b*) Montague fall
(*c*) Inglis fall (*d*) none of the above.

Q. 248. A meter fall of 1.8 m height is to be constructed on a canal carrying a discharge of 50 cumecs. The type of fall that you may recommend will be :
(*a*) Glacis fall (*b*) Montague fall
(*c*) Inglis fall (*d*) none of the above.

Q. 249. The energy dissipation in an Inglis fall is caused by :
(*a*) a pool of water (*b*) a hydraulic jump
(*c*) neither (*a*) nor (*b*) (*d*) both (*a*) and (*b*).

Q. 250. The length of the water cistern to be provided in a Sarda type fall, is :
(*a*) $5\,(y_2 - y_1)$ (*b*) $5\sqrt{H \cdot H_L}$
(*c*) $\frac{1}{4}(H \cdot H_L)^{2/3}$ (*d*) $1.84 \cdot H^{3/2}$.

where y_1, y_2, H, H_L have their usual meaning.

Q. 251. The concrete glacis of a glacis fall, usually slopes H : V, as :
(*a*) 2 : 1 (*b*) 1 : 2
(*c*) 1 : 5 (*d*) 5 : 1.

Q. 252. In a glacis fall, the hydraulic jump forms on :
(*a*) the horizontal floor (*b*) the sloping glacis
(*c*) either (*a*) or (*b*) (*d*) none of the above.

Q. 253. Friction blocks on the downstream side of the pucca canal fall, can be used to dissipate the left out energy in the falling water, in case of :
(*a*) a glacis fall (*b*) a vertical drop fall
(*c*) a baffle fall (*d*) (*a*) and (*b*) both
(*e*) (*a*), (*b*) and (*c*) all.

Q. 254. A trapezoidal crest in a Sarda type canal fall is preferred and used in comparison to a rectangular crest, when :
(*a*) the discharge is less than 14 cumecs
(*b*) the discharge is equal to or more than 14 cumecs
(*c*) the discharge is less than 30 cumecs
(*d*) the discharge is equal to or more than 30 cumecs.

Q. 255. A cylinder fall, popularly known as syphon well drop, is suitable and economical for :

(*a*) low discharges and low drops
(*b*) low discharges and high drops
(*c*) high discharges and low drops
(*d*) high discharges and high drops.

Q. 256. In a baffle fall, the hydraulic jump forms, on :

(*a*) the sloping glacis (*b*) the horizontal baffle platform
(*c*) either (*a*) or (*b*) (*d*) none of the above.

Q. 257. A baffle wall, as a sort of a low weir, is constructed in a baffle type canal fall at the end of the baffle platform and before the water cistern, inorder to head up water to its upstream side to :

(*a*) ensure formation of hydraulic jump to dissipate energy
(*b*) withstand actual impact of the high velocity jet to dissipate energy
(*c*) both (*a*) and (*b*)
(*d*) none of the above.

CHAPTER-13

Q. 258. The canal regulator, which is constructed at a diversion headworks, is called a :

(*a*) cross regulator (*b*) distributary head regulator
(*c*) canal module (*d*) none of the above.

Q. 259. The gated regulator, which is constructed in the parent canal near the site of an offtaking canal, is called a :

(*a*) canal head regulator (*b*) distributary head regulator
(*c*) cross regulator (*d*) none of the above.

Q. 260. A cross regulator helps in

(*a*) increasing supply in the parent channel downstream
(*b*) increasing supply in the offtaking channel
(*c*) increasing water depth in the parent canal, upstream
(*d*) both (*b*) and (*c*)
(*e*) all (*a*), (*b*) and (*c*) above.

Q. 261. Point out the choice among the following, which is not a function of a distributary head regulator :

(*a*) it serves as a meter for measuring discharge in the offtaking canal
(*b*) it serves to control silt entry into the offtaking canal
(*c*) it helps in controlling and regulating supplies in the entire downstream canal network.
(*d*) it helps in controlling supplies in the offtaking canal.
(*e*) none of the above.

Q. 262. Khosla's theory of independent variables is used in the design of :

(*a*) weirs and barrages (*b*) cross regulators and head regulators
(*c*) modules (*d*) both (*a*) and (*b*) above
(*e*) all (*a*), (*b*) and (*c*) above.

Q. 263. The channel constructed to bypass the excess water entering a canal, is called a :

(*a*) canal module (*b*) canal siphon
(*c*) canal escape (*d*) canal regulator.

Q. 264. The crest level of a distributary head regulator is usually kept :
(*a*) the same as the upstream bed level of the parent canal
(*b*) the same as the crest level of the cross-regulator at the site.
(*c*) 0.3 to 1.0 m higher than the upstream bed level of the parent canal
(*d*) none of the above.

Q. 265. The regulator constructed on a major distributary near its junction with a minor distributary, will be called a :
(*a*) distributary head regulator
(*b*) cross-regulator
(*c*) regulating head
(*d*) none of the above.

Q. 266. The arrangement made in a canal network, which acts as its safety valve, is:
(*a*) cattle crossing (*b*) canal ladder
(*c*) canal escape (*d*) canal module.

Q. 267. The arrangement provided in a canal to help the cattle safely cross the canal, is called a :
(*a*) canal ladder (*b*) canal crossing
(*c*) cattle crossing (*d*) all of the above.

Q. 268. Point out the correct statement :
(*a*) bed bars are constructed in canals at the toe of the canal lining, to prevent slippage of lining.
(*b*) cattle crossings, besides their usual function of helping the cattle to cross the canal, also provide safety to those who are swept away by currents.
(*c*) a canal escape helps in supplying irrigation water to the downstream water courses
(*d*) none of the above.

Q. 269. Canal modules help in :
(*a*) modulating and varying the canal discharge
(*b*) releasing desired quantity of water into the water courses
(*c*) releasing desired quantity of water into the minors
(*d*) all of the above.

Q. 270. Canal outlets are also called :
(*a*) canal escapes (*b*) canal modules
(*c*) canal offtakes (*d*) canal openings.

Q. 271. The type of irrigation module, which makes equitable distribution of water more difficult, is a :
(*a*) non-modular outlet (*b*) semi-module
(*c*) rigid module (*d*) none of them.

Q. 272. A good irrigation module is the one, which :
(*a*) draws heavy silt from the canal
(*b*) draws clear water from the canal
(*c*) draws fair share of silt from the canal
(*d*) none of the above.

Q. 273. If the rate of change of discharge from an irrigation outlet is equal to the rate of change of discharge in the distributary, then the outlet is called :
(*a*) flexible (*b*) proportional
(*c*) sensitive (*d*) none of these.

Q. 274. The rate of change of discharge through an irrigation outlet becomes equal to the rate of change of water depth in the channel, when its :

(*a*) flexibility is 1 (*b*) sensitivity is 1
(*c*) setting is 1 (*d*) sensitivity is zero.

Q. 275. If the sensitivity of an irrigation module is $\frac{1}{2}$, then 50% variation in canal water depth will cause x% variation in outlet discharge, where x is :

(*a*) 100% (*b*) 50%
(*c*) 25% (*d*) 12.5%.

Q. 276. A fully modular canal outlet has its :

(*a*) sensitivity = 1, and flexibility = 1
(*b*) sensitivity = 1, and flexibility = 0
(*c*) sensitivity = 0, and flexibility = 1
(*d*) sensitivity = 0, and flexibility = 0.

Q. 277. An irrigation outlet is said to be proportional, when its :

(*a*) sensitivity = 1 (*b*) flexibility = 1
(*c*) setting = 1 (*d*) all of the above.

Q. 278. If the percentage rise/fall in the outlet discharge of a canal distributary is more than the percentage rise/fall of discharge in the distributary, then the outlet is :

(*a*) super sensitive (*b*) sub sensitive
(*c*) hyper proportional (*d*) sub proportional.

Q. 279. An irrigation, outlet fixed at a height of 0.4 y above the canal DBL, where y is the full supply depth in the canal, will have its setting equal to :

(*a*) 0.4 (*b*) 0.6
(*c*) 0.2 (*d*) none of these.

Q. 280. An irrigation outlet is said to be proportional, when its :

(*a*) setting = outlet index/channel index
(*b*) setting = channel index/outlet index
(*c*) setting = channel index × outlet index
(*d*) none of the above.

Q. 281. For a wide trapezoidal channel, the channel index is :

(*a*) 2/3 (*b*) 5/3
(*c*) 1/3 (*d*) none of them.

Q. 282. A free pipe outlet is a :

(*a*) rigid module (*b*) flexible module
(*c*) non-modular module (*d*) all of the above.

Q. 283. The type of irrigation module, which is usually considered as the best in our country, is :

(*a*) Adjustable orifice semi module
(*b*) Punjab open flume outlet
(*c*) Khanna's rigid module
(*d*) Kennedy's gauge module.

Q. 284. The only irrigation semi module, through which the discharge is not proportional to $\sqrt{H}$, where H is the head causing flow through the module, is :

(*a*) kennedy's gauge outlet (*b*) open flume outlet
(*c*) free pipe outlet (*d*) adjustable orifice semi module.

CHAPTER 14

Q. 285. The hydraulic structure which is constructed at the junction of a canal and a drain to dispose of drainage without disturbing canal supplies, is known as a;

(*a*) cattle crossing (*b*) canal module
(*c*) canal regulator (*d*) canal crossing.

Q. 286. The type of canal alignment, which involves maximum cross-drainage works, is a :

(*a*) ridge canal (*b*) contour canal
(*c*) side slope canal (*d*) all of the above.

Q. 287. The irrigation structure which is constructed at the crossing of a canal and a drain, to avoid the mixing of their respective discharges, is :

(*a*) an aqueduct (*b*) a super passage
(*c*) a level crossing (*b*) (*a*) and (*b*) both
(*e*) (*a*), (*b*) and (*c*) all.

Q. 288. When an irrigation canal is taken over a drainage channel, the crossing is called :

(*a*) an aqueduct (*b*) a super passage
(*c*) a level crossing (*d*) none of them.

Q. 289. An irrigation canal, freely flowing under a drainage channel, is specifically called a :

(*a*) canal junction (*b*) canal crossing
(*c*) canal siphon (*d*) super passage.

Q. 290. An irrigation canal flowing freely above a drainage, which in turn is flowing under pressure, is specifically, called a :

(*a*) canal siphon (*b*) canal aqueduct
(*c*) siphon aqueduct (*d*) super passage.

Q. 291. The crossing arrangement, preferably made at the junction of a huge canal and a river stream carrying short lived-high flood discharge at almost equal bed levels, is a :

(*a*) super passage (*b*) aqueduct
(*c*) level crossing (*d*) canal siphon.

Q. 292. The drainage water is sometimes allowed to join the canal water to augment canal supplies, through a hydraulic structure, called a :

(*a*) canal outlet (*b*) canal inlet
(*c*) module (*d*) level crossing.

Q. 293. The DBL of a canal and the HFL of a drain at their crossing point are, respectively, 216 m and 214 m. The adopted cross-drainage work here, will be a :

(*a*) super passage (*b*) siphon
(*c*) aqueduct (*d*) siphon aqueduct.

Q. 294. Pinpoint the correct statement in relation to the relative merits of the two cross-drainage works :

(*a*) a super passage is preferred to an aqueduct
(*b*) a canal-syphon is preferred to a super-passage
(*c*) a super passage is preferred to a canal syphon
(*d*) a siphon aqueduct is inferior to a canal siphon, and is seldom used.

Q. 295. A cross-drainage work is called a siphon, when it carries the canal water :
(*a*) below the drainage under pressure
(*b*) below the drainage at atmospheric pressure
(*c*) above the drainage at atmospheric pressure
(*d*) none of the above.

Q. 296. In a syphon aqueduct, the worst condition of uplift on the roof occurs, when:
(*a*) the canal and drainage are flowing full
(*b*) the canal is flowing full and there is no drainage discharge
(*c*) the canal is empty and drainage is flowing full
(*d*) none of the above.

Q. 297.* In an aqueduct provided with a pucca bottom floor, the uplift will occur :
(*a*) on the roof slab
(*b*) on the bottom floor
(*c*) on both the roof slab as well as the bottom floor
(*d*) no where, since the flow is free in the canal as well as in the drainage channel.

Q. 298.* In a syphon aqueduct, the worst condition of uplift on the floor occurs, when:
(*a*) the canal is full and the drainage empty, with watertable at drainage bed
(*b*) the canal is empty and the drainage full, with watertable at drainage bed
(*c*) the canal and drainage both full
(*d*) the canal is full and drainage empty, with watertable below the floor.

Q. 299. Point out the incorrect statement :
(*a*) aqueducts and super passages are usually not provided with pucca bottom floors
(*b*) aqueduct syphons and canal syphons are usually provided with pucca bottom floors
(*c*) in a level crossing, a cross regulator is provided on the canal below the crossing
(*d*) canals or drainage channels are usually flumed to affect economy at the sites of crossings, wherein the contraction transitions are not to be steeper than $22\frac{1}{2}^\circ$ and the expansion transitions not to be steeper than 30°.
(*e*) none of the above.

Q. 300. The head loss through a siphon barrel is usually given by Unwin's formula, by neglecting velocity of approach, as :
$H_L = \left(1 + f_1 + f_2 \cdot \frac{L}{R}\right)\frac{V^2}{2g}$, **where f_1 and f_2 respectively represent the coefficient of head losses due to :**
(*a*) barrel friction and entry (*b*) entry and barrel friction
(*c*) barrel friction and exit (*d*) exit and barrel friction.

Q. 301. A syphon aqueduct is constructed at a canal crossing site, where drainage HFL was 212.2 m, by allowing an afflux of 0.4 m at high flood discharge. At this site, the water level, downstream of the crossing at the same high flood, will be :
(*a*) 212.2 m
(*b*) 212.6 m
(*c*) 211.8 m
(*d*) none of them, as it cannot be predicted.

Q. 302. The following data is available at the proposed site of a canal crossing :

Item	Drain	Canal
B.L. (m)	252.2	248.0
FSL/HFL (m)	253.2	253.0
Discharge (*cumecs*)	2	400

The most appropriate and economical cross-drainage work at the above site will be :

(*a*) an aqueduct (*b*) a super passage
(*c*) a syphon aqueduct (*d*) a syphon.

CHAPTER 15

Q. 303. Culverts are all those road bridges, whose spans are up to :

(*a*) 6 m (*b*) 8 m
(*c*) 10 m (*d*) 12 m.

Q. 304.* A road bridge is proposed to be constructed across an alluvial stream. The design discharge at the bridge site is estimated to be 2500 m^3/s. It is decided to adopt a total effective waterway equal to 300 m for this bridge, as against the regime width of 237.5 m, due to afflux considerations. If the HFL at the site is at RL 214.0 m, then the RL of the open foundation can be suggested at :

(*a*) 206.6 m (*b*) 207.2 m
(*c*) 208.5 m (*d*) 210.2 m.

Q. 305. The relation between Scour depth (*R*) and Regime scour depth (R_e) for an alluvial river, having regime width (W) and contracted width (B), is given by :

(*a*) $R = R_e\left(\frac{W}{B}\right)^{0.61}$ (*b*) $R = R_e\left(\frac{B}{W}\right)^{0.61}$

(*c*) $R = R_e\left(\frac{W}{B}\right)^{1/3}$ (*d*) $R = R_e\left(\frac{B}{W}\right)^{1/3}$

Q. 306. The Lacey's scour depth R, given by equation $R = 0.473\left(\frac{Q}{f}\right)^{1/3}$, is applicable, when the river width equals :

(*a*) actual river width (*b*) regime width
(*c*) any contracted width (*d*) all of the above.

Q. 307. The Lacey's scour depth R, given by the equation $R = 1.35\left(\frac{q^2}{f}\right)^{1/3}$ is applicable, when the river width equals :

(*a*) regime width (*b*) actual river width
(*c*) any contracted width (*d*) all of the above.

Q. 308. Proposed RCC road bridge, having total height of its abutments or piers from top to bottom as H, can be proposed to have economical spans, each equal to :

(*a*) $\frac{2}{3}$H (*b*) H
(*c*) 1.2 H (*d*) 1.5 H.

Q. 309. A bridge culvert is to be constructed across a proposed lined canal. The afflux at this site for a contracted canal width, can be computed by :

(*a*) Unwin's formula (*b*) Broad-crested weir formula
(*c*) Orifice formula (*d*) Molesworth formula.

Q. 310. While going to Tirupati Tirumala Devasthanam from Madras, highway road is found to be lowered at several places, over which natural drainage flow during rainy season. Such dips in the roads, are called :

(*a*) freeways (*b*) road dips
(*c*) causeways (*d*) broadways.

CHAPTER 16

Q. 311. An irrigation project is classified as a major project, when the culturable command involved in the project, is more than :

(*a*) 2,000 hectares (*b*) 5,000 hectares
(*c*) 10,000 hectares (*d*) none of the above.

Q. 312. A minor irrigation scheme, involves command area, equal to or less than :

(*a*) 100 hectares (*b*) 500 hectares
(*c*) 1000 hectares (*d*) 2000 hectares.

Q. 313. Energy is required in the utilisation of :

(*a*) ground water (*b*) surface water
(*c*) both (*a*) and (*b*) (*d*) none of the above.

Q. 314. In an underground profile, the zone of aeration does not include :

(*a*) soil water (*b*) capillary water
(*c*) ground water (*d*) none of them.

Q. 315. Ground water is found to occur in the geological formation, which is :

(*a*) porous (*b*) permeable
(*c*) both (*a*) and (*b*) (*d*) none of the above.

Q. 316. The property of a geological formation, which represents its water storage capacity, is :

(*a*) permeability (*b*) porosity
(*c*) both (*a*) and (*b*) (*d*) none of these.

Q. 317. The zone of aeration in a ground profile consists of :

(*a*) capillary fringe (*b*) soil water zone
(*c*) intermediate zone (*d*) all of them.

Q. 318. The zone of aeration in a ground profile does not include :

(*a*) capillary zone (*b*) soil water zone
(*c*) intermediate zone (*d*) saturation zone.

Q. 319. The area in our country, which is irrigated by ground water, as a fraction of the total irrigated area, is approximately equal to :

(*a*) $\frac{1}{2}$ (*b*) $\frac{2}{3}$
(*c*) $\frac{3}{4}$ (*d*) $\frac{1}{4}$.

Q. 320. The rate of flow of water through ground strata, can be estimated by :

(*a*) Manning's formula (*b*) Strickler's formula
(*c*) Dupuit's formula (*d*) Darcy's formula.

Q. 321. Darcy's law is valid, when the flow is :

(*c*) laminar (*b*) turbulent
(*c*) both (*a*) and (*b*) (*d*) none of these.

Q. 322. The relation between Transmissibility(T) and Permeability(K) for an aquifer of depth d, is :
(*a*) $K = Td$ (*b*) $T = K \cdot d$
(*c*) $T = K \cdot \log d$ (*d*) $T = \ln (Kd)$.

Q. 323.* The geological formation, which yields only insignificant quantity of ground water, is an :
(*a*) aquifer (*b*) aquifuse
(*c*) aquiclude (*d*) aquitard.

Q. 324.* The zeological formation, which may contain water, but does not yield any, is an :
(*a*) aquifer (*b*) aquifuse
(*c*) aquiclude (*d*) aquitard.

Q. 325.* The zeological formation, which contains and readily yields water to our tubewells, is an :
(*a*) aquifuse (*b*) aquitard
(*c*) aquiclude (*d*) aquifer.

Q. 326. The capillary fringe water is also called :
(*a*) suspended water (*b*) vadose water
(*c*) gravity water (*d*) all of the above.

Q. 327. The quantum of water contained in the soil pores, which cannot be extracted by gravity drainage, is called :
(*a*) pellicular water (*b*) capillary water
(*c*) hygroscopic water (*d*) none of these.

Q. 328. The cum of ground water, which can be extracted by gravity drainage from a soil stratum, when expressed as percentage fraction of the cum of the soil stratum, is called :
(*a*) pellicular water (*b*) available water
(*c*) specific yield (*d*) field capacity.

Q. 329. Field capacity of a ground aquifer, equals :
(*a*) specific yield (*b*) 100—specific yield
(*c*) $\frac{100}{\text{specific yield}}$ (*d*) none of the above.

Q. 330. Specific retention of ground water is larger in :
(*a*) coarse grained soils (*b*) fine grained soils.

Q. 331. Water wells excavated through unconfined aquifers are known as :
(*a*) gravity wells (*b*) water table wells
(*c*) non-artesian wells (*d*) all of them.

Q. 332. Water wells excavated through confined aquifers are known as :
(*a*) artesian wells (*b*) pressure wells
(*c*) both (*a*) and (*b*) (*d*) none of these.

Q. 333. In the case of a flowing well, the piezometric surface is always :
(*a*) below the ground level (*b*) above the ground level
(*c*) at the ground level (*d*) none of the above.

Q. 334. The line joining the static water levels in several wells, excavated through confined aquifer, is known as the :
(*a*) cone of depression (*b*) piezometric surface
(*c*) perched water-table (*d*) hypsometric curve.

Q. 335. If the piezometric surface along an unconfined aquifer declines to a level below the top of the aquifer, then the aquifer at this point is called :

(*a*) a perched aquifer (*b*) a leaky aquifer
(*c*) a flowing aquifer (*d*) an unconfined aquifer.

Q. 336.* The permeability of an aquifer (m/day) will :

(*a*) increase with increase in temperature of water flowing through the aquifer
(*b*) decrease with increase in temperature of water flowing through the aquifer
(*c*) not get affected by the change in temperature of water flowing through the aquifer
(*d*) increase upto 20°C and then decrease with the increase in the temperature of water flowing through the aquifer.

Q. 337.* The component of the permeability of a porous medium, which is usually expressed in units of darcy or m^2, is called :

(*a*) specific permeability (*b*) intrinsic permeability
(*c*) both (*a*) and (*b*) (*d*) none of the above.

Q. 338. The discharge per unit drawdown at a well is known as :

(*a*) specific yield (*b*) specific retention
(*c*) specific capacity (*d*) specific storage.

Q. 339. Storage coefficient (A) divided by aquifer depth (*d*) is known as :

(*a*) specific storage (*b*) specific capacity per unit depth of aquifer
(*c*) specific yield. (*d*) specific retention.

Q. 340.* Coefficient of storage (A) has the dimensions of :

(*a*) L^3 (*b*) L^2
(*c*) L (*d*) dimensionless.

Q. 341. The units of specific capacity of a well are :

(*a*) m^3/sec (*b*) m^2/sec
(*c*) m/sec (*d*) no units.

Q. 342. Specific capacity of a well is :

(*a*) constant over time after commissioning of the well
(*b*) increases with time after start of pumping
(*c*) decreases with time after start of pumping
(*d*) may increase or decrease, depending upon a particular aquifer.

Q. 343. Standard specific capacity of a well is the well discharge per unit of drawdown. This value of discharge should be obtained at :

(*a*) any drawdown
(*b*) a particular drawdown of 3 m
(*c*) the first 1 m of drawdown
(*d*) none of the above.

Q. 344.* The performance of a well is measured by its :

(*a*) specific capacity (*b*) specific yield
(*c*) storage coefficient (*d*) all of the above.

Q. 345. The clogging of well screens and consequent reduction in pump efficiency is indicated by :

(*a*) high value of well loss (*b*) low value of well loss
(*c*) variable value of well loss (*d*) none of the above.

Q. 346. The water level in a confined well :
(*a*) increases with increase in atmospheric pressure
(*b*) decreases with increase in atmospheric pressure
(*c*) does not undergo any change with change in the atmospheric pressure.
(*d*) all of the above are possible.

Q. 347. The water level in an unconfined well :
(*a*) increases with increase in atmospheric pressure
(*b*) decreases with increase in atmospheric pressure
(*c*) does not undergo any change with change in the atmospheric pressure
(*d*) all of the above are possible.

Q. 348.* "Specific yield" for an unconfined aquifer is :
(*a*) greater than porosity (*b*) less than porosity
(*c*) equal to porosity (*d*) unrelated to porosity. (*GATE 1989*)

Q. 349. A perched aquifer is essentially found within :
(*a*) an unconfined aquifer (*b*) a confined aquifer
(*c*) an aquiclude (*d*) none of these.

Q. 350. The most widely used type of a deep state tubewell in India is:
(*a*) cavity well (*b*) strainer tubewell
(*c*) slotted pipe gravel packed tubewell
(*d*) none of them.

Q. 351. The general average yield from standard tubewells is of the order of :
(*a*) 5 L/s (*b*) 50 L/s
(*c*) 500 L/s (*d*) 5000 L/s.

Q. 352. The types of pumps used in tubewells are :
(*a*) submergible pumps (*b*) centrifugal pumps
(*c*) turbine pumps (*d*) all of these.

Q. 353. The appropriate life of a tubewell in India is of the order of :
(*a* 1 year (*b*) 2 years
(*c*) 5 years (*d*) 15 years.

Q. 354. Corrosion of tubewell pipes may cause :
(*a*) reduced discharge from the tubewell
(*b*) excessive discharge of sand with water
(*c*) discharge of highly corrosive acidic water
(*d*) none of the above.

Q. 355. Incrustation of the tubewell pipes may lead to :
(*a*) reduced discharge from the tubewell
(*b*) excessive discharge of sand with water
(*c*) discharge of highly alkaline water
(*d*) none of the above.

Q. 356. The efficiency of the centrifugal pumps, commonly used for lifting water from wells, may be of the order of :
(*a*) 30% (*b*) 65%
(*c*) 85% (*d*) 95%.

Q. 357. Deep tubewells in soft alluviums can be best drilled by :
(*a*) rotary drilling rigs
(*b*) percussion drilling rigs
(*c*) down the hole hammer (DTH) rigs
(*d*) none of the above.

Q. 358. Deep tubewells in hard boulder-alluviums can be best drilled by :
(*a*) rotary drilling rigs
(*b*) percussion drilling rigs
(*c*) down the hole hammer (DTH) rigs
(*d*) none of the above.

Q. 359. Bored tubewells in rocky consolidated formations, as are encountered in South Indian States of our country, are usually drilled by :
(*a*) rotary drilling rigs
(*b*) percussion drilling rigs
(*c*) down the hole hammer (DTH) rigs
(*d*) none of the above.

Q. 360. Shallow tubewells upto 70 m depth in alluvial soils, are usually drilled by :
(*a*) light rigs (*b*) water jet methods
(*c*) cable tool drills (*d*) hand boring devices
(*e*) all of the above.

CHAPTER 17

Q. 361. Aswan dam on Nile river in Egypt was perhaps the first modern dam of the world. Its construction was completed in the year :
(*a*) 1664 (*b*) 1798
(*c*) 1902 (*d*) 1931.

Q. 362. The only modern dam among the following, which has failed, is :
(*a*) Koyna dam in India
(*b*) Vega de Tera dam in spain
(*c*) Mauvoisin dam in Switzerland
(*d*) Trinity dam in California, USA.

Q. 363. Earthen dams are :
(*a*) rigid dams (*b*) non rigid dams
(*c*) overflow dams (*d*) diversion dams.

Q. 364. When sand and gravel foundation strata is available at a proposed dam site of moderate height, the dam may be of the type :
(*a*) earthen dam or rockfill dam
(*b*) masonry gravity dam
(*c*) double arch dam
(*d*) concrete gravity dam.

Q. 365. The prominent modern dam, which has changed its name several times, is :
(*a*) Nagarjuna sagar dam (*b*) Hoover dam
(*c*) Bhakra dam (*d*) Idduki dam.

Q. 366. The highest concrete gravity dam of the world is Grand Dixence dam is Switzerland, whose height is :
(*a*) 325 m (*b*) 317 m
(*c*) 285 m (*d*) 272 m.

Q. 367. The only arch dam of our country is Idduki dam across Periyar river in Kerala State, whose height is :
(*a*) 81 m (*b*) 143 m
(*c*) 149 m (*d*) 169 m.

Q. 368. The highest concrete gravity dam of India, which is also the 2nd highest such dam of the world, is :
(*a*) 285 m in height (*b*) 256 m in height
(*c*) 226 m in height (*d*) 206 m in height.

Q. 369. Bhakra dam of our country is located in the State of :
(*a*) Punjab (*b*) Himachal Pradesh
(*c*) Uttar Pradesh (*d*) Madhya Pradesh.

Q. 370.* The highest stone masonry gravity dam of India as well as of the world, is:
(*a*) Ramganga dam (*b*) Nagarjuna Sagar dam
(*c*) Hirakud dam (*d*) Mettur dam.

CHAPTER 18

Q. 371. A dam reservoir, not provided with gate controls on its spillway and other sluices, is called a :
(*a*) detention basin (*b*) storage reservoir
(*c*) retarding basin (*d*) all of these.

Q. 372. A dam reservoir, catering to flood control, irrigation, and water supply, although basically designed for irrigation alone, is a :
(*a*) multipurpose reservoir (*b*) single purpose reservoir
(*c*) distribution reservoir (*d*) none of the above.

Q. 373. A hydel power project has been envisaged to serve the water supply and irrigation needs of the area at its inception stage. The dam reservoir, so constructed, will be known as :
(*a*) multipurpose reservoir (*b*) single purpose reservoir
(*c*) both (*a*) and (*b*) above (*d*) none of the above.

Q. 374. The 'surcharge storage' in a dam reservoir is the volume of water stored between :
(*a*) minimum and maximum reservoir levels
(*b*) minimum and normal reservoir levels
(*c*) normal and maximum reservoir levels
(*d*) none of the above.

Q. 375. The 'useful storage' in a dam reservoir is the volume of water stored between:
(*a*) minimum and maximum reservoir levels
(*b*) minimum and normal reservoir levels
(*c*) normal and maximum reservoir levels
(*d*) none of the above.

Q. 376. The 'dead storage' in a dam reservoir, is the available volume for collection of silt and sediment, between :
(*a*) bed level of the reservoir and minimum reservoir level
(*b*) bed level of the reservoir and the silt level in the reservoir
(*c*) bed level of the reservoir and the normal pool level
(*d*) none of the above.

Q. 377. 'Bank storage' in a dam reservoir :
(*a*) increases the computed reservoir capacity
(*b*) decreases the computed reservoir capacity
(*c*) sometimes increases and sometimes decreases the computed reservoir capacity
(*d*) has no effect on computed reservoir capacity.

Q. 378. The water stored in a reservoir below the minimum pool level is called :
(*a*) valley storage (*b*) bank storage
(*c*) surcharge storage (*d*) dead storage.

Q. 379. For a flood control reservoir, the effective storage is equal to :
(*a*) useful storage + valley storage
(*b*) useful storage + surcharge storage – valley storage
(*c*) useful storage + surcharge storage + valley storage
(*d*) useful storage – valley storage.

Q. 380. The volume V of water stored in a reservoir in a depth y above a given datum, can be easily represented by an equation of the type :
(*a*) $V = [\alpha + \beta y + \gamma y^2]$ where α, β, γ are constants
(*b*) $V = [\alpha y + \beta y^2 + \gamma y^3] + K$ K is reservoir capacity up to given datum
(*c*) neither (*a*) nor (*b*).
(*d*) both (*a*) and (*b*).

Q. 381. Average yield of a storage reservoir is the arithmetic average of its :
(*a*) firm yields over a long period
(*b*) secondary yields over a long period
(*c*) firm and secondary yields over a long period
(*d*) none of the above.

Q. 382. If A_1 and A_2 are surveyed and known areas at a reservoir site at elevations of E_1 and E_2, respectively ; then the area, A, at any intermediate elevation E, is interpolated as :
(*a*) $\sqrt{A} = \sqrt{A_1} + \frac{E - E_1}{E_2 - E_1}\left(\sqrt{A_2} - \sqrt{A_1}\right)$
(*b*) $\sqrt{A} = \sqrt{A_1} + \frac{E + E_1}{E_2 + E_1}\left(\sqrt{A_2} + \sqrt{A_1}\right)$
(*c*) $A = \sqrt{A_1} + \frac{E - E_1}{E_2 - E_1}\left[\sqrt{A_2} - \sqrt{A_1}\right]^2$
(*d*) none of the above.

Q. 383. While planning a water supply reservoir, as compared to an irrigation reservoir, the design yield may be kept :
(*a*) higher (*b*) lower
(*c*) equal
(*d*) lower or higher, as per designer's discretion.

Q. 384. Yield of a reservoir represents :
(*a*) the inflow into the reservoir
(*b*) the capacity of the reservoir
(*c*) the outflow demand on the reservoir
(*d*) none of the above.

Q. 385. The capacity of a storage reservoir can be decided by using :
(*a*) the mass curve of inflow
(*b*) the mass curve of outflow
(*c*) by both (*a*) and (*b*) together
(*d*) none of the above.

Q. 386. The maximum quantity of water that is estimated to remain available in a storage reservoir for supply, even during worst dry periods, is known as its:

(*a*) firm yield (*d*) design yield
(*c*) reservoir yield (*d*) primary yield.

Q. 387. Design yield of a storage reservoir is kept :

(*a*) higher than its firm or safe yield
(*b*) lower than its firm or safe yield
(*c*) equal to its firm or safe yield
(*d*) higher or lower than the firm yield, depending upon the designer's intuition.

Q. 388. The amount of water stored in a river channel without any artificial storage, is known as :

(*a*) bank storage (*b*) river storage
(*c*) valley storage (*d*) dead storage.

Q. 389. With the reduction in reservoir capacity over the passage of time, the trap efficiency

(*a*) increases
(*b*) decreases
(*c*) remains unaffected
(*d*) may increase or decrease, depending upon the reservoir characteristics.

Q. 390. Trap efficiency of a storage reservoir is defined as :

(*a*) $\frac{\text{Total annual sediment inflow}}{\text{Reservoir capacity}}$
(*b*) $\frac{\text{Total sediment deposited in a given period}}{\text{Total sediment inflow in that period}}$
(*c*) $\frac{\text{Total annual sediment deposited in the reservoir}}{\text{Dead storage capacity of the reservoir}}$
(*d*) none of the above.

Q. 391. Capacity-inflow ratio for a storage reservoir is defined as :

(*a*) $\frac{\text{reservoir capacity}}{\text{average annual flood inflow}}$
(*b*) $\frac{\text{reservoir capacity}}{\text{average annual sediment inflow}}$
(*c*) $\frac{\text{dead storage capacity of the reservoir}}{\text{average annual sediment deposited}}$
(*d*) none of the above.

Q. 392. With the increase in its capacity-inflow ratio, the trap efficiency of a reservoir:

(*a*) increases
(*b*) decreases
(*c*) remains unchanged
(*d*) may increase or decrease depending on the reservoir characteristics.

Q. 393. The capacity-inflow ratio for a reservoir :

(*a*) is a constant factor overtime
(*b*) increases with time
(*c*) decreases with time
(*d*) may increase or decrease with time.

Q. 394. As the height of a proposed dam is increased, the cost per unit of storage :
(*a*) increases
(*b*) decreases
(*c*) initially increases and then decreases
(*d*) initially decreases and then increases.

Q. 395. "Economical height of a Dam" is that height, for which the :
(*a*) cost per unit of storage is minimum
(*b*) benefit cost ratio is maximum
(*c*) net benefits are maximum
(*d*) none of the above.

Q. 396.* If 20% of the reservoir capacity is earmarked for dead storage in a storage reservoir of 30 M cum; and the average annual silt deposition in the reservoir is 0.1 M cum, then the useful life of the reservoir will start reducing after :
(*a*) 60 years (*b*) 120 years
(*c*) 240 years (*d*) 300 years.

Q. 397. The appropriate value of the estimated average life of a dam reservoir is :
(*a*) 25 years (*b*) 50 years
(*c*) 75 years (*d*) none of these.

CHAPTER 19

Q. 398. The vertical component of the earthquake wave, which produces adverse effects on the stability of a dam, is, when it is acting in;
(*a*) upward direction (*b*) downward direction
(*c*) both (*a*) and (*b*) (*d*) none of them.

Q. 399. The horizontal component of an earthquake wave, producing instability in a dam is the one, which acts :
(*a*) towards the reservoir (*b*) towards the dam
(*c*) both (*a*) and (*b*) (*d*) none of the above.

Q. 400. The vertical downward earthquake acceleration, $\alpha_v = 0.1$ g, acting on a gravity dam, will :
(*a*) increase the resisting weight of the dam by 10%
(*b*) decrease the resisting weight of the dam by 10%
(*c*) increase the uplift by 10%
(*d*) none of the above.

Q. 401. A gravity dam is subjected to hydro dynamic pressure, caused by :
(*a*) the rising waters of the reservoir when a flood wave enters into it
(*b*) the rising waves in the reservoir due to high winds
(*c*) the increase in water pressure, momentarily caused by the horizontal earthquake, acting towards the reservoir.
(*d*) the increase in water pressure, momentarily caused by the horizontal earthquake, acting towards the dam.

Q. 402. In a concrete gravity dam with a vertical upstream face, the stabilising force is provided by the :
(*a*) weight of the dam
(*b*) the water supported against the upstream slope
(*c*) both (*a*) and (*b*)
(*d*) none of the above.

Q. 403. In a concrete gravity dam, with a sloping upstream face, the resisting force is provided by the :
(*a*) weight of the dam
(*b*) weight of the water supported on the upstream slope
(*c*) both (*a*) and (*b*)
(*d*) none of these.

Q. 404. The factor among the following, which does not try to destabilise a masonry gravity dam, is :
(*a*) water seeping below the foundation of the dam
(*b*) generation of waves by high winds
(*c*) deposition of silt in dead storage zone of the reservoir
(*d*) water standing against the downstream face of the dam
(*e*) water standing against the upstream face of the dam.

Q. 405*. The horizontal destabilising force caused by the formation of waves in a storage reservoir having a fetch of 40 km, due to high wind of 160 km/h, is about :
(*a*) 30 kN (*b*) 60 kN
(*e*) 90 kN (*d*) 130 kN.

Q. 406*. According to IS 6512—1984, the wind set up in a reservoir having maximum fetch of F km, and average reservoir depth along this fetch length as D m, as caused by the wind velocity of V km/h, is given by :
(*a*) $\frac{V^2 F}{62000 D}$ (*b*) $\frac{V^2 F}{D}$
(*c*) $\frac{V^2 F}{6200 D}$ (*d*) $\frac{V^2 F}{620 D}$.

Q. 407*. A reservoir, extending 20 km upstream and having a design wind velocity of 100 km/h, should be provided with a free-board of :
(*a*) 1.2 m (*b*) 1.6 m
(*c*) 2.5 m (*d*) 3 m.

Q. 408. Development of tensile stresses in a concrete or masonry gravity dam are usually not allowed, because it may lead to development of tension cracks, ultimately causing failure of the structure, by :
(*a*) excessive seepage
(*b*) excessive tensile stresses
(*c*) excessive compressive stresses
(*d*) none of the above.

Q. 409. The bottom portion of a concrete or a masonry gravity dam is usually stepped, in order to :
(*a*) increase the overturning resistance of the dam
(*b*) increase the shear strength at the base of the dam
(*c*) decrease the shear stress at the base of the dam
(*d*) none of the above.

Q. 410. The governing compressive stress in a concrete gravity dam, which should not be allowed to exceed the permissible value of about 3000 kN/m^2, while analysing full reservoir case, is :
(*a*) the vertical maximum stress at the toe
(*b*) the major principal stress at the toe
(*c*) the shear stress at the toe
(*d*) none of the above.

Q. 411. The two dimensional stability analysis of gravity dams proves better for U-shaped valleys than for *V*-shaped valleys, because the transverse joints in the dam body are generally :

(*a*) not grouted in *U*-shaped valleys, but are keyed together in *V*-shaped valleys
(*b*) not grouted in *V*-shaped valleys, but are keyed together in *U*-shaped valleys.

Q. 412. The small openings made in the huge body of a concrete gravity dam, such as sluices and inspection galleries, can be assumed to be causing only local effects without any appreciable effect on the distribution of stresses, as per the principle of :

(*a*) Laplace (*b*) St. Venant
(*c*) Reynold (*d*) St. Francis.

Q. 413. For usuall values of permissible compressive stress and specific gravity of concrete, a high concrete gravity dam is the one, whose height exceeds :

(*a*) 48 m (*b*) 70 m
(*c*) 88 m (*d*) none of these.

Q. 414*. An elementary triangular concrete gravity dam, supporting 60 m height of reservoir water and full uplift, should have a minimum base width equal to:

(*a*) 36 m (*b*) 39 m
(*c*) 51 m (*d*) 61 m.

Q. 415. Inorder to economise on the provided section of a concrete gravity dam, attempts are made to reduce the uplift, by :

(*a*) providing drainage gallery to collect seepage water
(*b*) constructing cut-off under upstream face
(*c*) pressure grouting in dam foundation
(*d*) constructing drainage channels between the dam and its foundation
(*e*) all of the above methods.

Q. 416. Transverse joints in concrete gravity dams are the :

(*a*) horizontal construction joints at each lift height
(*b*) vertical construction joints of full height and width
(*c*) diagonal construction joints for torsion
(*d*) none of the above.

Q. 417. Leakage through the transverse joints in a gravity dam is prevented by :

(*a*) shear keys (*b*) key ways
(*c*) water stops (*d*) none of these.

Q. 418. Shear key is several times provided between the bottom of a masonry or concrete gravity dam and its foundation, to increase the frictional resistance of the dam against sliding. This key is usually provided :

(*a*) near the toe (*b*) near the heel
(*c*) near the individual seams in the bed rock
(*d*) none of the above.

CHAPTER 20

Q. 419. Point out the correct statement with reference to earthen dams :

(*a*) these dams are very costly as compared to other types
(*b*) they are less susceptible to failure as compared to rigid dams
(*c*) they can be constructed almost on every type of foundation
(*d*) highly skilled labour is generally not required.

Q. 420. The most preferred type of an earthen dam section is the one, in which the:
(*a*) entire embankment is made of one type of soil
(*b*) inner embankment is made of highly porous soil, surrounded by the outer shell of highly impervious soil, both separated by transition filter material of mediocre permeability
(*c*) inner embankment is made of highly impervious soil surrounded by the outer shell of highly pervious soil, both separated by transition filter material of mediocre permeability
(*d*) none of the above.

Q. 421. When seepage takes place through the body of an earthen dam, it leads to :
(*a*) development of pore pressures in the dam body
(*b*) reduction in the shear strength of the dam
(*c*) reduction in the developed shear stresses in the dam
(*d*) (*a*) and (*b*) both
(*e*) (*a*), (*b*) and (*c*) all.

Q. 422. The most preferred soil for the central impervious core of a zoned embankment type of an earthen dam, is
(*a*) highly impervious clay (*b*) highly pervious gravel
(*c*) coarse sand (*d*) clay mixed with fine sand.

Q. 423. Pure clayey soils are generally not preferred for the central impervious cores of zoned type of earthen dams, because :
(*a*) clays are highly impervious
(*b*) clays are highly pervious
(*c*) clays are susceptible to cracking
(*d*) none of the above.

Q. 424. During the construction of an earthen dam by hydraulic fill method, development of pore pressures become important in the :
(*a*) central impervious core (*b*) pervious outer shell
(*c*) both (*a*) and (*b*) (*d*) none of the above.

Q. 425. The process of laying and compacting earth in layers by power rollers under Optimum Moisture Conditions (OMC) for construction of earthen dams, is known as :
(*a*) rolled fill method (*b*) hydraulic fill method
(*c*) OMC method (*d*) none of the above.

Q. 426. Coordination between field and design engineers to ensure continuous field observations and modifications in design during construction, is more importantly required in case of :
(*a*) concrete gravity dams (*b*) masonry gravity dams
(*c*) arch dams (*d*) earthen dams.

Q. 427. When the water level standing against an earthen embankment, suddenly falls down, then there is an iminent risk of sliding failure, to the :
(*a*) upstream slope (*b*) downstream slope
(*c*) both (*a*) and (*b*) (*d*) none of these.

Q. 428. When the reservoir is full, the slope which is most likely to slide, is :
(*a*) the upstream slope (*b*) the downstream slope
(*c*) both (*a*) and (*b*) (*d*) none of them.

Q. 429. Provision of horizontal berms at suitable vertical intervals may be provided in the downstream face of an earthen dam, in order to :
(*a*) allow the movement of cattle
(*b*) allow the inspection vehicles to move
(*c*) reduce the erosion caused by the flowing rain water
(*d*) none of the above.

Q. 430. The only provision among the following, which can help control the seepage through the body of an earthen dam and, thus, to keep the phreatic line well within the dam width, is :
(*a*) upstream impervious cutoff
(*b*) drain trench along the downstream toe
(*c*) relief wells
(*d*) chimney drain.

Q. 431*. During the maintenance of an earthen dam, the apparent seepage through the foundation of the dam is best taken care of, by providing :
(*a*) a chimney drain
(*b*) a rock toe
(*c*) a drain trench along the downstream toe
(*d*) an upstream impervious cutoff.

Q. 432. The base width of a rock fill dam, incomparison to that of an earthen dam, is :
(*a*) much larger
(*b*) much smaller
(*c*) sometimes larger sometimes smaller
(*d*) almost equal.

Q. 433. On moderate foundations, and particularly in seismic areas, the type of dam which can preferably be considered for construction, is :
(*a*) masonry gravity dam (*b*) earthen dam
(*c*) rock fill dam (*d*) arch dam.

CHAPTER 21

Q. 434. The 'safety valve' of a dam is its :
(*a*) drainage gallery (*b*) inspection gallery
(*c*) spillway (*d*) outlet sluices.

Q. 435. A ungated spillway starts functioning, as soon as the water level in the reservoir, crosses the :
(*a*) maximum reservoir level
(*b*) minimum reservoir level
(*c*) maximum conservation level
(*d*) none of the above.

Q. 436. The top line of an ogee spillway, which is kept at RL equal to the full reservoir level, is precisely called :
(*a*) the crest (*b*) the apex of the crest
(*c*) the corbel (*d*) the chute.

Q. 437. When the gated sluices are provided through the body of a dam spillway, then the ogee spillway structure shall normally be of :
(*a*) corbel type (*b*) non corbel type
(*c*) either (*a*) or (*b*) (*d*) none of these.

Q. 438. The effective length of an ogee spillway will be different from its actual length, in case of :
(*a*) gated spillway
(*b*) ungated spillway
(*c*) both gated as well as ungated spillway
(*d*) none of these.

Q. 439. Which one of the following spillways is least suited to earthen dams ?
(*a*) ogee spillway (*b*) side channel spillway
(*c*) chute spillway (*d*) shaft spillway.

Q. 440. The spillway, which can be adopted with ease on gravity as well as earthen dams, is :
(*a*) ogee spillway
(*b*) chute spillway
(*c*) both ogee as well as chute spillway
(*d*) none of these.

Q. 441. The famous Bhakra dam of our country has been provided with :
(*a*) trough spillway (*b*) ogee spillway
(*c*) shaft spillway (*d*) syphon spillway.

Q. 442. Hydraulic jump is widely used for dissipation of energy in :
(*a*) ogee spillways (*b*) trough spillways
(*c*) side channel spillways (*d*) all of these.

Q. 443. The surplus reservoir water, after spilling over the spillway crest, flows on the chute :
(*a*) parallel to the crest in a trough spillway
(*b*) parallel to the crest in a side channel spillway
(*c*) perpendicular to the crest in a side channel spillway
(*d*) obliquely to the crest in a chute spillway.

Q. 444. When the crest of an ogee spillway is designed to be in accordance with the lower nappe of a free falling water jet over a duly ventilated sharp crested weir, then theoretically :
(*a*) the pressure on the spillway crest will always be zero (*i.e.* atmospheric pressure)
(*b*) the pressure on the spillway crest will be zero at design head only
(*c*) the pressure on the spillway crest will always be negative
(*d*) the pressure on the spillway crest will always be positive.

Q. 445. If the operating head on an ogee spillway is more than the design head, then:
(*a*) the pressure on the crest will be zero
(*b*) the pressure on the crest will be negative, causing cavitation
(*c*) the pressure on the crest will be positive
(*d*) the discharge coefficient of the spillway will be reduced.

Q. 446. In computing the spilling capacity of high ogee spillways, the velocity head is usually :
(*a*) very small, and hence neglected
(*b*) very large, and hence cannot be neglected
(*c*) neither (*a*) nor (*b*).

Q. 447. In the functioning of an ogee spillway, the operating head :
(*a*) frequently exceeds the design head
(*b*) rarely exceeds the design head
(*c*) never exceeds the design head
(*d*) has no connection with the design head.

Q. 448. The discharge passing over an ogee spillway, per unit length of its appex line, is proportional to :
(*a*) H (*b*) H^2
(*c*) $H^{1/2}$ (*d*) $H^{3/2}$.

where H is the head over the appex of its crest.

Q. 449*. If a flood enters a dam reservoir at F.R.L., the efficiency of its ogee spillway will :
(*a*) increase with the increasing head
(*b*) reduce with the increasing head
(*c*) remain constant with the increasing head
(*d*) vary with the increasing head.

Q. 450. The W.E.S. downstream profile of an ogee spillway can be represented by an equation :

$$\left(\frac{x}{H_d}\right)^n = K\left(\frac{y}{H_d}\right);$$

where the coefficients K and n for a vertical upstream spillway would, respectively, be :
(*a*) 2.0 and 1.85 (*b*) 0.5 and 1.85
(*c*) 2.0 and 0.85 (*d*) 0.5 and 0.85.

where (x, y) are the coordinates on the crest profile with crest appex as origin.

Q. 451. The portion of a chute spillway, which is known as its control structure, is :
(*a*) low ogee weir
(*b*) chute channel
(*c*) approach channel leading the water from the reservoir to the ogee weir.
(*d*) stilling basin at its bottom.

Q. 452. The spillway, which can be called as an 'overflow spillway', is essentially :
(*a*) an ogee spillway (*b*) shaft spillway
(*c*) chute spillway (*d*) syphon spillway.

Q. 453. The spillway, which may sometimes be called a 'waste weir', is :
(*a*) an ogee spillway (*b*) a trough spillway
(*c*) a shaft spillway (*d*) all of the above.

Q. 454. The spillways involving weir type spills, giving increased discharge with the increase in reservoir level, are :
(*a*) ogee spillways (*b*) chute spillways
(*c*) side channel spillways (*d*) all of these.

Q. 455. A shaft spillway is located :
(*a*) inside the body of a gravity dam
(*b*) inside the upstream reservoir
(*c*) inside the downstream reservoir
(*d*) on side flanks of the main dam.

Q. 456. The syphons installed within a gravity dam, to spill the surplus reservoir water, are known as :
(*a*) hooded type syphon spillway
(*b*) tilted outlet type syphon spillway
(*c*) both (*a*) and (*b*)
(*d*) none of these.

Q. 457. The syphons installed over overflow dams, constitute what are known as :
(*a*) tilted outlet type of syphon spillways
(*b*) hooded type of syphon spillways
(*c*) discharge carriers
(*d*) baby syphons.

Q. 458. Syphons installed over an overflow spillway will :
(*a*) decrease the effectiveness of the spillway
(*b*) increase the effectiveness of the spillway
(*c*) not affect the effectiveness of the spillway
(*d*) all the above are possible.

Q. 459. The only spillway among the following, through which the discharge does not increase as fast as it increases in all others, is :
(*a*) chute spillway (*b*) side channel spillway
(*c*) ogee spillway (*d*) shaft spillway.

Q. 460. The only spillway among the following, through which the discharge is almost at its capacity rate, even from the start of its functioning, is :
(*a*) chute spillway (*b*) side channel spillway
(*c*) ogee spillway (*d*) syphon spillway.

Q. 461*. An air vent is provided at F.R.L. to break the syphoning action at that level, in a :
(*a*) hooded type of syphon spillway
(*b*) tilted outlet type of syphon spillway
(*c*) saddle syphon spillway
(*d*) all of the above.

Q. 462. The energy dissipation at the toe of a spillway is affected basically by the use of hydraulic jump, in :
(*a*) a roller bucket
(*b*) a ski-jump bucket
(*c*) a sloping apron taken below the downstream river bed
(*d*) none of the above.

Q. 463. The most ideal condition for energy dissipation in the design of spillways is the one, when :
(*a*) the tail water rating curve lies above the jump rating curve at all discharges
(*b*) the tail water rating curve coincides with the jump rating curve at all discharges
(*c*) the tail water rating curve lies below the jump rating curve at all discharges
(*d*) the tail water rating curve either lies above or below the jump rating curve, depending upon the discharge.

Q. 464. When the tail water depths in the river, downstream of a spillway are quite low, such that the tail water curve, at all discharges, lies below the post jump depth curve, then the energy dissipation can be affected best by :

(*a*) a roller bucket (*b*) a ski-jump bucket
(*c*) either (*a*) or (*b*) (*d*) none of them.

Q. 465. Energy dissipation arrangements made at the bottom of an overfall spillway, usually depend upon the relative positions of tail water curve (T.W.C.) and jump height curve (J.H.C.) and also on Incoming Froude number. Such positions and suggested arrangements are given below in a haphazard manner, and you have to match them properly :

(*A*) T.W.C. coinciding with J.H.C. at all discharges	(*i*) ski-jump bucket at higher incoming Froude number (> 9); and sloping apron below the d/s river bed at lower froude numbers (4.5 to 9)
(*B*) T.W.C. lying below the J.H.C. at all discharges	(*ii*) a single horizontal apron at toe of the spillway at d/s river bed
(*C*) T.W.C. lying above the J.H.C. at all discharges	(*iii*) a roller bucket at higher Froude numbers, and sloping apron above the d/s river bed at lower values of *F*
(*D*) T.W.C. lying above the J.H.C. at low discharges, and below the J.H.C. at high discharges, and *vice versa*.	(*iv*) a sloping apron partly below and partly above the d/s river bed.

Q. 466. A troublesome and oscillating hydraulic jump is usually formed in flows, involving the incoming Froude number in the range of 2.5-4.5, which normally is met in cases of :

(*a*) weirs and barrages (*b*) overflow spillways of dams
(*c*) both (*a*) and (*b*) (*d*) none of them.

Q. 467. A very steady and stable hydraulic jump is usually formed in flows, involving the approaching Froude number in the range of 4.5-9.0, which normally is met in cases of :

(*a*) weirs and barrages (*b*) overflow spillways of dams
(*c*) both (*a*) and (*b*) (*d*) none of them.

Q. 468. In case of very high overflow spillways, the approaching Froude number of the flow at the toe of the spillway may exceed 9.0, bringing the hydraulic jump to be in the category of a 'strong jump'. In such a case, the energy dissipation arrangement preferably made at the toe of the spillway, may consists of :

(*a*) bucket type dissipator
(*b*) USBR stilling basin
(*c*) Indian standard stilling basin
(*d*) none of the above.

Q. 469. Standard USBR stilling basin II is useful for energy dissipation at the bottom of an overflow structure, if the approaching Froude number is :

(*a*) less than 4.5
(*b*) more than 4.5.

Q. 470. Standard stilling basin provided at the toe of a dam spillway for energy dissipation, is usually provided with auxiliary devices, like chute blocks and dentated sills, for the basic purpose of reducing the length of the stilling basin from about :

(*a*) $6y_2$ to $4y_2$ (*b*) $4y_2$ to $2y_2$
(*c*) $6y_2$ to $2y_2$ (*d*) $3y_2$ to y_2.

where y_2 is the post jump depth

Q. 471. The device which does not help in energy dissipation at the bottom of a hydraulic structure, over which water spills, is :

(*a*) chute block (*b*) dentated sill
(*c*) morning glory (*d*) none of them.

Q. 472. In a fixed roller of a spillway gate, the rollers are attached to the :

(*a*) gate (*b*) groove guide
(*c*) either (*a*) or (*b*) (*d*) none of them.

Q. 473. In vertical stoney spillway gate, the rollers are placed between the :

(*a*) gate and the u/s groove guide
(*b*) gate and the d/s groove guide
(*c*) either (*a*) or (*b*)
(*d*) none of them.

Q. 474. The spillway gate, which when lowered, can not be seen from a distance, is of the type :

(*a*) sliding gate (*b*) roller gate
(*c*) tainter gate (*d*) USBR drum gate.

Q. 475*. An ogee spillway of a concrete gravity dam having FRL of 325.0 m and MWL of 340.0 m, is provided with vertical gates between piers erected on the spillway. The effective length of the spillway is 60 m. The discharge through this spillway when gates are opened upto actual reservoir level of 331.0 m, will be :

(*a*) 850 cumecs (*b*) 1700 cumecs
(*c*) 2800 cumecs (*d*) none of them.

CHAPTER 22

Q. 476*. A 2 m diameter sluiceway at RL 300.0 m, is provided through a concrete overflow dam section to release water to the downstream, where tail water level is not more than 275.0 m. The discharge through this outlet, when the water level in the reservoir is at FRL of 330.0 m, is of the order of :

(*a*) 20 cumecs (*b*) 40 cumecs
(*c*) 50 cumecs (*d*) 80 cumecs.

Q. 477*. A 4 m diameter tunnel has been constructed through an earthen dam, with a bell mouth entry. The water levels on U/s and D/s sides of the dam are at RL 226 m and 210 m, respectively. The discharge through this outlet will be of the order of :

(*a*) 64 cumecs (*b*) 104 cumecs
(*c*) 170 cumecs (*d*) 216 cumecs.

Q. 478. A trash rack is not required at the entrance of a :

(*a*) syphon spillway (*b*) drum gate installation
(*c*) morning glory spillway (*d*) high head gate installation.

Q. 479. The outlet provided in a dam body, to release water for downstream water demand, is known as :

(*a*) spillway (*b*) sluiceway
(*c*) under-sluice (*d*) waterway.

Q. 480. Bar screens, used to cover dam outlets to prevent entry of debris or ice into the sluiceway conduits, are called :

(*a*) gate controlled ports (*b*) projecting collars
(*c*) trash racks (*d*) none of these.

Q. 481. Which of the following statement is false ?

(*a*) water can be withdrawn from any selected level of reservoir using dry intake towers
(*b*) Intake towers have to be designed for worst combination of hydrostatic, wind, earthquake, and wave forces
(*c*) Dry intake towers are lighter in construction as compared to wet intake towers
(*d*) none of the above.

Q. 482. A sluiceway conduit passing through an earthen dam has a length equal to L; and it has x projecting collars-each having projected length equal to x. The increased length of seepage path in this case is given by :

(*a*) $L + 2nx$ (*b*) $L + nx$
(*c*) $2nx$ (*d*) none of them.

Q. 483. Projecting collars are provided on sides of a rectangular tunnel of length (L), made through an earthen dam, to increase the seepage path. Their projection length (X) and numbers (N) are decided, as to provide increased seepage path equal to 2NX, where 2NX should generally be :

(*a*) greater than $\frac{L}{4}$ (*b*) greater than $\frac{L}{2}$
(*c*) greater than L (*d*) none of the above.

CHAPTER 23

Q. 484. The flow through Penstocks and Pressure conduits is, generally :

(*a*) laminar (*b*) turbulent
(*c*) both (*a*) and (*b*) (*d*) none of these.

Q. 485. The head lost in the flow of water through a penstock pipe of given length:

(*a*) increases with the increase in flow velocity
(*b*) decreases with the increase in flow velocity
(*c*) decreases with the increase in the roughness of the pipe surface
(*d*) decreases with time.

Q. 486. Hoop's reinforcement is provided in cement concrete pressure pipes, in order to counteract the;

(*a*) water hammer pressure
(*b*) internal water pressure
(*c*) stresses caused by external backfills
(*d*) both (*a*) and (*b*)
(*e*) (*a*), (*b*) and (*c*) all.

Q. 487. When a gate valve, installed in a pressure pipe, is suddenly closed, water hammer pressure is caused on the pipe shell, in the portion :
(*a*) downstream of the valve (*b*) upstream of the valve
(*c*) both (*a*) and (*b*) (*d*) none of the above.

Q. 488. Which of the following types of pressure conduits is preferably used for large heads?
(*a*) P.C.C. pipes (*b*) Cast iron pipes
(*c*) Prestressed concrete pipes (*d*) Asbestos pipes.

CHAPTER 24

Q. 489. A hydropower plant, developed at the site of a drop in an irrigation canal, is of the type :
(*a*) runoff river plant (*b*) storage plant
(*c*) pumped storage plant (*d*) tidal plant.

Q. 490. A nuclear power development scheme is essentially a :
(*a*) thermal power scheme (*b*) hydropower scheme
(*c*) neither (*a*) nor (*b*) (*d*) both (*a*) as well as (*b*).

Q. 491. Fossil fuels are used in :
(*a*) a nuclear power plant (*b*) an ordinary thermal power plant
(*c*) a hydro power plant (*d*) none of them.

Q. 492. Major hydropower generation in India is from :
(*a*) runoff river plants (*b*) storage plants
(*c*) pumped storage plants (*d*) tidal plants.

Q. 493. A run off river plant for hydro power generation is essentially a :
(*a*) high head scheme (*b*) low head scheme
(*c*) medium head scheme (*d*) any of these.

Q. 494. A storage hydro plant essentially involves :
(*a*) a barrage or a weir (*b*) a dam
(*c*) either (*a*) or (*b*) (*d*) neither (*a*) nor (*b*).

Q. 495. The natural highwater falls may be used for generation of :
(*a*) hydro-power (*b*) water-power
(*c*) both (*a*) and (*b*) (*d*) neither (*a*) nor (*b*).

Q. 496. The power plant among the following, which is a storage plant, is :
(*a*) Rance power station in France, located on sea
(*b*) Ganguwal power house, located on Nangal hydel channel in India
(*c*) Sarda power house, located on Sarda Canal in U.P. in India.
(*d*) none of these.

Q. 497. The only statement, which is incorrect in regard to hydropower, is :
(*a*) the system efficiency of a hydroplant is quite high
(*b*) the installation cost of a hydroplant is very high
(*c*) the running cost of a hydro power plant is very low
(*d*) the hydraulic turbines takes a lot of time in being put off and on.

Q. 498. Identify the correct statement in regard to hydropower :
(*a*) hydropower stations are generally labour oriented
(*b*) Gestation period for hydro-power plant is usually small
(*c*) the hydro generators give high efficiency over a wide range of load
(*d*) in a hydropower scheme, the firm power is usually high, as compared to total power.

Q. 499. The minimum power, which a hydro-power plant can generate throughout the year, is called :
(*a*) power plant capacity (*b*) power plant load
(*c*) firm power (*d*) water power.

Q. 500. 400 cumecs of water is being released from a dam storage to meet the downstream demand, through the turbines of the connected hydroplant. The effective head of water acting on the turbines is 50 m. The efficiency of the hydroplant is 0.8. The electrical power generated from this plant, would be:
(*a*) 1,56,800 MW (*b*) 156.8 MkW
(*c*) 156.8 MW (*d*) none of these.

Q. 501. If the peak load for a power plant equals the plant capacity, then the ratio of the capacity factor to load factor will be :
(*a*) 1 (*b*) 0
(*c*) <1 (*d*) >1
(*e*) – 1 (*f*) none of these.

Q. 502*. If the peak load on a power plant having a capacity of 100 MW is 70 MW during a given week, and the energy produced is 58,80,000 kwh, the capacity factor for the plant for the week, will be :
(*a*) 35% (*b*) 50%
(*c*) 70% (*d*) none of these.

Q. 503*. The load factor for the data given in the above question will be :
(*a*) 0.35 (*b*) 0.50
(*c*) 0.70 (*d*) none of these.

Q. 504*. The utilisation factor for the data given in Q. 502, will be :
(*a*) 0.35 (*b*) 0.50
(*c*) 0.70 (*d*) none of these.

Q. 505. Pondage requirement in a hydropower plant includes the need :
(*a*) to balance the varying demand
(*b*) to compensate for wastage and spillage
(*c*) to balance short time fluctuations in the flow
(*d*) all of these.

Q. 506. A Pelton's turbine is a :
(*a*) velocity turbine (*b*) reaction turbine
(*c*) pressure turbine (*d*) none of these.

Q. 507. An impulse turbine, like a pelton's wheel, is used for :
(*a*) low heads and high discharges
(*b*) high heads and low discharges
(*c*) low heads and low discharges
(*d*) high heads and high discharges.

Q. 508. An impulse turbine, like a Pelton's turbine, is :
(*a*) an open body exposed to atmosphere
(*b*) a closed body not exposed to atmosphere
(*c*) a partially open and partially closed body
(*d*) none of these.

Q. 509. In a reaction turbine, like a Francis turbine or Kaplan turbine,
(*a*) the potential head of water is totally changed into kinetic head
(*b*) the potential head of water is partially changed into kinetic head
(*c*) the potential head of water is not at all changed into kinetic head
(*d*) all of the above are possible.

Q. 510. In a Francis turbine, the water
(*a*) flows out openly
(*b*) flows out through a closed draft tube
(*c*) both (*a*) and (*b*), depending upon the type
(*d*) neither (*a*) nor (*b*).

Q. 511*. You have to select turbines for a hydropower plant, having 150 m head of stored water behind the dam. The water in the reservoir is clear, and load on the power house is not likely to fluctuate much. Which type of turbines will you generally recommend?
(*a*) Pelton's turbines (*b*) Francis turbines
(*c*) Kaplan turbines (*d*) Any of them.

Q. 512. You have to select turbines for a hydropower plant, working on 150 m head. The water is sandy and load on the plant is highly variable. Which type of turbines will you generally recommend ?
(*a*) Pelton's turbines (*b*) Francis turbines
(*c*) Kaplan turbines (*d*) Any of them will do.

Q. 513. If two identical pumps, each capable to deliver a discharge Q against a head H, are connected :
(*a*) in parallel, the resulting discharge is $2Q$ against a head H
(*b*) in parallel, the resulting discharge is Q against a head of $2H$
(*c*) in series, the resulting discharge is $2Q$ against a head H
(*d*) in series, the resulting discharge is Q against a head $2H$.

CHAPTER 25

Q. 514. The minimum depth of water required for safe and economical navigation is:
(*a*) 1 m (*b*) 2.0 m
(*c*) 2.7 m (*d*) 4.2 m

Q. 515 The transit time taken by a barge tow, moving upstream in a navigable river, increases, if :
(*a*) the flow velocity is more (*b*) the flow velocity is less.

Q. 516. Inorder to improve the navigability in a river, the cutoffs are :
(*a*) encouraged (*b*) discouraged.

Q. 517. The belt conveyor forming an essential part of a ladder dredger, used for excavating river sand deposits, is used for :
(*a*) excavation of sand
(*b*) transporting excavated sand to the surface, over small distances
(*c*) transporting excavated sand to the surface, over large distances
(*d*) none of these.

Q. 518. The dredger which excavates sand deposits of a river, and discharges the excavated material with water over long distances, is called a :
(*a*) ladder dredger (*b*) suction dredger
(*c*) dipper dredger (*d*) none of these.

Q. 519. Lock and dam constructions for creation of a series of slack water pools along a river, for promoting its navigability, is adopted only when :
(*a*) the river is perennial
(*b*) the river carries heavy sediment load
(*c*) the available water in the river is quite less
(*d*) none of the above.

Q. 520. Timber is generally transported in our country through :

(*a*) rail routes (*b*) road routes
(*c*) air routes (*d*) water routes.

Q. 521. The oldest and cheapest mode of transporting cargo is by :

(*a*) rail routes (*b*) road routes
(*c*) air routes (*d*) water routes.

Q. 522. Ships or Steamers can ply to carry cargo, in Indian waterways, extending to about :

(*a*) 500 km (*b*) 2500 km
(*c*) 10,000 km (*d*) 40,000 km.

CHAPTER 26

Q. 523. The depth of water stored in an irrigation tank, rarely exceeds :

(*a*) 4 m (*b*) 8 m
(*c*) 12 m (*d*) none of them.

Q. 524. The storage created on the upstream side of a smaller stream by construction of low earthen bunds, is technically known as :

(*a*) a reservoir (*b*) a tank
(*c*) a lake (*d*) none of these.

Q. 525. The usual arrangement made in a tank-bund scheme to spill the surplus excess water, is known as a :

(*a*) tank spillway (*b*) tank sluice
(*c*) tank weir (*d*) none of these.

Q. 526. The provision of suitable breaching sections become more important in irrigation tanks, which are :

(*a*) isolated (*b*) in groups
(*c*) both (*a*) and (*b*) (*d*) none of them.

Q. 527. The cubic metres of water that can be stored in an irrigation tank between FTL (Full Tank Level) and Sill level of the lowest supply sluice, is known as:

(*a*) the gross capacity (*b*) effective capacity
(*c*) the specific capacity (*d*) none of the above.

Q. 528. If A_1 represents the area of a tank bed at bottom, and A_2 at FTL, then the storage capacity of this tank of height H, is given as :

(*a*) $H\left[A_1 + A_2 + \sqrt{A_1 A_2}\right]$ (*b*) $\frac{H}{2}\left[A_1 + A_2 + \sqrt{A_1 A_2}\right]$
(*c*) $\frac{H}{3}\left[A_1 + A_2 + \sqrt{A_1 A_2}\right]$ (*d*) none of them.

Q. 529. The top level of a tank weir in a tank bund scheme, is kept at :

(*a*) MTL (*b*) FTL
(*c*) either (*a*) or (*b*) (*d*) none of these.

Q. 530. The capacity of an irrigation tank is sometimes increased by installing a temporary stone wall over the top of the tank weir. This fixture is known is :

(*a*) saddle (*b*) dam stones
(*c*) breast wall (*d*) divide wall.

Q. 531. The discharge through the sluice of a small irrigation tank is usually controlled by a :
(*a*) dam stone (*b*) plug
(*c*) shutter gate (*d*) none of these.

Q. 532. The mostly adopted earthen section for tank bunds is of the type :
(*a*) homogeneous embankment type
(*b*) zoned embankment type
(*c*) diaphragm type
(*d*) none of the above.

Q. 533*. The discharging capacity of a masonry waste weir of 50 m length and 1 m width provided in an earthen bund, for passing a flood with 0.8 m water depth over its crest, will be :
(*a*) 12.25 cumecs (*b*) 25.4 cumecs
(*c*) 59.4 cumecs (*d*) 65.8 cumecs.

CHAPTER 27

Q. 534. An arch dam, looks like a single arch, in :
(*a*) plan (*b*) front elevation
(*c*) side elevation (*d*) none of them.

Q. 535. An arch dam behaves as :
(*a*) a cantilever vertical retaining wall, standing up from its base
(*b*) an arch, transferring loads at the two ends by horizontal arch action
(*c*) both (*a*) and (*b*)
(*d*) none of the above.

Q. 536. A non-vertical arch dam is known as a :
(*a*) buttress dam (*b*) double curvature arch dam
(*c*) shell arch dam (*d*) both (*b*) and (*c*) above.

Q. 537. Arch dams are suitable in locations where the canyon slope factor (length of arch at top/height of dam) is :
(*a*) less than 8 (*b*) greater than 16
(U.P.S.C. Engg. Services)

Q. 538. The most economical type of arch dam in general is of :
(*a*) constant radius type (*b*) variable radius type
(*c*) constant angle type (*d*) none of these.

Q. 539. A constant angle arch dam, when compared to constant radius arch dam, utilises concrete quantity of about :
(*a*) 43% (*b*) 130%
(*c*) 230% (*d*) none of them.

Q. 540. Amongst the following types of dam sections, the thinnest and most economical section is expected for a :
(*a*) Constant angle arch dam (*b*) shell-arch dam
(*c*) constant radius arch dam (*d*) concrete gravity dam.

Q. 541. Which of the following forces is the least important in the design of arch dams?
(*a*) reservoir water force (*b*) uplift pressure
(*c*) temperature stresses (*d*) ice load
(*e*) yield stresses.

Q. 542. A constant radius arch dam is also sometimes called a :
(*a*) constant centre arch dam (*b*) constant angle arch dam
(*c*) variable angle arch dam (*d*) none of the above.

Q. 543. The most economical central angle of the arch rings in an arch dam, is :
(*a*) 123°—34′ (*b*) 133°—34′
(*c*) 143°—34′ (*d*) 153°—34′.

Q. 544. The most economical central angle of the arch rings of an arch dam, can be adopted only at one place, preferably at mid height in an arch dam of the type :
(*a*) constant angle arch dam (*b*) constant radius arch dam
(*c*) both (*a*) and (*b*) above (*d*) none of the above.

Q. 545. In an arch dam, the 'extrodos curves' refer to the arch rings corresponding to the :
(*a*) upstream face of the dam
(*b*) downstream face of the dam
(*c*) either (*a*) or (*b*)
(*d*) none of the above.

Q. 546. A V-shaped valley with stronger foundations can suggest the choice of an arch dam of the type :
(*a*) constant radius arch dam
(*b*) variable radius arch dam
(*c*) constant angle arch dam
(*d*) none of the above.

Q. 547. The type of an arch dam, which generally requires 'overhangs at abutments', is of :
(*a*) constant radius type (*b*) variable radius type
(*c*) constant angle type (*d*) none of the above.

Q. 548. The most accurate method to design arch dams is based upon :
(*a*) the thin cylinder theory (*b*) the theory of elastic arches
(*c*) the trial load method (*d*) all of the above.

Q. 549. Which among the following is not an assumption of the 'Trial Load Analysis' method of design of arch dams ?
(*a*) plane sections normal to the axis remain plane after flexure
(*b*) any horizontal arch ring is independent of the arch rings below and above
(*c*) modulus of elasticity of concrete and rock foundation is equal for compression and tension
(*d*) stresses are proportional to strains.

Q. 550. The thin cylinder theory for designing arch dams, ignores :
(*a*) temperature stresses (*b*) ice pressures
(*c*) yield stresses (*d*) all of the above.

Q. 551. The only force considered in the design of arch dams by thin cylinder theory is :
(*a*) water pressure (*b*) yield stresses
(*c*) shrinkage stresses (*d*) none of them.

Q. 552. The temperature stresses, producing worst effects in the design of arch dams, are caused by :
(*a*) rise of temperatures (*b*) fall of temperatures
(*c*) both (*a*) and (*b*) (*d*) none of the above.

Q. 553. The stresses due to rib-shortening become quite important in the case of :
(*a*) long thin arch dams (*b*) thick arch dams of small angle
(*c*) both (*a*) and (*b*) above (*d*) none of the above.

Q. 554. While designing a freely supported deck type of a buttress dam, the chosen spacing between buttresses is usually increased to achieve economy, from about 4.5 to 15 m or so, with the :
(*a*) decrease in dam height from about 45 m to about 15 m
(*b*) increase in dam height from about 15 m to about 45 m.

Q. 555. The load from the deck of a buttress dam is transmitted to the foundations, through the :
(*a*) corbels (*b*) buttresses
(*c*) lateral braces (*d*) none of them.

Q. 556. Which of the following components in a buttress dam are responsible for transmission of load from the sloping deck slab to the buttresses?
(*a*) corbels (*b*) lateral braces
(*c*) counterforts (*d*) none of them.

Q. 557. The most suitable foundation for buttress dams is :
(*a*) pile foundation (*b*) well foundation
(*c*) mat foundation (*d*) none of them.

Q. 558. The lateral braces are provided between buttresses in a buttress dam, to :
(*a*) transfer the load from the deck to the foundation
(*b*) transfer the load from the deck to the buttresses
(*c*) provide resistance against buckling of buttresses
(*d*) none of the above.

Q. 559. No reinforcement is necessary in the :
(*a*) fixed deck buttress dam
(*b*) simple deck buttress dam
(*c*) cantilever deck buttress dam
(*d*) massive head buttress dam.

Q. 560. Which of the following dam is also known as Ambursen dam?
(*a*) multiple arch buttress dam
(*b*) mushroom head buttress dam
(*c*) massive head buttress dam
(*d*) free deck buttress dam.

Q. 561. Which of the following component of a buttress dam is also called a counterfort?
(*a*) corbel (*b*) buttress
(*c*) lateral brace (*d*) sloping deck.

Q. 562. Which of the following is incorrect about buttress dam?
(*a*) it requires less construction material than a solid gravity dam of the same height
(*b*) it is less susceptible to deliberate damage or sabotage
(*c*) Ice pressure is not significant, because ice slides over the sloping deck
(*d*) the factor of safety is usually greater than for a gravity dam.

Q. 563. Multiple dome buttress dam is a type of :
(*a*) rigid buttress dam (*b*) deck slab buttress dam
(*c*) bulk head buttress dam (*d*) none of the above.

Q. 564. In a simple deck slab buttress dam of a given height, the buttress spacing can be increased by :

(*a*) increasing the upstream slope
(*b*) decreasing the upstream slope
(*c*) either (*a*) or (*b*)
(*d*) none of these.

Q. 565. The buttresses of a free deck buttress dam are provided @ 'x' m clear spacing, and buttress thickness is proposed to be 't' m. The buttress section can then be designed like the section of a concrete gravity dam, by considering the effective unit weight of water = surcharge factor(S) × actual unit weight of water (γ_w). The value of this surcharge factor is given as :

(*a*) $S = \frac{x+t}{x}$ (*b*) $S = \frac{x+t}{t}$

(*c*) $S = \left(\frac{x+t}{1+t}\right)$ (*d*) none of these.

Q. 566. Multiple arch dams are generally used for :

(*a*) high heights (*b*) low heights
(*c*) medium heights (*d*) all of the above.

ANSWERS OF THE OBJECTIVE QUESTIONS

1. (*d*)	2. (*a*)	3. (*d*)	4. (*a*)	5. (*c*)
6. (*a*)	7. (*b*)	8. (*b*)	9. (*d*)	10. (*a*)
11. (*b*)	12. (*c*)	13. (*a*)	14. (*b*)	15. (*b*)
16. (*b*)	17. (*b*)	18. (*d*)	19. (*b*)	20. (*c*)
21. (*a*)	22. (*c*)	23. (*a*)	24. (*c*)	25. (*b*)
26. (*b*)	27. (*d*)	28. (*b*)	29. (*a*)	30. (*c*)
31. (*c*)	32. (*a*)	33. (*c*)	34. (*b*)	35. (*b*)
36. (*b*)	37. (*b*)	38. (*b*)	39. (*c*)	40. (*a*)
41. (*c*)	42. (*b*)	43. (*c*)	44. (*c*)	45. (*b*)
46. (*c*)	47. (*c*)	48. (*c*)	49. (*b*)	50. (*a*)
51. (*d*)	52. (*d*)	53. (*c*)	54. (*c*)	55. (*c*)
56. (*b*)	57. (*d*)	58. (*d*)	59. (*a*)	60. (*a*)
61. (*b*)	62. (*c*)	63. (*b*)	64. (*d*)	65. (*a*)
66. (*c*)	67. (*d*)	68. (*c*)	69. (*b*)	70. (*c*)
71. (*c*)	72. (*a*)	73. (*a*)	74. (*b*)	75. (*d*)
76. (*b*)	77. (*b*)	78. (*d*)	79. (*b*)	80. (*d*)
81. (*c*)	82. (*c*)	83. (*b*)	84. (*a*)	85. (*b*)
86. (*c*)	87. (*a*)	88. (*c*)	89. (*c*)	90. (*d*)
91. (*d*)	92. (*c*)	93. (*a*)	94. (*a*)	95. (*c*)
96. (*c*)	97. (*b*)	98. (*d*)	99. (*b*)	100. (*c*)
101. (*c*)	102. (*b*)	103. (*b*)	104. (*c*)	105. (*b*)
106. (*a*)	107. (*b*)	108. (*a*)	109. (*b*)	110. (*d*)
111. (*b*)	112. (*b*)	113. (*b*)	114. (*b*)	115. (*b*)
116. (*a*)	117. (*c*)	118. (*c*)	119. (*a*)	120. (*c*)
121. (*b*)	122. (*b*)	123. (*c*)	124. (*c*)	125. (*c*)
126. (*a*)	127. (*b*)	128. (*c*)	129. (*c*)	130. (*d*)
131. (*a*)	132. (*a*)	133. (*c*)	134. (*b*)	135. (*c*)

136. (*c*)	137. (*d*)	138. (*d*)	139. (*d*)	140. (*d*)
141. (*d*)	142. (*d*)	143. (*e*)	144. (*c*)	145. (*d*)
146. (*c*)	147. (*a*)	148. (*c*)	149. (*d*)	150. (*a*)
151. (*a*)	152. (*b*)	153. (*c*)	154. (*c*)	155. (*d*)
156. (*b*)	157. (*c*)	158. (*d*)	159. (*b*)	160. (*a*)
161. (*c*)	162. (*a*)	163. (*b*)	164. (*a*)	165. (*d*)
166. (*b*)	167. (*b*)	168. (*b*)	169. (*b*)	170. (*c*)
171. (*a*)	172. (*b*)	173. (*a*)	174. (*a*)	175. (*c*)
176. (*a*)	177. (*a*)	178. (*c*)	179. (*b*)	180. (*d*)
181. (*c*)	182. (*c*)	183. (*b*)	184. (*d*)	185. (*e*)
186. (*a*)	187. (*c*)	188. (*e*)	189. (*d*)	190. (*b*)
191. (*d*)	192. (*d*)	193. (*a*)	194. (*a*)	195. (*d*)
196. (*c*)	197. (*d*)	198. (*c*)	199. (*b*)	200. (*c*)
201. (*b*)	202. (*b*)	203. (*b*)	204. (*d*)	205. (*b*)
206. (*d*)	207. (*b*)	208. (*b*)	209. (*b*)	210. (*a*)
211. (*a*)	212. (*c*)	213. (*b*)	214. (*d*)	215. (*b*)
216. (*b*)	217. (*c*)	218. (*a*)	219. (*a*)	220. (*a*)
221. (*d*)	222. (*a*)	223. (*c*)	224. (*c*)	225. (*b*)
226. (*d*)	227. (*b*)	228. (*c*)	229. (*a*)	230. (*b*)
231. (*b*)	232. (*d*)	233. (*c*)	234. (*d*)	235. (*d*)
236. (*b*)	237. (*c*)	238. (*c*)	239. (*c*)	240. (*c*)
241. (*c*)	242. (*b*)	243. (*d*)	244. (*d*)	245. (*c*)
246. (*d*)	247. (*a*)	248. (*c*)	249. (*d*)	250. (*b*)
251. (*a*)	252. (*b*)	253. (*e*)	254. (*b*)	255. (*b*)
256. (*b*)	257. (*c*)	258. (*d*)	259. (*c*)	260. (*d*)
261. (*c*)	262. (*d*)	263. (*c*)	264. (*c*)	265. (*b*)
266. (*c*)	267. (*c*)	268. (*d*)	269. (*b*)	270. (*b*)
271. (*a*)	272. (*c*)	273. (*b*)	274. (*b*)	275. (*c*)
276. (*d*)	277. (*b*)	278. (*c*)	279. (*b*)	280. (*a*)
281. (*b*)	282. (*b*)	283. (*a*)	284. (*b*)	285. (*d*)
286. (*b*)	287. (*d*)	288. (*a*)	289. (*d*)	290. (*c*)
291. (*c*)	292. (*b*)	293. (*c*)	294. (*c*)	295. (*a*)
296. (*c*)	297. (*b*)	298. (*a*)	299. (*e*)	300. (*b*)
301. (*a*)	302. (*d*)	303. (*b*)	304. (*a*)	305. (*a*)
306. (*b*)	307. (*d*)	308. (*d*)	309. (*d*)	310. (*c*)
311. (*c*)	312. (*d*)	313. (*a*)	314. (*c*)	315. (*c*)
316. (*b*)	317. (*d*)	318. (*d*)	319. (*a*)	320. (*d*)
321. (*a*)	322. (*b*)	323. (*d*)	324. (*c*)	325. (*d*)
326. (*d*)	327. (*a*)	328. (*c*)	329. (*b*)	330. (*b*)
331. (*d*)	332. (*c*)	333. (*b*)	334. (*b*)	335. (*d*)
336. (*a*)	337. (*c*)	338. (*c*)	339. (*a*)	340. (*d*)
341. (*b*)	342. (*c*)	343. (*c*)	344. (*a*)	345. (*a*)
346. (*b*)	347. (*c*)	348. (*b*)	349. (*a*)	350. (*b*)
351. (*b*)	352. (*d*)	353. (*d*)	354. (*b*)	355. (*a*)
356. (*b*)	357. (*a*)	358. (*b*)	359. (*c*)	360. (*e*)
361. (*c*)	362. (*b*)	363. (*b*)	364. (*a*)	365. (*b*)

366. (*c*)	367. (*d*)	368. (*c*)	369. (*b*)	370. (*b*)
371. (*c*)	372. (*b*)	373. (*a*)	374. (*c*)	375. (*b*)
376. (*a*)	377. (*a*)	378. (*d*)	379. (*b*)	380. (*b*)
381. (*c*)	382. (*a*)	383. (*b*)	384. (*c*)	385. (*c*)
386. (*a*)	387. (*b*)	388. (*c*)	389. (*b*)	390. (*b*)
391. (*a*)	392. (*a*)	393. (*c*)	394. (*d*)	395. (*a*)
396. (*a*)	397. (*c*)	398. (*b*)	399. (*a*)	400. (*b*)
401. (*c*)	402. (*a*)	403. (*c*)	404. (*d*)	405. (*d*)
406. (*a*)	407. (*c*)	408. (*c*)	409. (*b*)	410. (*b*)
411. (*a*)	412. (*b*)	413. (*c*)	414. (*d*)	415. (*e*)
416. (*b*)	417. (*c*)	418. (*b*)	419. (*c*)	420. (*c*)
421. (*d*)	422. (*d*)	423. (*c*)	424. (*a*)	425. (*a*)
426. (*d*)	427. (*a*)	428. (*b*)	429. (*c*)	430. (*d*)
431. (*c*)	432. (*b*)	433. (*c*)	434. (*c*)	435. (*c*)
436. (*b*)	437. (*b*)	438. (*a*)	439. (*a*)	440. (*b*)
441. (*a*)	442. (*d*)	443. (*b*)	444. (*b*)	445. (*b*)
446. (*a*)	447. (*b*)	448. (*d*)	449. (*a*)	450. (*a*)
451. (*a*)	452. (*a*)	453. (*b*)	454. (*d*)	455. (*b*)
456. (*b*)	457. (*b*)	458. (*b*)	459. (*d*)	460. (*d*)
461. (*d*)	462. (*c*)	463. (*b*)	464. (*b*)	

465. A (*ii*)
(B) (*i*)
(C) (*iii*)
(D) (*iv*)

466. (*a*)	467. (*b*)	468. (*a*)	469. (*b*)	470. (*a*)
471. (*c*)	472. (*a*)	473. (*b*)	474. (*d*)	475. (*b*)
476. (*c*)	477. (*c*)	478. (*b*)	479. (*b*)	480. (*c*)
481. (*c*)	482. (*c*)	483. (*a*)	484. (*b*)	485. (*a*)
486. (*d*)	487. (*b*)	488. (*c*)	489. (*a*)	490. (*a*)
491. (*b*)	492. (*b*)	493. (*b*)	494. (*b*)	495. (*c*)
496. (*d*)	497. (*d*)	498. (*c*)	499. (*c*)	500. (*c*)
501. (*a*)	502. (*a*)	503. (*b*)	504. (*c*)	505. (*d*)
506. (*a*)	507. (*b*)	508. (*a*)	509. (*b*)	510. (*b*)
511. (*b*)	512. (*a*)	513. (*a*)	514. (*c*)	515. (*b*)
516. (*a*)	517. (*b*)	518. (*b*)	519. (*c*)	520. (*d*)
521. (*d*)	522. (*b*)	523. (*c*)	524. (*b*)	525. (*c*)
526. (*b*)	527. (*b*)	528. (*c*)	529. (*b*)	530. (*b*)
531. (*b*)	532. (*a*)	533. (*c*)	534. (*a*)	535. (*b*)
536. (*d*)	537. (*a*)	538. (*c*)	539. (*a*)	540. (*b*)
541. (*b*)	542. (*a*)	543. (*b*)	544. (*b*)	545. (*a*)
546. (*c*)	547. (*c*)	548. (*c*)	549. (*b*)	550. (*d*)
551. (*a*)	552. (*b*)	553. (*b*)	554. (*b*)	555. (*b*)
556. (*a*)	557. (*c*)	558. (*c*)	559. (*d*)	560. (*d*)
561. (*b*)	562. (*b*)	563. (*a*)	564. (*a*)	565. (*b*)
566. (*a*).				

HINTS FOR SOLVING STARRED QUESTIONS

Q. 16. *Follow example 1.1, and compute :*

From eqn (1.1)

$$T = 2.3 \cdot \frac{y}{f} \log_{10}\left(\frac{Q}{Q - f \cdot A}\right)$$

where $A = 0.203 \text{ ha} = 2030 \text{ m}^2$

$Q = 0.043 \text{ cumecs} = 0.043 \times 60 \times 60 \text{ m}^3/\text{h}$

$= 154.8 \text{ m}^3/\text{h} = 5 \text{ cm/hr} = 0.05 \text{ m/h}$

$y = 6.35 \text{ cm} = 0.0635 \text{ m}$

$$T = 2.3 \times \frac{0.0635}{0.05} \log\left(\frac{154.8}{154.8 - 0.05 \times 2030}\right) = \mathbf{1.35\ h.} \quad \textbf{Ans.}$$

Q. 17. Combining wt $= \dfrac{\text{Atomic wt}}{\text{Valency}}$

$\therefore$ Combining wt of Na $= \dfrac{23}{1} = 23$

Combining wt of Ca $= \dfrac{40}{2} = 20$

Combining wt of Mg $= \dfrac{24}{2} = 12$

$$Na^+ = \frac{\text{Na in mg/l}}{23} = \frac{345}{23} = 15 \text{ milli equivalent per litre}$$

$$Ca^{++} = \frac{60}{20} = 3 \text{ milli equivalent per litre}$$

$$Mg^{++} = \frac{18}{12} = 1.5 \text{ milli equivalent per litre}$$

From eqn. (1.4) :

$$SAR = \frac{Na^+}{\sqrt{\dfrac{Ca^{++} + Mg^{++}}{2}}} = \frac{15}{\sqrt{\dfrac{3 + 1.5}{2}}} = \frac{15}{1.5} = \mathbf{10.} \quad \textbf{Ans.}$$

Q. 45. Specific weight means unit weight $= 14.7 \text{ kN/m}^3$.

Also, unit wt of water $= 9.8 \text{ kN/m}^3$:

Available moisture holding capacity of root zone

$$= \frac{\gamma d}{\gamma_w}\left(\text{F.C. m.c.} - \text{Wilting pt m.c.}\right)$$

$$= \left(\frac{14.7}{9.8}\right) \times (0.9\ m)\left[25\% - 15\%\right]$$

$$= 1.5 \times (0.9 \text{ m}) \times 10\% = 0.135 \text{ m} = \mathbf{13.5\ cm.} \quad \textbf{Ans.}$$

Q.47. Water depth required at canal

$$= \frac{\text{Water depth required in the field}}{\eta_a \cdot \eta_c}$$

$$= \frac{10 \text{ cm}}{0.8 \times 0.9} = 13.9 \text{ cm}$$

∴ Volume of water required for 10 hectare (10×10^4 m^2) field

$$= \frac{13.9}{100} \text{ m} \times 10^5 \text{ m}^2 = 13{,}900 \text{ m}^3 = \mathbf{13{,}900 \text{ kl.}} \quad \textbf{Ans.}$$

Q. 48. **Storage duty** or **Quantity duty** is the crop hectares, which can be fully grown/irrigated by 1 M. cum of water.

Thus, if Δ (m) is the water depth required by the crop for its maturity, then D hectares of crop will evidently require $(D \times 10^4)\,\Delta$ m^3 of water, which is 1 M. cum.

$$\therefore \quad (D \times 10^4\,\Delta \text{ m}^3 = 1 \text{ M cum} = 10^6 \text{ m}^3$$

or $$D \times \Delta = 100$$

or $$\Delta = \frac{100}{D}$$

where Δ is in m

D is in ha/M. cum.

Q. 55. I.I. for Kharif $= 100 - 72 = 28\%$

I.I. for Rabi $= 100 - 56 = 44\%$

Total I.I. $= 28 + 44 = \mathbf{72\%}$. **Ans.**

Q. 57. Such an area may be called as a seasonal cropped area, or seasonal sown area, such as *Rabi area or Kharif area.* Net sown area is the total cropped area (which is only cropped once) in the entire year, *i.e.* in both the seasons.

Q. 58. $G.1.A = 4.5 \times 0.9 + 2.5 \times 0.8 = 6.05$ M ha

$$A.I.I = \frac{G.I.A.}{CCA} = \frac{6.05}{5} = \mathbf{1.21\ i.e.\ 121\%}. \quad \textbf{Ans.}$$

Q. 66. Required discharge (mean) in the field

$= 0.4$ cumecs.

Peak discharge required

$$= \frac{\text{Mean discharge}}{\text{Capacity factor}} \qquad \left(\because \ C.F. = \frac{Q_{mean}}{Q_{peak}}\right)$$

$$= \frac{0.4}{0.8} = 0.5 \text{ cumecs}$$

Design discharge for distributary

$$= \frac{\text{Required discharge for crops}}{\text{Time factor}}$$

(Since channel runs for lesser days than the crop days, the required channel discharge should be more)

$$= \frac{0.5}{0.5} = \mathbf{1 \text{ cumecs.}} \quad \textbf{Ans.}$$

Q. 67. Discharge required at head of canal

$$= \frac{6.25 \text{ cumecs}}{0.75} = 8.33 \text{ cumecs}$$

Area irrigated $= 5100$ hectares

Duty on capacity

$$= \frac{\text{Area to be irrigated during base period}}{\text{Design full supply discharge at its head during peak demand}}$$

$$= \frac{5100}{8.33} = \mathbf{612\ ha/cumecs.}\quad \textbf{Ans.}$$

Q. 69. $G.C.A = 8000$ ha

$CCA = GCA -$ Reserved forests, etc. $= 8000 - 1600 = 6400$ ha

Pastures and fallow lands are included in CCA, and hence not to be subtracted.

Area under irrigation

$$= CCA \times \text{In. of Irr.} = 6400 \times 0.5 = 3200 \text{ hectares.}\quad \textbf{Ans.}$$

Q. 79.
$$\frac{\tau_0}{\gamma \cdot d\,(S_s - 1)} = 0.056$$

or $\tau_\theta \propto d.$ **Ans.**

Q. 80. For non-scouring

$$d < 11\,RS$$

or
$$R_{max} = \frac{d}{11S} \qquad ...(i)$$

$\therefore$
$$V = \frac{1}{N} \cdot R^{2/3} \cdot S^{1/2}$$

But
$$N = \frac{1}{24} d^{1/6}$$

$\therefore$
$$V = \frac{24}{d^{1/6}} \cdot R^{2/3} \cdot S^{1/2}$$

$$V_{max} = \frac{24}{d^{1/6}} \cdot R_{max}^{2/3} \cdot S^{1/2}$$

$$= \frac{24}{d^{1/6}} \cdot \left(\frac{d}{11S}\right)^{2/3} \cdot S^{1/2}$$

$$= \frac{24}{(11)^{2/3}} \cdot d^{2/3 - 1/6} \cdot S^{1/2 - 2/3}$$

$$= 4.85\, d^{1/2} \cdot S^{-1/6} \qquad \text{where } d \text{ is in m}$$

When d is in cm *i.e.* d_{cm}, then

$$d = \frac{d_{cm}}{100}$$

$\therefore$
$$V_{max} = 4.85 \left(\frac{d\,cm}{100}\right)^{1/2} \cdot S^{-1/6}$$

$$= 0.485\, d^{1/2} \cdot S^{-1/6} \qquad \text{where } d \text{ is in cm}$$

or $V_{max} \approx 0.48\, d^{1/2} \cdot S^{-1/6}.$ **Ans.**

Q. 81. N is given by Strickler's formula, as

$$N = \frac{1}{24} \cdot d^{1/6}$$

where d is representative grain dia in m.

Q. 85. Such channels are designed only for non-scouring. Hence, they must carry clear waters only. **Ans.**

Q. 86. For 1.5 H : 1 V,

$$\tan\theta = \frac{y}{x} = \frac{1}{1.5}$$

$$\sin\theta = 0.554$$

$$\frac{\tau_c'}{\tau_c} = \sqrt{1 - \frac{\sin^2\theta}{\sin^2\phi}} = \sqrt{1 - \left(\frac{0.554}{\sin 37°}\right)^2}$$

$$= \sqrt{0.153} = 0.391.$$

For stability on banks

$$0.75\,\gamma RS \le 0.391\,\tau_c$$

$$\le 0.391\,(\gamma RS)$$

For stability on bed,

$$RS = \frac{d}{11}$$

$$0.75\,\gamma \cdot RS \le 0.391 \cdot \gamma \cdot \frac{d}{11}$$

or $$RS \le \frac{0.391}{11 \times 0.75}\,d$$

or $$RS \le \frac{d}{21}$$

or $$R \le \frac{d}{21S}.$$ **Ans.**

Q. 89. If the actual velocity in the channel exceeds the optimum velocity (V_0), then scouring may occur ; and if the actual velocity is lesser than the optimum velocity (V_0), then silting may occur. This optimum velocity, thus, represents both maximum as well as minimum velocity.

Q. 104 & Q. 108. When a canal has a *fixed configuration* (*i.e.* its bed slope and cross-section do not change with time) due to its non-erodible surface, such as in a lined canal, then the channel is called in **permanent regime.** Besides no scouring, usually, there is no silting in such channels, because higher velocities are permitted, due to their inerodible surface. Hence, the silt amount leaving the channel is almost the same as the silt amount entering the channel. Such channels, whose boundaries are fixed and rigid, are called **rigid boundary canals,** and Lacey's theory is not applicable to them. *The resistance offered to the flowing water in such a channel, is a function of the nature of the boundary surface alone.* On the other hand, when the canals are constructed in erodible strata, their boundaries tend to change continuously with the flow of water in them. Such channels are evidently called **mobile boundary canals.** The material forming the boundaries of such channels are mostly loose soils, easily susceptible to erosion by flowing water. In such canals, higher velocities are usually not permissible, because this would lead to excessive scouring. Lower values of velocities will not be able to keep the entire silt load into suspension, which may lead to silting in them. Due to such scouring and silting of sediment, the sediment load transport at different sections along the channel, will usually be different. Silting and scouring may also result in modification of channel

cross-section. *The resistance offered to the flow of water in such channels depends on two factors. viz :*

(*i*) *nature of the surface of boundaries, and*

(*ii*) *condition of the bed and banks of the channel.*

Such channels need be designed for no-nsilting and non-scouring velocity. Rivers and unlined alluvial canals are examples of mobile boundary channels.

Q. 109. Scour depth $= 1.35\left(\frac{q^2}{f}\right)^{1/3} = 1.35\left(\frac{3^2}{1.2}\right)^{1/3} = 2.64$ m. **Ans.**

Q. 111. From eqn. (4.14) :

$$f = 1.75\sqrt{d_{mm}} = 1.75\sqrt{0.4} = 1.107$$

From eqn. (4.19) :

$$\text{Regime bed slope} = \frac{f^{5/3}}{3340\,Q^{1/6}} = \frac{(1.107)^{5/3}}{3340(40)^{1/6}} = \frac{1}{5200}.$$ **Ans.**

Q. 112.

$$1.0 = 1.75\sqrt{d_{mm}}$$

$$d_{mm} = \left(\frac{1}{1.75}\right)^2 = 0.326 \text{ mm} \approx 0.32 \text{ mm}.$$ **Ans.**

Q. 114.

$$P = b + \sqrt{5}\cdot y;$$

with $\frac{1}{2}$ H : 1 V slopes, which are generally attained in a regime canal, irrespective of Q and f

$$= 22 + \sqrt{5} \times 2.5 = 27.59 \text{ m}$$

$$P = 4.75\sqrt{Q} = 27.59$$

$$\therefore \quad Q = \left(\frac{27.59}{4.75}\right)^2 = 33.73 \text{ m}^3/\text{s} \approx 34 \text{ cumecs}.$$ **Ans.**

Q. 115. Given $\left[\begin{array}{l} Q_A = Q_B \\ f_A > f_B \end{array}\right.$

Now since, $V = \left(\frac{Qf^2}{140}\right)^{1/6}$

and $P = 4.75\sqrt{Q}$

$$V_A > V_B$$

$$P_A = P_B$$

Since Q is same, $A_A < A_B$

and since $R = \frac{A}{P}$

$$R_A < R_B \quad \text{or} \quad R_B > R_A$$

i.e. channel B has more hydraulic radius for the same wetted perimeter, and this happens only when the channel has more depth (deeper channels are more hydraulically efficient).

$$\therefore \quad y_B > y_A.$$

Hence, choice (*b*) is the correct choice. **Ans.**

Q. 122. By the geometry of Fig. 29.1, it can be seen that the points, B, B', etc. will lie on the bank slope line, only when the berm equals $(n_2 - n_1)\, d_1$,

where d_1 is the depth of cutting, which will vary according to ground level.

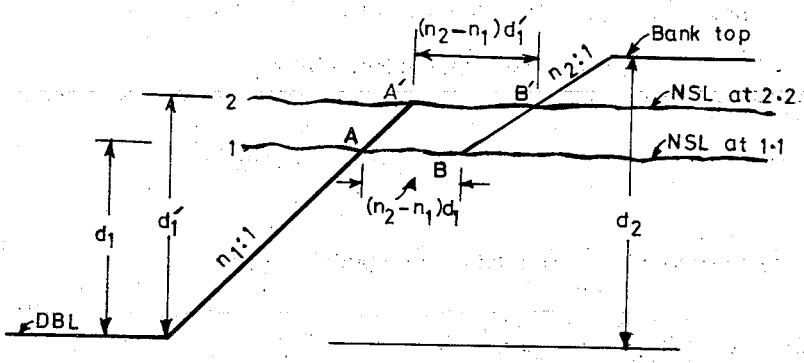


Fig. 29.1

Q. 123. It can be seen from Fig. 29.2 that when channel attains regime, $\frac{1}{2}$ H : 1 V side

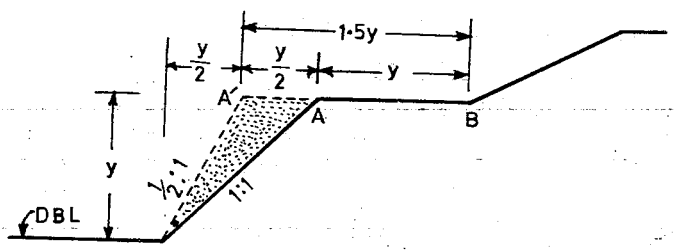


Fig. 29..2

slopes will be formed, thereby increasing the initial berm width AB to $A'B$, where

$$A'B = A'A + AB = 0.5\,y + y = 1.5\,y. \quad \textbf{Ans.}$$

Q. 126. *Assume 1 km length*

$$\text{Annual benefit} = 1000\text{ m} \times 20\text{ m} \times \frac{3}{10^6}\text{ cumecs} \times \text{Rs. 4 lakh/yr cumecs}$$

$$= \text{Rs. 24000 per yr}$$

$$\text{Annual cost} = \frac{100\text{ m} \times 20\text{ m}}{40\text{ yr}} \times \text{Rs. 40/m}^2 = \text{Rs. 20,000 per yr.}$$

$$\text{B.C. ratio} = \frac{24000}{20000} = 1.2. \quad \textbf{Ans.}$$

Q. 129. W.r.t. Fig. 29.3 ; $\tan\theta = \frac{1}{1.5}$

$$\theta = 0.588$$

$$A = y^2(\theta + \cot\theta)$$

$$= 2.5^2(0.588 + 1.5) = 13.05 \text{ m}^2$$

$$V = \frac{26}{13.00} \cong 2 \text{ m/s.} \quad \textbf{Ans.}$$

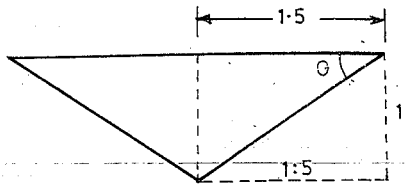


Fig. 29.3

Q. 147. The classification of saline and alkaline soils is as follows :

Sl. No.	*Classification*	*Electrical Conductivity (EC) in micro-mho/cm*	*Exchangeable Sodium Percentage (ESP)*	*pH value*
1.	**Saline soil** or *white alkali*	>4000	< 15	≤ 8.5
2.	**Alkaline soil** or *Non-saline alkali* or *sodic soil* or *black alkali*	< 4000	> 15	8.5 to 10.0
3.	**Saline-alkali soil**	> 4000	> 15	< 8.5

Q. 166. $M_B \approx 150 \cdot \sqrt{Q_a} = 150\sqrt{1600} = 6000$ m *i.e.* 6 km. **Ans.**

Q. 169. $M_B = 3 \text{ km} = 3000 \text{ m}$

Also, $M_B \approx 150 \cdot \sqrt{Q_{dominant}}$

$\therefore$ $3000 = 150\sqrt{Q_{dominant}}$

or $Q_{dominant} = 400$ cumecs

$\therefore$ $Q_{max} = \frac{16}{9} \times 400 = 711 \text{ cumecs} \approx 700 \text{ cumecs.}$ **Ans.**

Q. 176. Average thickness of apron

$$= T = 1.9t = 1.9 \times 0.06\,(5000)^{1/3}$$

$$= 1.95 \text{ m ; say } 2 \text{ m.} \quad \textbf{Ans.}$$

Average width of apron

$$= 1.5\,D$$

where $D = 1.5\,R - y$ (average), where

$$R = 0.47\left(\frac{Q}{f}\right)^{1/3}$$

$$= 0.47\,(5000)^{1/3} \text{ (assume } f = 1) = 8.04$$

$$= 1.5 \times (1.5 \times 8.04 - 5)$$

$$= 10.6 \text{ m} \approx 10 \text{ m (say).} \quad \textbf{Ans.}$$

Q. 178. Waterway required

$$= 1.2 \times 4.75 \cdot \sqrt{Q_{max}}$$

where 1.2 is safety factor of Lacey's regime width

$$= 1.2 \times 4.75 \times \sqrt{1600 \times \frac{16}{9}}$$

$$\left(\text{since } Q_{max} = \frac{16}{9} \cdot Q_{dom}\right)$$

$$= 304 \text{ m} \approx 300 \text{ m}. \quad \textbf{Ans.}$$

Q. 216. Only nominal thickness is required under upstream floor.

Q. 219. ∵ Fine sand will get lifted most easily.

Q. 297. Refer and study article 14.5.6.

Q. 298. Refer and study article 14.5.6.

Q. 304.

$$R = 1.35\left(\frac{q^2}{f}\right)^{1/3} = \left(\frac{2500}{300}\right)^{2/3} \qquad \text{(assume } f = 1)$$

$$= 5.55 \text{ m}$$

Foundation may be taken down below *HFL* by 1.33*R*, or $R + 1.2$ m, whichever is more.

(*i*) $1.33\,R = 7.38$ m

(*ii*) $R + 1.2 \text{ m} = 5.55 + 1.2 = 6.75 \text{ m} < 7.38 \text{ m}$

Hence, foundation level

$$= 214 - 7.38 = 206.62 \text{ m}$$

say 206.6 m. **Ans.**

Q. 323 to Q. 325. **Aquifuge** is that geological formation, which is neither porous nor permeable ; and hence it neither contains nor yields ground water. *Granite rock is an example of aquifuge.*

Aquitard is that geological formation, which does not yield water freely to wells due to its lesser permeability, although seepage is possible through it. The yield from such a formation is, thus, insignificant. *Sandy clay is an example of aquitard.*

Aquiclude is highly porous, containing large quantities of water, but essentially impervious, as not to yield water. *A clay layer is an example of aquiclude.*

Q. 336. The permeability coefficient, *K*, is given by the eqn.

$$K = C \cdot d^2 \frac{\gamma_w}{\mu} = \frac{C \cdot d^2 \rho_w g}{\rho \cdot \nu}$$

or

$$K = \frac{C d^2 \cdot g}{\nu} \qquad \text{...(29.1)}$$

where C = constant called **shape factor**, depending on porosity, packing and grain size distribution of the porous medium

d = mean particle size of porous medium

γ_w = unit weight of water flowing through soil

μ = dynamic viscosity of water.

Thus $\quad K \propto \frac{1}{\nu}$

ν decreases with increase in temperature (Table 1 at the end of the book), and hence *K* will increase with increase in temperature. **Ans.**

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Q. 337. From Eq. (30.1)

$$K = C \cdot d^2 \frac{g}{\nu}$$

where ν represents the property of fluid passing through the porous medium

C and d represents the properties of porous medium only.

The component of permeability which represents only the properties of the porous media, is called **intrinsic or specific permeability,** and represented by K_0,

$$\text{where } K_0 = C \cdot d^2$$

Its units are m^2 or darcy,

$$\text{where 1 darcy} = 9.87 \times 10^{-13}\ \text{m}^2.$$

The coefficient K_0 is generally used, when water/fluid temperature varies considerably or where more than one fluid is involved. However, in ground water flow, the temperature variation is small, and only water is involved as the flowing fluid, only K is considered.

Q. 340. A is defined as volume of water (m^3) released from an aquifer of 1 m^2 area and full depth d, per m fall of piezometric head. Its units will, thus, be $m^3/m^2/m$; *i.e.* dimensionless.

Q. 344. Specific yield and storage coefficient are related to a given aquifer and not to a given well.

Q. 348. Since Specific yield + Specific retention = Porosity, the specific yield, evidently, is less than porosity.

Q. 370. Ramganga dam, although 128 m high, is earthen and rockfill dam. Nagarjuna Sagar dam is 125 m high, Hirakud dam is 59 m high, and Mettur dam is 70 m high ; they are all stone masonry gravity dams.

Q. 396. Dead storage = 0.2 × 30 = 6 M. cum

Time during which 6 M. cum is filled by sediment

$$@0.1\ \text{M. cum/yr} = \frac{6}{0.1} = 60 \text{ years.} \quad \textbf{Ans.}$$

Q. 405. For $F > 32$ km, the wave ht is given by eqn. (19.12) as

$$h_w = 0.032 \cdot \sqrt{V \cdot F}\ \text{m}$$
$$= 0.032\sqrt{160 \times 40} = 2.56\ \text{m}$$

Force caused by waves P_w is given by eqn. (19.14) as

$$P_w = 19.62\, h_w^2\ \text{kN/m run of dam}$$
$$= 19.62 \times (2.56)^2\ \text{kN} = 128.6\ \text{kN}$$
$$\approx 130\ \text{kN. Hence, choice } (d) \text{ is correct.} \quad \textbf{Ans.}$$

Q. 406. The formula at choice (*a*) is correct. This may be remembered, as some numerical may be asked on this.

Q. 407. From eqn. (19.11),

wave ht $(h_w) = 0.032\sqrt{V \cdot F} + 0.763 - 0.271\,(F)^{1/4}$ for $F < 32$ km

$$\therefore \quad h_w = 0.032 \cdot \sqrt{100 \times 20} + 0.763 - 0.271\,(20)^{1/4}$$
$$= 1.431 + 0.763 - 0.573 = 1.62 \text{ m}$$

Free-board is generally provided equal to

$$1.5\,h_w = 1.5 \times 1.62 = 2.45 \text{ m, say } 2.5 \text{ m.} \quad \textbf{Ans.}$$

Q. 414. From eqn. (19.22) :

$$B = \frac{H}{\sqrt{S_s - c}} = \frac{H}{\sqrt{2.4 - 1}} = \frac{60}{\sqrt{1.4}}$$
$$= 50.7 \text{ m (with full uplift} = 1)$$

From eqn. (19.23) :

$$B = \frac{H}{\mu\,(S_s - c)} = \frac{60}{0.7\,(1.4)} = 61.22 \text{ m; say } 64 \text{ m.}$$

Chosen B should be greater of the two values, *i.e.* 61 m. **Ans.**

Q. 431. Cut off can be provided only during construction. But if any seepage appears afterwards, a drain with inverted filter along toe line is provided to collect soil-free water, to check piping.

Q. 447. Since an ogee spillway is designed for maximum head, up to *RL* of maximum reservoir level, there are almost no chances of operating head exceeding the design head. However, under rare emergency, when extraordinary flood is being passed near the end of monsoon season, it may become necessary to allow the reservoir level to even go above the Maximum Reservoir level, by encroaching upon the freeboard, in which case, the operating head may exceed the design head.

Q. 449. The efficiency of spillway indirectly means its coefficient of discharge, which increases with the increasing head above full reservoir level (*FRL*) till it becomes about 2.2 at full design head (*i.e.* at MRL). Since the spillway starts working when water level just crosses *FRL*, the coefficient of discharge in early stages, is only about 1.7, *i.e.* about 77% of its value at full design head. Hence, the spillway efficiency can be said to be increasing from 77% to 100%, as water level in the reservoir increases from F.R.L. to M.R.L.

Q. 461. A saddle syphon is nothing but a syphon spillway, and includes both of its types. Air vent is the essential requirement of both types of syphon spillways.

Q. 475. $Q = C_d \cdot A\,\sqrt{2gH_1}$

where C_d = Coefficient of discharge through orifice

H_1 = Water head over the centre line of opening

$= RL331 - RL328 = 3$ m

A = Area of opening

$= 60 \text{ m} \times 6 \text{ m} = 360 \text{ m}^2$

$$\therefore \quad Q = 0.62 \times 360\,\sqrt{2 \times 9.81 \times 3}$$
$$= 1712 \text{ cumecs} \approx 1700 \text{ cumecs.} \quad \textbf{Ans.}$$

Q. 476. $Q = C_d A \cdot \sqrt{2gH}$

H = Water head over centre line of sluice (since it is a free overfall on downstream side)

$= RL330\,(FRL) - RL300$ m (sluice centre line level)

$= 30$ m

$$A = \frac{\pi}{4} \times 2^2 = 3.14 \text{ m}^2$$

C_d = Coefficient of discharge ≈ 0.62 for free overfall

$$\therefore \quad Q = 0.62 \times 3.14 \cdot \sqrt{2 \times 9.81 \times 30}$$

$$= 47.23 \text{ cumecs}$$

say 50 cumecs (nearest appropriate choice given). **Ans.**

Q. 477. $$Q = C_d \cdot A \cdot \sqrt{2gH_L}$$

$$= 0.8 \times \left(\frac{\pi}{4} \times 4^2\right) \sqrt{2 \times 9.81 \times (226 - 210)}$$

$$= 0.8 \times 4\pi \sqrt{16.62 \times 16} = 164 \text{ cumecs}$$

≈ 170 cumecs (nearest appropriate choice). **Ans.**

To countercheck, let us see the absolute maximum discharge, which computes to be 204.8 cumecs at $C_d = 1$. In any case, even if all losses are neglected, Q cannot exceed 204.8 cumecs, and hence the last choice is certainly incorrect, other choices at (*a*) and (*b*) are giving very low-coefficient of discharge, such as $\frac{64}{204.8} = 0.3$ and $\frac{104}{204.8} = 0.5$, which are too low for submerged bell mouthed pipe flows.

Q. 502. From eqn. [17.6 (*a*)] :

Capacity factor (C.F.)

$$= \frac{\text{Average load}}{\text{Plant capacity}}$$

$$\text{where, average load} = \frac{\text{Energy produced}}{\text{Time in h}}$$

$$= \frac{58,80,000}{7 \times 24} \text{ kW}$$

$$= 35000 \text{ kW} = 35 \text{ MW}$$

$$\text{C.F.} = \frac{35 \text{ MW}}{100 \text{ MW}} = 0.35. \quad \textbf{Ans.}$$

Q. 503. Load factor $= \frac{\text{Average load}}{\text{Peak load}} = \frac{35 \text{ MW}}{70 \text{ MW}} = 0.5.$ **Ans.**

Q. 504. Utilisation Factor (*UF*)

$$= \frac{\text{Maximum power utilised}}{\text{Maximum power available}} = \frac{70 \text{ MW}}{100 \text{ MW}} = 0.7. \quad \textbf{Ans.}$$

Q. 511. Francis turbines are cheaper than Pelton's turbines, and there is no special need for Pelton's turbines here, because part load (variable loads) are not likely to come and water is not sandy, which may certainly need Pelton's turbines. Hence, Francis turbines will suffice to our needs. **Ans.**

Q. 533. $Q = C \cdot LH^{3/2} = 1.66 \times 50 \times (0.8)^{3/2}$

($\because$ When width of rectangular crest > 0.9 m, $C = 1.66$)

$= 59.4$ cumecs. **Ans.**

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Test Paper on Objective Questions

Q. 1. In a hot arid district of Rajasthan State, like Jodhpur, the preferred mode of irrigation, would be :

(*a*) flood irrigation (*b*) furrow irrigation
(*c*) drip irrigation (*d*) sprinkler irrigation,

Q. 2. The conjunctive use of water in a basin is defined as :

(*a*) the Evapotranspiration needs of the basin
(*b*) combined exploitation of surface and ground waters in the basin
(*c*) combined utilisation of surface waters for kharif as well as Rabi crops in the basin
(*d*) none of the above.

Q. 3. The ratio of the average value of the shear stress produced on the bed, and that on the banks of an alluvial canal, due to the action of the flowing water, is :

(*a*) 0.75 (*b*) 1.0
(*c*) 1.33 (*d*) 0.

Q. 4. Lacey's silt theory is not applicable when :

(*a*) silt grade consists of pure sand
(*b*) silt amount is of the order of 500 ppm
(*c*) the canal is lined
(*d*) none of the above.

Q. 5.* An alluvial irrigation canal has been excavated with 1 : 1 side slopes in cutting and $1\frac{1}{2}$: 1 (H : V) in filling. At a given cross-section of this canal, the cutting depth is found to be D, and water depth up to FSL as 1.2 D. The berm width left originally is D/2, which, when channel attains regime, will become :

(*a*) $\frac{D}{2}$ (*b*) D
(*c*) $1.2\,D$ (*d*) $1.3\,D$.

Q. 6.* The maximum area that can be irrigated by a stream discharge of 0.05 cumecs, if the average rate of infiltration is 5 cm/h, is :

(*a*) 0.36 hectares (*b*) 100 hectares
(*c*) 0.84 hectares (*d*) 10.26 hectares. (*Calcutta University*)

Q. 7. The ratio of the quantity of water stored in the root zone of the crops to the quantity of water actually delivered into the field, is called :

(*a*) efficiency of water distribution
(*b*) efficiency of water storage
(*c*) efficiency of water use
(*d*) efficiency of water application.

Q. 8. The hydraulic radius of a standard trapezoidal lined canal with 1.25 H : IV side slopes, 12 m bed width and 2 m FSD, is :

(*a*) 1.11 m (*b*) 1.41 m
(*c*) 1.61 m (*d*) none of them.

Q. 9. Safety ladders are provided to enable :

(*a*) the fish to move in water from downstream to upstream to their ancesstoral spawning grounds
(*b*) the fish to move in water from upstream to downstream in search of food
(*c*) both (*a*) and (*b*) above
(*d*) none of the above.

Q. 10. Which one of the following statements is not true about cement concrete lining ?

(*a*) that it develops frequent cracks due to temperature changes
(*b*) that it develops frequent cracks due to settlement of subgrade
(*c*) that it is likely to be damaged by alkaline water
(*d*) that it can be easily punctured by weed growth.

Q. 11.* River yamuna in the city reach of Delhi, has to carry a maximum discharge of about 5000 cumecs. The river has been jacketed on both sides by dikes at 2 km apart. These dikes come under frequent attacks of the meandering river, and need elaborate protective anti-erosion works, because the dikes are closely spaced, curtailing the original meander belt by about :

(*a*) 10% (*b*) 25%
(*c*) 50% (*d*) 75%.

Q. 12.* The meander width for an alluvial stream is estimated to be about 3.4 km. The normal width of this stream shall roughly be of the order of :

(*a*) 100 m (*b*) 200 m
(*c*) 30 m (*d*) 600 m.

Q. 13. An Indian irrigation project designed to serve a command of more than 2000 hectares and up to 10000 hectares, is known as a :

(*a*) major irrigation project
(*b*) minor irrigation project
(*c*) medium irrigation project
(*d*) none of them, since irrigation projects are classified on the basis of their costs.

Q. 14. The property of a zeological formation, which represents its capability to allow transmission of water through it, is called :

(*a*) porosity (*b*) permeability
(*c*) field capacity (*d*) yield.

Q. 15. The water consumed in irrigation, when compared with the total water used for all purposes in our country, is about :

(*a*) 30% (*b*) 50%
(*c*) 70% (*d*) 90%.

Q. 16. 1 Tonne of wheat produced, may consume water, as much as, say :

(*a*) 100 tonnes (*b*) 500 tonnes
(*c*) 2000 tonnes (*d*) 4000 tonnes.

Q. 17. The velocity of flow of water (v) through a soil stratum of permeability K and porosity n, as per Darcy's equation, is given by :

(*a*) $v = Ki$ (*b*) $v = Ki\,A$

(*c*) $v = K\,i \cdot n$ (*d*) $v = Ki \cdot \frac{1}{n}$.

Q. 18. Water required in producing 1 tonne of Rice is of the order of :

(*a*) 100 tonnes (*b*) 500 tonnes

(*c*) 1000 tonnes (*d*) 4000 tonnes.

Q. 19. The zeological formation, which does not contain any amount of ground water, is an :

(*a*) aquitard (*b*) aquifer

(*c*) aquiclude (*d*) aquifuse.

Q. 20. An example of aquitard stratum in the ground profile, is a layer of :

(*a*) granite rock (*b*) clay

(*c*) sandy clay (*d*) sand stone.

Q. 21. The appropriate percentage that represents the world's water resources existing in the form of ground water, is :

(*a*) 2% (*b*) 12%

(*c*) 22% (*d*) 47%.

Q. 22. In a meandering river reach, the shallowest portions of the river can be searched out, at :

(*a*) the crossings (*b*) the convex banks

(*c*) the concave banks (*d*) none of them.

Q. 23. The 'sinuosity' of a meandering river is defined as :

(*a*) $\frac{\text{meander length}}{\text{meander width}}$ (*b*) $\frac{\text{meander width}}{\text{meander length}}$

(*c*) $\frac{\text{length of the river}}{\text{axial length of the river}}$ (*b*) $\frac{\text{axial length of the river}}{\text{length of the river}}$.

Q. 24. 'Crossings' in a meandering river, are :

(*a*) straight channel reaches, between two consecutive clockwise and anticlockwise loops

(*b*) apex points of clockwise loops

(*c*) apex points of anticlockwise loops

(*d*) both (*b*) and (*c*).

Q. 25. A river reach downstream of a newly built dam, may behave as :

(*a*) aggrading (*b*) degrading

(*c*) virgin (*d*) all of them.

Q. 26. An unconfined aquifer in ground water profile is also known, as :

(*a*) an artesian aquifer (*b*) a perched aquifer

(*c*) a water table aquifer (*d*) none of the above.

Q. 27.* The upper limit of Reynold's number, for the Darcy's law to remain valid for ground water flow, is :

(*a*) 0 (*b*) 1

(*c*) 100 (*d*) ∞.

Q. 28. The safe depression head for open wells is generally taken to be x times the critical depression head, when x is :

(*a*) $\frac{1}{6}$ (*b*) $\frac{1}{3}$

(*c*) $\frac{1}{2}$ (*d*) $\frac{3}{4}$.

Q. 29. When groundwater flows into a stream to augment its discharge, then the stream is called :

(*a*) an influent stream (*b*) an effluent stream
(*c*) a perennial stream (*d*) an ephemeral stream.

Q. 30. A bridge is proposed to be constructed across the Ganga river at a site, where the design discharge is about 20,650 cumecs. The river width is proposed to be reduced at the bridge site to 800 m by constructing guide banks and marginal bunds, on either side. The length of each guide bank in this case will be about :

(*a*) 0.8 km (*b*) 1.2 km
(*c*) 1.4 km (*d*) 2.8 km.

Q. 31.* A guide bank is to be constructed for an alluvial river, having maximum design discharge of 6400 cumecs. The thickness of the stone pitching required on the river side of this guide bank, will be about :

(*a*) 0.6 m (*b*) 0.9 m
(*c*) 1.2 m (*d*) 1.5 m.

Q. 32. A fully developed pilot cut, excavated in a river, is called :

(*a*) a cut off (*b*) a short cut
(*c*) a meander (*d*) a river channel.

Q. 33. Retrogression occurring at the downstream reach of a weir site, is :

(*a*) a continuous scouring process
(*b*) a continuous silting process
(*c*) a reversible process, in which gradual scouring occurs during the initial 4-5 years of weir construction ; which later, changes into gradual silting, and recovery towards original levels, and even more, sometimes.
(*d*) a reversible process, in which gradual silting occurs during the initial 4-5 years of weir construction ; which then changes into gradual scouring, and recovery towards original levels and even more, sometimes.

Q. 34. A 'breast wall' is usually provided above the sill of a canal head regulator at a barrage site, between :

(*a*) sill level and pond level
(*b*) pond level and HFL of river
(*c*) pond level and FSL of canal
(*d*) none of the above.

Q. 35. The alkali salt, which is most injurious to plant growth, is :

(*a*) sodium corbonate (*b*) sodium chloride
(*c*) sodium sulphate (*d*) sodium nitrate.

Q. 36. The joints in the C.C. blocks laid over the inverted filter infront of the downstream end of concrete floor of a barrage, should be filled up with :

(*a*) C.C. 1 : 2 : 4 (*b*) cement mortar 1 : 4
(*c*) bajri or coarse sand (*d*) any of the above.

Q. 37. A deflecting groyne is :

(*a*) a small repelling groyne, giving only local protection, without repelling the river current
(*b*) a small attracting groyne
(*c*) a specially designed curved head groyne
(*d*) a tree groyne.

Q. 38. The spacing between the groynes constructed to protect a dike, in the absence of model testing, may be of the order of :
(*a*) 2 to 2.5 times their length
(*b*) 8 to 10 times their length
(*c*) 20 to 30 times their length
(*d*) 50 times their length.

Q. 39. Rigid boundary canals, whose bed and banks are made with non-erodible materials, are in :
(*a*) initial regime (*b*) final regime
(*c*) true regime (*d*) permanent regime.

Q. 40. Lacey's silt theory is not applicable to channels, which are in :
(*a*) final regime (*b*) permanent regime
(*c*) true regime (*d*) none of the above.

Q. 41. A perfectly lined canal is a :
(*a*) mobile boundary canal (*b*) rigid boundary canal
(*c*) neither (*a*) nor (*b*) (*d*) either (*a*) or (*b*).

Q. 42.* The total water requirement for certain irrigated area is 300 M cum. The reservoir losses and carry over losses are 10% and 5% respectively ; and dead storage is 10% of gross storage. The gross storage capacity of reservoir should be :
(*a*) 375 M cum (*b*) 383 M cum
(*c*) 395 M cum (*d*) 400 M cum.

Q. 43. The stone apron laid infront of a hydraulic structure in the river bed, to ensure its launching into any possible scour that may occur at the site, is called :
(*a*) launching apron (*b*) rip rap
(*c*) talus (*d*) all of these.

Q. 44. A pilot cut in a meandering river should be as deep as possible, and the ratio $\frac{R}{L^2}$ for the cut should be :
(*a*) lesser for the cut than for the main course
(*b*) greater for the cut than for the main course
(*c*) equal for the cut and the main course
(*d*) none of the above.

where R is the hydraulic radius
L is the channel length.

Q. 45. Cut off ratio for an alluvial river is always :
(*a*) < 1 (*b*) 1
(*c*) > 1 (*d*) none of them.

Q. 46. When other conditions are favourable, a natural cut off along an existing side channel may develop across a sharp meandering bend in an alluvial river, at a meander ratio of the order of :
(*a*) 0.5 to 1.0 (*b*) 1.0 to 1.5
(*c*) 1.7 to 3.0 (*d*) 10 to 20.

Q. 47. The arrangements made along a canal, to help the unfortunate victims being swept away by water current, are called :
(*a*) canal crossings (*b*) cattle crossings
(*c*) canal ladders (*d*) none of them.

Q. 48. The only irrigation module, which is not an orifice type but is of weir type, is :

(*a*) Kennedy's gauge outlet (*b*) open flume outlet
(*c*) pipe outlet (*d*) adjustable proportional module.

Q. 49. The one among the following, which is not a rigid module, is :

(*a*) Gibb's module (*b*) Foote module
(*c*) Khanna's module (*d*) Kennedy's module.

Q. 50. Flood waters of rivers are sometimes diverted through canals, called :

(*a*) feeder canals (*b*) inundation canals
(*c*) perennial canals (*d*) carrier canals.

Q. 51. A cut off across a meander loop in an alluvial river leads to :

(*a*) silting of sediment upstream and scouring downstream
(*b*) scouring upstream and silting downstream
(*c*) scouring in the upstream reach only
(*d*) silting in the downstream reach only.

Q. 52. The sill level of a canal head regulator is kept higher by about 1.2 to 1.5 m, above the :

(*a*) crest level of under-sluice ways
(*b*) crest level of barrage bays
(*c*) canal bed level
(*d*) none of these.

Q. 53. The tunnel openings provided parallel to the axis of the main canal head regulator and in front of that, are called :

(*a*) head sluices (*b*) under sluices
(*c*) silt ejector (*d*) silt excluder.

Q. 54. Out of the following, the measures which help in improving the duty of irrigation water, are :

(*a*) lining of canals to reduce conveyance losses of irrigation water
(*b*) minimising water losses in irrigation fields, by improving the efficiency of cultivation methods
(*c*) lining of water courses to avoid seepage of water
(*d*) educating the farmers with the harmful effects of using excess water in irrigation
(*e*) rotation of crops in farms
(*f*) all of the above (*g*) none of the above.

Q. 55. An irrigation canal, flowing under pressure below a drainage channel, is specifically called a :

(*a*) canal aqueduct (*b*) siphon aqueduct
(*c*) super passage (*d*) canal siphon.

Q. 56. A venturiflume, which is helpful in measuring canal discharge, is also called a :

(*a*) standing wave flume (*b*) parshall flume
(*c*) non-metering flume (*d*) none of the above.

Q. 57. The drainage water intercepting the canal can be displosed of by passing the canal above the drainage, in :

(*a*) an aqueduct or a siphon aqueduct
(*b*) a super passage or a canal siphon

(*c*) a level crossing
(*d*) none of the above.

Q. 58. A level crossing constructed at the junction of an irrigation canal and a natural drainage channel consists of :
(*a*) one regulator only
(*b*) two weirs only
(*c*) one weir and two regulators
(*d*) two weirs and two regulators.

Q. 59. Which of the following is not true about a standing wave flume, used for metering purposes in a canal ?
(*a*) it involves low head loss
(*b*) its discharge depends only upon the upstream water level
(*c*) for the same discharging head, its discharge is more than that of a venturi flume
(*d*) it involves the formation of a hydraulic jump downstream of the throat
(*e*) it is seldom constructed as independent structure solely for metering purposes
(*f*) it involves low energy loss.

Q. 60. Cut offs are promoted along a river course in plains, to shorten and straighten its course. This exercise helps us to :
(*a*) dissipitate excess valley slope of upper mountainous river reach
(*b*) remedy the deficit surface slope of the river reach in plain
(*c*) both (*a*) and (*b*)
(*b*) neither (*a*) nor (*b*).

Q. 61. Out of the following, which measure helps in reducing scouring and silting along an alluvial river course :
(*a*) to dissipate excess land slope of upper reach, by constructing weirs and barrages across the river in this reach
(*b*) to overcome the deficit slope in lower plain reaches, by promoting and affecting cut offs
(*c*) by promoting soil conservation measures in the upper catchment
(*d*) by creating storage on the upper reaches, to release somewhat constant outflow in the downstream river.
(*e*) all of the above.

Q. 62. The floor of a siphon aqueduct needs to be designed for conditions, when :
(*a*) the canal and drainage are flowing full
(*b*) the canal is full and there is no drainage discharge
(*c*) the canal is empty and drainage discharge is full
(*d*) (*a*) and (*b*) both
(*e*) (*b*) and (*c*) both
(*f*) (*c*) and (*a*) both.

Q. 63.* The discharge from a spillway on a horizontal floor is 8 cumecs per metre width. If the depth before the jump is 50 cm, the depth after the jump will be ... metres. (*GATE - 92*)

Q. 64.* 'Eddy viscosity', means that it is
(*a*) a physical property of the fluid
(*b*) same as kinematic viscosity (*c*) always associated with laminar flow
(*d*) an apparent viscosity due to turbulent nature of flow. (*GATE-90*)

Q. 65. In a siphon aqueduct provided with a depressed floor, the worst uplift acting on the floor is caused when there is no drainage flow, by the
(*a*) static head due to watertable rising up to the drain bed
(*b*) seepage head due to canal running full
(*c*) both (*a*) and (*b*)
(*b*) none of the above.

Q. 66.* The following hydraulic data is available at the proposed site of a canal crossing :

Item	*Drain*	*Canal*
B.L. (m)	207.4	209.4
HFL/FSL (m)	209.3	215.6
Discharge (m^2/s)	390	350

The appropriate economical cross-drainage work at the above site will be :
(*a*) an aqueduct (*b*) a super passage
(*c*) an aqueduct syphon (*d*) a canal syphon.

Q. 67. Multiple arch dam is an example of :
(*a*) arch dams (*b*) buttress dams
(*c*) shell-arch dams (*d*) all of the above.

Q. 68. An aquifer, which is confined at its bottom but not at the top, is called a :
(*a*) semiconfined aquifer (*b*) confined aquifer
(*c*) unconfined aquifer (*d*) none of the above.

Q. 69.* The permeability of a soil sample at standard temperature of 20°C was 0.01 cm/s. The permeability at 40°C temperature, when kinematic viscosity of water at 20°C and 40°C are given to be 1.009$\mu m^2/s$ and 0.654$\mu m^2/s$, will be:
(*a*) 0.01 cm/s (*b*) 0.015 cm/s
(*c*) 0.0065 cm/s (*d*) none of these.

Q. 70. The volume of water (cum) that can be extracted from a unit volume of aquifer material (1 cum) under the force of gravity, is called :
(*a*) storage coefficient (*b*) specific coefficient
(*c*) specific yield (*d*) specific capacity.

Q. 71.* Specific capacity of a confined well under equilibrium conditions and within the working limites of drawdown :
(*a*) is constant at all drawdowns
(*b*) decreases as the drawdown is increased
(*c*) increases as the drawdown is increased
(*d*) none of the above.

Q. 72. A soil sample, having exchangeable sodium percentage = 13%, pH = 7.5, and electrical conductivity = 6000 $\mu\Omega$/cm, can be classified as :
(*a*) white alkali (*b*) alkaline soil
(*c*) saline-alkali soil (*d*) none of the above.

Q. 73. The dead storage in a reservoir is created for :
(*a*) sedimentation (*b*) firm hydropower
(*c*) emergent water supply (*d*) none of these.

Q. 74. The silt suspended in water, settles down, approximately following the :
(*a*) Blaney criddle formula (*b*) Darcy's law
(*c*) Lacey's theory (*d*) Stoke's law.

Q. 75. Sedimentation over long periods of time, is responsible for reducing the reservoir capacity, earmarked as :

(*a*) dead storage (*b*) live storage
(*c*) both (*a*) and (*b*) (*d*) none of these.

Q. 76. The most appropriate value of the 'average estimated life' of a concrete gravity or an earthen dam, is :

(*a*) 50 years (*b*) 100 years
(*c*) 150 years (*d*) none of these.

Q. 77. The axis of a gravity dam is the :

(*a*) line of the crown of the dam on the downstream side
(*b*) line of the crown of the dam on the upstream side
(*c*) centre line of the top width of the dam
(*d*) line joining mid points of the base.

Q. 78. Tension cracks in gravity dams may sometimes lead to the failure of the structure, by :

(*a*) sliding of the dam at the cracked section
(*b*) overturning about the toe
(*c*) crushing of concrete, starting from the toe
(*d*) none of the above.

Q. 79. Presence of tail water in a gravity dam :

(*a*) increases the principal stress and decreases the shear stress
(*b*) increases the principal stress and increases the shear stress
(*c*) decreases the principal stress and increases the shear stress
(*d*) decreases the principal stress and decreases the shear stress.

Q. 80. Neglecting tail water effect, the major principal stress at the toe of a gravity dam under full reservoir condition, is given by :

(*a*) p_v (*b*) $p_v \sec^2 \theta$
(*c*) $p_v \tan^2 \theta$ (*d*) $p_v \sin^2 \theta$.

where p_v = direct vertical stress at the toe
θ = angle of the downstream face with the vertical.

Q. 81. The provision of a drainage gallery in a gravity dam helps in reducing :

(*a*) hydrostatic pressure (*b*) silt pressure
(*c*) seepage pressure (*d*) none of these.

Q. 82. The uplift pressure at the face of a drainage gallery in a gravity dam, is taken as equal to :

(*a*) hydrostatic pressure at the toe
(*b*) average of hydrostatic pressure at the toe and heel
(*c*) two third of the hydrostatic pressure at the toe plus one third of the hydrostatic pressure at the heel
(*d*) none of the above.

Q. 83. The pore pressure distribution under steady condition in an earthen dam, is:

(*a*) determined by considering linear variation from upstream to downstream water levels
(*b*) determined from the flownet
(*c*) the same as the upstream water level
(*d*) none of the above.

Q. 84. A rock toe and a horizontal filter is provided on the downstream base of an earthen dam in order to :
(*a*) prevent piping action in the dam body
(*b*) prevent piping action in the dam foundation
(*c*) to reduce the seepage quantity by blocking its flow
(*d*) to collect and drain out the seepage flow.

Q. 85. The one among the following, which helps to meet the downstream river water requirements in a dam reservoir project, is :
(*a*) dam spillway (*b*) dam galleries
(*c*) dam sluices (*d*) none of these.

Q. 86. The crest of a dam spillway is provided at R.L. equal to that of its :
(*a*) maximum reservoir level
(*b*) dead storage level
(*c*) maximum conservation level
(*d*) none of the above.

Q. 87. The seepage discharge in an earthen dam, as collected by a horizontal blanket or a rock toe, can be computed by :
(*a*) plotting the flow net below the dam body
(*b*) plotting the flow net through the dam body
(*c*) either (*a*) or (*b*)
(*d*) none of the above.

Q. 88. The hydraulic jump for energy dissipation is used in :
(*a*) shaft spillway
(*b*) syphon spillway
(*c*) side channel spillway
(*d*) none of them.

Q. 89. The formation of a hydraulic jump at the foot of a spillway is one of the common methods of energy dissipation services, because :
(*a*) it destroys more than 90% of the total energy by the turbulence produced in the jump
(*b*) it reduces kinetic energy by increasing the depth of flow
(*c*) its action is not understood. (*U.P.S.C. Engg. Services, 1977*)

Q. 90. The process by virtue of which water moves from regions of higher concentration to lower concentration, the two regions being separated by a selectively permeable membrane, is called :
(*a*) diffusion (*b*) osmosis
(*c*) transpiration (*d*) reverse osmosis.

Q. 91. The productivity of a soil will be maximum, when its pH value is of the order of :
(*a*) below 6 (*b*) 6.5—7.5
(*c*) 7.5—8.0 (*d*) 8.5— 11.0.

Q. 92.* The efficiency of sprinklar irrigation is the highest in :
(*a*) arid regions (*b*) semi arid regions
(*c*) humid regions (*d*) none of them.

Q. 93. The uniformity coefficient will be higher in case of :
(*a*) drip irrigation (*b*) sprinkler irrigation
(*c*) equal in both cases
(*d*) sometimes (*a*) and sometimes (*b*), depending on soil type.

Q. 94. The structure among the following, which is not used in a shaft spillway, is:
(*a*) a tunnel (*b*) a bridge
(*c*) radial gates (*d*) radial piers.

Q. 95. 'Morning glory' is the :
(*a*) special flared inlet of the shaft spillway of a dam of very small height
(*b*) special flared inlet of the shaft spillway of a large dam project
(*c*) horizontal tunnel constructed in a shaft spillway across the body of a gravity dam, to carry the surplus reservoir water to the downstream river
(*d*) horizontal tunnel constructed in the shaft spillway of an earthen dam through its foundation, to carry the surplus reservoir water to the downstream river.

Q. 96. A ski-jump bucket is also known as :
(*a*) flip bucket (*b*) solid roller bucket
(*c*) slotted roller bucket (*d*) none of these.

Q. 97. The most commonly used vertical lift gates in modern days is :
(*a*) sliding gates (*b*) free-roller gates
(*c*) stoney gates (*d*) fixed wheel gates.

Q. 98. The percentage of energy dissipation in a hydraulic jump :
(*a*) increases with the increase in the Froude number (of the incoming flow)
(*b*) decreases with the increase in the Froude number
(*c*) remains unaffected with the increase in the Froude number
(*d*) increases with the increase in. Froude number up to a limit, and then decreases with further increase in Froude number.

Q. 99. In case of dam spillways, the approaching Froude number F_1, usually lies in the range of :
(*a*) < 2.5 (*b*) 2.5—4.5
(*c*) 4.5—0.9, or sometimes even more
(*b*) none of them.

Q. 100. In case of weirs and barrages, the approaching Froude number F_1, usually lies in the range of :
(*a*) < 2.5 (*b*) 2.5—4.5
(*c*) 4.5—9.0 (*d*) more than 9.0.

ANSWERS FOR THE TEST PAPER

1. (*c*)	2. (*b*)	3. (*c*)	4. (*c*)	5. (*c*)
6. (*a*)	7. (*d*)	8. (*c*)	9. (*d*)	10. (*d*)
11. (*d*)	12. (*b*)	13. (*c*)	14. (*b*)	15. (*d*)
16. (*c*)	17. (*d*)	18. (*d*)	19. (*d*)	20. (*c*)
21. (*c*)	22. (*b*)	23. (*c*)	24. (*a*)	25. (*b*)
26. (*c*)	27. (*b*)	28. (*b*)	29. (*b*)	30. (*c*)
31. (*c*)	32. (*a*)	33. (*c*)	34. (*b*)	35. (*a*)
36. (*c*)	37. (*a*)	38. (*a*)	39. (*d*)	40. (*b*)
41. (*b*)	42. (*c*)	43. (*d*)	44. (*b*)	45. (*c*)
46. (*c*)	47. (*c*)	48. (*b*)	49. (*d*)	50. (*b*)
51. (*b*)	52. (*a*)	53. (*d*)	54. (*f*)	55. (*d*)
56. (*b*)	57. (*a*)	58. (*c*)	59. (*a*)	60 (*b*)

61. (*e*)	62. (*e*)	63. 4.86 m	64. (*d*)	65. (*c*)
66. (*c*)	67. (*b*)	68. (*c*)	69. (*b*)	70. (*c*)
71. (*b*)	72. (*a*)	73. (*a*)	74. (*d*)	75. (*c*)
76. (*c*)	77. (*b*)	78. (*c*)	79. (*d*)	80. (*b*)
81. (*c*)	82. (*c*)	83. (*b*)	84. (*d*)	85. (*c*)
86. (*c*)	87. (*b*)	88. (*c*)	89. (*b*)	90. (*b*)
91. (*b*)	92. (*c*)	93. (*a*)	94. (*c*)	95. (*b*)
96. (*a*)	97. (*d*)	98. (*a*)	99. (*c*)	100. (*b*).

HINTS FOR SOLVING STARRED QUESTIONS OF THE TEST PAPER

Q. 5. From Fig. 30.1, it can be seen that the original berm AB attains $A'B'$, when channel attains $\frac{1}{2}H:1\,V$ regime side slopes. From geometry of this Fig., $A'B'$ is found to be $1.2D$.

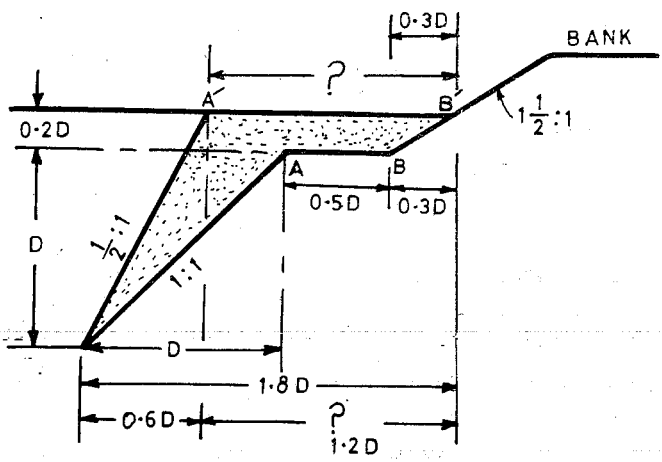

Fig. 30.1.

Q. 6. From eqn. (1.2)

$$A_{max} = \frac{Q}{f}$$

where $Q = 0.05$ cumec

$= 0.05 \times 60 \times 60\ \text{m}^3/\text{h}$

$= 180\ \text{m}^3/\text{h}$

$f = 5\ \text{cm/h} = 0.05\ \text{m/hr}$

$$= \frac{180}{0.05}\ \text{m}^2 = 3600\ \text{m}^2$$

$= 0.36$ hectare. **Ans.**

Q. 8.

$$\tan\theta = \frac{1}{1.25}$$

$$\theta = 0.675$$

From eqn. (5.8)

$$A = (B + y\theta + y \cot \theta)$$
$$= 2\,(12 + 2 \times 0.675 + 2 \times 1.25) = 31.7 \text{ m}^2$$

From eqn. (5.9)

$$P = B + 2y\theta + 2y \cot \theta$$
$$= 12 + 2 \times 2 \times 0.675 + 2 \times 2 \times 1.25 = 19.7 \text{ m}$$
$$R = \frac{A}{P} = \frac{31.2}{19.7} = \mathbf{1.61\ m.} \quad \textbf{Ans.}$$

Q. 11. $M \approx 150\sqrt{Q_{dom}}$

$$= 150 \times \sqrt{5000 \times \frac{9}{16}} \text{ m} \approx 8 \text{ km}$$

Spacing provided = 2 km

Meander belt cut off

$$= 8 - 2 = 6 \text{ km}$$
$$= \frac{6}{8} \times 100\% = 75\% \quad \textbf{Ans.}$$

Q. 12. $M_B = 3400 \text{ m} \approx 17 \text{ W}$

$\therefore$ $W = 200$ m. **Ans.**

Q. 27. $\because$ The flow becomes turbulent when R_e exceeds 1, and Darcy's law is valid only in laminar range.

Q. 31. From Eqn. (8.4) : $t = 0.06\,(6400)^{1/2}$

$$= 1.11 \approx 1.2 \text{ m.} \quad \textbf{Ans.}$$

Q. 42. Carryover (conveyance) loss is 5% (of head discharge)

$\because$ Water drawn from reservoir

$$= \frac{300}{0.95} \text{ M cum.}$$

Reservoir loss and dead storage

$$= 20\%$$

$\because$ Gross reservoir capacity

$$= \frac{\text{water drawn}}{0.8}$$
$$= \frac{300}{0.95} \times \frac{1}{0.8} = 394.7 \text{ M cum}$$
$$\approx 395 \text{ M cum} \quad \textbf{Ans.}$$

Q. 63. From eqn. (10.6) :

$$y_2 = -\frac{y_1}{2} + \sqrt{\frac{y_1^2}{4} + \frac{2q^2}{gy_1}}$$

where $q = 8$ cumecs/m

$y_1 = 0.5$ m

$$\therefore \quad y_2 = -\frac{0.5}{2} + \sqrt{\frac{0.25}{4} + \frac{2 \times 64}{9.81 \times 0.5}}$$
$$= 4.86 \text{ m.} \quad \textbf{Ans.}$$

Q. 64. See article 4.8.

Q. 66. Refer Example 14.5 (*b*).

Q. 69.

$$K \propto \frac{1}{\nu}$$

$$\therefore \quad \frac{K_{40°}}{K_{20°}} = \frac{\nu_{20°}}{\nu_{40°}} = \frac{1.009}{0.654}$$

$$\therefore \quad K_{40°} = \frac{1.009}{0.654} \times 0.01 \text{ cm/s} = 0.015 \text{ cm/s}. \quad \textbf{Ans.}$$

Q. 71. From eqn. (16.28) :

Specific capacity

$$= \frac{1}{C_1 + C_2 Q}.$$

This eqn. shows that specific capacity decreases as Q increases, and Q increases only when drawdown is increased by heavier pumping, and hence, specific capacity decreases as drawdown is increased. **Ans.**

Q. 92. Efficiency of sprinkler irrigation is the highest in humid or cool climates.

31

Important Information to Students on UPSC Competitions and Conventional Questions of the Past Years

The Union Public Service Commission, Dholpur House, Shahjahan Road, New Delhi-110011, conducts every year two competitions, in which, you as civil engineering graduates (even with exam. results awaited) can participate for securing a direct class I Govt. appointment.

The first competition is named as **Combined Engineering Services (CES) exam,** and the other competition is known as **Civil Services Exam.**, popularly called *IAS competition.*

Press notifications for obtaining and submitting application forms are released by Secretary UPSC, through news papers including "Employment News" sometimes in Feb. every year. The exams are usually held in August.

On the basis of the first competition, you are likely to be appointed as Asstt. Executive Engineer or Asstt. Director or on an equivalent post in civil engineering side of the various Government Departments like Railways, CPWD, P&T, Central Water Commission, etc. whereas, on the basis of the second competition, you are liable to become an IFS, IAS, IPS, or an officer on allied services, such as Asstt. Custom Collector or Asstt. Excise Collector, etc.

Most of the engineering degree students aspire to clear any one of these two competitions, for an excellent career full of promotional avenues, just after doing BE degree. The syllabus for both these competitions, now a days, provide full opportunities to Civil Engineering degree holders, to make through these competitions.

We will here discuss the syllabus and examination schemes as may be of use to you as civil engineering graduates, for both of these competitions.

COMBINED ENGINEERING SERVICES EXAM.
(Civil Engg.)

In this competition, there will be 5 papers in all, 3 being of objective type, and 2 of conventional type. Each paper carries 200 marks, with a total of 1000 marks. 200 marks are reserved for Personal Interview, making a grand total of 1200 marks, as below:

Section I — Objective Type Papers

1. *General Ability Test*	2 hr.	200 marks
2. *Civil Engineering Paper I*	2 hr.	200 marks
3. *Civil Engineering Paper II*	2 hr.	200 marks
Section II — Conventional Papers*		
4. *Civil Engineering Paper I*	3 hr.	200 marks
5. *Civil Engineering Paper II*	3 hr.	200 marks
	Total	1000 marks
*Personality Test cum Interview** (only for those candidates who qualify on the basis of the written examination)		200 marks
	Grand Total Marks	1200

The syllabus for these papers (revised wef year 2000) is given below :

Syllabus for General Ability Test (Objective Type)

Part A — *General English.* The question paper in General English will be designed to test the Candidate's understanding of English and workman like use of words.

Part B — *General Studies.* The paper in General studies will include knowledge of current events and of such matters as of every day observation and experience in their scientific aspects as may be expected of an educated person. The paper will include questions on History of India and Geography of a nature which candidates should be able to answer without special study.

Syllabus for Civil Engineering Paper I (Revised wef the year 2000)

(Same as for Objective as well as Conventional Papers)

1. **Building Materials**

Timber. Different types and species of structural timber, density-moisture relationship, strength in different directions, defects, influence of defects on permissible stress, preservation, dry and wet rots, codal provisions for design, Plywood.

Bricks. Types, Indian Standard classification, absorption, saturation factor, strength in masonry, influence of mortar strength on masonry strength.

Cement. Compounds of, different types, setting times, strength.

Cement Mortar. Ingredients, proportions, water demand, mortars for plastering and masonry.

Concrete. Importance of W/C Ratio, Strength, ingredients including admixtures, workability, testing for strength, elasticity, non-destructive testing, mix design methods.

2. **Solid Mechanics.** Elastic constants, stress, plane stress, Mohr's circle of stress, strains, plane strain, Mohr's circle of strain, combined stress; Elastic theories of failure; Simple bending, Shear, Torsion of circular and rectangular sections and simple members.

3. **Structural Analysis.** Analysis of determinate structures - different methods including graphical methods.

Analysis of indeterminate skeletal frames - moment distribution, slope-deflection, stiffness and force methods, energy methods, Muller-Breslau principle and application.

Plastic analysis of indeterminate beams and simple frames-shape factors.

* In the personality test special attention will be paid to assessing the candidates capacity for leadership, initiative and intellectual qualities, mental & physical energy, powers of practical application and integrity of character.

4. **Design of Steel Structures.** Principles of working stress method. Design of connections, simple members; Built-up sections and frames, Design of Industrial roofs. Principles of ultimate load design. Design of simple members and frames.

5. **Design of Concrete and Masonry Structures.** Limit state design for bending, shear, axial compression and combined forces. Codal provisions for slabs, beams, walls and footings. Working stress method of design of R.C. members.

Principles of prestressed concrete design, materials, methods of prestressing, losses. Design of simple members and determinate structures. Introductions to prestressing of indeterminate structures.

Design of brick masonry as per I.S. Codes.

6. **Construction Practice, Planning and Management.**

Concreting Equipment : Weight Batcher, Mixer, vibrator, batching plant, concrete pump. Cranes, hoists, lifting equipment.

Earthwork Equipment : Power shovel, hoe, dozer, dumper, trailers and tractor, rollers, sheep foot rollers, pumps.

Construction, Planning and Management : Bar chart, linked bar chart, work-break down structures, Activity - on - arrow diagrams. Critical path, probabilistic activity durations ; Event-based net-works.

PERT network : Time-cost study, crashing; Resource allocation.

Syllabus for Civil Engg. Paper II (Revised wef year 2000)

1. (*a*) **Fluid Mechanics, Open Channel Flow, Pipe Flow.** Fluid Properties, pressure, Thrust, Buoyancy; Flow Kinematics; Integration of flow equations; Flow measurement; Relative motion; Moment of momentum; Viscosity, Boundary layer and Control, Drag, Lift; Dimensional Analysis, Modelling; Cavitation; Flow oscillations; Momentum and Energy principles in Open channel flow, Flow controls, Hydraulic jump, Flow sections and properties; Normal flow, Gradually varied flow; Surges; Flow development and losses in pipe flows, Measurements; Siphons; Surges and Water hammer; Delivery of Power Pipe networks.

(*b*) **Hydraulic Machines and Hydropower.** Centrifugal pumps, types, performance parameters, scaling, pumps in parallel; Reciprocating pumps, air vessels, performance parameters; Hydraulic ram; Hydraulic turbines, types, performance parameters, Controls, choice; Power house, classification and layout, storage, pondage, control of supply.

(2) (*a*) **Hydrology.** Hydrological cycle, precipitation and related data analyses, PMP, Unit and synthetic hydrographs; Evaporation and transpiration; Floods and their management, PMF; Streams and their gauging; River morphology; Routing of floods; Capacity of Reservoirs.

(*b*) **Water Resources Engineering.** Water resources of the globe : Multipurpose uses of Water : Soil-Plant-Water relationships, Irrigation systems, Water demand assessment; Storages and their yields, Ground water yield and well hydraulics; Water logging, drainage design; Irrigation revenue; Design of rigid boundary canals, Lacey's and Tractive force concepts in canal design, Lining of canals; Sediment transport in canals; Non-Overflow and overflow sections of gravity dams and their design, Energy dis-

sipators and tail water rating; Design of headworks, distribution works, falls, cross-drainage works, outlets; River training.

(3) (*a*) **Water Supply Engineering.** Sources of supply, yields, Design of intakes and conductors; Estimation of demand; Water quality standards; Control of Water-borne diseases; Primary and secondary treatment, detailing and maintenance of treatment units; Conveyance and distribution systems of treated water, leakages and control; Rural water supply; Institutional and industrial water supply.

(*b*) **Waste Water Engineering.** Urban rain water disposal; Systems of sewage collection and disposal; Design of sewers and sewerage systems; Pumping; Characteristics of sewage and its treatment, Disposal of products of sewage treatment, Stream flow rejuvenation, Institutional and industrial sewage management; Plumbing Systems; Rural and semi-urban sanitation.

(*c*) **Solid Waste Management.** Sources, classification, collection and disposal; Design and Management of landfills.

(*d*) **Air and Noise Pollution and Ecology.** Sources and effects of air pollution, monitoring of air pollution; Noise pollution and standards; Ecological chain and balance, Environmental assessment.

(4) (*a*) **Soil Mechanics.** Properties of soil, classification and interrelationship; Compaction behaviour, methods of compaction and their choice; Permeability and seepage, flow nets, Inverted filters; Compressibility and consolidation; Shearing resistance, stresses and failure; soil testing in laboratory and in-situ; stress path and applications. Earth pressure theories, stress distribution in soil; soil exploration, samplers, load tests, penetration tests.

(*b*) **Foundation Engineering.** Types of foundations, Selection criteria, bearing capacity, settlement, laboratory and field tests; Types of piles and their design and layout, Foundations on expansive soils, swelling and its prevention, foundation on swelling soils.

5. (*a*) **Surveying.** Classification of surveys, scales, accuracy; Measurement of distances - direct and indirect methods; optical and electronic devices; Measurement of directions, prismatic compass, local attraction; Theodolites - types; Measurement of elevations - Spirit and trigonometric levelling; Relief representation; Contours; Digital elevation modelling concept; Establishment of control by triangulations and traversing - measurements and adjustment of observations, computation of coordinates; Field astronomy, Concept of global positioning system; Map preparation by plane tabling and by photogrammetry; Remote sensing concepts, map substitutes.

(*b*) **Transportation Engineering.** Planning of highway systems, alignment and geometric design, horizontal and vertical curves, grade separation; Materials and construction methods for different surfaces and maintenance. Principles of pavement design; Drainage.

Traffic surveys, Intersections, signalling : Mass transit systems, accessibility, networking.

Tunnelling, alignment, methods of construction, disposal of muck, drainage, lighting and ventilation, traffic control, emergency management.

Planning of railway systems, terminology and designs, relating to gauge, track, controls, transits, rolling stock, tractive power and track modernisation; Maintenance; Appurtenant works; Containerisation.

Harbours - layouts, shipping lanes, anchoring, location identification; Littoral transport with erosion and deposition; sounding methods; Dry and Wet docks, components and operational Tidal data and analyses.

Airports - layout and orientation; Runway and taxiway design and drainage management; Zoning laws; Visual aids and air traffic control; Helipads, hangers, service equipment.

Note 1. Calculators are not allowed in objective Type papers but are allowed in conventional type papers.

Note 2. In Question papers, wherever required, SI units will be used.

A perusal of this syllabus shows that **Paper I** covers the entire Structural Engineering field, including *Building construction, Applied mechanics, Strength of materials, Theory and design of Structures* including *R.C.C., Steel, Masonry* and *Timber structures.*

Similarly, **Paper II** covers *Fluid mechanics, Irrigation and Hydrology, Water Supply and Sewage, Soil mechanics, Surveying, and Transportation.*

Both the objective papers; *i.e. Paper I (objective)* as well as *Paper II (objective)* usually contain 120 questions each, to be solved in 120 minutes. All the subjects covered in these papers usually get almost equal weightage. The objective questions from Irrigation side (*i.e.* water resources side for the years 1993 to 2002 have been given in the next chapter.

Similarly, both the conventional papers; *i.e. Paper I (conventional)* as well as *paper II (consventional)* usually respectively contain 8 questions, out of which 5 questions are to be solved. This paper-II usually carries 2 questions from Irrigation and Hydrology side, which are fully covered through this book. The other 2 to 3 questions are covered from author's other books on "Water Supply Engineering", "Sewage Disposal and Air Pollution Engineering" and "Soil-Mechanics and Foundation Engineering", thus enabling you to fetch near about cent per cent marks in this paper, thus, opening your way for selection in this competition for an excellent professional career.

You, *my dear students*, would further feel enlightened, when we inform you that in the past years, many numerical questions on water resources engineering side, have been directly set from the solved examples of this book, besides setting almost all the theoretical questions from this book.

Say for example, *in the year 1980, only two numerical questions of 80 marks were set, and both from this book in toto, without any change even in the numerical data values. In the year 1978, only one question appeared from irrigation, and that too from this book. In the year 1983, $2\frac{1}{2}$ questions were set from Irrigation, and all these were from this book* (with very slight modifications in the numerical data values), *similarly, one numerical question on 'Economics of Lining' appeared in 1986, with exactly the same data as solved in this book. In 1989, one full question on determining gross irrigation requirement for wheat crop was set from this book (with slight modifications in rainfall values), besides a question on S-hydrograph, which was also fully covered by this book. In 1993 as well as in the year 2000, all the three numerical questions on irrigation side were set from the solved examples of this book.*

In other years also, questions which appeared in this UPSC paper, were fully covered by this book.

In order to enable you to check these facts, and for your facility and guidance, given below are the questions that appeared during the last 23 years since the year 1978, from Irrigation and Hydrology side in this UPSC competition.

Questions of the Past Years Relating to Water Resources Engg. from the Engg. Services Competitions (Civil Engineering Paper II)

Year 1978 *(Engineering Services)*

Q. 5. Wheat is to be grown in a field having field capacity equal to 27% and the permanent wilting point is 13%. Find the storage capacity at 80 cm depth of the soil, if

the dry unit weight of the soil is 1.5 gm/cc, if irrigation water is to be supplied when the average soil moisture falls to 18% , find the water depth required to be supplied to the field if the field application efficiency is 80%. What is the amount of water needed at the canal outlet if the water lost in the water courses and the field channel is 15% of the outlet discharge ? (40*)

[**Note.** Pl. refer solved Example 2.14. **This question was set from this book**].

Year 1979 *(Engineering Services)*

Q. 2. (*a*) Define critical tractive stress and state the main factors on which it depends. A wide unlined channel carrying silt free water has a depth of 2.0 m. The maximum tractive stress permissible on the bed to prevent scour is 0.20 kg/m^2. What is the maximum slope that can be given to the channel. (16)

[**Hint.**

$$\tau_0 = \gamma RS,$$

$$\therefore \quad \tau_{0\,(max)} = \gamma R.S_{max}$$

$$= 1000 \times 2.0 \times S_{max} = 0.20 \text{ kg/cm}^2$$

$$\therefore \quad S_{max} = 1 \text{ in } 10{,}000.$$ **Ans.**]

(b) Water discharging from an overfall spillway flows to a jump type basin. The discharge intensity is 30 $m^3/s/m$ and the water depth is 1.5 m. Find the conjugate depth for the jump to form. Give the sketch of the basin showing approximate dimensions and basin appurtenances considered desirable. (17)

[**Hint.** Conjugate depth

$$= y_2 = -\frac{y_1}{2} + \sqrt{\frac{y_1^2}{4} + \frac{2q^2}{gy_1}}$$

$$= -\frac{1.5}{2} + \sqrt{\frac{1.5^2}{4} + \frac{2 \times 30^2}{9.81 \times 1.5}}$$

$$= -0.75 + \sqrt{0.5625 + 122.324}$$

$$= -0.75 + 11.08$$

$$= 10.33 \text{ m}$$

Length of jump basin

$$= 5\,(y_2 - y_1) = 5\,[10.33 - 1.5] = 44.15 \text{ m.}$$ **Ans.**]

Or

The ordinates of a 3 hour unit hydrograph are as given :

Time, hours	0	3	6	9	12	15	18	21	24	27	30
Ordinates, m^3/s	0	10	25	20	16	12	9	7	5	3	0

Find the ordinates of 6 hour hydrograph for the same basin :

[**Note.** Please refer Example 7.33]

Year 1980 *(Engineering Services)*

In 1980, five questions, each of 40 marks, were to be attempted. There were two full questions on "Irrigation and Hydrology", which were set from this very book, as given below :

* Marks allotted for the question, out of maximum 200 marks of Civil Engg. Paper II.

Q. 3. *The rainfall runoff relation for the 655.2 sq. km. of drainage basin under dry soil conditions is given below :*

Rainfall (cm)	*Runoff (cm)*
1.5	*0.0*
5.0	*2.0*
7.5	*4.0*
10.0	*6.5*
12.5	*8.75*

A storm of 2 hr effective duration occurred over this basin at a time when the soil was dry. The resulting hydrograph was as follows :

Time (hr)	*Flow (cumecs)*
0	*0*
3	*300*
6	*500*
9	*400*
12	*250*
15	*75*
18	*0*

Determine the number of mm of runoff from this basin from this storm, assume zero base flow. Approximately how much rain was there in this storm ? What was the average coefficient of runoff during this storm ? (40)

[**Note.** Pl. refer solved Example 7.40. **This question was set from this book**. The value of C.A. has since been revised.]

Q. 6. (*a*) A 1.5 m Sarda type fall is provided at a canal with a discharge of 12 cumecs, with the following data :

Bed level upstream	= *103.0 m*
Side slope of channel	= *1 : 1*
Bed level downstream	= *101.5 m*
Full supply level upstream	= *104.5 m*
Bed width U/S & D/S	= *1.0 m*
Soil	= *good loam*
Bligh's coefficient	= *6*

Calculate the following elements of the fall :

(*i*) *Crest R.L.*

(*ii*) *Length of the crest and the thickness at the base.*

(*iii*) *Depth of the cistern.* (40)

Or

(*b*) Calculate the elements of the fall required on Q. 6 (*a*) using as much of the data as you require. Assume data not given.

[**Note.** Pl. refer solved Example 12.3. **This question was set from this book**].

Year 1981 *(Engineering Services)*

Q. 6. (*a*) Define specific energy and critical depth. (8)

(*b*) Draw the specific energy diagram and label its various parts with brief explanation. The critical depth may also be shown. Discharge remains constant. (12)

(*c*) A trapezoidal channel has a bottom width of 6.0 m and side slopes of 1 : 1. The depth of flow is 1.5 m at a discharge of 15 m^3/sec. Determine the specific energy. (12)

(*d*) If the critical depth is 0.9 m, discuss the type of flow very briefly, corresponding to the critical depth. (8)

Q. 8. Using Rational method, rainfall-intensity duration curves (Diagram I) and the data given on diagram II, compute the diameter of the outfall sewer. The length of lines, drainage areas, and inlet times are marked on diagram II itself.

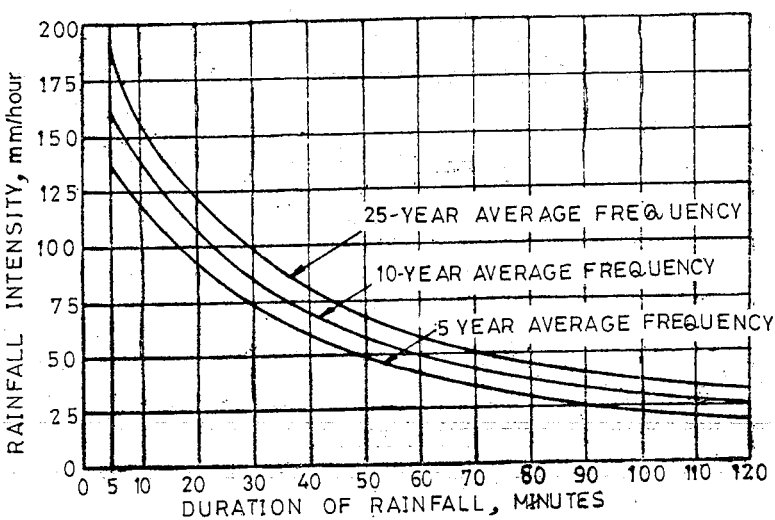

Diagram I (Fig. 31.1)

Assume :

(*i*) *Runoff coefficient for the entire area = 0.30.*

(*ii*) *Velocity of flow in sewers flowing full = 0.75 m/sec.*

(*iii*) *5 year average frequency curve may be used (Diagram II).*

(*iv*) *Table 1 gives the hydraulic elements for circular pipes flowing full.*

Table 1. Hydraulic Elements for a Circular Pipe Flowing Full

Quantity of flow, l/s	*Dia. of pipe, mm*	*Slope of pipe, m/m*	*Velocity m/sec*
400	450	0.025	2.7
600	525	0.020	2.8
690	1050	0.00055	0.75
1500	1350	0.001	1.55
2000	1450	0.001	1.20

(40)

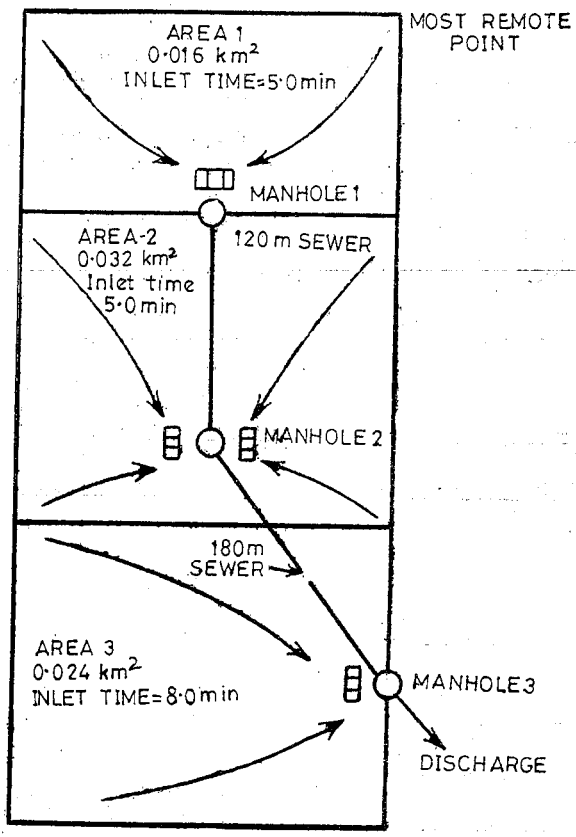

Diagram II (Fig. 31.2)

Hint. This question has been solved on Pages 790 to 791 in author's another book titled "Sewage Disposal and Air Pollution Engg." (11th Edition – 1998)

Year 1982 *(Engineering Services)*

Q. 2. *(a) Explain briefly the concept of a unit hydrograph.* (5)

(b) What is a S-curve ? What is its use ? (5)

(c) On a catchment area of 20 hectares, rainfalls of 7.5 cm, 2.0 cm and 5.0 cm occurred on three consecutive days. The average ϕ_{index} was 2.5 cm/day. The distribution graph percentages of surface runoff which extended over six days for every rainfall of such magnitudes are 5, 15, 40, 25, 10 and 5. Determine the ordinates of the discharge hydrograph and determine the peak discharge. Neglect base flow. (30)

[**Note.** Please refer Example 7.47]

Q. 3. *(a) List the various forces to be considered in the stability analysis of a gravity dam. Indicate the expressions to determine their value.* (15)

(b) Investigate the stability against overturning of a solid gravity dam at the base section. The dam is in the shape of a rightangled triangle with the upstream face vertical.

Ht. of dam	= *120 m*
Slope of d/s face	= *0.75 H : 1 V*
Ht. of water	= *120 m*
Free-board	= *0*
Ht. of tail water	= *0*
Unit wt. of concrete	= *2400 kg/m³*
Uplift intensity factor	= *0.5*
Allowable coeff. of friction	= *0.75*

Neglect silt pressure, earthquake forces, waves forces and its pressure. Assume any suitable data as required. (25)

Year 1983 *(Engineering Services)*

Q. 1. *(b) Design a concrete lined channel to carry a discharge of 500 cumecs at a slope of 1 in 4000. The side slopes of the channel may be taken as 1 : 1. The Manning's roughness coefficient for the lining is 0.014. Assume permissible velocity in the section as 2.5 m/sec.* (20)

[**Hint.** Follow Solved Example 5.4]

Q. 2. *(a) Describe briefly how the profile of ogee-shaped spillway section is designed.* (12)

(b) The tail water rating curve for the channel below an overflow spillway is below the "jump height curve" for all expected discharges. For this case, explain with the help of neat sketches the energy dissipation arrangements which can be provided below the spillway. (16)

(c) What are the functions and design considerations of canal head regulators (12)

Q. 4. *(a) A flood of a certain magnitude has a return period of 25 years. What is its probability of exceedence ? What is the probability that this flood may occur in the next 12 years ?* (10)

[**Note.** Please refer Example 7.52.]

(b) In a typical 4 hour storm producing 100 mm of runoff from a basin, the stream flows observed are as follows :

Time in hr.	0	2	4	6	8	12	16	20
Flow in cumecs	0	2.44	8.10	13.50	11.34	6.75	2.70	0.0

Estimate the peak flow in the stream and the time of its occurrence in a flood created by an 8 hr storm which produces 50 mm of runoff during the first four hours and 75 mm of runoff during the second four hours. Assume base flow to be negligible. (20)

[**Note.** Please refer solved Example 7.32. **This question was set from this book by just doubling the data values**].

(c) Explain how the average depth of precipitation over an area, due to a storm is computed from isohyetal map of the storm. (10)

Year 1984 *(Engineering Services)*

Q. 2. *(a) Mention the various types of canal falls commonly used indicating distinctive features, and describe briefly design principles of any one popular type.* (20)

(b) A weir has a solid horizontal floor length of 50 m with two lines of cutoff 8 m depth, below the river bed, at its two ends. The floor thickness is 1 m at the upstream end and 2 m at the downstream end, with its upper level being in flush with the river bed. For an effective head of 5 m over the weir, calculate the uplift pressure at the two inside corner points junction of bottom of floor with the cutoff), and also exit gradient. (20)

Q. 3. *(a) Describe briefly Kennedy's theory for the design of regime channels.* (10)

(b) Design a regime channel to carry a discharge of 100 cumecs in a soil having silt factor of 1.1, using Lacey's theory. Assume side slopes of 0.5 horizontal to 1.0 vertical. (18)

[**Note.** Please follow solved Example 4.9].

Q. 4. *(a) What are the basic propositions of unit hydrograph theory as propounded by L.K. Sherman ?* (10)

(b) The mass curve of precipitation resulted from the storm of 14th August 1983 gave the following values :

Hour	*Accumulated depth at the end of periods in mm*
22.00	0.0 (Beginning of storm)
22.05	10.2
22.10	20.8
22.15	33.0
22.20	47.2
22.25	55.8
22.30	64.0
22.35	71.6
22.40	78.8
22.45	85.4
22.50	91.4 (End of storm)

For the above storm, construct hydrograph, and draw maximum intensity duration curve. (20)

[**Note.** Please refer Example 7.11]

(c) Describe the function and design consideration of undersluices provided in diversion head works. (10)

Year 1985 *(Engineering Services)*

Q. 2 *(a) Describe briefly the main functions of a distributary 'Head Regulator' and 'Cross Regulator'. Mention the salient steps for their design.* (20)

(b) Design an irrigation channel by Kennedy's method to carry 50 cubic metres per second of discharge with base width to depth ratio as 2.5. The critical velocity ratio is

1.1. Assume Kutter's rugosity coefficient as 0.025 and side slopes of the channel as 0.5 horizontal to 1.0 vertical. (20)

[**Note.** Please follow solved Example 4.7].

Q. 3. *(a) How do you determine the storage capacity of a reservoir with the help of mass curve of runoff, if a constant or a variable demand is known ?* (12)

(b) Explain briefly the 'Infiltration Indices'. (8)

(c) In estimating the peak discharge of a river at X, the catchment area was divided into four parts A, B, C and D. The time of concentration and the area for different parts are as follows :

Part	*Time of concentration*	*Area in square metres*
A	*One hour*	600×10^4
B	*Two hours*	750×10^4
C	*Three hours*	1000×10^4
D	*Four hours*	1200×10^4

Records for a rain storm lasting for four hours as observed, and the runoff factor during different hours are as follows :

Time in hr.		*Rainfall (in mm)*	*Run-off factor*
From	*To*		
0	*1*	*25.0*	*0.50*
1	*2*	*50.0*	*0.70*
2	*3*	*50.0*	*0.80*
3	*4*	*23.5*	*0.85*

Calculate the maximum flow to be expected at X in cumecs, assuming a constant base flow of 42.5 cumecs. (20)

[**Note.** Please refer Example 7.31].

Q. 4. *(a) Explain briefly the functions and requirements of a good module. How are modules classified ? Mention common examples of each type.* (18)

(b) What are the various assumptions made in two dimensional design of gravity dams ? (12)

(c) Describe briefly uplift Pressure on Syphon Flooring in cross-drainage works. (10)

Year 1986 *(Engineering Services)*

Q. 3. *(a) Explain 'Retrogression of Downstream Levels' consequent to weir construction.* (8)

(b) Describe briefly causes of failure of earthen dams. (12)

(c) An unlined canal giving a seepage loss of 4.0 cumecs per million sq metres of wetted area is proposed to be lined with 12 cm thick cement concrete lining, which costs Rs. 20.0 per sq. m. Workout the economics of lining and show if the scheme is justified on the basis of the following data:

Annual revenue from crops = *Rs. 4,00,000 per cumec of water.*
Discharge of the channel = *50 square metre*
Wetted perimeter of the channel = *22.4 m*
Wetted perimeter of the lining = *22.0 m*
Annual maintenance cost of unlined channel = *12 paise per sq. metre*
Seepage loss in lined channel = *0.01 cumec per million sq. metre of wetted perimeter.*
Saving in annual maintenance as a consequence to lining = *40%*
Life of lining = *40 years*
Rate of interest = *6 per cent*
Assume additional suitable data, if required. (20)

[**Note.** Please refer solved Example 5.1. **This question was set from this book with slight modifications in numerical data values.**]

Q. 4. *(a) Explain how maximum intensity-duration frequency curve is obtained.* (12)

(b) Define :

(i) Maximum Probable Flood

(ii) Standard Project Flood (10)

(c) Estimate the average depth of precipitation from depth area curve, that may be expected over an area of 2400 sq km, due to the storm of 27th Sept. 1978, lasting for 24 hours, assuming the storm centre to be located at the centre of the area. The isohyetal map for the storm gave the areas enclosed between different isohyets as follows :

Isohyetal in mm	*21*	*20*	*19*	*18*	*17*	*16*	*15*	*14*	*13*	*12*
Enclosed area in sq km	*543*	*1345*	*2030*	*2545*	*2955*	*3280*	*3535*	*3710*	*3880*	*3915*

Hence, determine the depth of rainfall, due to the storm that may be expected to be recorded by a raingauge placed at the storm centre. (18)

[**Note.** Please refer Example 7.7].

Year 1987 *(Engineering Services)*

Q. 5. (*a*) A round crested spillway passes a design discharge of 1 m^3/sec per metre length. The coefficient of discharge may be taken as $C_d = 0.7$. If the height of the crest above the downstream stilling basin floor level is 10 m, design the

(*i*) depth, and

(*ii*) length of the stilling basin. Depth of flow in the stream on the downstream of the spillway is 1 m at the design discharge of 1 m^3/sec. Enquire if the bed of the stilling basin has to be depressed. (20)

Q. 6. (*b*) A railway embankment is 16 m wide with side slopes 2 : 1. Assume the ground to be level in the direction transverse to the centre line. Calculate the volume contained in a length of 100 m, the centre heights at 20 m intervals being in m : 2.0, 4.5, 4.0, 3.5, 2.5, 1.5. Use Trapezoidal formula. (20)

Year 1988 *(Engineering Services)*

Q. 1. *(a) A certain stretch of lined trapezoidal channel has one vertical side wall and the other 45° sloping wall. If it is to deliver water at 30 m^3/sec with a velocity of 1 m/sec, compute bed width and flow depth for minimum lining area.* (10)

Q. 6. *(a) Define unit hydrograph.*

Ordinates of 8 hour unit hydrograph for a drainage basin are given below. Obtain a 24 hour unit hydrograph by tabulation method. Neatly sketch it.

Time hours (1)	*Ordinates of 8 hr unit hydrograph* (2)	*Time hours (contd.)* (1)	*Ordinates of 8 hr unit hydrograph (contd.)* (2)
0	0.00	48	57.00
4	5.50	52	42.00
8	13.50	56	41.00
12	26.50	60	22.00
16	45.00	64	14.00
20	82.00	68	9.50
24	162.00	72	6.60
28	240.00	76	4.00
32	231.00	80	2.00
36	165.00	84	1.00
40	112.00	88	0.00
44	79.00		

(40)

[**Note.** Please refer Example 7.34].

Year 1989 *(Engineering Services)*

Q. 2. *For use in Hargreaves method, in respect of Group D wheat crop, the following K values are suggested :*

% age of crop growing season	0	5	10	15	20	25	30	35	40	45	50
K	0.08	0.08	0.15	0.19	0.27	0.33	0.40	0.46	0.52	0.58	0.65

% age of crop growing season	55	60	65	70	75	80	85	90	95	100
K	0.71	0.77	0.82	0.88	0.90	0.90	0.80	0.70	0.60	0.60

Wheat is grown between 1 November to 15 March. The effective rainfall in the individual months is taken as follows :

Nov.–0.5 cm ; Dec.–1.6 cm; Jan–3.2 cm; Feb.–2.7 cm ; March–Nil. Percolation losses are nil throughout, Field Irrigation Efficiency is 70% and Gross Irrigation Efficiency is 75%. The mean class A pan evaporation through the respective months are:

Nov.–13 cm; Dec.–11 cm; Jan.–7 cm; Feb.–15 cm; March–10cm. Compute the gross irrigation requirement for the whole crop period including 7 cm of net pre-sowing requirement. (40)

[**Hint.** Follow the sample calculations given for wheat crop in table 2.14, and compute G.I.R. as equal to 55.02 cm, as per table given on next page].

Solution Table for Q. 2 of 1989 (Engg. Services) : *Growth period* = 1 Nov. to 15 March (135 days)

Dates (1)	*No. of days upto mid point of interval from start* (2)	*%age of growing season* col. (2) $\times \frac{100}{135}$ (3)	*Pan evaporation* (E_p) *in cm* (4)	*Coefficient (K) interpolated values for %age values of col. (3) from given values* (5)	$E_t = KE_p = C_u$ *i.e. col. (4) × col. (5); consumptive use in cm* (6)	R_e = *Effective rainfall in cm (given)* (7)	*Net irr. requirement NIR* = $C_u - R_e$ = *col. (6) – col. (7)* (8)
Nov. 1—30	15	11	13	0.16	2.08	0.50	1.58
Dec. 1—31	46	34	11	0.46	5.06	1.60	3.46
Jan. 1—31	77	57	9	0.73	6.57	3.20	3.37
Feb. 1—28	106	78	15	0.90	13.50	2.70	10.80
March 1—15	128	95	10	0.60	6.00	Nil	6.0
					Z = 33.21 cm	8.00 cm	25.21 cm

$$\text{N.I.R.} = 25.21 \text{ cm}$$

$$\text{F.I.R.} = \frac{N.I.R.}{\eta_f} = \frac{25.21}{0.70} = 36.01 \text{ cm}$$

$$\text{G.I.R.} = \frac{F.I.R.}{\eta_c} = \frac{36.01}{0.75} = 48.02 \text{ cm}$$

Net presowing requirement before 1 Nov. = 7 cm.

∴ Total gross irrigation requirement = 48.02 + 7 = 55.02 cm. **Ans.**

Q. 8. *(b) A 2 hour unit hydrograph in a rather steep catchment is given below :*

Time, hour	0	2	4	6	8	10	12
Discharge, 100 m^3/s	0	0.54	1.75	1.27	0.58	0.25	0

Compute the 1 hour unit hydrograph for the catchment. (25)

[**Hint.** Please refer Solved Example 7.51].

Year 1990 *(Engineering Services)*

Q. 1. *(c) Given that the unit discharge in a rectangular channel is 18 m^3/sec and that the head loss across a hydraulic jump that forms in this channel is 1.1 m, estimate the prejump and post jump depths.* (15)

Solution. Using eqn. (10.5), and (10.12), we have

$$y_1 y_2 (y_1 + y_2) = \frac{2q^2}{g} = \frac{2 \times 18^2}{9.81} = 66.06 \quad ...(i)$$

[q = unit discharge *i.e.*, discharge per unit width]

and $$H_L = \frac{(y_2 - y_1)^3}{4 y_1 y_2}$$

where $H_L = 1.1$ m (given)

$$\therefore \quad (y_2 - y_1)^3 = 4.4\, y_1 y_2 \quad ...(ii)$$

Putting $y_2 = m y_1$ in (*ii*), we get

$$(m-1)^3 \cdot y_1^3 = 4.4 \cdot m y_1^2$$

$$\therefore \quad y_1 = \frac{4.4m}{(m-1)^3} \quad ...(iii)$$

Putting $y_2 = m y_1$ and this value of y_1 in (*i*), we get

$$y_1 \cdot m \cdot y_1 (y_1 + m \cdot y_1) = 66.06$$

or $$m(m+1) \cdot y_1^3 = 66.06$$

or $$m(m+1)\left[\frac{4.4m}{(m-1)^3}\right]^3 = 66.06$$

or $$\frac{m(m+1)(m)^3}{(m-1)^9} = \frac{66.06}{4.4^3}$$

or $$\frac{m^4(m+1)}{(m-1)^9} = 0.775$$

Solving by hit and trial, we get

$$m = 2.93$$

Substituting it in (*iii*), we get

$$y_1 = \frac{4.4 \times 2.93}{(1.93)^3} = \mathbf{1.79\ m}$$

$$\therefore \quad y_2 = 2.93 \times 1.79 = \mathbf{5.25\ m}$$

Ans.

Q. 2.. *Work out the irrigation schedule based on the soil moisture concept, given the following information. Also extract the data on the total depth of irrigation water required and the respective dates of irrigation water supply :*

(a) The crop is grown in an appropriate soil with no restrictive layers within the top 1.5 m depth of soil.

(b) Normal root zone depth of the crop is 1.2 m.

(c) Bulk density of soil is 1.35.

(d) Field capacity is 18% and permanent wilting point is 7%.

(e) Moisture level in the soil is to be maintained at not less than one-third of available retention. Irrigation will then be done over a duration of 2 days at a uniform rate of supply and at a uniform rate of advance to fully and just compensate for the depletion.

(f) No extra water is ever required for leaching.

(g) Sowing is done on 1 November when the soil moisture is left just at field capacity in the entire root zone.

(h) For the crop, at the location, the average evapotranspiration rates are :

1 Nov. — 30 Nov.	:	*1.1 mm/day*
1 Dec. — 31 Dec.	:	*1.7 mm/day*
1 Jan. — 31 Jan.	:	*2.4 mm/day*
1 Feb. — 28 Feb.	:	*1.5 mm/day*
1 March—25 March	:	*3.5 mm/day*

(i) Harvesting is done an or after 26 March.

(j) There is expected an effective rainfall of 24 mm during 4 January to 19 January, both days inclusive with uniform intensity.

(k) By the end of the crop growth season, only the minimum water needed to be left unused in the root zone.

[**Hint.** Pl see Solved Example 2.16]

Q. 3. *(c) An equipment used in construction activity costs Rs. 40,000 at first purchase. The average annual expenditure thereon for OMR is estimated to be Rs. 9,000 for the next three years for which funds be procured at the beginning of each year. At the end of the third year, it can be scrapped at a then value of Rs. 10,000. If the acceptable rate of return is 15% per annum, what should be the annual income by this machine as reckoned at the end of each year ?*

[**Note.** Please refer Solved Example 20.2 in "*Hydrology and Water Resources Engineering*" (11th-2000 Edition) by the same author].

Q. 8. *(a) The average rainfall values over a catchment in three successive 2-hour intervals are 3, 5 and 2 cm, respectively. The ϕ index for the catchment is taken as 0.2 cm/hr. The 2 hr unit hydrograph ordinates are given in table. Base flow can be taken as 7 m^3/sec at the beginning of the storm linearly increasing to 9 m^3/sec at 2 hours after the direct runoff peak discharge, and later linearly decreasing to 8 m^3/sec at 4 hours after the end of the direct runoff. Compute the resulting flood hydrograph.*

Time, hours	0	2	4	6	8	10	12	14	16
Discharge, m^3/sec	0	10	40	70	50	20	10	6	0
			(Peak)						

(15)

[**Solution.** Unit hydrograph (U.H.) ordinates are tabulated in col. (2) of the solution table given below. The discharge hydrographs are now produced in three successions, by 3 successive storms, each containing effective rainfalls equal to $R - \phi$; *i.e.* $3 - 2 \times 0.2 = 2.6$ cm ; $5 - 2 \times 0.2 = 4.4$ cm ; and $2 - 2 \times 0.2 = 1.6$ cm, respectively. Hydrograph ordinates due to these three rains are computed by multiplying U.H. ordinates by each, because U.H. is produced by 1 cm rain. These values are worked out in col. (3), (4) and (5) respectively. All of them are then added in col. (6) to obtain direct runoff hydrograph (D.R.H.). Base flow is then computed, and added to obtain hydrograph of resulting discharge as given in table below..

Solution Table for Q. 8 (*a*) of Year 1990 (Engg. Services)

Time from start, hr (1)	*U.H. ordinates* (2)	*Discharge ordinates caused by 1st rain = col. (2) × 2.6* (3)	*Discharge ordinates caused by 2nd rain (at 2 hr lag) = col. (2) × 4.4* (4)	*Discharge ordinates caused by 3rd rain = col. (2) × 1.6 at 4 hr lag* (5)	*Total D.R.H. = col. (3) + (4) + (5)* (6)	*Base flow* (7)	*Total flow ordinates in m^3/sec = col. (6) + (7)* (8)
0	0	0	—	—	0	7	7.0
2	10	26	0	—	26	7.4	33.4
4	40	104	44	0	148	7.8	155.8
6	70	182	176	16	374	8.2	382.2
8	50	130	308	64	502	8.6	510.6*
10	20	52	220	112	384	9	393.0
12	10	26	88	80	194	8.85	202.85
14	6	6	44	32	82	8.71	90.71
16	0	0	26.4	16	42.4	8.57	50.97
18	—	—	0	9.6	9.6	8.43	18.03
20	—	—	—	0	0	8.29	8.29
22	—	—	—	—	—	8.15	8.15
24	—	—	—	—	—	8	8.0

* Peak = 510.6 m^3/sec.

Year 1991 (*Engineering Services*)

Q. 5. *(a) Enumerate the assumptions on which Dupuit-Theim theory of ground water flow is based and state the range of validity of the law governing the flow. Define the following terms :*

(i) Storage coefficient, (ii) Specific yield, (iii) Specific retention, (iv) Porosity, stating clearly the relationship which exists among some of these terms. (20)

Year 1992 (*Engineering Services*)

Q. 4. *(b) The following data pertains to healthy growth of a crop :-*

(i) Field capacity of soil	*= 30%*
(ii) Permanent wilting percentage	*= 11%*
(iii) Density of soil	*= 1300 kg/m³*
(iv) Effective depth of root zone	*= 700 mm*
(v) Daily consumptive use of water for the given crop	*= 12 mm*

For healthy growth, moisture content must not fall below 25% of the water holding capacity between the field capacity and the permanent wilting point. Determine the watering interval in days. (20)

[**Solution.** Max. water holding capacity of soil

= Field capacity m.c. – Permanent wilting pt. m.c.
= 30% – 11% = 19%

25% of Max. holding capacity = 0.25 × 19% = 4.75%

The left out available m.c. must at no stage be allowed to fall below this value and watering should be applied as soon as the left out m.c. falls to such a low level. Hence lower limit to which m.c. can be allowed to fall = 4.75% above wilting point m.c. (at which plants fade away) = 4.75% + 11% = 15.75%.

In other words, the m.c. will be allowed to vary between 30% and 15.75%. Water depth stored between these two limits

$$= \frac{\gamma d}{w}[30\% - 15.75\%]$$

$$= \frac{1300}{1000} \times 0.7\,[0.30 - 0.1575]$$

$$= 0.1297 = 12.97 \text{ cm.}$$

Since consumptive use is 12 mm/day ;

12 mm of water is consumed in = 1 day

∴ 12.97 cm of water will be consumed in

$$= \frac{1}{12} \times (12.97 \times 10) = 10.8 \text{ days.}$$

The watering should, therefore, be applied after every 10 days.

Note. 10.8 days cannot be rounded to 11 days, as otherwise the left out m.c. will fall down below the minimum required. **Ans.**]

Q. 7. *(b) A sluice across a channel 6 metre wide discharges a stream 1 meter deep. What is the flow rate when the depth upstream of the sluice is 7 metre ? On the downstream side, concrete blocks have been placed to create conditions for hydraulic jump to occur. Determine the force on the blocks if the downstream depth is 3 metre.* (15)

[**Solution.** Discharge through the sluice gate

$$Q = C_d\,(6 \times 1)\,\sqrt{2gH}$$

as width of channel = 6 m

area of flow = 6 m × 1 m

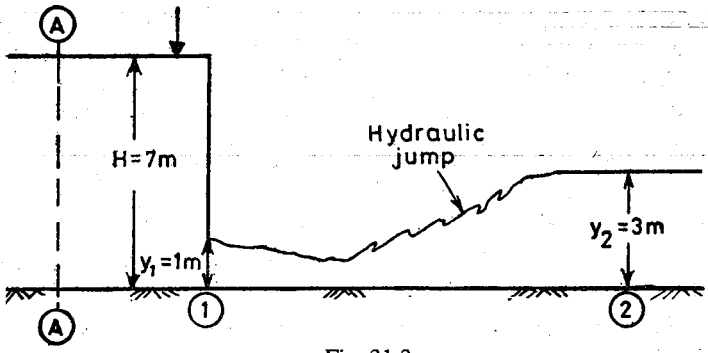

Fig. 31.3

where $C_d = C_c \cdot C_v$

Assuming $C_v = 0.95$

$C_c = 0.62$

$C_d = C_v \cdot C_c = 0.59$

$$Q = 0.59 \times 6 \times 1 \times \sqrt{2 \times 9.81 \times 7}$$

$$= 41.49\ \text{m}^3/\text{s} \quad \textbf{Ans.}$$

(*b*) Force acting on concrete blocks is given by the rate of change of momentum per unit channel width.

$$= \left(\frac{w}{g}\right) q\,(V_1 - V_2)\ \text{kg}$$

where $q = \dfrac{Q}{6} = \dfrac{41.49}{6} = 6.91\ \text{m}^2/\text{s}$

$$V_1 = \frac{q}{C_c \cdot y_1} = \frac{6.91}{0.95 \times 1}\ \text{m/s}$$

$$= 7.28\ \text{m/s}$$

$$V_2 = \frac{q}{y_2} = \frac{6.91}{3} = 2.3\ \text{m/s}$$

$$= \frac{1000 \times 6.91}{9.81}\,(7.28 - 2.3)$$

$$= 3504\ \text{kg} = 3504 \times 9.81\ \text{Newton} = 34.37\ \text{kN}. \quad \textbf{Ans.}]$$

Year 1993 *(Engineering Services)*

Q. 1. *(a) A water course commands an irrigated area 1000 hectares. The intensity of irrigation of rice in this area is 70%. The transplantation of rice crop takes 15 days and during the transplantation period, the total depth of water required by the crop on the field is 500 mm. During the transplantation period, the useful rain falling on the field is 120 mm. Find the duty of irrigation water for the crop on the field during transplantation at the head of the field, and also at the head of the water course*

assuming losses of water to be 20% in the water course. Also calculate the discharge required in the water course. (20)

[**Note :** Please follow exactly similar solved Example 3.5, and Compute : (*i*) 341 ha/cumec; (*ii*) 273 ha/cumec ; and (*iii*) 2.57 cumecs. **Ans.**]

Q. 2. *(c) A precipitation station X was inoperative for some time during which a storm occurred. At three stations A, B and C surrounding X, the total precipitation recorded during this storm are 75, 58 and 47 mm respectively. The normal annual precipitation amounts at stations X, A, B and C are respectively 757, 826, 618 and 482 mm. Estimate the storm precipitation for station X.* (10)

[**Note.** Please follow Exactly Similar Solved Example 7.1, and compute ppt. to be **71.2 mm. Ans.**]

Q. 3. *(c) During a recuperation test, the water in an open well was depressed by pumping by 2.5 m and it recuperated 1.8 m in 80 minutes. Calculate the yield from a well 4 m diameter under a depression head of 3 m.* (10)

[**Note.** Please follow exactly similar solved example 16.9, and compute yield as **10 l/sec. Ans.**]

Year 1994 *(Engineering Services)*

Q. 2. *(b) The information available from an isohyetal map of 1100 sq km basin is as follows :*

Zone	*Area* km^2	*Rain gauge station*	*Normal annual rainfall, cm*
I.	*85*	*A*	*120*
II.	*290*	*B* *C*	*95* *96*
III.	*395*	*D* *E* *F*	*60* *65* *70*
IV.	*230*	*G*	*45*
V.	*65*	*H*	*21*
VI.	*35*	—	—

How many additional rain gauge stations will be required if the desired limit of error in the mean value of rainfall is not to exceed 10 cm? Suggest how you propose to distribute these stations.

What factors will you consider in locating the additional rain gauge stations between different isohyetals ? (20)

[**Solution.** Follow Solved Example 7.3, and compute

$$\text{Mean rainfall} = \overline{P} = \frac{120+95+96+60+65+70+45+21}{8} = \frac{572}{8} = 71.5 \text{ cm}$$

$$\overline{P}^2 = \frac{\Sigma p^2}{n}$$

$$= \frac{(120)^2+(95)^2+(96)^2+(60)^2+(65)^2+(70)^2+(45)^2+(21)^2}{8} = 5979$$

$$\sigma = \sqrt{\frac{n}{n-1}[\overline{P^2} - (\overline{P})^2]}$$

$$= \sqrt{\frac{8}{7}(5979 - (71.5)^2} = 31.47$$

$$C_v = \frac{100\sigma}{\overline{P}} = \frac{100 \times 31.47}{71.5} = 44.019$$

Optimum No. of Rain gauges

$$= N = \left(\frac{C_v}{E}\right)^2$$

where E = % error allowed in mean rainfall

$$= \frac{10 \text{ cm}}{\overline{P}} = \frac{10}{71.5} \times 100 = 13.986$$

$$\therefore \quad N = \left(\frac{44.019}{13.986}\right)^2 = 9.91 : \textbf{say 10}$$

Additional gauges required

$= 10 - 8 =$ **2 No. Ans.]**

One of these two additional gauges can be installed in Zone IV, which is having only one gauge, although covering an area of 230 km^2, *i.e.* about 20% of total catchment, and thus may need about 20% × 10 gauges = 2 gauges. The other additional gauge can be installed in Zone VI, which at present, is not having any gauge at all. **Ans.]**

Q. 5. *(c) The base period, intensity of irrigation, and duty of water for various crops under the canal system are given. Determine the reservoir capacity if the culturable commanded area is 40,000 hectares, canal losses are 25%, and reservoir losses are 15%.*

Crop	*Base period days*	*Duty of the water at the field hec/cumec*	*Intensity of irrigation %*
Wheat	*120*	*1800*	*20*
Sugarcane	*360*	*1700*	*20*
Cotton	*180*	*1400*	*10*
Rice	*120*	*800*	*15*
Vegetables	*120*	*700*	*15*

(15)

[Solution. Follow Example 3.6, and compute Areas of different crops under irrigation by multiplying *CCA* (40,000 hectares) with intensity of irrigation, as given in column (5) of solution table. Water required by each crop in cumecs is then determined in column (6) by dividing crop area in hec by duty in hec/cumec. The volume of water required by each crop in *B* days is finally computed in column (7) by multiplying base period in secs with water required in m^3/sec *i.e.* column (2) × 24 × 60 × 60 × column (6).

Solution Table for Q. 5 (c) of Year 1994 *(Engg. Services)*

Crop	*Base period (B) in days*	*Duty at the field in hec/cumec*	*I.I. %*	*Area of each crop under irrigation CCA × I.I. = 40,000 × Col. (4)*	*Water required by each crop in m^3/sec = Col. (5) / Col. (3)*	*Volume of water required in m^3 in (B) days = Col. (2) × 24 × 60 × 60 × Col. (6)*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*
Wheat	120	1800	20	8,000	4.444	46.08 M-m^3
Sugarcane	360	1700	20	8,000	4.706	146.37 M-m^3
Cotton	180	1400	10	4,000	2.857	44.43 M-m^3
Rice	120	800	15	6,000	7.50	77.76 M-m^3
Vegetables	120	700	15	6,000	8.571	88.87 M-m^3
Σ			80%	32,000		403.51 M-m^3

Hence, total water required at the field

$$= 403.51 \text{ Mm}^3.$$

Canal losses $= 25\%$

∴ Water required at Canal Head

$$= \frac{403.51}{0.75} \text{ Mm}^3 = 538.013 \text{ Mm}^3$$

Reservoir losses $= 15\%$

∴ Gross Reservoir Capacity $= \dfrac{538.013}{0.85} \text{ Mm}^3 = 632.96 \text{ Mm}^3$

say **633 Mm³** **Ans.]**

Year 1995 *(Engineering Services)*

Q. 1. *(b) Calculate the bed width for an irrigation channel to carry a discharge of 5 cumec. Side slopes of the channel are 1/2 horizontal to 1 vertical. Assume critical velocity ratio as 0.8. Take first trial depth of flow as 1 m. The channel has a bed slope of 0.2 m per kilometre. Using Kutter's equation $V = C \cdot \sqrt{R \cdot S}$, check the depth. The value of C for the given conditions is computed as 30. V is the mean velocity of flow in m/sec, and R is the hydraulic mean radius in metre.*

State the factors on which the value of C will depend. (15)

[**Hint.** Using $y = 1.0$ m (*assumed in 1st trial as given*), compute $V_0 = \mathbf{0.44\ m/s}$;

and $A = \dfrac{Q}{V_0} = \dfrac{5}{0.44} = 11.36 = \left(B + \dfrac{1.0}{2}\right) 1.0$ or $B = 10.86$ m;

$P = 10.86 + \sqrt{5} \times 1.0 = 13.10$, $R = \dfrac{11.36}{13.10} = 0.867$.

Then $V = 30 \cdot \sqrt{0.867 \times \dfrac{0.2}{1000}} = \mathbf{0.395\ m/s}$. To reduce V_0 reduce y in second trial to about 0.7 m, when we get $V \approx V_0 \approx 0.35$ and $B = 20$ m. **Ans.**]

Q. 2. *(a) A stream has a width of 30 m, depth of 3 m and a mean velocity of 1.25 m/sec. Find the height of a weir to be built on the stream floor to raise the water level by 1 m. Assume value of discharge coefficient as 0.95.* (15)

[**Hint.** If x is the height of the weir, then head over the weir causing flow $= (4 - x)$, because D/s water depth is 3 m, and U/s water depth is 4m.

$$Q = \frac{2}{3} \cdot C_d \cdot \sqrt{2g} \cdot B \cdot h^{3/2} = 30 \times 3 \times 1.25$$

or $$\frac{2}{3} \times 0.95 \sqrt{2 \times 9.81} \times 30 \times (4 - x)^{3/2} = 30 \times 3 \times 1.25$$

or $x = \mathbf{2.79\ m}$ **Ans.**]

Q. 3. *(b) An aquifer of 20 m average thickness is overlain by an impermeable layer of 30 m thickness. A test well of 0.5 m diameter and two observation wells at a distance of 10 m and 60 m from the test well are drilled through the aquifer. After pumping at a rate of 0.1 m^3/sec for a long time, the following drawdowns are stabilised in these wells : first observation well, 4 m, second observation well 3 m. Show the arrangement in a diagram. Determine the coefficient of permeability and drawdown in the test well. State the validity of Darcy's Law.* (15)

[**Hint.** Solve the question using Thiem's formula for confined aquifers, and determine $K = \mathbf{1.425 \times 10^{-3}}$ **m/sec Ans.**

Then use r_w in place of r_1 and s_w in place of s_1 in this formula and compute $s_w = \mathbf{6.059}$ **m. Ans.**]

Q. 5. *(c) A farmer wishes to have his own pump set for the following cropping pattern to be followed in five hectares of his land. Calculate the right size of the centrifugal pump he should have, litres/sec.*

Season	*Crop*	*Area to be irrigated, ha*	*Intensity of irrigation, cm*	*Rotation period, days*
Rabi	*Wheat*	*2.0*	*7.5*	*12*
	Cotton	*0.4*	*7.5*	*20*
	Vegetables	*0.4*	*7.5*	*10*
	Mustard	*2.2*	*5.0*	*40*

For each crop, duration of pumping hours per day is 10. (10)

[**Hint.** It can be seen from the given data that the water is required for different crops as below :

(*i*) *for wheat* $= (2 \times 10^4\ m^2) \times \frac{7.5}{100}\ m = 300\ m^3$ @ every 12th day in 10 hrs.

(*ii*) *for cotton* $= (0.4 \times 10^4\ m^2 \times \frac{7.5}{100}\ m = 300\ m^3$ @ every 20th day in 10 hrs.

(*iii*) *for vegetables* $= (0.4 \times 10^4\ m^2) \left(\frac{7.5}{100}\ m\right) = 300\ m^3$ @ every 10th day in 10 hrs.

(*iv*) *for mustard* $(2.2 \times 10^4 \text{ m}^2)\left(\frac{5}{100} \text{ m}\right) = 1100 \text{ m}^3$ @ every 40th day in 10 hrs.

This shows that maximum water is required for wheat, *i.e.* 1500 m^3 in 10 hrs.

∴ Water required in l/s to be pumped

$$= \frac{(1500 \times 1000)\, l}{(10 \times 3600) \text{ sec}} = \mathbf{41.67\ l/s} \quad \textbf{Ans.}$$

Q. 7. *(b) A masonry dam 10 m high is trapezoidal in section with a top width of 1 m and bottom width of 8.25 m. The face exposed to water has a batter of 1 : 10. Calculate (i) the factor of safety against overturning; (ii) Factor of safety against sliding; (iii) Shear friction factor. Assume coefficient of friction as 0.75. Unit weight of masonry as 2240 kg/cu.m. Permissible shear stress of joint = 14 kg/cm². Based on the results give your remarks.* (15)

[**Hint.** Draw the section of the dam, and compute

$\Sigma V = 103.6$ t ; $\Sigma H = 50$ t

$\Sigma M_V = 528$ tm ; $\Sigma M_H = 166.5$ t.m.

$$\therefore \text{ F.S. against overturning} = \frac{\Sigma M_V}{\Sigma M_H} = \frac{528}{166.5} = \mathbf{3.17} \quad \textbf{Ans.}$$

$$\text{F.S. against sliding} = \frac{\mu \cdot \Sigma V}{\Sigma H} = \frac{0.75 \times 103.6}{50} = \mathbf{1.55} \quad \textbf{Ans.} \quad (> 1\ ;\ \text{OK})$$

$$\text{S.F.F.} = \frac{\mu \Sigma V \cdot B \cdot q}{\Sigma H} = \frac{0.75 \times 103.6 + 8.25 \times 14}{50}$$

$$= \mathbf{3.86} \quad \textbf{Ans.} \quad (> 3\text{–}5\ ;\ \text{OK})$$

The structure is safe. **Ans.**]

Year 1996 (*Engineering Services*)

Q.1. *(a) For a date of maximum recorded flood of a river, the mean and standard deviation are 4200 m³/s and 1705 m³/s, respectively. Using Gumbel's extreme value distribution, estimate the return period of a design flood of 9500 m³/s. Assume an infinite sample size.* (20)

[**Hint.** Use eqn = (7.142) as :

$$X = \overline{X} + k \cdot \sigma$$

where $\overline{X} = 4200 \text{ m}^3/\text{s}$

$\sigma = 1705 \text{ m}^3/\text{s}$

$X = 9500 \text{ m}^3/\text{s}$

$\therefore \quad K = 3.1085$

Now, use eqn. (7.148) as :

$$K = \frac{y_{(T)} - 0.577}{1.2825} \quad \text{for infinite sample size as given } i.e.\ N \to \alpha$$

$\therefore \quad y_{(T)} = 4.5637$

Now, use eqn. (7.146) as :

$$y_{(T)} = -\left[\ln \ln \frac{T}{T-1}\right]$$

$$\therefore \quad (-)\,4.5637 = \left[\ln \ln \frac{T}{T-1}\right]$$

or $$\frac{T}{T-1} = 1.0104$$

or $$\frac{T-1}{T} = 1 - \frac{1}{T} = 0.9896$$

or $$T = 96.43$$

Hence, the return period of design flood of 9500 m^3/s = **96.43 Yrs. Ans.**]

Q.2. *(b) The following is the set of observed data for successive 15 minute period of 105 minutes storm in a catchment.*

Duration (Min)	*Rainfall cm/hr.*
15	*2.0*
30	*2.0*
45	*8.0*
60	*7.0*
75	*1.25*
90	*1.25*
105	*4.5*

If the value of ϕ_{index} is 3.0 cm/hr, estimate the net runoff, the total rainfall and the value of W_{index} (20)

[**Hint.** Follow exactly 'similar solved example 7.24 and compute :

$$\text{Total runoff } (Q) = (8-3)\frac{15}{60} + (7-3)\frac{15}{60} + (4.5-3)\frac{15}{60}$$

or $$Q = \frac{5}{4} + \frac{4}{4} + \frac{1.5}{4} = \frac{10.5}{4} = \mathbf{2.625\ cm} \quad \textbf{Ans.}$$

Total rainfall = P

$$= 2 \times \frac{15}{60} + 2 \times \frac{15}{60} + 8 \times \frac{15}{60} + 7 \times \frac{5}{60} + 1.25 \times \frac{15}{60} + 1.25 \times \frac{15}{60} + 4.5 \times \frac{15}{60}$$

$$= 26 \times \frac{1}{4} = \mathbf{6.5\ cm} \quad \textbf{Ans.}$$

$$W_{index} = \frac{P-Q}{Tr} = \frac{6.5 - 2.625}{\left(\frac{105}{60}\right)} = \mathbf{2.214\ cm/hr} \quad \textbf{Ans.}$$]

Q.3. *(a) During a recuperation test, the water in an open well was depressed by pumping by 2.1 m and it recuperated by 1.6 m in 90 minutes. Find the diameter of a well to yield 10 litre/sec under a depression head of 2m.* (20)

[**Hint.** Follow exactly similar solved example 16.10, and compute :

$$\frac{C'}{A} = \frac{2.3}{T} \log \frac{s_1}{s_2} \qquad \text{(Eqn 16.39)}$$

$$= \frac{2.3}{90 \times 60} \log \frac{2.1}{2.1 - 1.6}$$

$$= 2.64 \times 10^{-4} \text{ m}^3/\text{sec/m}^2/\text{m}$$

Now use eqn. (16.40) as :

$$Q = \left(\frac{C'}{A}\right) \cdot A \,.\, s$$

$$\therefore \quad \frac{10}{1000} \text{ m}^3/\text{s} = (2.654 \times 10^{-4}) \times A \times 2$$

or $\qquad A = 18.83$

$\therefore$ diameter of well giving 10 $l/s = d =$ **4.9 m Ans.**]

Q 5. *(b) Enumerate the various types of energy dissipation devices which may be recommended below spillway in relation to the relative position of tail water rating curve and jump height rating curve.* (30)

[**Hint.** See article 21.6]

Q. 6. *(a) Design a lined canal to carry 100 m³/s on a slope of 1 in 2500. The maximum permissible velocity is 2 m/s, n = 0.013 in Manning's formula, and the side slope = 1.25 H : 1.0 V.* (20)

[**Hint.** Follow exactly similar solved example 5.4 and compute.

$$2 = \frac{1}{0.013} \times R^{2/3} \cdot \frac{1}{\sqrt{2500}}$$

or $\qquad R = 1.482 \text{ m} \qquad$ ...(*i*)

$$\cot \theta = 1.25$$

$$\theta = 0.675 \text{ radians}$$

From eqn = (5.8) ; $A = y\,(B + 1.925y) = \frac{Q}{V} = \frac{100}{2} = 50$

or $\qquad (B + 1925y) = \frac{50}{y}$

or $\qquad B = \left(\frac{50}{y} - 1.925y\right) \qquad$ ...(*ii*)

From eq. 5.9 ; $P = B + 3.85\,y$

$$R = \frac{A}{P} = \frac{50}{(B + 3.85\,y)} = 1.482 \qquad \text{...(iii)}$$

$\therefore \qquad B + 3.85y = 33.74$

or $\qquad B = 33.74 - 3.85y \qquad$ ...(*iv*)

Equating (*ii*) and (*iii*)

$$\frac{50}{y} - 1.925y = 33.74 - 3.85\,y$$

or $$50 - 1.925y^2 = 33.74\,y - 3.85\,y^2$$

$$1.925y^2 - 33.74y + 50 = 0$$

or $$y = \frac{33.74 \pm \sqrt{33.74^2 - 4 \times 50 \times 1.925}}{2 \times 1.925} = \frac{33.74 \pm 27.45}{3.85}$$

$$\left.\begin{array}{l} y = 1.634\text{ m} \\ B = 27.45\text{ m} \end{array}\right] \quad \textbf{Ans.}$$

Q.8. *(a) Calculate the seepage through an earthen dam resting on an impervious foundation. The relevant data are given below :*

Height of the dam	= 60.0 m
Upstream slope	= 2.75 : 1 (H : *V*)
Downstream slope	= 2.50 : 1 (H : *V*)
Freboard	= 2.5 m
Crest width	= 8.0 m
Length of drainage blanket	= 120.0 m

Coefficient of permeability of the embankment material in

X-direction = 8×10^{-7} m/s

Y-direction = 2×10^{-7} m/s

(20)

[**Hint.** As a matter of fact such problems have generally been solved in "Soil Mechanics", and hence the students will have to refer and study article 7.7.1.1 in authors another book titled "Geotech Engineering – Soil Mechanics and Foundation Engineering – 3rd 1999 edition)", before attempting to solve this question.

According to the theory explained in that article, we have by eqn. (7.30)

q = discharge through dam = K.S.

where $K = \sqrt{K_H \cdot K_V}$

S = Focal distance of base parabola forming the top seepage line in the dam body.

S is further given by eq. (7.28) as

S = Focal distance w.r. Fig. 7.22 in "Soil Mechanics"

$$= \sqrt{b^2 + H^2} - b$$

where H = Max. water depth = 60 – 2.5 = 57.5 m

= 47 + 8 + 30 (Please see Fig. 31.4)

= 85 m

$$\therefore\ S = \sqrt{85^2 + 57.5^2} - 85 = 17.62\text{ m}$$

$$q = \sqrt{K_H \cdot K_v} \cdot S$$

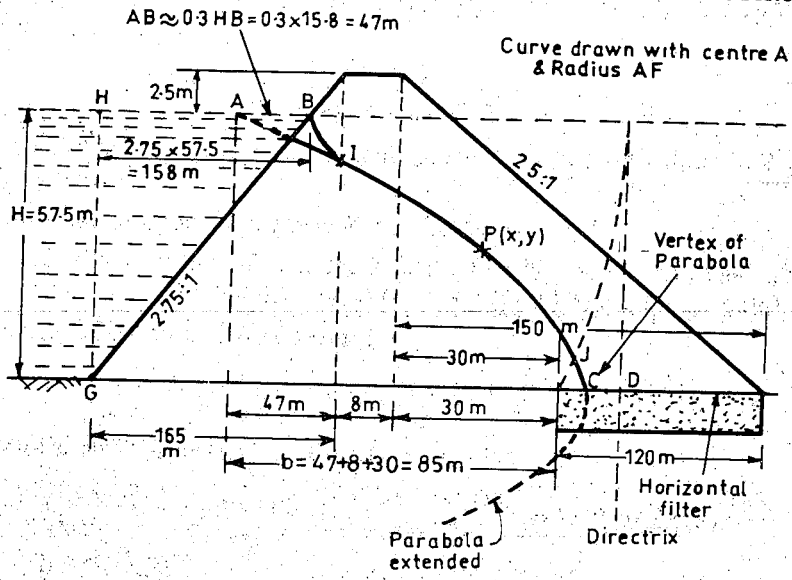

Fig. 31.4.

$= \sqrt{8 \times 10^{-7} \times 2 \times 10^{-7}} \times 17.62\ m^3/sec$ per m width of dam

$= 4 \times 10^{-7} \times 17.62\ m^3/s.m$

$= 70.48 \times 10^{-7}\ m^3/sec.m = 0.610\ m^3/day.\ m$

= 610 litres per day per *m* width of dam. **Ans.**

Year 1997 (*Engineering Services*)

Q. 1. *(a) A large sample of peak floods data was available for a river. Flood frequency computations using Gumbel's method, yield the following results.*

Return Period T (Years)	*Peak Flood m^3/sec*
50	*30,800*
100	*36,300*

Estimate the magnitude of a flood for this river with a return period of 200 years. (15)

[**Hint.** Follow exactly similar solved Example 7.57 and determine $X_{(200)} = 41780\ m^3/sec$ **Ans.**]

Q. 1. *(c) A most efficient trapezoidal section is required to give a maximum discharge of 21.5 m^3/s of water. The slope of the channel bottom is 1 in 2500. Taking C = 70 $m^{1/2}/s$ in Chezy's equation, determine the dimensions of the channel. Also determine the value of Manning's 'n' taking the value of velocity of flow as obtained for the channel by Chezy's equation.* (15)

[**Hint.** Pl refer Solved Example 4.5]

Q. 2. *(c) Show that the drawdown in the case of a partially penetrating well in a confined aquifer (well just penetrating to the top of an infinite porous medium) is given by the equation.*

$$S_w = \frac{Q_s}{2\pi \cdot r_w \cdot K}$$

where Q_w = discharge from well under steady conditions

r_w = radius of the well

K = Coefficient of permeability

S_w = drawdown at the well

Also find an expression for $\frac{Q_s}{Q_r}$

where Q_r = discharge from a fully penetrating well in confined aquifer under steady state (derivation for Q_r not required)

(10)

[**Hint.** Please refer article 16.13]

Q. 3. (*a*) Sugarcane (root zone depth 1.8 m) is grown in a particular area where the ground watertable is 2.0 m below ground. If the size of the soil pores is 0.08 mm in diameter and surface tension $\sigma = 0.054$ N/m, is the field water-logged ? If so, determine the vertical location of closed dreius below ground, spaced at 15 m. Take drainage coefficient as 0.116 *cumecs/km*^{2}, Coefficient of permeability as 10^{-6} m/s, and the impervious stratum to occur at 7.0 m below ground. Assume the capillary rise from ground watertable not to interfere with the root zone of plant. (15)

[**Hint.** Please refer Solved Example 6.5]

Q. 6. (*c*) The following figure gives profile of a gravity dam with reservoir level as shown. If the coefficient of friction is 0.75, is the dam safe against sliding ? Take weight density of concrete = 2.4 Tonnes/m^3.

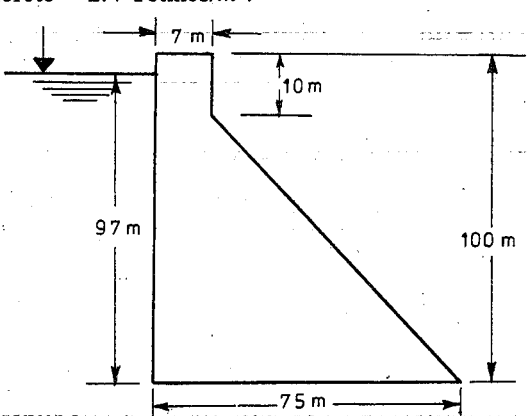

Fig. 31.5

(15)

Year 1998 (*Engineering Services*)

Q. 3. (*c*) What are the factor affecting evaporation from water surfaces ? Describe briefly a method of estimating the evaporation from weather data. (10)

Q. 6. (*c*) *Compute and draw the storm hyetograph and the intensity duration curve for the following storm (of a given-frequency) on a drainage basin.*

Duration (Minutes)	*Accumulated Precipitation (cm)*
0	–
30	*5.0*
60	*7.5*
90	*8.5*
120	*9.0*

(15)

[**Hint.** Follow exactly similar solved numerical Example 7.11]

Q. 7. *(a)* A soil 50 cm deep over rock has two horizons, the first being a fine sandy loam 20 cm thick and the second a clay loam 30 cm thick. The field capacity, wilting point and volume weight for the first horizon are 20 percent, 10% percent and 1.5 gm/cm^3, respectively. The corresponding values for the second horizon are 25 percent, 13 percent and 1.2 gm/cm^3. Determine the available moisture storage capacity of the soil profile. If consumptive use requirements of a crop in a particular season is 0.5 mm/day and the soil is initially at field capacity, how long will the crop survive without irrigation. (15)

[**Solution.**

(*i*) Moisture holding capacity of first horizon (fine sandy loam of 20 cm thick)

$$= \frac{\gamma_d \cdot d}{\gamma_w} [F.C. - W.P]$$

$$= \frac{1.5}{1} \times 20 \text{ cm } [20\% - 10\%] = 3 \text{ cm} \quad ...(i)$$

(*ii*) Moisture holding capacity of 2nd horizon (clay loam of 30 cm depth)

$$= \frac{1.2}{1} \times 30 \text{ cm } [25\% - 13\%] = 4.32 \text{ cm} \quad ...(ii)$$

Total max available moisture *i.e.* storage capacity of the soil

$$= (i) + (ii)$$

$$= 3 \text{ cm} + 4.32 \text{ cm} = \mathbf{7.32 \text{ cm}} \quad \textbf{Ans.}$$

Consumptive use requirement per day

$$= 0.5 \text{ mm/day} = 0.05 \text{ cm/day}$$

Max. No. of days in which the entire moisture storage capacity will be utilised *i.e.* the absolute No. of days for which crop will survive

$$= \frac{\text{Moisture Storage capacity in cm}}{\text{Consumptive use in cm/d}}$$

$$= \frac{7.32 \text{ cm}}{0.05 \text{ cm/day}} = \mathbf{146.4 \text{ days.}} \quad \textbf{Ans.}$$

Q. 7. *(c) A vertical lift gate 5m × 2.5 m size weighing 0.5 Tonnes slides along guides (coefficient of friction 0.25) fitted on the side walls of an overflow spillway and its crest. What force will have to be exerted at the hoisting mechanism to lift the gate when the head of water over the crest is 2 m ?* (10)

[**Solution.** In article 24.14.3, while describing the vertical lift gate, we have clearly stated that when such a gate is sliding between groove guides not provided with rollers, force will be required to lift the gate at the hoisting mechanism for not only lifting the self wt. of the gate but also

to overcome the friction that will be developed between the gate and the d/s groove guides due to the hydrostatic force caused by the water standing against the gate

(*i*) self weight of gate = 0.5 tonne wt.

$$= 0.5 \times 9.81 \text{ kN}$$
$$= 4.91 \text{ kN} \qquad ...(i)$$

(*ii*) Hydrostatic force acting on the gate of ht 2.5 m & 5 m span with 2 m water height

$$P_H = \frac{1}{2} \times (9.81 \times 2) \times 2 \text{ m} \times 5 \text{ m} = 98.1 \text{ kN}$$

$$\mu = 0.25$$

∴ Friction force (downward)

$$= \mu \cdot P_H = 0.25 \times 98.1 \text{ kN}$$
$$= 24.53 \text{ kN} \qquad ...(ii)$$

Total Upward force required to lift the gate

$$= (i) + (ii) = 4.91 + 24.53 = 29.44 \text{ kN}.$$

Hence, the force required at hoisting to lift the gate

$$= 29.44 \text{ kN} = \frac{29.44}{9.81} \text{ t force} \quad \textbf{Ans.}$$

$$= 3 \text{ t} \cdot \textbf{force} \quad \textbf{Ans.}$$

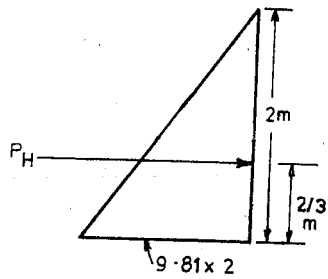

Fig. 31.6

Q. 8. *(a) An irrigation canal takes off from a perennial river. Sketch a typical layout of a diversion headworks. Indicate therein the various components of the headworks.* (15)

Year 1999 (Engineering Services)

Q. 4. (*a*) *The amounts of water flowing from a certain catchment area at the proposed dam site are as tabulated below :*

Month	*Inflow*
January	*2.83*
February	*4.25*
March	*5.66*
April	*18.40*
May	*22.64*
June	*22.64*
July	*19.81*
August	*8.49*
September	*7.10*
October	*7.10*
November	*5.66*
December	*5.66*

Determine :

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

(*ii*) *the initial storage required to maintain the uniform demand as above* (*15*)

[**Hint.** Follow exactly similar solved example 18.8, and compute

(*i*) Min capacity of reservoir = 40.08×10^5 cum
(*ii*) Initial storage reqd. = 19.82×10^5 cum.] **Ans.**

Q. 5. (*a*) *A river discharges 1000 cum/sec of water at high flood level of RL. 103 m. A weir is constructed for flow diversion with a crest length of 255m and total length of concrete floor as 40m. The weir has to sustain the under-seepage at a maximum static head of 2.4m. The silt factor and the safe exit gradient for the river bed material are 1.1 and 1/6, respectively. Determine the depth of cut off required at the downstream end of the concrete floor. Take the level of downstream concrete floor as RL 100.0m. Check for exit gradient.* (15)

[**Solution.** High flood discharge = Q = 1000 m^3/s.

Length of weir = L = 255 m

$$q = \frac{Q}{L} = \frac{1000}{255} = 3.92 \text{ m}^2/\text{s}$$

f = 1.1 (given)

$$R = 1.35\left(\frac{q^2}{f}\right)^{1/3} = 1.35\left(\frac{3.92^2}{1.1}\right)^{1/3} = 3.25 \text{ m}$$

Provide D/s cutoff up to 1.5 R below d/s water level.

$$1.5\,R = 1.5 \times 3.25 = 4.88 \text{ m}$$

D/s W.L. = ?

U/s WL = U/s HFL = 103.00 m (given)

H = Max. Static head causing seepage = 2.4 m (given)

∴ D/s W.L. = U/s HFL − H = 103.0 − 2.4 = 100.6 m

Hence, R.L. of bottom of d/s cutoff = 100.6 − 4.88 = 95.72 m.

R.L. of D/s floor = 100.0 m (given)

∴ depth of d/s cutoff = 100.0 − 95.72 = 4.28 m.

Hence d = **4.28 m** **Ans.**

Exit gradient $G_E = \frac{H}{d} \cdot \frac{1}{\pi\sqrt{\lambda}}$

$$\text{where } \lambda = \frac{1+\sqrt{1+\alpha^2}}{2}$$

$$\text{where } \alpha = \frac{b}{d} = \frac{\text{Length of weir floor}}{\text{depth of d/s cutoff}}$$

where, b = 40 m (given)

d = 4.28 m (calculated above)

$$\therefore \quad \alpha = \frac{40}{4.28} = 9.346$$

$$\therefore \quad \lambda = \frac{1+\sqrt{1+9.346^2}}{2} = 5.2$$

$$\therefore \quad G_E = \frac{2.4 \text{ m}}{4.28 \text{ m}} \frac{1}{\pi.\sqrt{5.2}} = \mathbf{0.078} \textbf{ Ans.}$$

$$= \frac{1}{12.77} < \frac{1}{6} \text{ (Safe exit gradient)}$$

Hence, the weir is safe from exit gradient considerations with bottom of d/s cut off at RL 95.72 m. **Ans.**

Q. 6. (*c*) *What are the assumptions and limitations of Sherman's Unit Hydrograph Theory ?* (10)

[**Hing.** Pl See article 7.45. 1].

Q. 8. (*b*). *A sandy loam soil holds water at 140 mm/m depth between field capacity and permanent wilting point. The root depth of the crop is 30 cm and the allowable depletion of water is 35%. The daily water use by the crop is 5 mm/day. The area to be irrigated is 60 ha and water can be diverted at 28 lps. The surface irrigation application efficiency is 40%. There are no rainfall and ground water contribution.*

Determine

(*i*) *allowable depletion depth between irrigations.*

(*ii*) *frequency of irrigation*

(*iii*) *net application depth of water*

(*iv*) *Volume of water required*

(*v*) *time to irrigate 4 ha plot* (15)

[**Hint:** Pl see Solved Example 2.16]

Year 2000 (*Engineering Services*)

Q.5(*a*). *The regression analysis of a 30-year flood data at a point on a river yielded sample mean* $\bar{x} = 1200\ m^3/sec$ *and standard deviation* $s_x = 650\ m^3/sec$. *For what discharge would you design the structure at this point to provide 95% assurance that the structure would not fail in the next 50 years ? Use Gumbel's method. The value of mean and standard deviation of the reduced variate for* $n = 30$ *are 0.53622 and 1.11238, respectively.* (15)

[**Solution.** Follow article 7.50 3.2 and solved examples 7.58 & 7.55 and compute as under:

$\bar{y}_n$ = reduced mean for $n = 30$

$= \mathbf{0.53622}$

s_n = reduced standard deviation for $n = 30$

$= \mathbf{1.11238}$

$\bar{X}$ *i.e.* $\bar{x} = 1200\ m^3/sec.$ (given)

σ = standard deviation of variate x

$= 650\ m^3/sec$ (given under symbol s_x)

$X_{(T)}$ = Discharge for return period T years = ?

$T = ?$

Return period T has to be such that the structure does not fail in the next 50 years with 95% assurance. To find this value of T, please refer solved example 7.55, and compute as under :

The structure has not to fail in the next 50 years with 95% assurance means that it can tolerete a risk of 5% ; *i.e.* $R = 5\% = 0.05$ in 50 years (*i.e.* $n = 50$ years]

Using $R = [1 - (1 - p)^n]$, we get

where $n = 50$ years

$$0.05 = [1 - (1 - p)^{50}]$$

or $$(1 - p)^{50} = 1 - 0.05 = 0.95$$

or $$1 - p = (0.95)^{1/50} = (0.95)^{0.02} = 0.9989746$$

or $$p = 1 - 0.9989746 = 0.001025$$

$$\therefore \quad T = \frac{1}{p} = \frac{1}{0.001025} = \mathbf{975.29\ yrs.}$$

For this value of T, we have to determine $X_{(T)}$ as below

$$X_T = \bar{X} + K_{(T)}\,\sigma$$

where $K_{(T)} = \dfrac{y_{(T)} - \bar{y}_n}{s_n}$ (*i.e.* Eq. 7.148*a*)

For $T = 975.29$ year, $y_{(T)}$ is given by eqn. (7.146) as :

$$y_{(T)} = -\left[I_n \cdot I_n \cdot \frac{T}{T-1}\right]$$

$$y_{(975.29)} = -I_n \cdot I_n \cdot \left(\frac{975.29}{974.29}\right) = 6.882222$$

$$\therefore \quad K_{(975.29)} = \frac{6.882222 - 0.53622}{1.11238} = 5.70488$$

The expected peak discharge having frequency of 975.29 years is now given as

$$X = \bar{X} + K.\sigma$$

or $$X_{(979.29)} = 1200 + 5.70488 \times 650$$

$$= 1200 + 3708 = \mathbf{4908\ m^3/sec}\ \textbf{Ans.}$$

Q. 6(*a*). *In order to compute the flood discharge in a stream by the slope-area method, the following data have been obtained :*

	u/s Section	*Middle Section*	*d/s Section*
Area	108.6 m^2	103.1 m^2	99.8 m^2
Wetted Perimeter	65.3 m	60.7 m	59.4 m
Gauge Reading	316.8 m	—	316.55 m

Determine the flood discharge assuming Manning's n = 0.029 and length between d/s and u/s sections as 250 m (15)

[Hint. *Follow exactly similar solved numerical example 7.23.* The only point here is that the values of Area (A) and wetted perimeter (P) given here at mid section are not to be used any where, except to infer that the stream is a *gradually contracting* one, since the area of cross-section as well as wetted perimeter values are gradually decreasing. Additionally, in solved example 7.23, the values of hydraulic mean depths(R) were given ; here they can be worked out as $R = \dfrac{A}{P}$; otherwise this numerical example can be solved similarly, by assuming $h_e = K_e\left(\dfrac{V_2^2}{2g} - \dfrac{V_1^2}{2g}\right)$; where $K_e = 0.1$. The computed value of flood discharge initially works out to 159.89 m^3/sec. which on correction for velocity head & eddy loss h_e, works out to **154** m^3/sec **Ans.]**

Q. 7(*b*). *For border strip method of irrigation, determine the time required to irrigate a strip of land of 0.04 hectares in area from a tube-well with a discharge of 0.02 cumec. The infiltration capacity of the soil may be taken as 5 cm/hr. and the average depth of flow on the field as 10 cm. Also determine the maximum area that can be irrigated from this tubewell.* (15)

[**Hint.** Refer solved example 1.1. **This numerical question was set from this book]**

Q. 8(*a*). *Fig given below shows the section of a gravity dam (non-overflow portion) built of concrete. Calculate the maximum vertical stresses at the heel and toe of the dam. Assume weight of concrete at 23.5 kN/m³. Neglect earthquake effects.*

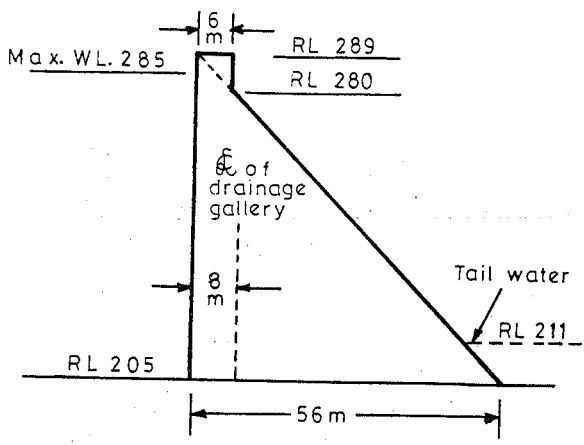

Fig. 31.7

[**Note.** Pl refer solved example 19.1 (*i*) **This numerical question was set from this book]**

Year 2001 (*Engineering Services*)

Q. 2. (*b*) *An unlined irrigation canal has its bed and sides composed of cohesionless material having mean diameter 6 mm. Angle of repose of the material is 40°. The bed width of the canal is 5 m and the side slope 1.5 (H) : 1 V. Determine the maximum discharge that can be admitted into the canal without any sediment movement. Longitudinal slope of the canal is 1 in 5000, and Manning's n = 0.025.* (15)

[**Solution.** Follow & refer solved Example 4.3, and compute as under :

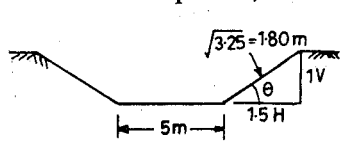

Fig. 31.8

$\phi = 40°$ (given) $\qquad \tan\theta = \dfrac{1}{1.5}$

$\sin\phi = \sin 40° = 0.6428 \qquad \sin\theta = \dfrac{1}{\sqrt{3.25}} = \mathbf{0.5547}$

$$\frac{\tau_c'}{\tau_c} = \sqrt{1 - \frac{\sin^2\theta}{\sin^2\phi}} = \left[1 - \frac{(0.5547)^2}{(0.6428)^2}\right]^{1/2} = \sqrt{0.255} = 0.505.$$

∴ Min. shear stress required to dislodge the soil grain on side slope of canal is given by

$$\tau_c' = 0.505\, T_c$$

Hence, for stability, the shear stress actually going to be generated on slopes of the channel of given R & S must be less than or equal to 0.507 τ_c ; *i.e.*

$$\tau_0' \le 0.507\, \tau_c$$

The actual shear stress actually going to be generated on the side slopes of a channel of given R & S is given by Eqn. (4.17) as :

$$\tau_0 = 0.75\, \gamma_w \cdot R.S.$$

$$\therefore \quad 0.75 \cdot \gamma_w \cdot R.S. \le 0.507 \cdot \tau_c$$

But

$$\tau_c = \gamma_w \cdot R.S. = \gamma_w \cdot \frac{d}{11}$$

$$\therefore \quad 0.75 \cdot \gamma_w \cdot R.S. \le 0.507\, \gamma_w \cdot \frac{d}{11}$$

or $$RS \le \frac{0.507}{0.75 \times 11}\, d \quad \text{or} \quad RS \le 0.0615\, d$$

where d = mean particle dia = 6 mm = 0.006 m

S = 1 in 5000

or $$RS \le 0.0615 \times 0.006 = 3.69 \times 10^{-4}$$

or $$R \times \frac{1}{5000} \le 3.69 \times 10^{-4} \quad \text{or} \quad R \le 1.845 \text{ m.}$$

using 20% factor of safety, we have

max. R = 1.47 m.

For 5 m wide trapezoidal channel with 1.5 (H) : 1 (V) side slopes, we have

$$R = \frac{A}{P} = \frac{(5 + 1.5y)\, y}{(5 + 2 \times 1.8y)} = \frac{(5 + 1.5y)\, y}{(5 + 3.6y)}$$

$$\therefore \quad 1.47 = \frac{(5 + 1.5y)\, y}{5 + 3.6\, y}$$

Solve by Hit & Trial, and find

$$y \approx 2.3 \text{ m.}$$

$$\therefore \quad A = (5 + 1.5 \times 2.3)\, 2.3 = 19.44$$

$$P = (5 + 2 \times 1.8 \times 2.3) = 13.28$$

$$R = \mathbf{1.46} \approx 1.47 \text{ (ok)}$$

$$Q_{\max.} = \frac{1}{n} \cdot A \cdot R^{2/3} \cdot \sqrt{S}$$

$$= \frac{1}{0.025} \times 19.44 \times (1.46)^{2/3} \frac{1}{\sqrt{5000}} = \mathbf{14.15\ m^3/sec.} \quad \textbf{Ans.}$$

Q. 3. (*b*) ***The following are the ordinates for a flood hydrograph resulting from an isolated storm of 6 hours duration.***

Time (hr.)	0	12	24	36	48	60	72	84	96
Ordinates of Flood hydrograph (cumecs)	5	15	40	80	60	50	25	15	5

Determine the ordinates of 1 cm-6 hr. unit hydrograph, if the catchment area is 450 sq. km. (15)

[**Hint :** Follow similar Solved Example 7.39, Assuming base flow = 5 m^3/s, list the ordinates of flood hydrograph as 0, 10, 35, 75, 55, 45, 20, 10 & 0. Compute vol. of water contained in this hydrograph as :

$$= \frac{12 \times 60 \times 60}{3}\left[\frac{0+0}{2} + 4\,(10+75+45+10) + 2\,(35+55+20)\right]$$

$$= 11.232 \times 10^6 \text{ m}^3.$$

Catchment area $= 450 \times 10^6 \text{ m}^2$

$\therefore$ Depth of water in the hydrograph $= \dfrac{11.232 \times 10^6}{450 \times 10^6}$ m = 2.5 cm.

Ordinates of U.H. (of 1 cm depth) can be obtained by dividing the above ordinates of flood hydrograph by 2.5. Hence, the ordinates of U.H. are : 0, 4, 14, 30, 22, 18, 8, 4, 0 **Ans.**]

Q. 4. (*c*). *A lined irrigation canal with trapezoidal cross-section has 5 m bed width, 2.5 m depth and 1.5 (H) : 1(V) side slope, longitudinal bed slope of the canal is 1 in 1000 and Manning's n = 0.016. What is the maximum carrying capacity of the canal ? What area of land in hectares the canal can irrigate if the crop has 150 mm field irrigation requirement in a core period of 10 days.* (10)

[**Solution.**

For a lined trapezoidal channel, corners are rounded, due to which A & P are computed by Eqn. 5.8 & 5.9 as :

$$A = y\,(B + y\,\theta + y \cot\theta)$$

$$P = B + 2y\,\theta + 2y \cot\theta$$

Here for 1.5 (H) : 1(V) side slopes, we have

$$\tan\theta = \frac{1}{1.5} = 0.8,$$

$$\cot\theta = 1.5, \quad \theta = 0.588 \text{ radians}$$

$$y = 2.5 \text{ m}, \quad B = 5 \text{ m}$$

$$\therefore \quad A = 2.5\,(5 + 2.5 \times 0.588 + 2.5 \times 1.5) = 25.55$$

$$P = (5 + 2 \times 2.5 \times 0.588 + 2 \times 2.5 \times 1.5) = 15.44$$

$$R = \frac{A}{P} = \frac{25.55}{15.44} = 1.655$$

$$Q = \frac{1}{n} \cdot A \cdot R^{2/3} \cdot \sqrt{S}$$

$$= \frac{1}{0.016} \times 25.55\,(1.65)^{2/3}\,\frac{1}{\sqrt{1000}} = 70.65 \text{ m}^3/\text{s}$$

Hence, max. carrying capacity of channel = **70.65 m^3/sec. Ans.**

During core period of 10 days, vol. of water which can be supplied by the channel

$$= 70.65 \times (10 \times 24 \times 60 \times 60) \text{ m}^3 = 61.04 \times 10^6 \text{ m}^3.$$

Area which can be irrigated (A) × Depth of water required

= Vol. of water available

$\therefore \quad A \times 0.15 \text{ m} = 61.04 \times 10^6 \text{ m}^3$

or $\quad A = \dfrac{61.04 \times 10^6}{0.15} \text{ m}^2 = 407 \times 10^6 \text{ m}^2 =$ **407 sq. km Ans.]**

Q. 7 (*b*). *Following particulars were recorded from a barrage :*

(*i*) Maximum reservoir level	= 212 m
(*ii*) Pond level	= 211 m
(*iii*) Downstream high flood level in the river	= 210 m
(*iv*) Maximum design flood discharge	= 3500 m^3/s
(*v*) Crest level of the barrage	= 207 m
(*vi*) Crest level of the head regulator	= 208 m
(*vii*) Coefficient of discharge	= 2.10 $m^{1/2}$/sec for barrage
	= 1.50 $m^{1/2}$/sec for head regulator
(*viii*) River bed level	= 205 m
(*ix*) Design discharge of main canal	= 500 m^3/sec

Determine the number of gates required for the barrage and the head regulator if each gate has 10 m clear span. Neglect :

(1) end contractions due to piers and abutments ;

(2) velocity of approach.

If a stilling basin is provided downstream of the barrage for the energy dissipation, find the length and RL of the basin floor. Assume that the length of basin is 5 times the conjugate depth required for hydraulic jump. Neglect losses due to friction. (20)

Solution. (1) **Waterway for Barrage :**

High flood discharge of 3500 m^3/s has to pass through the gated openings provided at the barrage site, with the following data.

Max. water level at high flood U/S of barrage = 212.00 m

Crest level of barrage = 207.00 m

Opening available or Head causing flow above the crest of barrage

= 212.00 – 207.00 = 5.00 m.

(ignoring Vel. head)

Now, $\quad Q = C \cdot L H^{3/2}$

where C = coeff. of discharge through the barrage

= 2.10 (given)

L = Effective waterway of barrage (openings)

H = 5.00 m

$\therefore \quad 3500 = 2.10 \times L \times (5)^{3/2}$

or $\quad L = \dfrac{3500}{2.1 \times (5)^{3/2}} = 149.1$ m ; say 150 m

Effective water way = Total water way (ignoring end contractions)

Hence, provide 15 bays, each of 10 m clear span.

$\therefore$ **No. of gates required for barrage = 15 Ans.**

(2) **Water way for Canal Head Regulator.**

The discharge through canal regulator = 500 m^3/s

Coefficient of discharge for regulator opening = 1.5

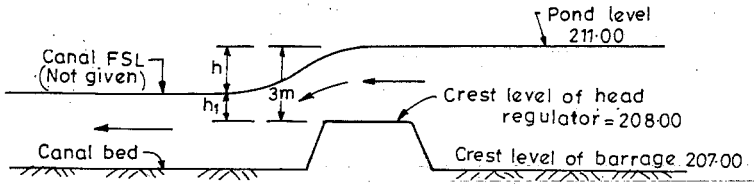

Fig. 31.9.

Head over the crest of regulator at pond Level = 211.00 – 208.00 = 3.00 m.

$$Q = C_1 \cdot L_1 \cdot H_1^{3/2}$$

$$\therefore \quad 500 = 1.5 \times L_1 \times (3)^{3/2}$$

or

$$L_1 = \frac{500}{1.5 \times (3)^{3/2}} = 64 \text{ m.}$$

Provide 7 bays each of 10 m clear span, thereby providing 70 m water way.

No. of gates required for canal head regulator each of 10 m span = **7 Ans.**

(3) **Design of Stilling Basin.**

$U/S\ HFL = 212.00$

$D/S\ HFL = 210.00$

$U/S\ TEL = 212.00$ (ignoring vel. of approach)

$D/S\ TEL = 210.00$ (– do –)

$H_L = 212.00 - 210.00 = \mathbf{2\ m}$

q = Discharge intensity through barrage bays

$= 3500/150 = \mathbf{23.34}\ m^3/s/m.$

E_{f_2} for $H_L = 2$ m & $q = 23.34$, from Blench curves – (Plate 10.1) is found to be = **7.2** m

∴ Level at which jump will form

$$= D/S\ TEL - E_{f_2} = 210.00 - 7.2 = \mathbf{202.8\ m}$$

y_2 corresponding to E_{f_2} of 7.2 m from plate 10.2 = 6.6 m.

Length of cistern $= 5y_2 = 5 \times 6.6 = 33$ m.

Hence, provide 33 m long stilling basin at level say *202.6 m* (slightly lower than the calculated value of 202.8 m) as shown in Fig. 31.10 below.

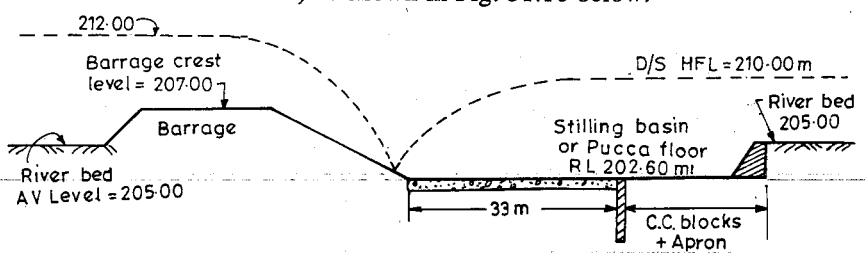

Fig. 31.10.

Alternatively, if Blench curves are not supplied in the Examination Hall, then we can compute the value of y_2 as follows :

$$y_c = \sqrt[3]{\frac{q^2}{g}} \qquad \text{(from Eqn. 10.3)}$$

$$= \sqrt[3]{\frac{23.3^2}{9.81}} = \mathbf{3.81\ m.}$$

Now let $Y = y_2/y_c$ (Eqn. 10.5)

and $Z = H_L/y_c$ (Eqn. 10.6)

Compute Value of $Z = \dfrac{2\text{ m}}{3.81\text{ m}} = \mathbf{0.5248}$

Use Eqns. 10.9 & 10.10 to compute the value of Y for known value of Z.

Here $Z < 1$, hence Eqn. (10.9) is applicable

$\therefore$ $Y = 1 + 0.93556\,Z^{0.368}$ (for $Z < 1$)

or $Y = 1 + 0.93556\,(0.5248)^{0.368} = \mathbf{1.738}$

$\therefore$ $Y = y_2/y_c = 1.738$

or $y_2 = 1.738\,y_c = 1.738 \times 3.81\text{ m} = \mathbf{6.6\ m.}$

$5y_2 =$ Length of cistern $= 5 \times 6.6 = \mathbf{33\ m.}$ **Ans.**

Also, E_{f_2} can be calculated by Eq. (10.13) as :

$$\frac{E_{f_2}}{y_c} = Y + \frac{1}{2Y^2} = 1.738 + \frac{1}{2\,.\,(1.738)^2}$$

or $E_{f_2} = 1.904 \times 3.81 = \mathbf{7.25\ m}$

Level at which jump will form

$= D/S\ TEL - E_{f_2}$ $D/S\ TEL = D/S\ HFL$

$= D/S\ HFL - E_{f_2}$ (on neglecting vel. of approach)

$= 210.00 - 7.25 = 202.75$ m

Provide cistern at RL **202.6 m** **Ans.**

Q. 7. (*c*) *During re-cuperation test, the water level in an open well was depressed by pumping up to 2.5 m. The water level was raised by 1 m within 60 minutes, just after stopping the pumping. Determine the yield from the well of 2m diameter when the depression head in 3 m.* (10)

[**Hint.** Follow exactly similar Solved Example 16.11 and compute :

$$\frac{C'}{A} = \frac{2.3}{3600}\log_{10}\frac{2.5}{1.5} = 1.417 \times 10^{-4}$$

$Q =$ yield from a well of 2m dia with 3m depression head

$$= \left(\frac{C'}{A}\right) \cdot A \cdot s = (1.417 \times 10^{-4}) \times \left(\frac{\pi}{4} \times 2^2\right) \times 3\text{ m}^3/\text{s}$$

$= \mathbf{1.335\ l/s}$ **Ans.**]

Q. 8. (*b*) *Explain the purpose of providing groins. State the factors which affect the selection of type and performance of groin. Mention about the orientation, desirable length and spacing of groins in a groin system.* (10)

[Please refer article 8.5.3 (3)]

Year 2002 (*Engineering Services*)

Q. 2. (*b*) *For a drainage basin of 640 km², isohyetals based on a storm event yield the following data :*

Isohyetal interval (cm)	*14-12*	*12-10*	*10-8*	*8-6*	*6-4*	*4-2*	*2-0*
Inter isohyetal area (km²)	*90*	*140*	*125*	*140*	*85*	*40*	*20*

Estimate the average depth of precipitation over the basin.

[**Hint :** Please follow exactly similar solved example 7.5, and compute]

$$P = \frac{90 \times 13 + 140 \times 11 + 125 \times 9 + 140 \times 7 + 85 \times 5 + 40 \times 3 + 20 \times 1}{90 + 140 + 125 + 140 + 85 + 40 + 20 = 640}$$

$= \mathbf{8.41\ cm}$ **Ans.**]

Q5. (*b*) *Design a regime channel for a discharge of 35 m³/s with silt factor of 0.9 by Lacey's theory, taking side slopes as 1 H : 2 V.*

[**Hint :** Please follow exactly similar solved Question 4.9 ; and compute

$B = 24$ m, $y = 1.83$ m, $S_o = 1$ in 7200 **Ans.**]

Q7. (*a*) *At an energy dissipater structure below a low spillway, the discharge is 19 m²/s, and the energy loss is 1 m at the hydraulic jump forming therein. Determine the depth of flow at both ends of the jump.*

Solution. $q = 19\ \text{m}^2/s$; $H_L = 1$ m ; $y_1 = ?$; $y_2 = ?$

Use equations as :

$$y_1 y_2 (y_2 + y_1) = \frac{2q^2}{g} \qquad \text{...(Eqn. 10.5)}$$

and

$$H_L = \frac{(y_2 - y_1)^3}{4\, y_1 y_2} \qquad \text{...(Eqn. 10.12)}$$

Solve the above equations for two unknowns, y_1 and y_2, as follows :

From Eqn. (10.5), we get

$$y_1 y_2 (y_2 + y_1) = \frac{2 \times 19^2}{9.81} = 73.6 \qquad \text{...(}i\text{)}$$

From Eqn. (10.12), we get

$$1 = \frac{(y_2 - y_1)^3}{4 \cdot y_1 y_2} \qquad \text{...(}ii\text{)}$$

Let $\dfrac{y_2}{y_1} = x$

$\therefore$ $y_2 = y_1 \cdot x$

Substituting in (*i*), we get

$$y_1 \cdot y_1 x (y_1 x + y_1) = 73.6$$

or $$y_1^2 \cdot x \cdot y_1 (x + 1) = 73.6$$

or $$x \cdot y_1^3 (x + 1) = 73.6 \qquad \text{...(}iii\text{)}$$

Also from (*ii*),

$$1 = \frac{(y_1 x - y_1)^3}{4 \cdot y_1 \cdot x \cdot y_1} = \frac{y_1^3 (x-1)^3}{4 \cdot x \cdot y_1^2} = \frac{y_1}{4x} (x-1)^3$$

or $$4x = y_1 (x-1)^3$$

or
$$y_1 = \frac{4x}{(x-1)^3} \qquad ...(iv)$$

Substituting in (*iii*), we get

$$x \cdot \left[\frac{4x}{(x-1)^3}\right]^3 [\,x+1\,] = 73.6$$

Solve by hit and trial, for x, as :

$$x = \mathbf{2.548}$$

$$\therefore \quad y_1 = \frac{4 \times 2.548}{(2.548-1)^3} = \mathbf{2.748\ m}$$

$$y_2 = y_1\, x = 2.748 \times 2.548 = \mathbf{7.0\ m}$$

Hence, the depth at both ends, of jump, y_1 and y_2 will be 2.748 m and 7.0 m, respectively. Ans.

Q8. (*a*) *The daily flows in a river for three consecutive years are given in the table by class interval along with the number of days the flow belonged to this class. What are the 50% & 75% dependable flows for the river ?*

Daily mean discharge m^3/s		*100-90.1*	*90-80.1*	*80-70.1*	*70-60.1*	*60.50.1*
No. of days in each class interval	1981	0	16	27	21	43
	1982	6	19	25	60	51
	1983	10	16	38	67	58

Daily mean discharge m^3/s		*50-40.1*	*40-30.1*	*30-20.1*	*20-10.1*	*10-Negligible*
No. of days in each class interval	1981	59	64	22	59	54
	1982	38	29	48	63	26
	1983	38	70	29	26	13

[**Hint :** Please refer Solved Example 18.3]

Year 2003 (Engineering Services)

Q2. (*b*) *During the passage of a flood the following data was estimated at two sections 500 m apart :*

Sections	*Water surface elevation, m*	*Area of flow section, m^2,*	*Hydraulic mean depth, m*
Upstream, *P*	85.233	91.746	2.835
Downstream, *R*	85.176	84.354	2.917

The eddy loss coefficient for gradual contraction is to be taken as 0.1 and for gradual expansion as 0.35. Estimate the flood discharge passing through the reach. n = 0.022. (15)

[**Hint.** Follow exactly similar solved example 7.23]

Q5. (*b*) *Water discharges at the rate of 298 m³/sec over a spillway 14 m wide into a stiling basin of the same width. The water level behind the spillway is 61 m over datum. The river water level in the downstream is 29.2 m over datum. The river bed is paved flush with the bed level to just adequate length to form the jump basin. Assuming that no energy is dissipated in the flow down the spillway, find the basin invert level required for a hydraulic jump to be formed within the basin* (15)

[**Solution.** Here $H_L = (61.0 - 29.2)$ m = 31.8 m

$$q = \frac{Q}{B} = \frac{298}{14} = 21.29 \text{ m}^3/\text{s/m}$$

This is a case where H_L and q are known, and we have to find the value of y_2, since the invert level of jump basin will be equal to (RL 29.2 – y_2). To determine y_2, proceed as given in article 10.3.1—when Blench curves are not available.

From

$$\text{Determine } y_c = \sqrt[3]{\frac{q^2}{g}} \text{ (i.e. Eq. 10.3)} = \sqrt[3]{\frac{(21.29)^2}{9.81}} = 3.59 \text{ m}$$

Now use Eq. (10.6) as :

$$Z = \frac{H_L}{y_c} = \frac{31.8}{3.59} = 8.86$$

Now use Eq. (10.10) as :

$$Y = 1 + 0.93556 \,.\, Z^{0.24} \text{ for } Z > 1$$

For $Z = 8.86$ (> 1) $\quad Y = 1 + 0.93556 \times (8.86)^{0.24} = 2.58$

Now, use Eq. (10.5) as :

$$\frac{y_2}{y_c} = Y$$

or $$\frac{y_2}{3.59} = 2.58 \qquad \text{or} \quad y_2 = 9.26 \text{ m}$$

The basin invert level = 29.2 – 9.26 = **19.94 m** above datum. **Ans.**]

Q6. (*b*) *A channel is 40 m wide, 2.6 m deep with slope of 1 in 5000. The representative bed material size D_{65} is 0.64 mm, and mean diameter of the bed material is 0.38 mm. Mannings n = 0.021. Neglecting the side friction, find the bed load transported by the channel in tonnes per day by Meyer-Peter equation, considering B/D as large and τ_c = 0.075 d.*

Solution. Meyer-Peter's equation, giving bed load transport in N/m.s is given by Eq. (4.51) as :

$$g_b = 0.417\left[\tau_0\left(\frac{n'}{n}\right)^{3/2} - \tau_c\right]^{3/2}$$

where $\tau_0 = 0.97\ \gamma_w$ R.S. for the channel

where γ_w = unit wt. of water

$= 9.81\ \text{kN/m}^3 = 9.81 \times 10^3\ \text{N/m}^3$

$R = \frac{A}{P} = \frac{B\,.\,y}{B} = y$ for wider channels having B/y large

$= 2.6$ m

$$S = \frac{1}{5000}$$

$$\therefore \quad \tau_0 = 0.97 \times (9.81 \times 1000) \times 2.6 \times \frac{1}{5000} = \mathbf{4.948\ N/m^2}$$

$n' = 1/24\,.\,d^{1/6}$ where d is the representative size of the bed material in m, usually taken as median size (d_{50}) or d_{65}, which is given as equal to 0.64 mm.

$\therefore \quad d = 0.64 \times 10^{-3}$

$$\therefore \quad n' = \frac{1}{24}(0.64 \times 10^{-3})^{1/6} = \mathbf{0.01223}$$

$\boldsymbol{n = 0.021}$ **(given)**

τ_c is usually given as $\tau_c = 0.687\,.\,d_a$ when τ_c is in N/m^2, and d_a is in mm.

Here it is given that $\tau_c = 0.075d$. Eventually, the given eqn. is meant for τ_c, when τ_c is expressed in kg/m^2. For S.I. units, the given eqn. will hence become.

$\tau_c = 0.075\,d_a \times 9.81\ \text{N/m}^2$

$= 0.736\,d_a\ (\text{N/m}^2)$

d_a is mean size of the bed particles, which is given as equal to 0.38 mm.

$$\therefore \quad \tau_c = 0.736 \times 0.38\ \text{N/m}^2 = \mathbf{0.279\ N/m^2}$$

Now, $$g_b = 0.417\left[4.948\left(\frac{0.01223}{0.021}\right)^{3/2} - 0.279\right]^{3/2}$$

$$= 0.417\,[2.199 - 0.279]^{3/2} = 0.417\,(1.92)^{1.5}$$

or $$g_b = 1.109 \text{ N/m.s} = \frac{1.109}{9.81} \text{ kg/s/m}$$

∴ Bed load transported by 40 m wide channel

$$= \frac{1.109}{9.81} \times 40 \text{ kg/sec} = 4.522 \text{ kg/sec}$$

$$= \frac{4.522 \times 24 \times 3600}{1000} \text{ t/day} = \mathbf{390.69 \text{ tonnes/day. Ans}}]$$

Q7. (*a*) *Flood frequency computations for a flashy river at a point 50 km upstream of a bund site indicated the following :*

Return period, T_{year}	*50*	*100*
Peak flood, m^3/sec	*20,600*	*22,150*

Estimate the flood magnitude in the river with a return period of 500 year through use of Gumbel's method (15)

[**Hint.** Follow exactly similar Solved Example 7.58.]

Q8. (*a*) *Given below are the monthly rainfall, P, and the corresponding run off values, R, for a period of 10 months for a catchment. Develops a correlation between R and P :*

Month No.	*P, cm*	*R, cm*
1	*4*	*0.2*
2	*22*	*7.1*
3	*28*	*10.9*
4	*15*	*4.0*
5	*12*	*3.0*
6	*8*	*1.3*
7	*4*	*0.4*
8	*15*	*4.1*
9	*10*	*2.0*
10	*5*	*0.3*

[**Hint.** To obtain a linear relation between rainfall P and runoff Q (here represented by R), we have to plot a curve taking R on X-axis and P on Y-axis, as to plot a curve like the one obtained in Fig. 7.105. The slope of straight line portion is then determined to compute the desired relation between P and R. This relation is worked out as

$R = 0.47P$ in Fig. 3.11. **Ans.**

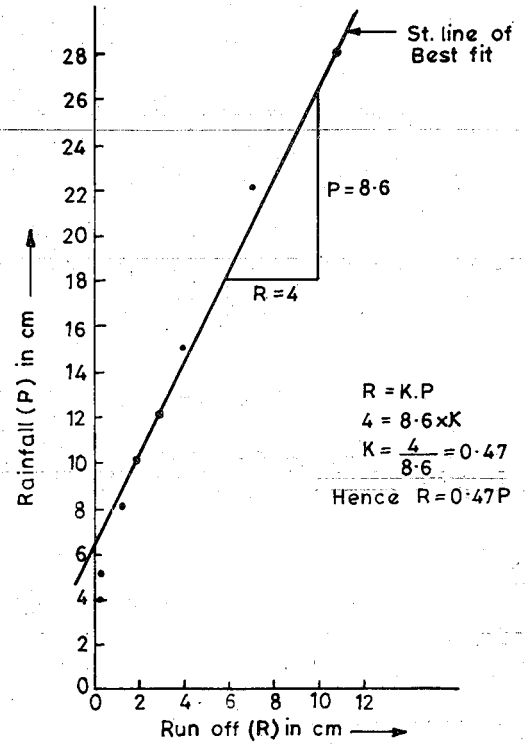

Fig. 31.11

Year 2004 (*Engineering Services*)

Q3. (*a*) *A watershed has five non-recording rain gauges, installed in its area. The amount of rainfall recorded for one of the years is given below :*

Rain gauge station	I	II	III	IV	V
Annual rainfall in cm	100	120	190	95	125

Find the required optimum number of non-recording and recording rain gauges for this watershed. Assume an error of 10% in the estimation of mean rainfall. (15)

[**Hint.** Pl. follow exactly similar solved example 7.3]

Q6. (*a*) *Explain briefly the functions of the following for a typical canal headworks :*

(*i*) *divide wall,*

(*ii*) *guide bank,*

(*iii*) *sediment ejector.*

Explain briefly how does seepage endanger the safety of a hydraulic structure on permeable foundation. (15)

Q7. (*a*) *In a catchment, the average rainfall for a storm at two successive 6 hr intervals was 3.0 cm and 6.0 cm respectively. The abstraction losses ϕ-index, were estimated to be 0.20 cm/hour. For the same catchment, the calculated data for a 6 hour unit hydrograph is available and is given below. Find the direct run-off hydrograph due to the storm.*

Time (hours)	0	6	12	18	24	30	36	42	48	54	60
Unit hydrograph ordinate (m^3/s)	0	10	25	40	100	150	100	75	25	15	0

(20)

[**Hint.** Pl. follow similar solved question 7.35. The excess rainfall here, in two successive 6 hr rainfall periods, will be [3.0 – 6 × 0.2 = 1.8 cm] and [6 – 6 × 0.2 = 4.8 cm], respectively. The ordinates of D.R.H. for first rain will be calculated by multiplying the given ordinates of U.H. by 1.8. While the ordinates of DRH caused by 2nd rain will be obtained by multiplying the given ordinates of U.H. by 4.8. The ordinates due to 2nd rain will start after 6 hr from those of the first, as usual. The summation of the two set of ordinates will give ordinates of total S.R. hydrograph.]

Q8. (*a*) *Define, duty, delta and base period of a crop and express the relationship connecting them. Explain briefly with the help of a diagram the concept of 'frequency of irrigation'.* (10)

(*b*) *A loam soil has a field capacity of 25 per cent and wilting coefficient of 10 per cent. The dry unit weight of soil is 1.5 g/cc. If the root zone depth is 60 cm, determine the storage capacity of the soil. Irrigation water is applied when moisture content falls to 15 per cent. If the water application efficiency is 75 per cent, determine the water depth required to be applied in the field.* (10)

[**Solution.** Pl. follow exactly similar solved question 2.15 and compute as :

Max. storage capacity in root zone of depth, d

$$= \frac{\gamma_d \cdot d}{\gamma_w}\left[\frac{\text{Field capacity mc}}{100} - \frac{\text{Wilting pt. mc}}{100}\right]$$

$$= \frac{\rho_d \cdot d}{\rho_w}\left[\frac{\text{F.C.}}{100} - \frac{\text{W.P.}}{100}\right]$$

$$= \frac{1.5\,\frac{\text{gm}}{\text{cc}} \times 60\text{ cm}}{1\,\frac{\text{gm}}{\text{cc}}}\left[\frac{25}{100} - \frac{10}{100}\right]$$

$$= 90\left[0.25 - 0.10\right]\text{ cm} = \mathbf{13.5\ cm.}\quad \textbf{Ans.}$$

Irrigation water is to be supplied when *mc* falls to 15%. It means that m.c. is allowed to vary between 25% (F.C. mc) and 15%. The deficiency of water depth created during this fall of m.c.

$$= \frac{\rho_d \cdot d}{\rho_w}[0.25 - 0.15]$$

$$= 1.5 \times 60\text{ cm} \times 0.10 = \mathbf{9\ cm.}$$

Hence, 9 cm depth of water is the net irrigation requirement.

Water depth required in the field

$$= \frac{\text{Net irrigation requirement}}{\text{Water application efficiency}}$$

$$= \frac{9\text{ cm}}{0.75} = \mathbf{12\ cm.}\quad \textbf{Ans.}]$$

CIVIL SERVICES COMPETITION (IAS)

This competition is open to all graduates including Engineering graduates. The age limits vary between 21 to 30 years as on 1st August of the year of exam.

In this competition, you would have to, first of all, appear in a screening test, called the **Preliminary Examination.** This exam. will consists of *objective type questions*, having the following marks :

(*i*) *General Studies*	:	150 Marks
(*ii*) *One subject, to be selected from out of 23 optionals, listed below, such as Civil Engineering (for you)*	:	300 Marks
	Total	450 Marks

List of optionals for Prelimes :

(*i*) *Agriculture;* (*ii*) *Animal Husbandry & Veterinary Science;* (*iii*) *Botany;* (*iv*) *Chemistry;* (*v*) *Civil Engineering;* (*vi*) *Commerce;* (*vii*) *Economics;* (*viii*) *Electrical Engg;* (*ix*) *Geography;* (*x*) *Geology;* (*xi*) *Indian History;* (*xii*) *Law;* (*xiii*) *Mathematics;* (*xiv*) *Mechanical Engg;* (*xv*) *Medical Science;* (*xvi*) *Philosophy;* (*xvii*) *Physics;* (*xviii*) *Political Science;* (*xix*) *Psychology;* (*xx*) *Public Administration;* (*xxi*) *Sociology;* (*xxii*) *Statistics; and* (*xxiii*) *Zoology.*

The syllabus (**revised wef year 2000**) prescribed for "General Studies" and "Civil Engineering" objective type papers, is given below :

Preliminary Exam
Paper I — General Studies (Max. Marks 150)

The paper on General Studies will include Questions covering the following field of knowledge

General Science.
Current events of national and international importance.
History of India and Indian National Movement.
Indian and World Geography
Indian Polity and Economy.
General Mental Ability

Questions on General Science will cover General appreciation and understanding of science including matters of everyday observation and experience, as may be expected of a well educated person who has not made a special study of any particular scientific discipline. In current events, knowledge of significant national and international events will be tested. In History of India, emphasis will be on broad general understanding of the subject in its social, economic and political aspects. Questions on the Indian National Movement will relate to the nature and character of the nineteenth century resurgence, growth of nationalism and attainment of Independence. In Geography, emphasis will be on Geography of India. Questions on the Geography of India will relate to physical, social and economic Geography of the country, including the main features of Indian agricultural and natural resources. Questions on Indian Polity and Economy will test knowledge of the country's political system and Constitution of India, Panchayati Raj, Social Systems and economic developments in India. On general mental ability, the candidates will be tested on reasoning and analytical abilities.

Paper II — Civil Engineering (Max. Marks 300)
PART-A

1. **Engineering Mechanics.** Units and dimensions, SI Units, Vectors, Concept of

Force, Concept of particle and rigid body. Concurrent, nonconcurrent and parallel forces in a plane, moment of force and Varignon's theorem, free body diagram, conditions of equilibrium, Principle of virtual work, equivalent force system.

First and Second Moment of area, Mass moment of Inertia.

Static Friction Inclined plane and bearings.

Kinematics and Kinetics : Kinematics in cartesian and polar co-ordinates, motion under uniform and non-uniform acceleration, motion under gravity. Kinetics of particle : Momentum and Energy principles, D Alembert's Principle, Collision of elastic bodies, rotation of rigid bodies, simple harmonic motion.

2. **Strength of Materials.** Simple Stress and Strain, Elastic constants, axially loaded compression members, Shear force and bending moment, theory of simple bending, Shear Stress distribution across cross-sections, Beams of uniform strength, Leaf spring, Strain Energy in direct stress, bending and shear.

Deflection of beams. Macaulay's method, Mohr's moment area method, Conjugate beam method, unit load method. Torsion of Shafts, Transmission of power, closecoiled helical springs, Elastic stability of columns : Euler's, Rankine's and Secant formulae. Principal Stresses and Strains in two dimension, Mohr's Circle. Theories of Elastic Failure, Thin and Thick cylinders : Stresses due to internal and external pressures-Lame's equation.

3. **Structural Analysis.** Analysis of pin jointed plane trusses, deflection in trusses. Three hinged and two hinged arches, rib shortening, temperature effects, influence lines in arches. Analysis of propped cantilevers, fixed beams, continuous beams and rigid frames. Slope deflection, moment distribution, Kani's method and Matrix method : Force and Displacement methods. Rolling loads and influence lines for determinate beams and pin jointed trusses.

PART-B

4. **Geotechnical Engineering.** Types of soil, field identification and classification, phase relationships, consistency limits, particle size distribution, classification of soil, structure and clay mineralogy.

Capillary water and structural water, effective stress and pore water pressure, Darcy's Law, factors affecting permeability, determination of permeability, permeability of stratified soil deposits.

Seepage pressure, quick sand condition, compressibility and consolidation, Terzaghi's theory of one dimensional consolidation, consolidation test. Compaction of soil, optimum moisture content, proctor Density.

Subsurface exploration, methods of boring, sampling, types of sampler, filed tests.

Shear strength of soils, Mohr-Coulomb failure theory, shear tests, Earth pressure at rest, active and passive pressures, Rankine's theory, Coulomb's wedge theory, earth pressure on retaining wall.

Bearing capacity, Terzaghi and other important theories, net and gross bearing pressure, Immediate and consolidation settlement.

Load carrying capacity of pile groups.

Stability of slope-Conventional method of slices, stability numbers.

5. **Transportation Engineering.** Highway alignment, choice of layout and capacity of highways, location survey, geometric design of highways-various elements, curves, grade separation and segregation of traffic, Intersection design, highway materials and testing subgrade and pavement components, types of pavements, road drainage, elements of airport engineering.

Railway engineering-elements of permanent track-rails, sleepers, ballast and rail fastenings, tractive resistance, elements of geometric design-gradients and grade compensation on curves, cant transition curves and vertical curves, stresses in railway tracks, points and crossings, signalling and interlocking, maintenance of railway track. Culverts and small bridges.

PART-C

6. **Fluid Mechanics.** Fluid properties, fluid statics, forces on plane and curved surfaces, stability of floating and submerged bodies.

Kinematics. Velocity, streamlines, continuity equation, accelerations irrotational and rotational flow, velocity potential and stream functions, flownet, separation.

Dynamics. Euler's equation along streamline, control volume equation, continuity, momentum, energy and moment of momentum equation from control volume equation, applications to pipe flow, moving vanes, moment of momentum, Dimensional analysis. Boundary layer on a flat plate, drag and lift on bodies. Laminar and Turbulent Flows. Laminar and turbulent flow through pipes, friction factor variation, pipe networks, water hammer, and surge tanks.

Open Channel Flow. Energy and momentum correction factors, uniform and non-uniform flows, specific energy and specific force, critical depth, Friction factors and roughness coefficients, flow in transitions, free overfall, weirs, hydraulic jump, surges, gradually varied flow equations, surface profiles, moving hydraulic jump.

PART-D

7. **Environmental Engineering**

Water supply. Estimation of surface and subsurface water resources, predicting demand for water, impurities of water and their significance, physical, chemical and bacteriological analysis, water borne diseases, standards for potable water.

Intake of water. pumping and gravity schemes, water treatment : principles of coagulation, floculation and sedimentation; slow-, rapid-, pressure-, filters; chlorination, softening, removal of taste, odour and salinity.

Water storage and distribution : storage and balancing reservoir types, location and capacity. Distribution systems : layout, hydraulics of pipe lines, pipe fittings, valves including check and pressure reducing valves, meters, analysis of distribution systems, leak detection, maintenance of distribution systems, pumping stations and their operations.

Sewerage systems. Domestic and industrial wastes, storm sewage-separate and combined systems, flow through sewers, design of sewers, sewer appurtenances, manholes, inlets, juctions, siphon. Plumbing in Public buildings.

Sewage characterisation. BOD, COD, solids, dissolved oxygen, nitrogen and TOC. Standards of disposal in normal water course and on land.

Sewage treatment. Working principles, units, chambers, sedimentation tank, trickling filters, oxidation ponds, activated sludge process, septic tank, disposal of sludge, recycling of waste water.

9. **Construction Management.** Elements and principles of Activity on Arrow (AOA) and Activity on Node (AON) networks and work breakdown structure. Interfaces. Ladder networks. Activity time. Time computations and floats. ATC and PTC trade-off. Work study and sampling. Scheduling principles-material schedules. ABC and EOQ analysis of inventory. Budgeting with barcharts. Working capital, PERT, probability of completion.

Elements of Engineering Economics, methods of appraisal, present worth, annual-cost, benefit-cost, incremental analysis. Economy of scale and size. Choosing between alternatives including levels of investments, Project profitability.

This preliminary examination is meant to serve as a screeining test only. The marks

obtained in the preliminary Exam by the candidates who are declared qualified for admission to Main Examination will not be counted for determining their final order of merit. The number of candidates to be admitted to the Main Examination will be about twelve to thirteen times the total approximate number of vacancies to be filled in various Services & Posts. Only those candidates who are declared by the commission to have qualified in the Preliminary Examination in a year will be eligible for admission to the Main Examination of that year, provided they are otherwise eligible for admission to the Main Examination.

Main Examination

The main examination consists of one qualifying paper in *English* and another qualifying paper in *one of the Indian Languages* to be selected from the languages included in the eighth schedule of constitution, such as Hindi, Tamil, Bangali, Sanskrit or any other regional language. These two papers, each carrying 300 marks, will be of matriculation standard, and of qualifying in nature *i.e.* the *marks obtained in these papers will not be counted in your final ranking in the competition.*

Besides the above two qualifying papers (Paper I and II), there will be 7 other papers (Paper III to IX), full marks of which will count in your final ranking at this competition. These 7 papers will consists of ;

Compulsory Subjects

Paper III	Essay	200 marks
Paper IV	General Studies I	300 marks
Paper V	General Studies II	300 marks
Paper VI	Paper I of chosen Optional I	300 marks
Paper VII	Paper II of chosen Optional I	300 marks
Paper VIII	Paper I of chosen Optional II	300 marks
Paper IX	Paper II of chosen Optional II	300 marks
		2000 marks
	Interview and Personality Test	300 marks
	Grand Total	**2300 marks**

Note : *On the basis of merit list drawn out of 2000 marks of the Main Examination, cendidates will be summoned for Interview. The no. of condidates called for Interview will be about Twice the number of vacancies to be filled. The final select list will be drawn on the basis of merit for total marks obtained out of 2300 marks.*

Obviously, all the civil engineering students are expected to offer *'Civil Engineering'* subject as one of their two chosen Optionals. Hence, the subject will cover 600 marks out of the grand total of 2300 marks at this competition. You may offer 2nd optional as either "Geology" or "Mathematics" or any other subject of your interest.

The syllabuses (**revised wef year 2000**) for both the papers on *'General Studies'* as well as on the subject of *'Civil Engineering'* are given below for your ready reference.

A perusal of the syllabus of 'Civil Engineering' subject further reflects that **Paper I** will cover four subjects like *Theory of Structures, Structural Designs, Fluid Mechanics* and *Soil Mechanics.* **Paper II** will cover the subjects of *Construction Technology, Survey & Transportation, Hydrology, Water Resources, Irrigation,* and *Environmental Engineering.* Both the papers will thus cover virtually the entire field of civil engineering.

ESSAY

Candidates will be required to write an essay on a specific topic. The choice of subjects will be given. They will be expected to keep closely to the subject of the essay to arrange their ideas in orderly fashion, and to write concisely. Credit will be given for effective and exact expression.

GENERAL STUDIES (MAIN EXAM.)

Note. *The nature and standard of questions in the General studies papers will be such that a well-educated person will be able to answer them without any specialized study. The questions will be such as to test a candidate's general awareness of a variety of subjects, which will have relevance for a career in Civil Services.*

Syllabus for General Studies Paper I (Main Exam)

(*a*) *History of Modern India and Indian Culture.* The History of Modern India will cover history of the Country from about the middle of nineteenth century and would also include questions on important personalities who shaped the Freedom Movement and Social reforms. The part relating to Indian Culture will cover all aspects of Indian Culture from the ancient to modern times.

(*b*) *Geography of India.* In this part, questions will be on the physical, economic and social geography of India.

(*c*) *Indian Polity.* This part will include questions on the Constitution of India, Political system and related matters.

(*d*) *Current National issues and topics of social relevance.* This part is intended to test the Candidate's awareness of current national issues and topics of social relevance in the present-day India, such as the following.

Demography & Human Resource & related issues. Behavioural & Social issues & Social Welfare problems, such as child labour, gender equality, adult literacy, rehabilitation of the handicapped and other deprived segments of the society, drug abuse, public health, etc.

Law enforcement issues, human rights, corruption in public life, communal harmony etc. Internal Security and related issues.

Environmental issues, ecological preservation, conservation of natural resources and national heritage.

The role of national institutions, their relevance and need for change.

Syllabus for General Studies Paper II (Main Exam)

(*a*) *India and the World.* This part is intended to test candidate's awareness of India's relationship with the world in various spheres, such as the following :-

(*i*) Foreign Affairs (*ii*) External Security and related matters

(*iii*) Nuclear Policy (*iv*) Indians abroad

(*b*) *Indian Economy.* In this part, questions will be on the planning and economic development in India, economic & trade issues, Foreign Trade, the role and functions of I.M.F., World Bank, W.T.O. etc.

(*c*) *International Affairs & Institutions.* This part will include questions on important events in world affairs and on international institutions.

(*d*) *Developments in the field of science & technology, Communications and space.* In

this part, questions will test the candidate's awareness of the developments in the field of science & technology, communications and space and also basic ideas of computers.

(*e*) *Statistical analysis, graphs and diagrams.* This part will include exercises to test the candidate's ability to draw common sense conclusions from information presented in statistical, graphical or diagrammatical form and to point out deficiencies, limitations or inconsistencies therein.

OPTIONAL SUBJECTS FOR MAIN EXAMINATION

Total number of questions in the question papers of optional subjects will be eight. All questions will carry equal marks. Each paper will be divided into two parts, viz. Part A and Part B, each part containing four questions. Out of eight questions, five questions are to be attempted. One question in each part will be compulsory. Candidates will be required to answer three more questions out of the remaining six questions, taking at least one question from each part. In this way, at least two questions will be attempted from each Part *i.e.* one compulsory question plus one more.

You have to choose 2 optionals for the Main Exam, out of the following subjects :

Optionals for Main Exam (2 optionals to be chosen from 50 subjects)

(*i*) Agriculture
(*ii*) Animal Husbandry and Veterinary Science
(*iii*) Anthropology
(*iv*) Botony
(*v*) Chemistry
(*vi*) Civil Engineering
(*vii*) Commerce and Accountancy
(*viii*) Economics
(*ix*) Electrical Engineering
(*x*) Geography
(*xi*) Geology
(*xii*) History
(*xiii*) Low
(*xiv*) Arabic
(*xv*) Bengali
(*xvi*) Chinese
(*xvii*) English
(*xviii*) French
(*xix*) German
(*xx*) Gujarati
(*xxi*) Hindi
(*xxii*) Kannada
(*xxiii*) Kashmiri
(*xxiv*) Konkani
(*xxv*) Malayalam
(*xxvi*) Manipuri
(*xxvii*) Marathi
(*xxviii*) Nepali
(*xxix*) Oriya
(*xxx*) Pali
(*xxxi*) Persian
(*xxxii*) Punjabi
(*xxxiii*) Russian
(*xxxiv*) Sanskrit
(*xxxv*) Sindhi
(*xxxvi*) Tamil
(*xxxvii*) Telgu
(*xxxviii*) Urdu
(*xxxix*) Management
(*xxxx*) Mathematics
(*xxxxi*) Mechanical Engineering
(*xxxxii*) Medical Science
(*xxxxiii*) Philosophy
(*xxxxiv*) Physics
(*xxxxv*) Political Science and International Relations
(*xxxxvi*) Psychology
(*xxxxvii*) Public Administration
(*xxxxviii*) Sociology
(*xxxxix*) Statistics
(*xxxxx*) Zoology

Since all the civil Engg. students are expected to offer "Civil Engineering" as one of their two optionals, we are giving below the syllabus of this subject :

Syllabus (*Revised wef year 2000*) for Civil Engineering Paper I (Main Exam.)

Part A — *Engineering Mechanics, Strength of Materials and Structural Analysis*

1. **Engineering Mechanics.** Units and Dimensions, SI Units, Vector, Concept of Force, Concept of particle and rigid body. Concurrent, Non Concurrent and parallel forces in a plane, moment of force and Varignon's theorem, free body diagram, conditions of equilibrium, Principle of virtual work, equivalent force system.

First and Second Moment of area, Mass moment of Inertia.

Static Friction, Inclined Plane and bearings. Kinematics and Kinetics :

Kinematics in Cartesian and Polar Co-ordinates, motion under uniform and non-uniform acceleration, motion under gravity. Kinetics of particle : Momentum and Energy principles, D' Alembert's Principle, Collision of elastic bodies, rotation of rigid bodies, simple harmonic motion, Flywheel.

2. **Strength of Materials.** Simple Stress and Strain, Elastic constants, axially loaded compression members, Shear force and bending moment, theory of simple bending, Shear Stress distribution across cross-sections, Beams of uniform strength, Leaf spring. Strain Energy in direct stress, bending & shear.

Deflection of beams : Meaculay's method, Mohr's Moment area method, Conjugate beam method, unit load method. Torsion of Shafts, Transmission of power, close coiled helical springs, Elastic stability of columns, Euler's, Rankine's and Secant formulae. Principal Stresses and Strains in two dimensions. Mohr's Circle, Theories of Elastic Failure, Thin and Thick cylinder : Stresses due to internal and external pressure - Lame's equations.

3. **Structural Analysis.** Castiglianio's theorems I and II, unit load method of consistent deformation applied to beams and pin jointed trusses. Slope deflection, moment distribution, Kani's method of analysis and Column Analogy method applied to indeterminate beams and rigid frames.

Rolling loads and Influence lines : Influence lines for Shear Force and Bending moment at a section of beam. Criteria for maximum shear force and bending Moment in beams traversed by a system of moving loads. Influence lines for simply supported plane pin jointed trusses.

Arches : Three hinged, two hinged and fixed arches, rib shortening and temperature effects, influence lines in arches.

Matrix methods of analysis : Force method and displacement method of analysis of indeterminate beams and rigid frames.

Plastic Analysis of beams and frames : Theory of plastic bending, plastic analysis, statical method, Mechanism method.

Unsymmetrical bending : Moment of inertia, product of inertia, position of Neutral Axis and Principle axes, calculation of bending stresses.

Part B — *Design of structures. Steel, Concrete and Masonry Structures*

1. **Structural Steel Design.** Structural Steel : Factors of safety and load factors. Rivetted, bolted and welded joints and connections. Design of tension and compression members, beams of built up section, rivetted and welded plate girders, gantry girders, stancheons with battens and lacings, slab and gussetted column bases. Design of highway and railway bridges : Through and deck type plate girder. Warren girder, Pratt truss.

2. **Design of Concrete and Masonry Structures.** Concept of mix design. Reinforced Concrete : Working Stress and Limit State method of design-Recommendations

of I.S. codes. Design of one way and two way slabs, stair-case slabs, simple and continuous beams of rectangular, T and L sections. Compression members under direct load with or without eccentricity, Isolated and combined footings.

Cantilever and Counterfort type retaining walls.

Water tanks : Design requirements for Rectangular and circular tanks resting on ground.

Prestressed concrete : Methods and systems of prestressing, anchorages, Analysis and design of sections for flexure based on working stress, loss of prestress.

Design of brick masonry as per I.S. Codes. Design of masonry retaining walls.

Part C. *Fluid Mechanics, Open Channel Flow and Hydraulic Machines*

1. **Fluid Mechanics.** Fluid properties and their role in fluid motion, fluid statics including forces acting on plane and curve surfaces. Kinematics and Dynamics of Fluid flow : Velocity and accelerations, streamlines, equation of continuity, irrotational and rotational flow, velocity potential and stream functions, flownet, methods of drawing flownet, sources and sinks, flow separation, free and forced vertices.

Control volume equation, continuity, momentum, energy and moment of momentum equations from control volume equation, Navier-Stokes equation, Euler's equation of motion, application to fluid flow problems, pipe flow, plane, curved, stationary and moving vanes, sluice gates, weirs, orifice meters and Venturi meters.

Dimensional Analysis and Similitude. Buckingham's Pi- theorem, dimensionless parameters, similitude theory, model laws, undistorted and distorted models.

Laminar Flow. Laminar flow between parallel, stationary and moving plates, flow through tube.

Boundary layer. Laminar and turbulent boundary layer on a flat plate, laminar sublayer, smooth and rough boundaries, drag and lift.

Turbulent flow through pipes : Characteristics of turbulent flow, velocity distribution and variation of pipe friction factor, hydraulic grade line and total energy line, siphons, expansion and contractions in pipes, pipe networks, water hammer in pipes and surge tanks.

2. **Open Channel Flow.** Uniform and non-uniform flows, momentum and energy correction factors, specific energy and specific force, critical depth, resistance equations and variation of roughness coefficient, rapidly varied flow, flow in contractions, flow at sudden drop, hydraulic jump and its applications, surges and waves, gradually varied flow, classification of surface profiles, control section, step method of integration of varied flow equation, moving surges and hydraulic bore.

3. **Hydraulic Machines and Hydropower.** Centrifugal pumps-Types, characteristics, Net Positive Suction Height (NPSH), specific speed. Pumps in parallel.

Reciprocating pumps, Air vessels, Hydraulic ram, efficiency parameters, Rotary and positive displacement pumps, diaphragm and jet pumps.

Hydraulic turbines, types classification, Choice of turbines, performance parameters, controls, characteristics, specific speed.

Principles of hydropower development. Type, layouts and Component works. Surge tanks, types and choice. Flow duration curves and dependable flow. Storage and pondage. Pumped storage plants. Special features of mini micro-hydel plants.

Part D — *Geo-technical Engineering*

Types of soil, phase relationships, consistency limits, particles size distribution, classifications of soil, structure and clay mineralogy.

Capillary water and structural water, effective stress and pore water pressure, Darcy's Law, factors affecting permeability, determination of permeability, permeability of stratified soil deposits.

Seepage pressure, quick sand condition, compressibility and consolidation, Terzaghi's theory of one dimensional consolidation, consolidation test.

Compaction of soil, field control of compaction. Total stress and effective stress parameters, pore pressure coefficients.

Shear strength of soils, Mohr Coulomb failure theory, Shear tests.

Earth pressure at rest, active and passive pressures, Rankine's theory, Coulomb's wedge theory, earth pressure on retaining wall, sheet pile walls, Braced excavation.

Bearing capacity, Terzaghi and other important theories, net and gross bearing pressure.

Immediate and consolidation settlement. Stability of slope, Total Stress and Effective Stress methods, Conventional methods of slices, stability number.

Subsurface exploration, methods of boring, sampling, penetration tests, pressure meter tests.

Essential features of foundation, types of foundation, design criteria, choice of type of foundation, stress distribution in soils, Boussinessq's theory, Newmarks's chart, pressure bulb, contact pressure, applicability of different bearing capacity theories, evaluation of bearing capacity from field tests, allowable bearing capacity, Settlement analysis, allowable settlement.

Proportioning of footing, isolated and combined footings, rafts, buoyancy rafts, Pile foundation, types of piles, pile capacity, static and dynamic analysis, design of pile groups, pile load test, settlement of piles, lateral capacity. Foundation for Bridges. Ground improvement techniques-preloading, sand drains, stone column, grouting, soil stabilisation.

Syllabus (*Revised wef year 2000*) for Civil Engineering Paper II (Main Exam.)

Part A — *Construction Technology, Equipment, Planning and Management*

1. **Construction Technology.** Engineering Materials : Physical properties of construction materials : Stones, Bricks and Tiles ; Lime, cement and Surkhi Mortars; Lime Concrete and Cement Concrete, Properties of freshly mixed and hardened concrete, Flooring Tiles, use of ferro-cement, fibre-reinforced and polymer concrete, high strength concrete and light weight concrete. Timber : Properties and uses; defects in timber; seasoning and preservation of timber. Plastics, rubber and damp-proofing materials, termite proofing, Materials, for Low cost housing.

Construction. Building components and their functions; Brick masonry : Bonds, jointing. Stone masonry. Design of Brick masonry walls as per I.S. codes, factors of safety, serviceability and strength requirements; plastering, pointing. Types of Floors & Roofs. Ventilators, Repairs in buildings.

Functional planning of buildings : Building orientation, circulation, grouping of areas, privacy concept and design of energy efficient building; provisions of National Building Code. Building estimates and specifications; Cost of works; valuation.

2. **Construction Equipment.** Standard and special types of equipment, Preventive maintenance and repair, factors affecting the selection of equipment, economical life, time and motion study, capital and maintenance cost.

Concreting equipments. Weigh batcher, mixer, vibration, batching plant, Concrete pump.

Earth-work equipment. Power shovel hoe, bulldozer, dumper, trailers, and tractors, rollers, sheep foot roller.

3. **Construction Planning and Management.** Construction activity; schedules, job layout, bar charts, organization of contracting firms, project control and supervision. Cost reduction measures.

Net work analysis. CPM and PERT analysis, Float Times, cashing of activities, contraction of network for cost optimization, up dating, cost analysis and resource allocation.

Elements of Engineering Economics, methods of appraisal, present worth, annual cost, benefit-cost, incremental analysis. Economy of scale and size. Choosing between alternatives including levels of investments, Project profitability.

Part B — *Survey and Transportation Engineering*

Survey. Common methods of distance and angle measurements, plane table survey, levelling traverse survey, triangulation survey, corrections, and adjustments, contouring, topographical map. Surveying instruments for above purposes. Techeometry. Circular and transition curves. Principles of photogrammetry.

Railways. Permanent way, sleepers, rail fastenings, ballast, points and crossings, design of turn outs, stations and yards, turntables, signals, and interlocking, level-crossing. Construction and maintenance of permanent ways : Super elevation, creep of rail, ruling gradient, track resistance, tractive effort, relaying of track.

Highway Engineering. Principles of highway planning, Highway alignments. Geometrical design : Cross section, camber, Super elevation, horizontal and vertical curves. Classification of roads : low cost roads, flexible pavements, rigid pavements. Design of pavements and their construction, evaluation of pavement failure and strengthening.

Drainage of roads : Surface and sub-surface drainage.

Traffic Engineering : Forecasting techniques, origin and destination survey, highway capacity. Channelised and unchannelised intersections, rotary design elements, markings, sign, signals, street lighting; Traffic surveys. Principle of highway financing.

Part C — *Hydrology and Water Resources Engineering*

1. **Hydrology.** Hydrological cycle, precipitation, evaporation, transpiration, depression storage, infiltration, overland flow, hydrograph, flood frequency analysis, flood estimation, flood routing through a reservoir, channel flow routing-Muskingam method.

Ground water flow. Specific yield, storage coefficient, coefficient of permeability, confined and unconfined aquifers, aquifers, aquitards, radial flow into a well under confined and unconfined conditions, tube wells, pumping and recuperation tests, ground water potential.

2. **Water Resources Engineering.** Ground and surface water resource, single and multipurpose projects, storage capacity of reservoirs, reservoir losses, reservoir sedimentation, economics of water resources projects.

3. **Irrigation Engineering.** Water requirements of crops : consumptive use, quality of water for irrigation, duty and delta, irrigation methods and their efficiencies.

Canals : Distribution systems for canal irrigation, canal capacity, canal losses, alignment of main and distributary canals, most efficient section, lined canals, their design, regime theory, critical shear stress, bed load, local and suspended load transport, cost analysis of lined and unlined canals, drainage behind lining.

Water logging : causes and control, drainage system design, salinity.

Canal structures : Design of cross regulators, head regulators, canal falls, aqueducts, metering flumes and canal outlets.

Diversion head work : Principles and design of weirs of permeable and impermeable foundation, Khosla's theory, energy dissipation, stilling basin, sediment excluders.

Storage works : Types of dams, design, principles of rigid gravity and earth dams, stability analysis, foundation treatment, joints and galleries, control of seepage.

Spillways : Spillway types, crest gates, energy dissipation.

River training : Objectives of river training, methods of river training.

Part D — *Environmental Engineering*

1. **Water Supply.** Estimation of surface and subsurface water resources, predicting demand for waters, impurities of water and their significance, physical, chemical and bacteriological analysis, waterborne diseases, standards for potable water.

Intake of water. pumping and gravity schemes. Water treatment : principles of coagulation, flocculation and sedimentation; slow-, rapid-, pressure-, filters; chlorination, softening, removal of taste, odour and salinity.

Water storage and distribution. storage and balancing reservoirs : Types, location and capacity. Distribution system : layout, hydraulics of pipe lines, pipe fittings, valves including check and pressure reducing valves, meters, analysis of distribution systems, leak detection, maintenance of distribution systems, pumping stations and their operations.

2. **Sewerage systems.** Domestic and industrial wastes, storm sewage-separate and combined systems, flow through sewers, design of sewers, sewer appurtenances, manholes, inlets, junctions, siphon. Plumbing in public buildings.

Sewage characterisation. BOD, COD, solids, dissolved oxygen, nitrogen and TOC. Standards of disposal in normal water course and on land.

Sewage treatment. Working principles, units, chambers, sedimentation tanks, trickling filters, oxidation ponds, activated sludge process, septic tank, disposal of sludge, recycling of waste water.

Solid waste. Collection and disposal in rural and urban contexts, management of long-term ill-effects.

3. **Environmental pollution.** Sustainable development. Radioactive wastes and disposal. Environmental impact assessment for thermal power plants, mines, river valley projects. Air pollution, Pollution control acts.

Past Years Questions of Civil Services Competition

The various questions that have been set during the last several years since 1979 in this exam. from the subject of Irrigation and Hydrology, are reproduced here. *You will be glad to notice that many of the numerical questions were directly set from this book.*

Year 1979 (*Civil Services*)

Paper I

Q. 5. *Describe with sketches where necessary how a weir based on Khosla's theory is designed. Discuss the limitations of this theory.*

Q. 8. *(b) A project involves construction of a canal of most economical section with side slopes 2 horizontal to 1 vertical has to discharge 15000 litres per sec with a slope of 10 cm in a kilometre. When unlined C = 45 and when lined C = 60. If the cost of per cub-metre of excavation is one third the cost per sq. metre of lining, which would be cheaper and what would be their dimensions ?*

Year 1980 (*Civil Services*)

Paper I

Q. 5. *Describe briefly various types of spillways and method used for dissipating the energy of the discharge from them.*

Paper II

Q. 11. *Wheat is to be grown in field having a field capacity equal to 27% and the permanent wilting point is 13%. Find the storage capacity in 80 cm depth of the soil, if*

the dry unit weight of the soil is 1.5 gm/cc. If the irrigation water is to be supplied when the average soil moisture falls to 18%, find the water depth required to be supplied to field if the field application efficiency is 80%, what is the amount of water needed at the canal outlet if the water lost in the water course and the field channel is 15% of the outlet discharge ?

[**Note.** Please refer Solved Example 2.14. **This question was set from this book.**]

Q. 12. *An intense rain is assumed to be falling at a uniform rate of 5 cm per hr. for a period of 80 min. on a drainage basin of 550 hectares. The average infiltration capacity during the entire rain period has been worked out to be 1.4 cm per hr. Determine the maximum runoff rate, if the peak percentage based on 10 minutes interval from the distribution graph for this basin is 18%.* (60)

[**Note.** Please refer Solved Example 7.44. **This question was set from this book.**]

Q. 13. *(a) Find the maximum safe width for an elementary profile of a gravity dam of 20 m height. Specific gravity of the dam material is 2.25 and the uplift factor is 0.45.* (20)

(b) Enumerate atleast four types of spillways. Describe any two types in detail illustrated with sketches. (20)

(c) A flood of certain magnitude has a return period of 25 years. What is the probability of exceedence ? What is the probability that this flood may occur in the next 12 years ? (20)

[**Note.** Please refer Solved Example 7.52]

Q. 24. *(a) Discuss the following :*

(i) Wilting point ;

(ii) Evapotranspiration and net Irrigation requirements. (15)

(b) Draw the neat cross-sections of canal in

(i) cutting,

(ii) filing, and

(iii) partial cutting. (15)

(c) Draw a neat L-section of a glacis type fall with baffle wall and platform and clearly label various parts indicating slopes and energy line, etc. (15)

Q. 15. *(a) Briefly discuss the types of rivers.* (15)

(b) Explain with sketches the meandering ratio. (15)

(c) Explain with sketches the theory of river meandering. (15)

(d) Enumerate river training works. Discuss one of these with neat sketches. (15)

Year 1981 (*Civil Services*)

Paper I

Q. 6. Give a brief outline of the procedures for the design of the non-overflow section of a low gravity dam indicating clearly the forces (acting on the dam) being considered for the design. (30)

Paper II

Q. 9. Discuss the following terms bringing out their meaning, significance and write the mathematical equation if any :

(i) Duty of crop

(ii) Delta of a crop

(iii) Optimum utilization of irrigation water

(iv) Consumptive use

(v) Net irrigation requirements ; and

(vi) Field capacity.

(60)

Q. 16. A pump is installed on a well to lift water and to irrigate rice crop, sown over a 3 hectares of land. If duty for rice is 864 hectares/cumec on the field and pump efficiency is 48% ; determine the minimum H.P. of the pump if the lowest well water level is 8.0 m below the highest portion of the field. Field channel losses are neglected.

(60)

[**Note.** Please refer Solved Example 3.7. **This question was set from this book.**]

Q. 11. Given Q = 50 cumec ; Silt factor = 1.1, side slope = $\frac{1}{2}$: 1, using Laceys theory , determine :

(i) Velocity ;

(ii) area of cross-section ;

(iii) Hydraulic radius ;

(iv) Perimeter, and

(v) Longitudinal slope.

(40)

[**Note.** Please refer Solved Example 4.6. **This question was set from this book.**]

(*b*) Discuss regime concept of Lacey. (20)

Q. 12. Results to determine the infiltration capacity equation in the exponential form are tabulated below :

Time in hours	*f in cm/hr*	$f-f_c$	$log_{10}(f-f_c)$
0	10.4	9.4	0.973 or + 0.973
0.25	5.6	4.6	0.663 or +0.663
0.50	3.2	2.2	0.342 or + 0.342
0.75	2.1	1.1	0.042 or + 0.042
1.00	1.5	0.5	$\bar{1}$.699 or (–) 0.301
1.25	1.2	0.2	$\bar{1}$.301 or –0.669
1.50	1.1	0.1	$\bar{1}$.00 or –1.00
1.75	1.0	0.0,	
2.00	1.0	0.0,	

Plot $log_{10}(f-f_c)$ against time and determine the infiltration exponential equation. Given that $log_{10} e = 0.4343$.

(60)

[**Note.** Please refer Solved Example 7.13. **This question was set from this book.**]

Year 1982 (*Civil Services*)

Paper I

Q. 6. *The concrete floor of a head regulator is level with the channel bed (except for the short crest hump) and is 13 m long. The floor is provided with cut off walls at its upstream and downstream ends. The depth of upstream cutoff is 1.5 m (below the floor level) and that of the downstream wall is 2.0 m. Using Khosla's theory (see Fig. for definition, sketch and formula), determine the thickness of the floor at its mid-length and also at its junction with the upstream and downstream cutoff walls. The floor thickness may not be less than 30 cm any where. The upstream FSL is 1.5 m above the floor level. If the permissible exit gradient is 0.18, is the floor safe against failure by piping ?*

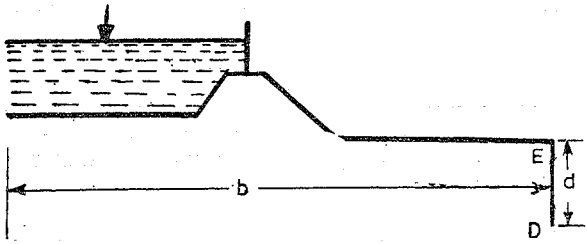

Fig. 31.11. Definition sketch for Khosla's theory for uplift pressure.

$$\phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda - 2}{\lambda}\right)$$

$$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2}$$

$$\alpha = \frac{b}{d}. \qquad (60)$$

[**Note.** Please refer Solved Example 11.4].

Paper II

Q. 9. *Write brief but relevant answers for any three parts :*

(a) Outlining the factors governing runoff, describe their interaction. Also describe briefly the methods for estimating runoff.

(b) Outlining the factors which lead to water-logging and salinity of soils in irrigated areas, discuss the respective remedial measures to reclaim them.

(c) Describe the stages of a river during its course. State the objectives of rive training. Also explain briefly how bunds and spurs function.

(d) Discuss the methods adopted to control seepage through and under-neath earth rock dams. Evaluate their performance.

Q. 10. *(a) Define the duty of a crop. Illustrate the relationship between duty and delta. Also describe how the efficiency of irrigation is determined.*

(b) Describe the procedure for determining the design discharge of an irrigation canal. Discuss why the kor demand of crops should be considered in place of average demand while computing the capacity.

(c) The transplantation of paddy takes 20 days and the total depth of water required by the crop is 25 cm on the field. During the transplantation period, there is about 10 cm of useful rain. Find the duty of irrigation water required for paddy during transplan-

tation period. Also find the duty of water at the head of water course and at the head of the distributary. Assume 25% losses of water in water courses and 15% losses from the distributary head to the water course head.

[**Note.** Please follow Example 3.5. **This question was set from this book by slightly modifying the numerals.**]

Q. 11. *(a) Describe hydrologic cycle and explain its importance.*

(b) Describing the method of determining the yield of a tubewell, discuss the factors contributing to the progressive reduction of its yield.

(c) Explain step by step, the method of construction of unit hydrograph from storm of unit duration stating the basic assumptions.

(d) Discuss general principles governing planned and optimal use of water resources.

Q. 12. *(a) Discuss the various design provisions which are intended to provide additional safety of earth-rock dams against earthquake forces.*

(b) Explain how cracking of embankments leading to piping failure become an important and difficult problem in the present day design of modern earth-rock dams.

(c) Explain how the treatment of foundations including abutments become relevant to the design of hydraulic structures of the most competitive types (namely arch dams and earth-rock dams).

(d) Stating the role of spillways, classify them. Also state the conditions for which each type is appropriate.

Year 1983 (*Civil Services*)

Paper I

Q. 6. (*a*) What is an "economical channel section" ? Work out the conditions for such a channel when the section is trapezoidal having bottom width *b*, depth *d* and side slopes *l/n*.

Part II

Question 9 is compulsory.

Q. 9. Write answers for any three sub-divisions :

(a) Define (i) Delta , (ii) Duty, and (iii) Base period and derive the relationship among them.

Also explain the influence of several factors which affect duty. (20)

(b) Explain the method of estimating maximum flood discharge in a river using unit hydrograph approach. (20)

(c) What is water-logging ? What are the ill-effects of water-logging? How will you reclaim a water-logged area ? (20)

(d) How do you differentiate between a high dam and a low dam ? Explain any two types of spillway in a masonry dam with sketches. (20)

Q. 10. *(a) A reservoir with a live storage capacity of 360 million cubic meters is able to irrigate an ayacut of 40,000 hectares with two fillings each year. If the crop season is 120 days, calculate the duty.* (20)

(b) Explain the isohyetal and the Theiesen's polygon method for finding out the average rainfall of a given basin. (20)

(c) Explain briefly the various factors which affect runoff. Mention the empirical formulae for the estimation of flood flows. (20)

Q. 11. *(a) What do you understand by a regime channel ? Compare the silt theories of Kennedy and Lacey.* (20)

(b) (i) Differentiate between a weir and a barrage. Explain with neat sketches their salient components.

(ii) List the advantages and disadvantages of a Butterss dam. (20)

(c) What are the forces that you will consider while designing a masonry dam in an earthquake region ? How will you account for these forces in the design ? Explain consolidation grouting. (20)

Q. 12. *(a) Discuss the object of the classification of river training work. Draw a practical section of a canal.* (20)

(b) Name the various types of cross-drainage works. Draw neat sketches to show the plan, the cross-section and sectional elevation of these. (20)

(c) What are different ways of aligning a canal ? Where will you use each of them ? Explain with suitable sketches. (20)

Year 1984 (*Civil Services*)

Paper I

Q. 2. *(b) Explain Lacey's regime channel, initial regime and final regime.*

Write the equations as per concept of Lacey for design procedure of a true regime channel carrying a fixed discharge and transporting a given silt grade in an unlimited incoherent alluvium. (20)

Paper II

Q. 9. *Write answers for any three subdivisions :*

(a) What is runoff ? How is it related to the rainfall ? What are the factors that influence the runoff from a catchment ? (20)

(b) Explain how Lacey's theory is an improvement over Kennedy's theory with reference to 'Regime' in channels. What are the steps involved in designing a channel through Lacey's theory ? (20)

(c) How will you reclaim waterlogged land ? Draw neat sketches of (i) carrier drains, and (ii) intercepting drains. (20)

(d) What is Retrogression ? Explain how river weirs are different from river barrages. Explain with neat sketches Syphon well drops and label various parts. (20)

Q. 10. *(a) What are different types of hydroelectric power plant ? Explain the various components of a high head hydroelectric project with a neat layout plan.* (30)

(b) What are the different types of earth dams ? Explain them with neat sketches. (30)

Q. 11. *(a) What are different types of irrigation ? Explain them. Define 'flow duty' and 'quantity duty'.* (15)

(b) Calculate the discharge flowing through a channel of trapezoidal section of bed width 18 m and depth 3 m with $\frac{1}{2}$ to 1 side slopes. The channel has a bed slope of 25 cm for every km and the coefficient of rugosity is 0.03. Given N = 0.9875 and D = 0.8760. (25)

(c) Describe ogee spillway and syphon spillway with neat sketches explaining how

they function. Also explain the general features of energy dissipation devices used in the toe of a spillway. (20)

Q. 12. *(a) List the factors to be considered before selecting a site for constructing a dam. Derive an expression for the elementary profile of a gravity dam in terms of base width and height of the dam.* (30)

[**Note.** Please refer Eqn. 19.23 and Article 19.6.]

(b) Explain the concept of Exit gradient. Also explain the different steps involved in the design of the apron of a weir on permeable foundation using Khosla's theory. (30)

Year 1985 (*Civil Services*)

Paper I

Q. 2. *(a) (i) Explain briefly the method of designing a canal using Lacey theory.*

(ii) Design an irrigation canal in alluvial soil according to Lacey silt theory ; given: Lacey silt factor is 1, canal side slope is 1 horizontal to 2 vertical and full supply discharge is 10 m^3/sec.

[**Note.** Please refer Solved Example 4.9.]

Paper II

Question 9 is compulsory.

Q. 9. *Answer any three sub-divisions :*

(a) Define consumptive use of water by a crop and explain how it is determined under natural conditions. (20)

(b) Describe how a unit hydrograph is prepared from an isolated storm. List and evaluate the basic assumptions. (20)

(c) Explaining waterlogging and salinity of soils in irrigated areas, discuss the causative factors and reclamation process. (20)

(d) Explain why earth-rock dams are increasingly adopted in preference to other types of dams. (20)

Describe the nature of foundation problems in hydraulic structures and their treatment. Illustrate the answer citing case histories. (20)

Q. 10. *(a) The figures below represents streamflow in cumecs from a 1000 acre drainage area. Construct a unit hydrograph for this basin.*

Time (hrs)	2	4	6	8	10	N noon	2	4	6
Flow (cumecs)	5.1	6.5	7.4	10.2	8.8	7.4	5.9	5.4	4.8

[**Note.** Please refer Solved Example 7.39]. (25)

(b) Describing the isohyetal method for finding average rainfall over a drainage basin, bring out its merits relative to station average method. (15)

(c) Discuss the factors that affect the runoff from a catchment. (10)

(d) Describe the design consideration of under-sluices provided in diversion head works. (10)

Q. 11. *(a) Construct a mass diagram for the stream having the following monthly discharges during the 2 year period of record. What is the reservoir size required to provide a constant downstream flow of 3 cumecs?*

Month	*Monthly discharge ha.m*	
	In 1st year	*In 2nd year*
January	*57.4*	*10.2*
February	*65.5*	*30.8*
March	*28.6*	*43.1*
April	*32.8*	*53.1*
May	*36.9*	*38.9*
June	*24.6*	*28.9*
July	*10.2*	*16.4*
August	*2.1*	*12.3*
September	*2.1*	*12.3*
October	*2.1*	*4.1*
November	*4.1*	*18.2*
December	*8.2*	*2.1*

(30)

(b) Which slope (s) (upstream or downstream or both) of a zoned earth dam is (are) analysed for the three independent conditions ? Name the three conditions and state with reason the normal safety factor recommended for each condition. (9)

(c) State the design criteria for filter material. (6)

(d) Classify spillways stating the conditions favourable for each type. Evaluate the merits and demerits of syphon spillway. (15)

Q. 12. *The command area of a canal is planted with ragi. The base period for the crop is 120 days and delta is 50 cm. If 33% of water is lost in conveyance, compute the area which can be irrigated per 100 Ips let into the canal from the reservoir. If the actual amount of evapotranspiration of the crop is 35 cm. compute the overall irrigation efficiency.* (20)

(b) Describe how the longitudinal section of an irrigation canal is fixed. (8)

(c) Discuss Lacey's theory of flow in alluvial canals. (10)

(d) Describe how the yield of a well is determined by a recuperation test. (10)

(e) Describe the method of analysis of gravity dam and demonstrate how the stability is enhanced by foundation treatment (grout curtain and drainage wells.) (12)

Year 1986 (*Civil Services*)

Paper-I

Q. 5. *(a) What is a canal outlet ? Explain the requirements to be satisfied by a canal outlet. Distinguish between a hyper-proportional outlet and a sub-proportional outlet.* (30)

(b) Water flows over a concrete spillway at the rate of 300 cumecs. After flowing over the spillway it passes over a level concrete apron (n = 0.015). The velocity of water at the toe of the spillway is 15 m/sec. and the width of the apron is 50 m. The tail water depth is 3 m. Calculate the length of the apron to contain the jump. Also calculate the energy dissipated. (30)

Q. 6. *(a) Explain the effects of construction of a weir on the regime of a river. Explain with neat sketches the working of a silt excluder.* (30)

(b) Write an explanatory note on filter criteria for earthen dams. Explain the method of checking the stability of earth dam foundation against shear failure. (30)

Paper II

Q. 9. *Attempt any three sub-divisions :*

(a) Derive Seddon's law using both Chezy's equation and Manning's equation for the passage celerity of a flood wave in a wide rectangular open channel. (20)

(b) (i) What is meant by the basic profile of a low gravity dam ? (5)

(ii) Develop the basic profile of a low concrete gravity dam where the specific gravity of the concrete of the dam is 2.5, and uplift is credited with an intensity factor of 0.5 and an area factor also of 0.5. Allowable vertical compressive stress (in the presence of shear stresses — i.e. the vertical stress is not the principal stress necessarily) is 8.5 kg/cm^3. (15)

(c) Bring out features of the hydrological cycle indicating the interrelationships also. All major stages or phases should be clearly emphasized including the anomalies to be cautioned against. (20)

(d) What are the major components of a canal irrigation system ? What are the functions that each performs ? What are the main appurtenances in, across, or along each component ? (20)

Q. 10. *(a) (i) Mention any three objectives for which river training works are constructed. What are the different types of works or constructions adopted in this connection ?* (12)

(ii) Draw a typical plan view of a Bell's guide bund showing the flow direction, the angles and the nominal lengths also; and name the several parts and locations. (8)

(b) (i) Mention six different types of spillways. Also mention four different types of gates used on spillways. (5)

(ii) What are the several features that affect the hydraulic design of an ogee spillways ? (15)

(c) Design a channel by Lacey's theory for 40 cumec capacity. The side slopes may be assumed to be 1 : 1. The average size of the bed material may be taken as 0.8 mm. (20)

Q. 11. *(a)(i) A canal takes off from a reservoir to irrigate the areas as shown in the table. 40% of the water required for irrigation is assumed to be available directly from precipitation. Channel conveyance losses are 15%. Reservoir losses are 10%. What would be the capacity of the reservoir needed ? (The reservoir is to be filled only once a year).* (12)

Crop	*Base period, days*	*Duty of the field, ha/cumec*	*Area under crop, ha*
Wheat	*120*	*1800*	*500*
Sugarcane	*320*	*800*	*600*
Rice	*120*	*900*	*300*
Cotton	*200*	*1400*	*1200*
Bajra	*100*	*1200*	*500*

[Ans. 2890 ham]

(ii) What are the advantages of the sprinkler system of irrigation ? (8)

(b) Derive the Dupuit's equation for the centre-to-centre spacing of a circular tile drains battery below ground for alleviating water-logging. You can take L as the spacing between the drains forming the drainage system, a as the height of the pipe system above the impervious layer, b as the maximum height of the watertable above the impervious layer and q as the rate of discharge per unit area of land surface. (20)

(c) An unregulated stream provides the following volumes through each successive 4-day period over a 40-day duration at a possible reservoir site. What would be the reservoir capacity needed to ensure maintaining the average flow for these 40 days if

Day	0	4	8	12	16	20	24	28	32	36	40
Runoff volume, $m^3 \times 10^6$	0	9.6	5.4	2.3	3.5	2.3	2.2	1.4	6.4	12.4	10.9

the reservoir is full to start with? What is this average flow ? What would be the approximate quantity of water wasted in spillage in this case ? (20)

Q. 12. *(a) Discuss the several relative conditions that can occur below spillways by the comparative locations of the tail water rating curve and the jump height curve. What are the methods adopted for efficient energy dissipation in each case ? By what mechanism is energy dissipated in each method ?* (30)

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

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

Derive from these the Muskingum equation and incidentally determine the coefficients therein. What is the sum of the coefficients ? (10)

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

Hours	0	6	12	18	24	30
I m^3/sec	35	55	92	130	160	140

(10)

[**Note.** Please refer Solved Example 13.3 in "Hydrology and Water Resources Engineering".]

Year 1987 (*Civil Services*)
Paper I

Q. 5. For the following data, make necessary hydraulic calculations for a vertical drop weir using the Bligh's theory :

Maximum flood discharge	= *2800 cumecs*
HFL before construction	= *285.0 m*
Minimum water level	= *278.0 m*
Downstream bed level	= *278.0 m*
F.S.L. of canal	= *284.0 m*
Allowable afflux	= *1 m*
Coefficient of creap	= *12*
Permissible exit gradient	= *1/6*

(60)

Q. 6. *(a) Explain the different types of spillways used in dam construction. State briefly the design principles of an ogee spillway in sequence.* (30)

(b) Design an irrigation canal for a discharge of 15 cumec, N = 0.225, m = 1 and $\frac{B}{D} = 5.7$. *Use the Kennedy theory.* (30)

Paper II

Q. 9. *Attempt any three subdivisions :*

(a) Derive the basic probability of an N-year event defined in the conventional way ; indicate the definition also. Accordingly, derive the probability of a flood with a return period of 20 years, occurring in a particular 3-year period. (20)

(b) (i) What are the factors (list at least six) that affect the pattern of sediment deposition in a reservoir. (10)

(ii) A proposed reservoir has a capacity of 50 ha-m. The catchment area is 125 km^2 and the annual streamflow averages 12 cm of run-off. If the annual sediment production is 0.03 ha-m/km^2, 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, n (%) and capacity-flow 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

[**Note.** Please refer Solved Example 18.16]. (20)

(c) What are the several natural and man-made causes of water-logging ? What are the ill-effects due to water-logging? What are the several measures practicable for minimising water-logging effects ? (20)

(d) What are the purposes for which foundation treatment for hydraulic structures is undertaken ? What are the several features that are treated in this manner ? Write briefly on each. (20)

Q. 10. *(a) Classify the common methods of irrigation and indicate the suitability and advantages of each main method besides indicating how each method is practised.* (30)

(b) How are river training works classified ? What is the propose achieved in each of these classes of training works and how is it achieved? (15)

(c) Define the following : (i) Alkali soil, (ii) Alkaline soil, (iii) Leaching, Sodium absorption ratio. (15)

Q. 11. *(a) A multi-purpose reservoir for providing irrigation water, flood control and low-flow augmentation for water quality control is under consideration. The following curves have been generated by proper analysis : (i) Reservoir cost curve, (ii) Reservoir yield curve, (iii) Annual irrigation demand, (iv) Annual flood control benefits, (v) Water quality benefits from low-flow augmentation and (vi) Water requirement for low augmentation. Draw the general shape of each of these curves indicating the entities represented on the coordinate axes.* (25)

(b) A project has been assessed for different levels of investment and is to be evaluated for a 10 year life at 10% return on its capital. The consecutive levels of

investment and the corresponding net annual returns are assessed to be as under (all in lakhs of rupees) :

Investment :	*500*	*600*	*700*	*800*	*900*	*1000*
Annual returns :	*100*	*110*	*122*	*132*	*141*	*149*

Considering the appropriate aspects of evaluation, determine how large an investment should be made. SFF for 10 years at 10% = 0.06275. (20)

(c) Calculate the maximum discharge required at the outlet of a distributary having the following cropped area and duties of water : (15)

Crop	*Area (in ha)*	*Crop Season*	*Duty (in '000 ha/cumec)*
Sugarcane	*200*	*Throughout*	*0.730*
Other perennials	*1000*	*Throughout*	*1.100*
Hot weather fodder	*3000*	*Hot weather*	*2.000*
Rice	*1500*	*Kharif*	*0.775*
Other Kharif	*1500*	*Kharif*	*1.500*
Rabi	*5000*	*Rabi*	*1.800*

Q. 12. *(a) A concrete gravity dam has the following design features :*

Top of dam	*= 200 m RL*
Width at top	*= 7 m*
Bottom of top rectangular position	*= 190 m RL*
Downstream battery	*= 0.7 h on 1 v*
Upstream battery is below 150 m RL and has a value	*= 1 h on 10 v*
Drainage gallery at 10 m from u/s vertical face ; Uplift reduction at gallery	*= 0.6*
Tail water depth	*= 5 m*
Wt. of concrete	*= 2.4 T/m*^{3}
Water stands upto	*= 195 m RL*
Base of dam	*= 100 m RL*

Determine the vertical normal stresses at each end of the base when the reservoir is full as indicated. (30)

(b) What are the criteria for safe design of earth dams ? (15)

(c) A storm with a 15.0 cm precipitation produced a direct runoff of 8.7 cm. The time distribution of the storm is as follows :

Time from start hr.	*1*	*2*	*3*	*4*	*5*	*6*	*7*	*8*
Incremental rainfall in each hour, cm	*0.6*	*1.35*	*2.25*	*3.45*	*2.7*	*2.4*	*1.5*	*0.75*

Estimate the ϕ_{index} *of the storm.* (15)

[**Note.** Please refer Solved Example 7.26.]

Year 1988 (*Civil Service*)

Paper I

Q. 5. *(a) A concrete dam 30m high retains water to a maximum height of 25 m on upstream side. The upstream face is vertical for the upper 10 m of the dam and then sloping towards upstream at 1 horizontal to 10 vertical up to the base. The top width*

of the dam is 4 m for a height of 5 m and then splays in the downstream giving a total base width of 20 m. There is tail water for a depth of 5 m and the axis of drainage gallery is at 4 m from the heel. Assuming the specific gravity of concrete as 2.4, calculate the factor of safety against overturning and the stress at the toe and the heel under reservoir full condition. (30)

(b) Describe briefly the function and design of a silt excluder provided in a diversion head work. Supplement your answer with neat sketches. (30)

Q. 6. *(a) Design the section of an unlined canal in a loamy soil to carry a discharge of 50 cum/sec with a permissible velocity of 1 m/sec. Assume the side slopes as 2 : 1 and B/D ratio as 6.0. Using Manning formula, calculate the bed slope of the canal, N = 0.0225.* (15)

(b) Discuss briefly the different types of failure of earth dams and measures to prevent them. (30)

(c) Explain the technical and financial justification for lining of existing canals. (15)

Paper II

(Part C — *Water Resources Engineering***)**

Q. 9. *(Compulsory) Answer any three subdivisions :*

(a) What is runoff ? Discuss the factors influencing the runoff of a catchment area. (20)

(b) What is duty of water ? Explain the influence of several factors affecting duty. (20)

(c) Enumerate the different methods of river training. Describe the method of marginal embankments. (20)

(d) Discuss the factors influencing the selection of a site for a proposed dam. (20)

Q. 10. *(a) What is unit hydrograph ? How is it obtained ? Explain its use in the construction of flood hydrograph resulting from rainfalls.* (20)

(b) Explain flood routing through reservoirs. Describe stepwise the procedure adopted for flood routing computations by the trial and error method. (20)

(c) Write explanatory notes on the different types of irrigation. (20)

Q. 11. *(a) Explain the governing considerations for fixing up the alignment, the full supply level, and the full supply discharge of an irrigation canal.* (20)

(b) The gross command area for a distributary is 10,000 hectares, 75% of which can be irrigated. The intensity of irrigation for Rabi season is 60% and that for Kharif season is 30%. If the average duty at the head of the distributary is 2500 hectares per cumec for Rabi season, and 1000 hectares per cumec for Kharif season, determine the discharge required at the head of the distributary from average demand consideration. (20)

[**Note.** Please refer and follow Example 3.1 and determine the requisite discharge as 2.25 cumecs. **Ans.**]

(c) Explain the principal causes and the effects of water-logging in a canal irrigated farm. State the precautions and measures adopted to prevent water-logging of irrigated lands. (20)

Q. 12. *(a) State the general conditions of stability of gravity dams. Explain stepwise the procedure of analysing high gravity dams.* (20)

(b) Enumerate the different types of earth dams indicating the situation where each type is more suitable. Explain in sequence the steps involved in determining the stability of earth dams by the Swedish slip circle method. (20)

(c) State the governing considerations for the selection of different types of spillways. Explain the design principles of ogee spillway. (20)

Year 1989 *(Civil Services)*

Paper I

[**Note.** ***Due to revision in syllabus w.e.f. 1989, Paper I did not contain any question on Irrigation, as it contained questions on Structural Engg., Fluid Mechanics, and Soil Mechanics only, in 3 sections.***]

Paper II

(**Part C** — *Water Resources and Irrigation Engineering)*

Q. 9. *(Compulsory). Attempt any three sub-divisions :*

(a) (i) Explain briefly what you understand by the terms :

1. Design flood

2. Standard project flood

3. Probable maximum flood

4. Design storm. (10)

(ii) What are the recommended design floods for :

1. Spillways of major projects

2. Permanent barrages

3. Waterways of aqueducts

4. Pick-up weirs. (10)

(b) (i) Describe briefly the factors affecting the evaporation from water bodies. Describe a commonly used evaporimeter. (10)

(ii) An isolated 3 hour storm occurred over an area of 120 ha as below:

Partial area of catchment (ha)	ϕ index (cm/h)	Rainfall (cm)		
		1st hour	2nd hour	3rd hour
36	0.90	0.6	2.4	1.3
18	1.10	0.9	2.1	1.5
66	0.50	1.0	2.0	0.9

What is the total rainfall on the catchment in this storm ? Estimate the runoff from the catchment. If the ϕ index were to be remain at the same value, what runoff would be produced by a uniform rainfall of 3.3 cm in 3 hours uniformly spread all over the catchment ? (10)

[**Note.** Please refer Solved Example 7.28].

(c) Describe briefly the forces considered in the design of gravity dams: what combination of these forces is critical in the design of non-overflow section of the dam under reservoir empty condition ? (20)

(d) (i) Explain very briefly the following terms used in groundwater flow :

1. Specific yield

2. Storage coefficient

3. Barometric efficiency

4. Specific capacity. (10)

(ii) Write short notes on :

1. Irrigation water quality

2. Leaching. (10)

Q. 10. *(a) What are the advantages of lining a canal ? Explain how the benefit cost ratio for lining an existing canal can be estimated. Indicate clearly the assumptions, if any, involved.* (15)

(b) Write brief notes on :

(i) Bed load

(ii) Suspended load

(iii) Critical shear stress in sediment transport. (15)

(c) The following table gives the monthly inflow into and contemplated demand from a proposed reservoir :

Month	*Jan.*	*Feb.*	*March*	*April*	*May*	*June*	*July*	*Aug.*	*Sept.*	*Oct.*	*Nov.*	*Dec.*
Monthly inflow M.m³	50	40	30	25	20	30	200	225	150	90	70	60
Monthly demand M.m³	70	75	80	85	130	120	25	25	40	45	50	60
Evapo-ration, cm	6	8	13	17	22	22	14	11	13	12	7	5
Rainfall cm	1	0	0	0	0	19	43	39	22	6	2	1

Assume the average reservoir area as 30 km². The runoff coefficient of the area flooded by the reservoir is 0.4. Estimate the minimum storage that is needed to meet the demand. (30)

[**Hint.** Follow Example 8.2 and compute the reqd. storage capacity = 133.08 ha. m, as per computations carried out in table given on next page. **Ans.**]

Q. 11. *(a) (i) Distinguish between :*

1. Hydrologic and hydraulic methods of flood routing.

2. Hydrologic storage routing and hydrologic channel routing. (10)

[**Hint.** Flow routing is a procedure by which we can estimate/trace the flow through a *hydrologic system* (like a river or a reservoir, or a canal), downstream of a given input point. This can be done either by adopting *hydrologic routing techniques* or by *hydraulic routing techniques.*

In **hydrological routing,** the flow is calculated as a *function of time alone at a particular location* ; whereas in a **hydraulic routing,** *the flow is calculated as a function of space and time both,* throughout the system.

Hydrologic storage routing has been adopted by us in Chapter 18, to compute the levels likely to be achieved in a reservoir due to entry of a flood hydrograph, and to work out the outflow hydrograph, spilling over the spillway. *Hydrologic channel routing* has similarly been used in Muskingum's method for determining the shape of flood hydrograph at a particular known point along a river channel downstream of a given input point upstream, as explained in Chapter 13 of "Hydrology and Water Resources Engineering".

Solution Table for Q. 10 (*c*) of 1989 (Civil Services)

Month	*Inflow in Mm^3*	*Evaporation cm*	*Rainfall in cm*	*Demand Mm^3*	*Evaporation loss* $= \frac{30\ km^2 \times col.\ (3)}{100}\ m$ $= 0.3 \times col.\ (3)$ *in $M.m^3$*	*Rainfall in Mm^3; only 60% rainfall is now counted, because 40% was already counted as runoff in the observed inflows before construction of dam* $= \frac{0.6 \times col.\ (4)}{100}\ m \times 30\ km^2$ $= 0.18 \times col.\ (4)$	*Adjusted inflow in $M.m^3$ col. (2) – col. (6) + col. (7)*	*Water reqd. from storage in $M.m^3$ col. (5) – col. (8) (only +ve values)*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*	*(8)*	*(9)*
Jan.	50	6	1	70	1.8	0.18	48.38	21.62
Feb.	40	8	0	75	2.4	0.00	37.60	37.4
March	30	13	0	80	3.9	0.00	26.10	53.9
April	25	17	0	85	5.1	0.00	19.90	5.10
May	20	22	0	130	6.6	0.00	13.70	6.30
June	30	22	19	120	6.6	3.42	26.82	3.18
July	200	14	43	25	4.2	7.74	203.54	Nil
August	225	11	39	25	3.3	7.02	228.72	Nil
Sept.	150	13	22	40	3.9	3.96	150.06	Nil
Oct.	90	12	6	45	3.6	1.08	87.48	2.52
Nov.	70	7	2	50	2.1	0.36	68.26	1.74
Dec.	60	5	1	60	1.5	0.18	58.68	1.32

Reqd. storage from reservoir = $\Sigma = 133.08\ Mm^3$. **Ans.**

The **hydraulic flood routing techniques,** such as those by the use of *Saint Venant Equations,* and *Dynamic wave routing,* involve complicated mathematics, and is generally used only when flows are to be analysed on a computer model. Such hydraulic routing models prove better than the hydrologic models in representing the actual unsteady non-uniform nature of flow in the channel, and also give reliable results even for routing *smaller flows,* such as for routing water in irrigation canals.]

Q. 11. *(a) (ii) Route the following flood through a river reach for which the Muskingum coefficients K and X are 22 h and 0.25 respectively. At time t = 0, the outflow discharge is 40 m^3/s :*

Time (h)	*0*	*12*	*24*	*36*	*48*	*60*	*72*
Inflow (m^3/s)	*40*	*65*	*165*	*250*	*240*	*205*	*170*

(10)

[**Note.** Please refer Solved Example 13.4 in "Hydrology and Water Resources Engineering".]

Q. 11. *(b) Describe the salient features and design considerations of the hydraulic jump type energy dissipators used below overflow spillways.* (20)

Q. 11. *(c) Describe the functions of 'Distributary Head Regulators' and 'Cross Regulators'. Sketch a typical section of a Distributary Head Regulator and indicate the salient parts. List the design principles for a Distributary Head Regulator.* (20)

Q. 12. *(a) (i) A weir across an alluvial river has a horizontal floor of length 60 m and retains 6 m of water under full pond condition. If the downstream sheet pile is driven to a depth of 6 m below the average bed level, calculate the exit gradient. Further, assuming a porosity of 30% and the relative density of soil particles as 2.70, estimate the critical exit gradient and the factor of safety of the system with respect to the exit gradient.* (10)

(ii) Write explanatory notes on the following terms connected with a barrage :

(1) Scouring sluices

(2) Divide wall

(3) Downstream loose protection. (15)

Q. 12. *(b) List the various objectives of river training and indicate against each objective the methods of training that can be adopted to achieve the same.* (20)

Q. 12. *(c) List the essential requirements of an ideal irrigation outlet. Describe with the aid of sketches, the open flume semi-modular outlet.* (15)

Year 1990 (*Civil Services*)

Q. 9. *(Compulsory) Attempt any three sub-divisions :*

(a) Describe the salient characteristics of precipitation in India. (20)

(b) A certain crop is grown in an area of 3000 hectares, which is fed by a canal system. The data pertaining to irrigation are as follows :

Field capacity of soil	= *26%*
Optimum moisture	= *12%*
Permanent wilting point	= *10%*
Effective depth of root zone	= *80 cm*
Apparent relative density of soil	= *1.4*

If the frequency of irrigation is 10 days and the overall irrigation efficiency is 22%, find :

(i) the daily consumptive use ; and

(ii) the water discharge in m^3/s required in the canal feeding the area. (30)

[**Hint.** Follow example 2.13 and 2.14, and determine :

Depth of water used by plants for growth, which is supplemented by irrigation after every 10 days

$$= \frac{\gamma d}{w}[\text{Field capacity m.c.} - \text{Optimum m.c.}]$$

$$= \frac{1.4 \times 0.8 \text{ m}}{1}\left[0.26 - 0.12\right] = 0.1568 \text{ m} = 15.68 \text{ cm.}$$

(*i*) Daily water consumed by plants ; *i.e.*, daily consumptive use

$$= \frac{15.68}{10} \text{ cm} = 1.568 \text{ cm.} \quad \textbf{Ans.}$$

(*ii*) Total irrigation water required *i/c* losses in field and conveyance

$$= \frac{\text{N.I.R.}}{\eta_{irrigation}} = \frac{1.568}{0.22} \text{ cm/day} = 7.127 \text{ cm/day}$$

Field area $= 3000 \text{ ha} = 3000 \times 10^4 \text{ m}^2$

$\therefore$ Discharge required in irrigation canal

$$= \frac{7.127}{100} \times \frac{3000 \times 10^4}{24 \times 60 \times 60} \text{ m}^3/\text{s} = \mathbf{24.75 \text{ m}^3/\text{s}} \quad \textbf{Ans.}]$$

(c) Describe the causes and effects of water-logging in canal irrigated areas. Indicate the measures to be adopted to prevent water-logging. (20)

(d) Write short notes on :

(i) Reservoir sedimentation.

(ii) Silt ejectors.

(iii) Guide bunds.

(iv) Well-loss. (20)

Q. 10. *(a) For a river, the estimated flood peaks for two return periods by use of Gumbel's method are as follows :*

Return period (years)	*Peak flood (m^3/s)*
100	*485*
50	*445*

What flood discharge in this river will have a return period of 1000 years. (30)

[**Note.** Please refer Solved Example 7.57.]

(b) Write short notes on :

(i) Launching apron *(ii) Groynes*

(iii) Silt excluder *(iv) Recuperence test for a well*

(v) Piping failure of a weir floor *(vi) Tainter gate.* (30)

Q. 11. *(a) An impervious floor of a weir on permeable soil is 16 m long and has sheet piles at both the ends. The upstream pile is 4 m deep and the downstream pile is 5 m deep. The weir creates a net head of 2.5 m. Neglecting the thickness of the weir floor, calculate the uplift pressures at the junction of the inner faces of the pile with the weir floor, by using Khosla's theory.* (30)

[**Note.** Please refer Solved Example 11.3].

(b) Describe briefly the energy budget method of estimating evaporation from a lake. (20)

(c) Describe with the help of a neat sketch the salient features of a Sarda type canal fall. (10)

Q. 12. *(a) Write short notes on :*

(i) Maximum-Depth-Area-Duration Curves.

(ii) Probable Maximum Precipitation

(iii) Potential Evapotranspiration.

(iv) Infiltration process. (20)

(b) A commonly used shape for concrete lined canals, called the standard lined trapezoidal canal section, consists of a trapezoidal section with each of its two corners being rounded off by a circular arc of a radius equal to the full supply depth. Determine the dimensions of such a standard lined trapezoidal canal to carry a discharge of 200 m^3/s at a bed slope of 1 in 4000. The side slopes are 2 horizontal : 1 vertical, and Manning's n = 0.014. The limiting velocity in the canal is 2.0 m/s. (20)

[**Hint.** Follow exactly similar solved example 5.4 and determine

$$R = 2.66 \text{ m}; \quad \cot\theta = 2; \quad \theta = 0.46 \text{ radians.}$$

$$A = 100 = y\,[B + 0.46\,y + 2y]$$

or $$B = \left[\frac{100}{y} - 2.46\,y\right] \quad ...(i)$$

Also $$R = 2.66 = \frac{100}{P}, \text{ where}$$

$$P = B + 0.92y + 2y \times 2 = \frac{100}{2.66}$$

or $$B = 37.59 - 4.92y \quad ...(ii)$$

Solving (*i*) and (*ii*)

$$\left.\begin{aligned} y &= 3.44 \text{ m} \\ B &= 20.7 \text{ m.} \end{aligned}\right] \textbf{ Ans.}$$

(c) Explain briefly a procedure for calculating the stability of slopes of an earth dam. What are the critical loading conditions at which the stability of an earthen dam is tested during design ? (20)

Year 1991 (*Civil Services*)

Q. 9. *Attempt any three sub-divisions.*

(a) Describe briefly the ground water resources of India and its utilization. (20)

(b) Explain how seismic forces are taken into account in the design of solid gravity dams. (20)

(c) (i) Describe briefly the dilution technique of stream flow measurement. (10)

(ii) A trapezoidal channel with side slopes 1.5 horizontal : 1 vertical is required to carry 15 m^3/s of flow with a bed slope of 1 in 4000. If the channel is lined, the Manning's coefficient n will be 0.014, and it will be 0.028 if the channel is unlined. Calculate the average boundary shear stress if hydraulically efficient lined channel is adopted. What percentage of earthwork is saved in a lined section-relative to an unlined section, when hydraulically efficient section is used in both the cases ? The freeboard can be assumed to be 0.75 m in both the cases and the lining can be assumed to be up to the top of the section. (10)

[**Note.** Please refer Solved Example 4.4]

(d) Write short notes on :

(i) Drip irrigation

(ii) Bed load

(iii) Meanders (20)

Q. 10. *(a) List the different types of spillway crest gates in common use. Describe with neat sketches the salient features of radial gates and vertical lift gates.* (30)

(b) (i) What are the assumptions in the unit hydrograph theory

(ii) The ordinates of 4-hour hydrograph of a catchment are given below :

Time (h)	0	4	8	12	16	20	24	28	32	36	40	44
Ordinate of 4-h U.H. (m^3/s)	0	20	60	150	120	90	70	50	30	20	10	0

Derive the flood hydrograph in the catchment due to the storm given below :

Time from start of storm (h)	0	3	8	12
Accumulated rainfall (cm)	0	5.0	5.8	8.8

The ϕ-index for the catchment can be assumed to be 0.25 cm/h and a constant base flow of 20 m^3/s be appropriate. (25)

[**Note.** Please refer Solved Example 7.37].

Q. 11. *(a) Explain briefly the concept of critical shear stress as used in sediment transport studies. Sketch and label the salient features of Shields curve relating to the critical shear stress. Show that in an unlined canal of hydraulic radius R and bed slope, S_o constructed in a non-cohesive material, the minimum size d of the sand grain that will not be moved by the flow could be expressed as $d = 11\ R.S_0$.* (20)

(b) With reference to a water resources development project, explain briefly :

(i) Engineering economy study

(ii) Project formulation. (30)

[**Note.** Please refer chapter 20 in "Hydrology and Water Resources Engineering"].

(c) Explain briefly the guidelines currently used in India for selection of design floods for dams and barrages. (10)

Q. 12. *(a) A well is located in 30 m thick confined aquifer of permeability 35 m/day and storage coefficient of 0.004. If the well is pumped at the rate of 1500 litres per minute, calculate the drawdown at a distance of 40 m from the well after 20 hours of pumping.* (20)

[**Note.** Use Jacob's formula and compute drawdown as 0.94 m. **Ans.**]

(b) Describe the use of hydraulic jump as an energy dissipator on the downstream side of overflow spillways. (25)

(c) In a wide stream, a suspended load sample taken at a height of 0.30 m from the bed indicated a concentration of 1000 ppm of sediment by weight. The stream is 5.0 m deep and has a bed slope of 1/4000. The bed material can be assumed to be of uniform size with a fall velocity of 2.0 cm/s. Estimate the concentration of the suspended load at mid-depth. (15)

[**Note.** Please refer Solved Example 4.10].

Year 1992 (*Civil Services*)

Q. 9. *(Compulsory) Answer any three of the following :*

(a) Discuss the considerations to be made for selection of site of rain gauge stations.

How would you estimate the missing rainfall data at a rain gauge station for a particular rain storm by Normal Ratio method ? Under what conditions is it adopted ? (20)

(b) Explain the terms duty, delta, and base period in connection with irrigation. Establish a relation between Duty of water, Delta and Base period.

Discuss the factors on which duty of water depends. (20)

(c) Discuss the relative advantages and disadvantages of a Barrage over a Weir. Explain the functions of Undersluices in a barrage. (20)

(d) Draw a typical cross-section of a masonry dam. Briefly explain the principle of design of this cross-section. Explain the functions of drainage gallery. (20)

Q. 10. *(a) What is unit hydrograph ? Discuss the assumptions involved in Unit Hydrograph Concept.* (20)

(b) Discuss in details the general procedure of deriving a unit hydrograph. Explain your answer with a neat diagram. (20)

(c) Name some statistical methods of forecasting peak flood discharge. Describe any one of these methods : (20)

Q. 11. *(a) Explain the terms :*

(i) Specific yield (ii) Coefficient of transmissibility, (iii) Storage coefficient, (iv) Infiltration gallery. (20)

(b) A well penetrates into an unconfined aquifer having a saturated depth of 15 m.

The discharge is 8000 litres/hour at 6 m drawdown. Assuming equilibrium flow conditions and a homogeneous aquifer, compute the discharge at 9 m drawdown. The radius of influence may be taken as equal in both cases. Deduce the formula that you would use. (20)

[**Solution.** Using equation 16.17, we have

$$Q = \frac{\pi K (d^2 - h_w^2)}{2.3 \log_{10} \frac{R}{r_w}}$$

where $h_w = d - s$

where d = saturated depth of stratum

s = drawdown

h_w = water head in well.

$$\therefore \quad Q_1 = \frac{\pi K (d^2 - h_{w1}^2)}{2.3 \log_{10} \frac{R}{r_w}} \quad \text{and} \quad Q_2 = \frac{\pi K (d^2 - h_{w2}^2)}{2.3 \log_{10} \frac{R}{r_w}}$$

$$\therefore \quad \frac{Q_2}{Q_1} = \frac{(d^2 - h_{w2}^2)}{(d^2 - h_{w1}^2)} = \frac{d^2 - (d - s_2)^2}{d^2 - (d - s_1)^2}$$

Using given values of $Q_1 = 8000\ L/h$, $s_1 = 6$ m, $d = 15$ m and $s_2 = 9$ m, we get

$$Q_2 = \frac{15^2 - (15-9)^2}{15^2 - (15-6)^2} \times Q_1 = \frac{15^2 - 6^2}{15^2 - 9^2} \times 8000\ L/h$$

$$= \frac{21 \times 9}{24 \times 6} \times 8000\ L/h = 10{,}500\ L/h. \quad \textbf{Ans.}]$$

(c) Describe briefly the ground water resources of India and its utilization. (20)

Q. 12. *(a) What are the different types of alignment of irrigation canals? Mention the situation where each type is suitable.* (15)

(b) Explain the phenomenon of hydraulic jump. Discuss its use in the design of hydraulic structures.

(c) Write short notes on :

(i) Bed load and suspended load.

(ii) Critical shear stress in sediment transport.

(iii) Guide-bunds.

(iv) Launching apron.

(v) Piping failure of a weir floor. (5 × 6)

Year 1993 (*Civil Services*)

Q. 9. *(Compulsory) Answer any three of the following :*

(a) What are the objectives of command area development ? Explain how these may be achieved. (20)

(b) (i) What is meant by consumptive use of water for a crop ? On what factors does this depend ? (10)

(ii) During a particular stage of the growth of a crop, consumptive use of water is 2.5 mm/day. Determine the interval in days between irrigations, and the depth of water to be applied when the amount of water available in the soil is 50% of the maximum depth of available water in the root zone, which is 80 mm. Assume irrigation efficiency to be 60%. (10)

[**Solution.** *Study Solved Example 2.13, and compute :*

Readily available moisture in root zone

$$= 50\% \times 80 \text{ mm} = 4 \text{ cm.}$$

Consumptive use = 2.5 mm/day = 0.25 cm/day

∴ 0.25 cm of water is consumed by crop in = 1 day

4 cm of water will be consumed by crop in

$$= \frac{1}{0.25} \times 4 = \mathbf{16\ days.}$$

Also, the depth of water to be recouped through irrigation = 4 cm.

Actual water required to be applied to the field

$$= \frac{\text{Water required to be recouped or absorbed}}{\text{Efficiency of irrigation application}}$$

$$= \frac{4 \text{ cm}}{60\%} = \frac{4 \text{ cm}}{0.6} \text{ cm} = \mathbf{6.67\ cm.} \quad \textbf{Ans.}$$

Hence, 6.67 cm depth of irrigation water shall have to be applied to the fields at an interval of 16 days. **Ans.**

(c) (i) Explain how maximum flood discharge can be obtained from frequency analysis of streamflow data. (8)

(ii) The records of annual rainfall at a rain gauge station expressed in mm in chronological sequence from 1981 to 1992 are given below :

360, 285, 557, 815, 273, 229, 707, 478, 309, 176, 285, 651

Using Hazen's method, estimate the maximum annual rainfall which has a recurrence interval of 10 years. (12)

[**Hint.** Follow Solved Example 7.8 and use Hazen's probability equation (eq. 7.139) as $T = \frac{N}{m - 0.5}$; *i.e.*

$$10 = \frac{12}{m - 0.5}$$

$m = 1.7$; Therefore choose the 1st severest storm

= **815 mm.** **Ans.**]

(d) Discuss the salient features of Khosla's method of design of weir on pervious foundation, against sub-surface flow. Point out the merits and limitations of the method. (20)

Q. 10. *(a) Discuss the common causes of failure of tubewells. Indicate the possible remedial measures against each of these. What are the criteria of failure of tubewells?* (30)

(b) Two identical tube-wells penetrating fully a 12 m thick aquifer are located at 180 m apart. The tubewells have diameter of 30 cm, radius of influence of 300 m, and the coefficient of permeability of aquifer is 10^{-3} m/sec. Compute :

(i) Discharge of tube-well when only one is working with a drawdown of 5 m.

(ii) Percentage decrease in discharge of the well, if both are working with a drawdown of 5 m. (30)

[**Solution.** (*a*) Discharge of single tube-well is given by eqn. (16.20) as :

$$Q_1 = \frac{2\pi KHs}{2.3 \log_{10} \frac{R}{r_w}}$$

$$= \frac{2 \times 3.14 \times 10^{-3} \times 12 \times 5}{2.3 \log_{10} \frac{300}{0.15}} = \mathbf{0.0496\ m^3/s} \quad \textbf{Ans.}$$

(*b*) Discharge of each identical tube-well, when both are working, is given by eqn. (16.24) as :

$$Q_2 = \frac{2\pi KHs}{2.3 \log_{10} \left(\frac{R^2}{r_w \cdot B} \right)}$$

$$= \frac{2 \times 3.14 \times 10^{-3} \times 12 \times 5}{2.3 \log_{10} \left(\frac{300^2}{0.15 \times 180} \right)} = \mathbf{0.0465\ m^3/s.}$$

Percentage decrease in discharge

$$= \frac{0.0496 - 0.0465}{0.0496} \times 100 = \mathbf{6.3\%.} \quad \textbf{Ans.}]$$

Q. 11. *(a) What are the true regime conditions in an alluvial channel as stipulated by Lacey ?*

(ii) A stable channel is to be designed for a discharge of 40 m^3/sec and silt factor of unity. Calculate the dimensions of the channel using Lacey's regime equations. Also calculate the dimensions of the channel if it were to be designed on the basis of Kennedy's method with critical velocity ratio equal to unity, and the ratio of bed width to depth of flow the same as obtained from Lacey's method. (40)

[**Hint.** Follow *Solved Example 4.8*, and design the channel dimensions on Lacey's theory to be y = **1.84 m**, and b = **25.92 m.** **Ans.** Then, design channel dimensions on the basis of Kennedy's theory by using $\frac{B}{D}$ ratio as equal to

$\frac{25.92}{1.84} = 14.09$, and by following *Solved Example 4.6*, to compute D = **1.53 m** and B = **21.50 m. Ans.**]

(b) Write short notes on :

(i) Sprinkler irrigation *(ii) Drip irrigation*

(iii) Canal lining *(iv) Silt ejector.* (20)

Q. 12. *(a) Explain with sketches the various channel patterns of a river. Discuss the influence of these patterns on planning of a hydraulic structure.* (20)

(b) What are the objectives of river training ? Describe the following river training methods indicating their merits and demerits :

(i) Spurs *(ii) Guide banks*

(iii) Revetments. (20)

Year 1994 *(Civil Services)*

Q. 9. *(Compulsory) Answer any three sub divisions.*

(a) (i) How is the inconsistency of rainfall data of a rain gauge station ascertained ? Discuss briefly the technique and also enumerate the possible causes for such inconsistency.

(ii) What are monolayer films ? How effective are they in retarding evaporation from reservoirs. (20)

(b) (i) 800 m^3 of water is applied to a farmer's rice field of 0.6 hectares. When the moisture content in the soil falls to 40% of the available water between the field capacity (36%) of soil and permanent wilting point (15%) of the soil crop combination, determine the field application efficiency. The root zone depth of rice is 60 cm. Assume porosity = 0.4.

[**Note.** Please refer Solved Example 2.15].

(ii) Differentiate between "Potential Evapotranspiration" and "Actual Evapotranspiration". Do the characteristics of soil and available moisture content affect them ? If yes, how ? (20)

(c) (i) In what manner does seepage endanger the safety of a structure on permeable foundation ? How will you ensure the safety of structures against seepage effects.

(ii) Sketch the typical cross-section of channel, when it is (i) Fully in filling (ii) Partly in filling and partly in cutting. (20)

(d) (i) In a silt excluder for a barrage, why the tunnels in general are of different lengths ? Explain.

(ii) Give a typical layout of the Hydropower development "shortcircuiting long reaches of a circuitous river with steep slope".

(iii) Find the base width of elementary profile of a gravity dam from considerations of no tension. (20)

Q. 10. *(a) Compare the characteristics of Instantaneous unit Hydro-graph (IUH) with the characteristics of Conventional unit Hydrograph. Can S-curve hydrograph be used to derive an IUH of a catchment ? If so, how ?*

(b) The ordinates of a 6-hour unit hydrograph for a basin are given below :

Time (hours)	0	3	6	9	12	18	24
6-h unit hydrograph ordinates (m^3/s)	0	150	250	450	600	800	700
Time (hours)	30	36	42	48	54	60	66
6-h unit hydrograph ordinates (m^3/s)	600	450	325	200	100	50	0

A storm had three successive 6 hour interval of rainfall magnitudes of 3.0, 5.0 and 4.0 cm respectively. Assuming a ϕ-index of 0.20 cm/hr and a base flow of 30 m^3/s, determine the resulting hydrograph of flow. (40)

[**Hint.** Follow similar Solved Example 7.35].

Q. 11. *(a) What are "partially penetrating wells". Discuss their characteristics.* (10)

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

[**Hint.** Derive and use Thiem's equation (*i.e.* eq. 16.9) for unconfined aquifers. Then to solve the given numerical example, assume the water table to be at the top of the aquifer, *i.e.* the depth of aquifer being equal to the saturated depth (d). Then using the eq. (16.9) we have

$$K = \frac{2.3\, Q \log \frac{r_2}{r_1}}{\pi\,(h_2^2 - h_1^2)} = \frac{2.3 \times \left(\frac{1.5}{60}\right) \text{m}^3/\text{s} \times \log_{10} \frac{75}{25}}{\pi\,[(40-3.5)^2 - (40-2)^2]}$$

$$= 0.781 \times 10^{-4} \text{ m/s}$$

$$T = K.d = 0.781 \times 10^{-4} \times 40$$

$$= \mathbf{3.126 \times 10^{-4}\ m^2/sec} \quad \textbf{Ans.}$$

Now, to determine drawdown, in main well s_w, use r_w in place of r_1, and h_w in place of h_1 in the above equation, as :

$$K = 0.781 \times 10^{-4} = \frac{2.3 \times 0.025\ \text{m}^3/\text{s} \times \log_{10} \frac{75}{0.15}}{\pi\,[36.5^2 - h_w^2]}$$

or $\quad h_w = \mathbf{13.55\ m}$ **Ans.**]

(c) What are the different steps necessary to create an Engineering project ? Enumerating conservation purposes, differentiate between single purpose and multipurpose projects. (20)

[**Hint.** Please refer article 21.1 & 21.2 in "Hydrology and Water Resources Engineering — 1998 ed.]

Q. 12. *(a) Describe various modes of sediment transport in an alluvial channel.*

Design a Lacey's regime channel, flowing through material of average diameter 0.328 mm, for the following data :

Culturable commanded area = *1,00,000 hectare*

Intensity of irrigation :

Rabi = *40%*

Kharif = *30%*

Outlet discharge factor :

Rabi = *1,800 hec/cumec*

Kharif = *800 hec/cumec*

Assume conveyance losses to be 10%. (30)

[**Hint.** Area irrigated in Rabi = 40,000 hec

$$\therefore\ Q \text{ for Rabi} = \frac{40{,}000\,\text{hec}}{1800\,\text{hec/cumec}} = 22.2\ \text{m}^3/\text{s}$$

$$\text{Similarly } Q \text{ for Kharif} = \frac{30{,}000\,\text{hec}}{800\,\text{hec/cumec}} = 37.5\ \text{m}^3/\text{s}$$

Hence, design the channel on Lacey's theory for Q = 37.5 m^3/s and silt factor $f = 1.75\sqrt{d_{mm}} = 1.75\sqrt{0.328} = 1.0$, by following the solved example 4.8, and compute $b = 25.06$ m, $y = 1.8$ m and $S = 1$ in 6110 for a channel having side slopes of $\frac{1}{2}H : IV$. **Ans.**].

(c) What are the objectives of river training ? Describe the following river training methods :

(i) Spurs *(ii) Guide banks.* (20)

Year 1995 (*Civil Services*)

Q. 9. *(Compulsory). Answer any three subdivisions :*

(a) In a basin six rain gauge stations are reporting rainfall. In a year the annual rainfall recorded by the gauges are as follows :

Station	A	B	C	D	E	F
Rainfall (cm)	41	51	32	55	50	68

For 8% error in estimation of the mean rainfall, calculate the optimum number of rain gauge stations in the basin. Give criteria to install them, if there number is more than the existing number. (20)

[**Hint.** Follow Example 7.3 and compute :

$$\bar{p} = \frac{41+51+32+55+50+68}{6} = 49.5\ \text{cm}$$

$$\overline{p^2} = \frac{\Sigma P^2}{n} = \frac{41^2+51^2+32^2+55^2+50^2+68^2}{6} = 2575.83$$

$$\sigma = \sqrt{\frac{n}{n-1}\left[\overline{p^2} - (\bar{p})^2\right]} = \sqrt{\frac{6}{5}\left[2575.83 - (49.5)^2\right]}$$

$$= \sqrt{150.7} = 12.276$$

$$C_v = \frac{100\,\sigma}{\bar{p}} = \frac{100 \times 12.276}{49.5} = 24.80$$

$$N = \left(\frac{C_v}{E}\right)^2 = \left(\frac{24.80}{8}\right)^2 = 9.61 \textbf{ Say 10 No.}$$

∴ Optimum No. of rain gauges = **10 No.** **Ans.**

4 additional gauges are, thus, required to be installed in the given catchment. *These additional gauges should be distributed in the different zones (caused by isohyets) in proportion to their areas.* This will ensure even distribution of the gauges over the entire basin. **Ans.**]

Q. 9. *(b) What are cross-drainage works ? The type of cross-drainage work on the main canal at a drainage can be changed by suitably changing the alignment of main canal. Explain giving schematic sketch.* (20)

Q. 9 *(c) What information is needed to route a flood through a reservoir? Discuss the salient features of the outflow hydrograph as it differs from the inflow hydrograph.* (20)

Q. 9 *(d) "Seismic forces are taken into account in the design of solid gravity dams". If the statement is correct, explain how ?* (20)

Q. 10 *(a) Briefly discuss the limitations of unit hydrograph theory.* (10)

Q. 10 *(b) The IUH (Instantaneous Unit Hydrograph) of a drainage basin is a triangle with base 36 hours and a peak of 20 m^3/s at 8 hours from the start. Determine 2 hour unit hydrograph* (20)

[**Hint.** Instantaneous Unit Hydrograph is a unit hydrograph (containing 1 cm effective rainfall) of infinitely small duration. It is only a theoretical concept, and is explained in author's another book titled "Hydrology and Water Resources Engg" under article 9.11.

The ordinates of a I.U.H. at D hr interval can be used to work out ordinates of unit hydrograph of D hr duration, by averaging the two adjoining coordinates. Say for example, mean of cordinates of IUH at 0 hr and D hr will represent the ordinate of U.H. at D hr. Similarly, the mean of the ordinates of IUH at D hr. and 2 D hr. will give the ordinate of the U.H at 2D hr. Hence, to work out ordinates of U.H. of 2 hr duration, the ordinates of IUH at 2 hr interval are first of all written down in col (2) of the solution table (Figs worked out from shape of IUH triangle).

Solution Table for Q. 10 (*b*)

Time (*1*)	*Ordinates of IUH at 2 h interval* (*2*)	*Ordinates of 2 h Unit hydrograph (Computed by averaging)* (*3*)	
0	0	0	
2	5	2.5	(a.v of 0 + 5)
4	10	7.5	(av. of 5 and 10)
6	15	12.5	(av. of 10 and 15)
8	20	17.5	(av. of 15 and 20)
10	18.57	**19.28**	
12	17.15	17.85	
14	15.73	16.43	
16	14.30	15.01	
18	12.87	13.58	
20	11.44	11.15	
22	10.00	10.72	
24	8.57	9.28	
26	7.14	7.85	
28	5.71	6.43	
30	4.28	5.00	
32	2.85	3.56	
34	1.43	2.14	
36	0	0	

The ordinates of unit graph are finally written down in Col (3) using averaging method clarified in the table **Ans.**]

Q. 10. *(c) The following flood hydrograph was recorded in a drainage basin due to a storm having three successive 6 hour interval of rainfall magnitudes of 2.9 cm, 4.9 cm, and 3.9 cm respectively. Assume base flow = 20 m^3/s, and storm loss = 0.15 cm/hour, determine a 6 hour unit hydrograph*

Time hours	*0*	*3*	*6*	*9*	*12*	*15*	*18*	*21*
Flood graph ord (m^3/s)	*20*	*50*	*92*	*140*	*199*	*202*	*204*	*144*
Time (hours)	*24*	*27*	*30*	*33*				
Flood graph ord (m^3/s)	*84.5*	*45.5*	*29.0*	*20.0*				

[**Hint.** This question is the reverse type of solved examples like 7.38 and can be solved by assuming ordinates of 6 hr unit graph as *a, b, c, d, e, f,* etc. and then computing the flood graph caused by three successive effective rainfalls amounting to (*i*) $2.9 - 0.15 \times 6 = 2$ cm: ; (*ii*) $4.9 - 0.15 \times 6 = 4$ cm ; and (*iii*) $3.9 - 0.15 \times 6 = 3$. cm, respectively. The second rainfall to occur at 6 hr lag, and third one at 12 hr lag. The additive hydrograph ordinates will equal the flood graph ordinates *minus* base flow. The computations are given below in the table.

Solution Table for Question 10 (*c*) of Year 1995

Time (hr)	*Assumed ordinates of U.H. of 6 hr m^3/s*	*Flood graph ordinate caused by effective of rainfall of 2.9 – 0.9 = 2 cm*	*Flood graph ordinate caused by effective rainfall = 4.9 – 0.9 = 4 cm at a lag of 6 hr*	*Flood graph ordinate caused by eff. rainfall = 3.9 – 0.9 = 3 cm at a lag of 12 hr*	*Σ Total flood graph (3) + (4) + (5) = Given value of flood graph ordinate — base flow of 20 m^3/s*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	
0	0	0			$0 = 0$
3	a	$2a$			$2a = 30^*$; $a = 15$
6	b	$2b$	0		$2b = 72^{**}$; $b = 36$
9	c	$2c$	$4a$		$2c + 4e = 120$; $c = 30$
12	d	$2d$	$4b$	0	$2d + 4b = 179$; $d = 17.5$
15	e	$2e$	$4c$	$3a$	$2e + 4c + 3a = 182$; $e = 8.5$
18	f	$2f$	$4d$	$3b$	$2f + 4d + 36 = 184$; $f = 3$
21	g	$2g$	$4e$	$3c$	$2g + 4e + 3c = 124$; $g = 0$
24			$4f$	$3d$	$4f + 3d = 64.5$
27			$4g$	$3e$	$4g + 3e = 25.5$
30				$3f$	$3f = 9$
33				$3g$	$3g = 0$

* 50 – 20 = 30
** 92 – 20 = 72

The required ordinates of unit graph are thus finally worked out at 3 hr interval between 0 to 21 hrs, as :

0, 15, 36, 30, 17.5, 8.5, 3, 0 m^3/s. **Ans.**]

Q. 11. *(c) A 30 cm diameter well completely penetrates a confined aquifer of permeability 45 m/day. The length of the strainer is 20 m. Under steady state of pumping, the drawdown at the well was found to be 3.0 m, and the radius of influence was 300 m. Calculate the discharge. With increase in diameter of well by 50%, what will be the increase in % of discharge ? Derive the equation used* (30)

[**Hint.** For confined stratum, eqn. (16.20) is applicable, which states :

$$K = \frac{2.3\, Q \log_{10} \dfrac{R}{r_w}}{2\pi\, H.S}$$

where $K = 45 \text{ m/day} = \dfrac{45}{24 \times 3600} \text{ m/s}$

$$= 0.052 \times 10^{-2} \text{ m/s}$$

$$R = 300 \text{ m}$$

$$r_w = 0.15 \text{ m}$$

$$H = 20 \text{ m}$$

$$s = 3.0 \text{ m}$$

$$\therefore \quad Q = \frac{2 \cdot \pi H.S.K}{2.3 \log_{10} \frac{R}{r_w}} = \frac{2 \times 3.14 \times 20 \times 3 \times 0.052 \times 10^{-2}}{2.3 \times \log_{10} \frac{300}{0.15}}$$

$$= 0.02581 \text{ m}^3/\text{sec} = 25.81 \text{ L/sec} \quad \textbf{Ans.}$$

If well dia increases by 50%, then r_w will become = 1.5 r_w ; then

$$Q' = \frac{2\pi\, H.S.K}{2.3 \log \frac{R}{1.5\, r_w}}$$

$$\therefore \quad \frac{Q'}{Q} = \frac{\log_{10} \frac{R}{r_w}}{\log_{10} \frac{R}{1.5\, r_w}}$$

$$\text{\% age increase in discharge} = \frac{Q' - Q}{Q} \times 100 = \left(\frac{Q'}{Q} - 1\right) 100$$

$$= \left[\frac{\log_{10} \frac{300}{0.15}}{\log_{10} \frac{300}{0.15 \times 1.5}} - 1\right] 100 = \left[\frac{\log_{10} 2000}{\log_{10} \frac{2000}{1.5}} - 1\right] 100$$

$$= \left[\frac{3.3010}{3.1249} - 1\right] 100 = \mathbf{17.61\%} \quad \textbf{Ans.}]$$

Q. 11. *(b) Write short explanatory notes on :*

(i) Flood frequency analysis

(ii) Guide bunds and Launching aprons (30)

Q. 12. *(a) Design a cement plaster lined irrigation channel (n = 0.012) for 30 m³/s discharge at a slope of 22.5 cm/km. Take the side slope as $1\frac{1}{2}$ H : 1 V.*

[**Hint.** This question can be solved by following the solved example 5.4, and by assuming a limiting velocity of say 1.5 m/sec in the channel]

Q. 12. *(b) Calculate the seepage through an earth dam resting on an impervious foundation for the relevant data given below*

Height of the dam	*= 60.0 m*
Upstream slope	*= 2.75 : 1 (H : V)*
Downstream slope	*= 2.50 : 1 (H : V)*
Free board	*= 2.5 m*
Crest width	*= 8.0 m*
Length of drainage blanket	*= 120.0 m*

Coefficient of permeability of the embankment material

$$X\text{- direction} = 4.0 \times 10^{-7} \text{ m/s}$$

$$Y\text{-direction} = 1 \times 10^{-7} \text{ m/s} \qquad (20)$$

[**Hint.** Exactly similar question has appeared in Engg services 1995 Exam, which has been solved on page there itself. Students may follow through, and compute

$$q = 35.24 \times 10^{-7} \text{ m}^3/\text{sec. m}$$

$$= \mathbf{305\ l \text{ per day per m width of dam}} \quad \textbf{Ans.}$$

Q. 12. *(c) Why is it necessary to control sediment entering a canal ? Describe the working of a silt ejector giving neat sketches. On what basis is it designed ?* (20)

Year 1996 (*Civil Service*)

Q. 9. *(Compulsory) Answer any three sub divisions.*

(a) What are the causes of infertility of water-logged areas ? Derive an expression to determine the spacing of drains in open drainage system. Under what conditions this system may be recommended as an antiwater logging measure. (20)

(b) Sketch a 'runoff cycle'. Discuss direct runoff and base flow components of the cycle in details (20)

(c) Find the base width of the elementary profile of a gravity dam from the considerations of no tension, and safety in sliding. Assume reservoir full condition. (20)

(d) Define critical shear stress for a sediment bed. Discuss the use of Shield's diagram to calculate the value of critical shear stress if sediment and fluid properties are known. (20)

Q. 10. *(a) Define a unit hydrograph. Enumerate steps to develop a unit hydrograph from a given flood hydrograph for a catchment. The peak of a flood hydrograph due to a 6 hour storm is 470 m³/s. The average depth of rainfall is 8.0 cm. Assume an infiltration loss of 0.25 cm/hour and a constant base flow of 15 m³/s. Estimate the peak discharge of a 6 hour unit hydrograph for the catchment.* (35)

[**Hint.** For subjective part, please refer text. For numerical question, the effective rainfall is = 8.0 – 6 × 0.25 = 6. 5 cm. The U.H. will be obtained by dividing the ordinates a flood graph by 6.5. Hence, the peak of U.H.

$$= \frac{\text{Peak of F. graph} - \text{Base flow}}{6.5}$$

$$= \frac{470 - 15}{6.5} = 70 \text{ m}^3/\text{s}$$

Add constant base flow of 15 m³/s

∴ Peak discharge of U.H. of 6 hr duration

$$= 70 + 15 = \mathbf{85\ m^3/s} \quad \textbf{Ans.}]$$

Q. 10. *(b) How would you determine a 2 hour synthetic unit hydrograph by Snyder's method ? Discuss the situations in which this method is used* (25)

[**Hint.** Please refer article 9.9 in author's another book titled "Hydrology and Water Resources Engineering"]

Q. 11. *(a) In a frequency analysis of rainfall based on years of data of 10 minutes storm, the following values were obtained :*

Arithmetic mean of data = 1.65 cm

Standard deviation = 0.45 cm

Find using Gumbel's extreme value distribution the recurrence interval of a storm of ten minute duration and depth equal to 3 cm. (20)

[**Hint.** Use Eq. (7.142) as

$$X_{(T)} = \bar{X} + K \cdot \sigma$$

or $$3 = 1.65 + K \times 0.45$$

$$K = 3$$

Use eq. (7.148) as

$$K = \frac{y_{(T)} - 0.577}{1.2825} \text{ for } N \to \alpha$$

Since the analysed data is for a number of years, it can be presumed to be for an infinite no. of yrs ; *i.e.* $N \to \alpha$

Hence, $$3 = \frac{Y_{(T)} - 0.577}{1.2825}$$

$$Y_{(T)} = 4.4245$$

Finally use eq. (7.146) as

$$Y_{(T)} = -\left[\ln \ln \frac{T}{T-1}\right] \quad \text{or} \quad -4.4245 = \ln \ln \frac{T}{T-1}$$

or $$\frac{T}{T-1} = 1.012052 \quad \text{or} \quad \frac{T-1}{T} = 0.98809$$

or $$T = 83.97 ; \quad \text{say } \mathbf{84 \text{ yrs Ans}}]$$

Q. 11. *(b) Draw a schematic sketch of a diversion headworks with canals taking off from both sides. Indicate the functions of undersluices portion and canal head regulator.* (25)

Q. 11. *(c) Discuss the use of hydraulic jump, as an energy dissipation device in the design of hydraulic structures* (15)

Q. 12. *(a) Explain the terms :*

(i) Specific yield *(ii) Specific capacity*

A well penetrates into an unconfined aquifer having a saturated depth of 100 m. The discharge is 250 litres per minute at 12 m draw down. Assuming equilibrium flow conditions and a homogeneous aquifer, estimate the discharge at 18 m drawdown. The radius of influence may be taken as equal in both cases. Derive the formula used (30)

[**Hint.** Please refer solved example 16.3. **This question was set from this book]**

Q. 12. *(b) Write short notes on :*

(i) Piping failure of a weir floor and remedial measures

(ii) Spherical flow in a well (30)

Year 1997 (*Civil Services*)

Q. 9. *(Compulsory). Answer any three sub divisions*

(a) Explain briefly interception, transpiration and consumptive use. A 3-hour storm occurs over a 63 sq. km area. From the following data, estimate the rainfall excess for the entire area and its hourly distribution :

Sub area	*ϕ Index*	*Hourly rain*		
	cm/hr	*1st*	*2nd*	*3rd*
14.0	2.0	1.5	5.0	0.8
21.0	3.5	1.5	5.0	0.9
28.0	1.0	1.7	6.0	1.5

(20)

[**Hint.** Follow similar solved Example 7.28, and determine.

Total rainfall excess (runoff) = Σ (partial runoff volumes)

or $R = R_1 + R_2 + R_3$

$$R_1 = 14 \text{ km}^2 [0 + (5.0 - 2.0) + 0] = 42 \text{ km}^2 \cdot \text{cm}$$

$$R_2 = 21 \text{ km}^2 [0 + (5.0 - 3.5) + 0] = 31.5 \text{ km}^2 \cdot \text{cm}$$

$$R_3 = 28 \text{ km}^2 [(1.7 - 1.0) + (6.0 - 1.0) + (1.5 - 1.0)]$$

$$= 28 \text{ km}^2 [0.7 + 5 + 0.5] = 173.6 \text{ km}^2 \cdot \text{cm}$$

$$\therefore \quad R = R_1 + R_2 + R_3 = 42 + 31.5 + 173.6 = 247.1 \text{ km}^2 \cdot \text{cm}$$

∴ Total runoff as volume from 63 km^2 area = 247.1 km^2 · cm

∴ Total runoff as depth $= \dfrac{247.1}{63}$ cm = **3.92 cm** **Ans.**

Hourly values of total runoff can also similarly be computed by considering hourly values separately rather than in consolidated form as below :

$$\text{Total runoff in 1st hour} = \frac{14 \times 0 + 21 \times 0 + 28\,(1.7 - 1.0)}{63}$$

$$= \mathbf{0.31\ cm} \qquad ...(i)$$

$$\text{Total runoff in 2nd hour} = \frac{14 \times (5.0 - 2.0) + 21\,(5.0 - 3.5) + 28 \times (6.0 - 1.0)}{63}$$

$$= \frac{14 \times 3 + 21 \times 1.5 + 28 \times 5.0}{63}$$

$$= \mathbf{3.39\ cm} \qquad ...(ii)$$

$$\text{Total runoff in 3rd hour} = \frac{14 \times 0 + 21 \times 0 + 28\,(1.5 - 1.0)}{63}$$

$$= \frac{28 \times 0.5}{63} = \mathbf{0.22\ cm} \qquad ...(iii)$$

Total runoff = (*i*) + (*ii*) + (*iii*)

= 0.31 + 3.39 + 0.22 = **3.92 cm** (*O.K*) **Ans.**

(b) Write a note on the main elements of water resources development. Enumerate the steps followed in planning. (20)

(c) Explain the various causes of water logging. Describe various methods adopted as anti-water logging measures. (20)

(d) Describe in brief various types of groynes used for river training. Draw a section of a groyne. (20)

Q. 10. *(a) What do you understand by infiltration index ? How do you determine it?* (15)

(b) Describe with the help of a neat sketch any three methods of separation of base flow from the hydrograph of runoff indicating the situations under which you advocate them. (20)

(c) The following are the ordinates of a 3 hour unit hydrograph. Derive the ordinates of a 6 hour unit hydrograph and plot the same.

Time (hr.)	*3 hr. UGO (cumec)*	*Time (hr.)*	*3 hr. UGO (cumec)*
0	0	15	9.4
3	1.5	18	4.6
6	4.5	21	2.3
9	8.6	24	0.8
12	12.0	27	0.0

(25)

[**Hint.** Follow exactly similar solved Example 7.33]

Q. 11. *(a) A 30 cm well penetrates 50 m below the static watertable. After a long period of pumping at a rate of 1800 lpm, the draw-downs in the wells at 15 m and 45 m from the pumped well were 1.7 m and 0.8 m respectively. Find the transmissibility of the aquifer. What is the draw-down in the pumped well ?* (30)

[**Hint.** Follow exactly similar solved Example 16.5]

(b) Derive an expression for discharge from a well fully penetrating a confined aquifer. (15)

(c) What do you understand by 'priming' and 'depriming' ? Describe any three devices used for early priming in Saddle Siphon Spillway. (15)

Q. 12. *(a) What is berm ? Why is it provided in canals ? Using Lacey's theory, design an irrigation channel for the following data :* (30)

Discharge Q = *50 cumec*

Silt factor f = *1*

[**Hint.** Follow solved Example 4.8]

Q. 12. *(b) Write short notes on :*

(*i*) *Bed forms in alluvial channels*

(*ii*) *Silt excludes*

(*iii*) *Divide wall and fish ladder.* (30)

Year 1998 (*Civil Services*)

Q. 9. *(Compulsory). Answer any three sub divisions :*

(a) Rainfall intensities in mm/h at half-hour intervals during a 4-hr storm were : 5,

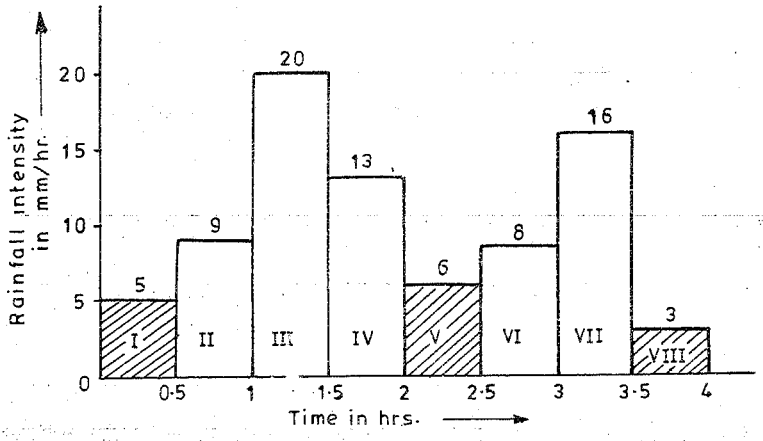

Fig. 31.12

9, 20, 13, 6, 8, 16 and 3 mm/hr. If the corresponding observed runoff is 27.45 million m^3 from a basin having an area of 1830 km^2, determine the ϕ_{index} for the storm. (20)

[**Solution.** Follow Solved Example 7.26, and compute as follows : Plot rainfall hyetograph as shown below.

$$\text{Total runoff } (Q) = \frac{27.45 \times 10^6 \text{ m}^3}{(1830 \times 10^6) \text{ m}^2} = \frac{27.45}{1830} \times 1000 \text{ mm} = \mathbf{15\ mm}$$

$$\text{Total rainfall}(P) = 0.5 \text{ hr } [5 + 9 + 20 + 13 + 6 + 8 + 16 + 3] \frac{\text{mm}}{\text{hr}}$$

$$= \mathbf{40\ mm}$$

$$W_{index} = \frac{P - Q}{t_r} = \frac{(40 - 15) \text{ mm}}{4 \text{ hr}} = \mathbf{6.25\ mm/hr}$$

Since ϕ_{index} has to be somewhat more than W_{index}, we can conclude that ϕ_{index} would be a little more than 6.25 mm/hr. When this is so, evidently, the Ist, Vth and VIIIth (half hr) rainfalls would become ineffective in producing excess rain, because the rain intensity in those periods would be less than ϕ_{index}.

In other words, the period (t_e) during which excess rain occurs, would be only $4 - 1.5 = 2.5$ hr. In that case, using

Eq. (7.102), we have

$$\phi_{index} = \frac{\text{Total in filtration during the period of excess rainfall}}{\text{Period of excess rainfall}}$$

$$= \frac{\text{Total infiltration} - \text{Infiltration during the time when no excess rain occurs}}{t_e}$$

$$= \frac{25 \text{ mm} - 0.5\,[5 + 6 + 3]}{2.5 \text{ hr}}$$

$$\therefore \phi_{index} = \frac{25 - 0.5 \times 14}{2.5} = \frac{25 - 7}{2.5} = \frac{18}{2.5} = 7.2 \text{ mm/hr.}$$

With this computed value of ϕ_{index}, no other period excepts the periods marked as II, VI & VIII (already accounted) become infructous to yield excess rain. Hence, the computations done do not need further revision. Hence, ϕ_{index} = **7.2 mm/hr Ans.**

Q. 9. *(b) Distinguish between*

(i) Drainage divide and ground water divide

(ii) Consumptive and non-consumptive uses of water (20)

Q. 9. *(c) Briefly explain water-logging and its ill-effects.* (20)

Q. 9. *(d) Draw a sketch of a composite earth dam showing its components. Also state the functions of these components.* (20)

Q. 10. *(a) During a recuperation test, the water level in an open well depressed by 2.5 m during pumping and it recuperated 2 m in 100 minutes. Find :*

(i) Yield from a well of 4 m diameter under a depression head of 2.4 m; and

(ii) Diameter of a well to yield 12 l/s under a depression head of 2.4 m. (20)

[**Hint.** Follow exactly similar Solved Example 16.10]

Q. 10. *(b) Describe the zone method of design of high gravity dams.* (20)

Q. 10. *(c) What is a silt excluder ? Explain its working with the help of neat sketches.* (20)

Q. 11. *(a) The ordinates of a 3-hr unit hydrograph are as follows :*

Time (hr.)	*3-hr unit graph ordinates (m^3/s)*
0	0
3	3
6	8
9	15
12	13
15	10
18	6
21	2
24	0

Assuming a constant base flow of 15 m^3/s and an average storm loss of 0.20 cm/hr, obtain the ordinates of the resultant hydrograph for three successive storms of 3.6, 6.0 and 4.2 cm, each of 3 hours duration. (20)

[**Hint.** Follow exactly similar Solved Example 7.38]

(b) Describe various methods used for energy dissipation below spillways. (20)

(c) Compare Kennedy's and Lacey's silt theories. Which of them would you prefer for designing an irrigation channel ? Why ? (20)

Q. 12. *(a) Explain the terms 'duty' and 'delta'. State the different factors that affect duty.* (20)

(b) Draw a neat sketch of a syphon aqueduct and state the conditions under which it is recommended. (20)

Q. 12. *(c) Write short notes on :*

(i) Reservoir sedimentation ; and (ii) River training. (20)

Year 1999 (*Civil Services*)

Questions 9 is compulsory

Q. 9. Answer any three sub-divisions

(*a*) *Define a flood plain. Briefly describe and discuss structural and non-structural measures of flood control.* (20)

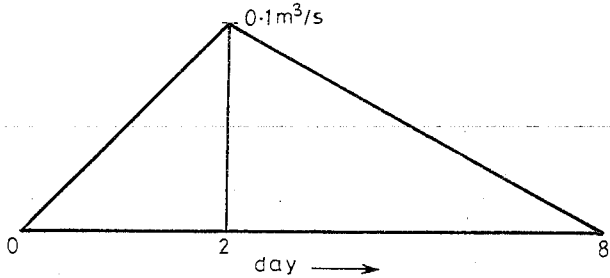

Fig. 31.13

(*b*) *Define a unit hydrograph. What are its assumptions ?*

The unit hydrograph of a drainage basin for unit duration of 1 hour and effective rainfall of 1 cm/hour is given in the form of a triangle with the base as 8 days and peak flow of 0.1 m³/sec occurring on the second day. Estimate the area of the drainage basin.

[**Solution.** Area of unit hydrograph = 1 cm of rain on catchment of A sq. m. .

$$\text{Area of U. graph} = \frac{1}{2} \times 0.1 \frac{m^3}{s} \times 8 \text{ days}$$

$$= \frac{1}{2} \times 0.1 \frac{m^3}{s} \times (8 \times 24 \times 60 \times 60) \,.\, s$$

$$= \frac{1}{2} \times 0.1 \times 8 \times 24 \times 3600 \text{ m}^3$$

or $$\frac{1}{2} \times 0.1 \times 8 \times 24 \times 3600 \text{ m}^3 = \left(\frac{1}{100} \text{ m}\right) \times A \text{ (m}^2\text{)}$$

or $$A = 345.6 \times 10^4 \text{ m}^2 = 345.6 \text{ ha}$$ **Ans.**]

(*c*) *What are reservoir routing and channel routing ? What are the shortcomings of the Muskingum method of flood routing ?* (20)

[**Hint :** Please refer chapter on Flood Forecasting in "Hydrology and water Resources" by the same author.]

(*d*) *What are the important features for the design of cross-drainage works ? Sketch a typical siphon aqueduct.* (20)

Q. 10. (*a*) (*i*) *State and discuss the various factors affecting run off from a drainage basin.*

(*ii*) *How will you test the consistency of the rainfall record of a station, if the annual precipitation data of a group of stations including the one in question are available for a long period ?* (20)

Q. 10. (*b*) *At a certain point in an unconfined aquifer of 4 km² area, the water-table was at an elevation of 120.0 m. Due to natural recharge in a wet season, its level rose to 121.2 m A volume of 2 million cubic metres of water was then pumped out of the aquifer, causing the water-table to reach a level of 119.2 m. Assuming the water-table in the entire aquifer to correspond in a similar way, estimate*

(*i*) *the specific yield of the aquifer; and*

(*ii*) *volume of recharge during the wet season.* (20)

[**Solution.** Water pumped in lowering W.T. by 121.2 – 119.2 = 2 m is 2M.m³.

Vol of Aquifer drained in lowering W.T. by 2m

$$= \text{Area} \times 2 \text{ m depth} = (4 \times 10^6 \text{ m}^2) \times 2 \text{ m} = 8 \text{ M.m}^3$$

(*i*) Sp. yield of the aquifer (Eq. 16.8)

$$= \frac{\text{Vol. of water drained by gravity drainage}}{\text{Total vol. of aquifer drained}}$$

$$= \frac{2 \text{ M.m}^3}{8 \text{ M.m}^3} = 0.25 = \mathbf{25\%} \quad \textbf{Ans.}$$

(*ii*) During rainy season, the W.T. rose by 121.2 – 120 = 1.2 m. Since 2m lowering of W.T. equals 2 M.m³ of water withdrawal, 1.2 m rise will be equal to **1.2 M.m³ of recharge. Ans.**]

Note. This question was earlier set at AMIE Exam., 1994 (Summer) and its solution was available in Author's another book titled "Hydrology and Water Resources Engg."

Q. 10. (*c*) *Briefly discuss different methods of economic evaluation of water resources projects. What is meant by project-induced economic growth ?* (20)

Q. 11. (*a*) *State the method of images as used in the analysis of ground-water flow.*

A 0.5 m diameter well fully penetrates a 30 m thick unconfined aquifer of hydraulic conductivity 20 m/day. Due to continuous pumping, if a drawdown of 1.0 m is registered in the well, what is the rate of pumping when the well is located at a distance of 50 m from a perennial stream ?

[**Hint.** Pl refer solved example 16.9] (30)

Q. 11. (*b*) *What are the factors affecting the evaporation process ? Briefly describe the Penman approach of estimating reservoir evaporation.* (15)

Q. 11. (*c*) *What is the purpose of providing drainage in an irrigated system ? What are the effects of poor drainage on soil and plants.* (15)

Q. 12. (*a*) *Enumerate the various forces acting on a gravity dam. Also indicate their points of application. Why are construction joints provided in a dam ?* (20)

Q. 12. (*b*) *Briefly describe and discuss the various methods of lining irrigation canals. Give a cross-section of a lined canal.* (20)

Q. 12. (*c*) *Suggest a suitable device for energy dissipation at the toe of a spillway with the following data :*

Discharge per unit width	= *10 cumecs.*
Depth of water at the toe	= *80 cm*
Margin of safety to be adopted against sweep out	= *7%*
Tail water depth	= *6.0 m* (20)

[**Hint.** Here $y_1 = 0.8$ m ; T.W.D. = 6 m ; $q = 10$ m^3/s

Using Eq. (10.4) as :

$$y_2 = -\frac{y_1}{2} + \sqrt{\frac{y_1^2}{4} + \frac{2q^2}{gy_1}}, \text{ we get}$$

$$y_2 = -\frac{0.8}{2} + \sqrt{\frac{0.8^2}{4} + \frac{2 \times 100}{9.81 \times 0.8}} = 4.664 \text{ m.}$$

With 7% margin for sweep out caused by jumping water on tail water, y_2 becomes equal to $1.07 \times 4.664 = 4.99$ m.

Since tail water depth = 6m, is much more than $y_2 = 4.99$ m, the jump to be formed at the toe of the spillway shall be drowned out by tail water, and proper energy dissipation will not occur.

Hence, the point of jump formation will have to be raised by providing a *sloping glacis* as shown in fig. 21.31 (b_1) - allowing the jump to form on the apron at RL above the river bed by as much as TWD–$y_2 = 6 - 4.99 = 1.01$ m. Alternatively a *roller bucket* as shown in Fig. 21.31 (b_2) may be provided for energy dissipation. **Ans.**]

Year 2000 (*Civil Services*)

(Max. 3 Questions can be solved from this section)

Section B

Q. 5. *is compulsory in five parts :*

Q. 5(*a*). *Briefly discuss the factors affecting run off from a drainage basin* (12)

Q. 5(*b*). *Describe the chemical constituents which affect the suitability of water for irrigation.* (12)

Q. 5(*c*). *How will you estimate the benefits of*

(*i*) *on irrigation project ; and* (*ii*) *a flood control project* (12)

Q. 6.(*a*). *Define storage coefficient and transmissibility coefficient of an aquifer. A well is 30 cm in diameter and fully penetrates 50 m below the static water-table. After 36 hours of pumping at a steady state rate of 4 m^3/minute the water level in a test well 200 m apart is lowered by 1.2 m; and in a well 40 m away, the draw down is 2.7 m. What is the transmissibility of the aquifer ?* (20)

[**Hint.** Pl. follow exactly similar Solved Question 16.5.]

Q. 6(*c*). *Briefly discuss the causes of water logging in irrigated areas. Describe the procedure for the design of the drainage for water-logged soils.* (20)

Q. 7(*a*). *Describe methods of measuring rainfall and snowfall of a basin. How is consistency of rainfall data determined.* (20)

Q. 7(c). *Describe salient features of meandering of rivers in alluvial plains. Describe suitable methods of training such rivers.* (20)

Q. 8(a). *Briefly describe Lacey's theory of transportation of silt in channels*

Design an irrigation channel in alluvial soil according to Lacey's Regime theory. The following data are given :

Full supply discharge = 15 cumecs *Lacey's silt factor = 1.0 ;*

Channel side slopes = 1/2 : 1. (20)

[**Hint.** Pl refer exactly *Similar Solved Example* 4.9]

Q. 8(c). *Briefly discuss various methods of determining flood frequency analyses. Develop a computer program to determine flood frequency by any one method.* (20)

Year 2001 (*Civil Services*)

Section B. (*Max. 3 questions can be solved from this section*)

Q. 5. *is compulsory in five parts from Irr. & Env. Engg. side.*

Q. 5 (a). *What is the difference between a 'confined aquifer' and an 'unconfined aquifer' ? Define and describe Darcy's law of flow of water through permeable soils.* (12)

Q. 5 (b). *What is the water requirements of crops ? What is the meaning of the terms 'Duty' and 'Delta' in connection with crop water requirements ? What are the factors affecting duty ? What is average 'delta value' in millimetres for sugarcane ?* (12)

Q. 5 (c). *What are the various types of cross drainage works ? What is the purpose of these C. D. Works ? Describe the use of syphon in cross drainage works.* (12)

Q. 6 (a). *What are the various surface water sources used for irrigation ? How is the storage capacity of a large reservoir fed by a river for a large irrigation project determined ?* (20)

Q. 6 (b). *For reducing seepage of water through the body of an earthen dam, what provisions are made ? What is piping in earthen dams ? How is it prevented ?* (20)

Q. 6 (c). *Discuss hydrological cycle and water budget. How can we determine yield from a catchment for various uses from natural rainfall ?* (20)

Year 2002 (Civil Services - Paper II)

Section - B. *(Maximum of 3 Questions can be solved from this section as against a total of 5 Questions).*

Q5 is compulsory in five parts from this section from Irrigation and Environment Engg side.

Q5. (a) *List the different types of canal lining in common use. Draw a neat sketch of a typical cross-section of a canal carrying a discharge of 60 m^3/s and lined with brick in cement mortar. Mark the salient features on the sketch.* (12)

Q5. (b) (i) *What are the basic premises of unit hydrograph theory ? What are the limitations of unit hydrographs ?*

(ii) A 6-hour unit hydrograph is triangular in shape and has a peak ordinate of 10 m^3/s. The base is 72 hours with 12 hours of time up to the peak. What is the area of the catchment represented by this unit hydrograph. (12)

[**Hint :** Vol. of water contained in $U \cdot H$

$$(1 \text{ cm})\, A\,(m^2) = \frac{1}{2} \times 72\, hrs \times 10 \frac{m^3}{s}$$ where A is catchment area in m^2.

or $$\left(\frac{1}{100}\text{m}\right) A\,(m^2) = \frac{1}{2} \times (72 \times 3600)\, s \times 10 \frac{m^3}{s}$$

or $$A = (36 \times 36000 \times 100)\, m^2 = 12960 \times 10^4\, m^2 = 12960 \text{ hectares}$$ Ans.]

Q5. (*c*) *In Khosla's theory, how is the exit gradient of a weir in permeable foundation estimated ? Using the theory, how can the factor of safety of a weir design against piping be estimated ?* (12)

[**Hint :** Exit gradient at downstream end of a weir having depth of d/s cut off $= d$ is given by equation (11.4) as :

$$G_E = \frac{H}{d} \frac{1}{\pi \sqrt{\lambda}}$$

where $$\lambda = \frac{1 + \sqrt{1 + \alpha^2}}{2} \text{ and } \alpha = \frac{b}{d}\ ;$$ where b = Total floor length

$$F \cdot S = \frac{1}{\text{Exit gradient}}.$$

Q5. (*f*) *Write the Shield's curve for incipient motion of sediment. Mark clearly the ordinate and abscissa of this curve and level the salient features.*

[**Hint :** Please refer article 4.5 and Fig. 4.4]

Q6. (*b*) *In the context of sediment transport in alluvial channels, describe briefly the following :*

(*i*) *Bed load,* (*ii*) *Bed forms and* (*iii*) *Regime channel* (20)

Q6. (*c*) *List the different methods of application of irrigation water to farm land. In a system which relies on supply of water through field channels, indicate methods of achieving high efficiency of water use.* (20)

Q7. (*a*) (*i*) *Write the Rouse's equation for distribution of suspended sediment in the vertical of a channel and label all the terms.*

In a wide alluvial channel having suspended sediment load, the depth of flow is 2.8 m. If a suspended load sampler indicated a concentration of 700 ppm at a point 30 cm below water surface, estimate the concentration at a point 10 cm above the bed in the same vertical. The exponent in Rouse's equation can be assumed as 0.4. (20)

[**Solution.** Rouse's equation is given by equation (4.40) as :

$$\frac{c}{c_a} = \left[\frac{a\,(D - y}{y\,(D - a)}\right]^{\frac{w_o}{KV*}} \qquad \text{...(Eqn. 4.40)}$$

In the given question, the exponent $\frac{w_o}{KV*}$ is given as equal to 0.4.

Hence, the equation becomes :

$$\frac{c}{c_a} = \left[\frac{a\,(D - y}{y\,(D - a)}\right]^{0.4}$$

where c_a is known concentration at a distance 'a' from the bed c is concentration at distance y from load.

Here $D = 2.8$ m.

$c_a = 700$ ppm, $a = (2.8 - 0.3) = 2.5$ m

$c = ?$ $y = 0.1$ m

$$\therefore \quad \frac{c}{700 \text{ ppm}} = \left[\frac{2.5\,(2.8-0.1)}{0.1\,(2.8-2.5)}\right]^{0.4} = \left[\frac{2.5 \times 2.7}{0.1 \times 0.3}\right]^{0.4} = (225)^{0.4} = 8.727$$

or $c = 6109$ ppm Ans.

Q7. (*c*) *Explain the difference between hydraulic routing and hydrological routing of a flood. Describe method of routing of flood wave through a reservoir with uncontrolled spillway outlet ?* (20)

[**Hint :** Please see article 18.10].

Q8. (*a*) *Describe the principle used in the development of Ogee spillway profile. Describe the discharge characteristics of Ogee spillways with vertical upstream face under free flow condition.* (20)

[**Hint :** Please see articles 21.6.2 & 21.6.4 to 21.6.6.]

Q8. (*c*) *Discuss the problem of reservoir sedimentation ? What is the impact of sedimentation on the performance of a multipurpose reservoir ? What remedial measures can be adopted to reduce the rate of siltation ?* (20)

[**Hint :** Please see article 18.12 & 18.12.5.]

Year 2003 (*Civil Services Paper II*)

Q5. (*a*) **Compulsory :** *Briefly state the various steps needed for planning an irrigation project.* (12)

Q5. (*b*) *List the objectives of various of water resources development in the context of the lesser developed countries.* (12)

Q5. (*c*) *Briefly describe the basic methods of applying irrigation water to fields describing the situations when each method is useful.* (12)

Q6. (*a*) *In a lake of 70.8 ha surface, the inflow was 1.5 m^3/s in a certain 30 day month. The dam on the lake regulated the outflow (discharge) from the lake to 1.25 m^3/s in that month. If the recorded precipitation in that month was 7.62 cm and the storage volume increased by an estimated 6,50,000 m^3, what is the estimated evaporation in m^3 and cm ? Assume that no water infiltrates out of the bottom (and sides) of that lake.* (20)

[**Hint.** Inflow = 1.5 m^3/s ; Outflow = 1.25 m^3/s

Increase in storage (rate) = 0.25 m^3/s

Increase in storage in the month of 30 days

$$= 0.25 \times (30 \times 24 \times 3600)\ m^3 = \mathbf{6{,}48{,}000\ m^3}.$$

Precipitation = 7.62 cm

Precipitation volume = 7.62 cm × Lake area of 70.8 ha

$$= \frac{7.62}{100}\ m \times (70.8 \times 10^4\ m^2) = \mathbf{53{,}950\ m^3}.$$

Total gross storage $= 6{,}48{,}000 + 53{,}950 = 7{,}01{,}950\ m^3$.

Actual net increase in storage = 6,50,000 cum

Total loss of water volume from the lake

$$= 7{,}01{,}950 - 6{,}50{,}000 = 51{,}950 \text{ cum}$$

Since loss due to infiltration is Nil, the total loss is the loss due to evaporation

= 51,950 cum. Ans.]

Q7. (*a*) *Assuming the initial infiltration rate of 10 mm/h and final infiltration rate of 5 mm/h and the constant value (describing the rate of decay of the difference between the initial and final infiltration rates) as 0.95 h^{-1}, calculate the total infiltration depth for a storm lasting 6 h.* (20)

[**Hint.** Please refer Solved Example 7.13 (*a*).]

Q8. (*a*) (*iii*) *Briefly state the criteria for selection of gravity dam site.* (10)

Year 2004 (*Civil Services Paper II*)

Q5. (*a*) *Why are canal falls provided in irrigation channels ? With the help of sketches, describe and illustrate a trapezoidal notch fall, labelling all the salient features and their functions in flow management and safety.* (12)

(*b*) *What is meant by river training ? What are its objectives ? What are the in-situ features that help in this context ?* (12)

(*c*) *Estimate the depth and frequency of irrigation required for a certain crop, given :*

Root zone depth	:	*90 cm*
Field capacity	:	*22%*
Wilting point	:	*12%*
Apparent sp. gr. of soil	:	*1.5*
Consumptive use	:	*22 mm/day*
Efficiency of irrigation	:	*60%*

Assume 50% depletion of available moisture as the indiator to begin application of irrigation water.

[**Solution.** Please follow similar solved example 2.14 and compute as :

Available moisture = Field capacity m.c. – Wilting point m.c.

$$= 22\% - 12\% = 10\%$$

Since irrigation water is to be supplied on 50% depletion of available moisture, we have

Readily available moisture = 50% × 10% = 5%

It means that irrigation water will be supplied as soon as m.c. falls to 17% from 22%. Thus, m.c. will be allowed to vary between 22% to 17%.

Depth of irrigation water required to raise m.c. from 17% to 22%

$$= \frac{\gamma_d \cdot d}{\gamma_w}\left[\text{Field capacity m.c.} - \text{Optimum m.c.}\right]$$

$$= \frac{\gamma_w \cdot G_a^* \cdot d}{\gamma_w}\left[22\% - 17\%\right]^*$$

d = depth of root zone = 90 cm

$$= 1.5 \times 90 \text{ cm } [5\%] = \frac{1.5 \times 90 \times 5}{100} = \mathbf{6.75\ cm}$$

Water depth reqd. in the field

$$= \frac{6.75 \text{ cm}}{0.6} = 11.25 \text{ cm.} \quad \textbf{Ans.}$$

Depth of water available to plants for evapotranspiration = 6.75 cm

Consumptive use = 2.2 cm/day

2.2 cm of water depth is utilised by plants in = 1 day

$\therefore$ 6.75 cm of water depth will be utilised by plants in $= \frac{1}{2.2} \times 8.75$ day

= 3.07 days.

Hence, 11.25 cm water depth will be supplied to fields at every 3 days, which represents the irrigation frequency. **Ans.**

Q6. (*a*) *For a station 1, the recorded annual 24-hr maximum rainfall are given below. Compute the necessary data to estimate : (a) 24 hr maximum rainfall with a specified return period ; and (b) probability of a rainfall of not less than a specified depth occurring in 24 hours at station 1. Indicate the above features on a sketch (not necessarily plotted to exact data) prescribing the coordinate parameters.*

1 Year	1960	61	62	63	64	65	66	67	68	69
2 Rainfall, cm	14.2	13.1	7.5	14.9	12.4	14.2	7.0	16.1	8.3	9.2

1 Year	70	71	72	73	74	75	76	77	78	79	80
2 Rainfall cm	7.0	13.4	7.5	14.0	15.2	6.5	9.7	13.1	8.8	14.7	11.2

Solution. Please follow similar solved example 7.9 and compute Recurrence interval (T) values for all the given precipitations arranged in decreasing order, as shown in table below, using Eq. $T = \frac{N}{m}$, where N = Total no. of years of record = 21, and m is the ranking of storm :

* G_a is apparent sp gravity, which is the sp gravity of dry soil.

Solution Table for Q6 (a) of Year 2004

Year	*Maximum 24 hr rainfall in cm (arranged in decreasing order)*	*Ranking of storm (m)*	*Recurrence interval (T) in yrs.* $T = \frac{N}{m} = \frac{21}{col.\ (3)}$
(1)	*(2)*	*(3)*	*(4)*
1967	16.1	1	21.00
1974	15.2	2	10.5
1963	14.9	3	7.00
1979	14.7	4	5.25
1960	14.2	5	4.20
1965	14.2	6	3.50
1973	14.0	7	3.00
1971	13.4	8	2.63
1977	13.1	9	2.33
1961	13.1	10	2.10
1964	12.4	11	1.91
1980	11.2	12	1.75
1976	9.7	13	1.62
1969	9.2	14	1.50
1978	8.8	15	1.40
1968	8.3	16	1.31
1972	7.5	17	1.24
1962	7.5	18	1.17
1970	7.0	19	1.11
1966	7.0	20	1.05
1975	6.5	21	1.00

A graph can now be plotted between values of T (on log scale on x-axis) and values of rainfall (cm) on log scale on Y-axis using log-log graph paper. (Refer Fig. 7.30).

(*b*) To find probability of a rainfall of not less than a specified depth, say D cm, we can read out Recurrence interval (T) value for the given rainfall depth from the plotted graph. The inverse of this T value will give the required probability value p $\left(\because\ p = \frac{1}{T}\right)$.]

Q6. (b) *Draw a schematic diagram (in plan view) of a barrage on an alluvial river, with provision for overflow and fish passage, for diversion of flow into a main canal, including for sediment extraction. On the diagram, label the several features and the schematic flow paths.*

Q7. (a) *Ordinates of a 4-hr unit hydrograph (u.h.g.) on a catchment are tabulated. Derive the ordinates of a 2-hr u.h.g. for the same catchment.*

Time (hr)	0	4	8	12	16	20	24	28
Ordinate of 4-hr u.h.g. (m^3/s)	0	23	54	60	25	14	6	4

(20)

[**Hint.** Please follow similar solved example 7.52.]

Q8. (a) *In a subsurface pipe drainage system, it is desired to keep the highest level of the water table at 1.5 m below the ground surface. The depth of impervious layer from land surface is 10.0 m and the depth of the drain below the land surface is 2.0 m. The mean annual rainfall in the area is 96 cm ; and the coefficient of permeability is 6 × 10⁻⁶ m/s. Design the spacing the the drain pipes.* (20)

Solution. Please follow similar solved example 6.2 and compute as under :

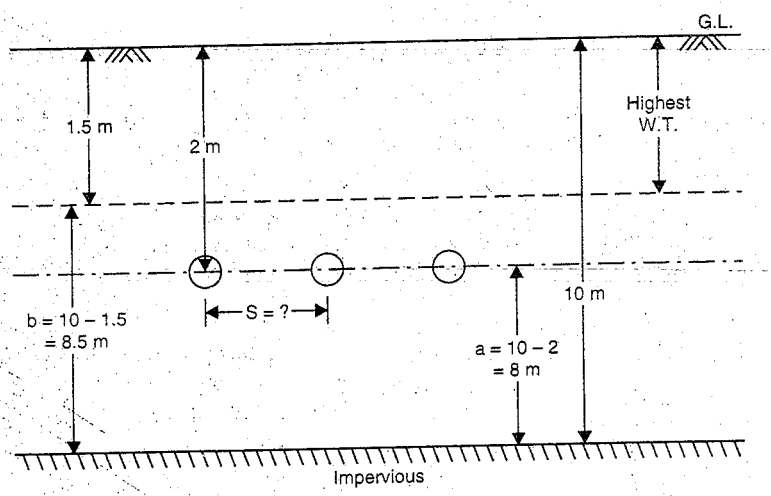

Fig. 31.14

From Fig. 31.14, we have

$$a = 10 - 2 = 8 \text{ m}$$

$$b = 10 - 1.5 = 8.5 \text{ m}$$

$$K = 6 \times 10^{-6} \text{ m/s}$$

$$P_{AA} = \text{Annual rainfall} = 96 \text{ cm} = 0.96 \text{ m}$$

$$S = ?$$

We have to use eqn. (6.9) as :

$$q = \frac{4K}{S}\left[b^2 - a^2\right]$$

Where q is the infiltration discharge into the ground, which should be removed by the drains in m³/sec.

Assuming that 1% of the average annual rainfall of the considered place, shall be removed by the tile drains in 24 hours (a common assumption), we get

$$q = \frac{P_{AA} \times 1\%}{24 \times 60 \times 60} \times (S \times 1) \text{ cumec/m length of drains}$$

or

$$q = \frac{0.01 \times 0.96 \text{ m}}{24 \times 60 \times 60} \times \left[S \times 1\right] \text{ m}^3\text{/s per m length of drains} \quad ...(i)$$

Also

$$q = \frac{4 \times 6 \times 10^{-6}}{S}(8.5^2 - 8^2) \quad ...(ii)$$

Equating (*i*) and (*ii*), we get

$$\frac{4 \times 6 \times 10^{-6} \times 8.25}{S} = \frac{0.01 \times 0.96 \times S}{24 \times 3600}$$

or $$S^2 = \frac{4 \times 6 \times 10^{-6} \times 8.25 \times 24 \times 3600}{0.01 \times 0.96} = 1782 \text{ m}^2$$

or $$S = \mathbf{41.21\ m.}\quad \textbf{Ans.}$$

Q. 8. (b) (i) In single as well as multiple purpose water resources projects, how are benefits classified ? Give brief explanation for each ; also indicate the distinction between "with/without" and "before/after" considerations.

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Objective Questions of Engineering Services Exams.

Year 1993 (*Objective Paper II of Engg. Services*)*

Q. 1. The ordinates of a 3-hour hydrograph for a small catchment are given below:

Time (hour)	*Unit graph ordinates (m^3/s)*
0	0
3	1.5
6	4.5
9	18.6
12	12.0
15	9.4
18	4.6
21	2.3
24	0.8
27	0

If the design storm produces net rainfall depths of 4.6 cm and 3.5 cm in successive unit periods, and if the base flow is 2 m^3/s, then the peak flood flow (in m^3/s) will nearly be :

(*a*) 270 (*b*) 130

(*c*) 90 (*d*) 86.

Q. 2. A 4-hour rainfall in a catchment of 250 km^2 produces rainfall depths of 6.2 cm and 5.0 cm in successive 2-hour unit periods. Assuming the ϕ-index of the soil to be 1.2 cm/hour, the runoff volume in ham will be:

(*a*) 16 (*b*) 22

(*c*) 1600 (*d*) 2200.

* U.P.S.C. has, with effect from the year 1993, allowed students to take back home the Objective Papers, which it was not allowing earlier.

Q. 3. *A 3-hour unit hydrograph (UH) differs from a 6-hour U.H., for any catchment, in that :*

(*a*) both the time of rise and the peak ordinate for the former are less than that for the latter

(*b*) both the time of rise and the peak ordinate for the former are greater that that of the latter

(*c*) the time of rise is less but the peak ordinate is greater for the former as compared to that of the latter

(*d*) the time of rise is greater but the peak ordinate is less for the former as compared to that of the latter.

Q. 4. *As per the recommendations of the ISB (NBS), the shape of a lined canal is:*

(*a*) trapezoidal (*b*) semicircular
(*c*) parabolic (*d*) elliptic.

Q. 5. *Which of the following pairs are correctly matched ?*

1.	Device to receive and eject drainage from canal	inlet and outlet
2.	Cross-drainage structure when canal bed level and drain bed level are same	canal siphon
3.	Cross-masonary work to facilitate road transport.	bridge

Codes :

(*a*) 1, 2 and 3 (*b*) 1 and 3
(*c*) 2 and 3 (*d*) 1 and 2.

Q. 6. Application efficiency of irrigation is best defined as the ratio of:

(*a*) water for normal consumptive requirement to water depleted from the root zone

(*b*) water actually stored in the root zone to water delivered to the farm

(*c*) water reaching the farm to water released from the head works

(*d*) water actually delivered to the farm to water actually reaching the farm.

Q. 7*. Match List I with List II and select the correct answer using the codes given below the lists :

List I	*List II*
A. Contour bunding	1. High rainfall
B. Bench terrace with outward slope	2. Low rainfall
C. Land slope = 100%	3. 45°
D. Drop inlet spillway	4. 90°
	5. Gully control.

Codes :

	A	*B*	*C*	*D*
(*a*)	1	2	3	4
(*b*)	3	5	2	1
(*c*)	2	1	3	5
(*d*)	3	4	5	1.

ANSWERS

1. (*b*) 2. (*c*) 3. (*c*) 4. (*a*)
5. (*b*) 6. (*b*) 7. (*c*).

Year 1994 *(Objective Paper II of Engineering Services)*

Q. 1. Consider the following statements regarding confined aquifers :

1. The aquifer is bound at top and below by impervious strata
2. The pressure of water is greater than atmospheric pressure
3. A tubewell sunk in such an aquifer starts flowing always by itself.
4. The aquifer is fully saturated .

Of the above statements :

(*a*) 1, 2 and 3 are correct (*b*) 2, 3 and 4 are correct
(*c*) 1, 2 and 4 are correct (*d*) 1, 3 and 4 are correct.

Q. 2. The given diagram I shows the curve of a hydrograph. Which of the following would cause the peak of the curve to shift to the right ?

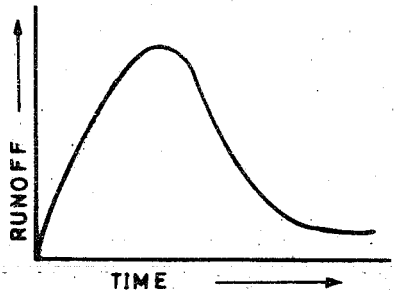

Diagram I. (Fig. 32.1)

(1) When the length of the overland flow is more
(2) When the slope of the land surface is less
(3) When the runoff is more
(4) When the rainfall is moderate.

Select the correct answer using the codes given below :

Codes :

(*a*) 3 and 4 (*b*) 2 and 4
(*c*) 1 and 2 (*d*) 2, 3 and 4.

Q. 3. A tropical cyclone in the northern hemisphere is a zone of :

(*a*) low pressure with clockwise wind
(*b*) low pressure with anticlockwise wind
(*c*) high pressure with clockwise wind
(*d*) high pressure with anticlockwise wind.

Q. 4. If one wants to be 90% sure that the design flood in a dam project will not occur during the design life period of 100 years, the recurrence interval for such a flood would be :

(*a*) about 90 years (*b*) equal to 100 years

(*c*) about 110 years (*d*) roughly 1000 years.

Q. 5.* For the estimation of a flood with a return period of T years by Gumbel's method, which of the following sets of data regarding the annual flood series would be required ?

(*a*) Mean and Standard deviation (SD) of the flood flow series

(*b*) Mean flood flow, SD of the flood flow series, and the Length of record

(*c*) SD of the flood flow series and Length of record

(*d*) Mean, SD, and coefficient of skew of the flood flow series.

Q. 6.* A channel designed by Lacey's theory has a mean velocity of one m/s. The silt factor is unity. The hydraulic mean radius will be :

(*a*) 2.5 m (*b*) 2 m

(*c*) 1 m (*d*) 0.5 m.

Q. 7. If D = scour depth measured below high flood level, and d = depth of scour below the original bed level, then the width of the launching apron at toe of guide bank will generally be :

(*a*) 1.5*D* (*b*) 1.5*d*

(*c*) 2.0 *D* (*d*) 2.0 *d*.

Q. 8. Match list I with list II and select the correct answer using the codes given below the lists :

List I	*List II*
A. Land capability	1. Drip irrigation
B. Sandy soil	2. Parshall flume
C. Consumptive use of a crop	3. Contour strip cropping
D. Water flow measurement	4. Blanly Criddle formula
	5. U.S. Bureau of reclamation classification.

Codes :

	A	*B*	*C*	*D*
(*a*)	5	1	4	2
(*b*)	1	2	4	3
(*c*)	5	3	1	2
(*d*)	1	2	5	3

ANSWERS

1. (*c*)	**2.** (*c*)	**3.** (*b*)	**4.** (*d*)
5. (*d*)	**6.** (*a*)	**7.** (*b*)	**8.** (*a*)

Hints for Solving Starred Questions of Year 1994

Q. 5. Gumbel's method uses the eqn :

$$Q_p = \overline{Q}_p + k\,\sigma$$

where $\overline{Q}_p$ = Mean flood flow

k = Coefficient of skew

σ = Standard Deviation.

Hence for estimation of flood, we need Mean flood flows, S.D. (Standard Deviation) and Coefficient of skew (k). Hence, choice (d) is correct.

Q. 6. Use eqn. 4.15 as :

$$R = \frac{5}{2}\left(\frac{v^2}{f}\right) = \frac{5}{2}\left(\frac{1^2}{1}\right)$$

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

Hence choice (a) is correct. **Ans.**

Year 1995 *(Objective Paper II of Engineering Services)*

Q. 1. Which one of the following constitutes the basic assumption of the unit hydrograph theory :

(a) Non-linear response and time invariable.

(b) Non-linear time variance and linear response

(c) Linear response and linear time variance

(d) Time invariance and linear response.

Q. 2. The following parameters relate to the design of weirs of permeable foundations

1. Scour depth
2. Exit gradient
3. Uplift pressure
4. Unbalanced head.

Design of the downstream end pile of the weir depends upon :

(a) 1 and 2 (b) 1 and 4

(c) 2 and 3 (d) 3 and 4.

Q. 3. Given that the base period is 100 days and the duty of the canal is 1000 hectares per cumec, the depth of water will be

(a) 0.864 cm (b) 8.64 cm

(c) 86.4 cm (d) 864 cm.

Q. 4. The most economical method of soil conservation is to :

(a) construct check dams (b) construct contour bunds

(c) drain the soil (d) aforest the soil.

Q. 5. A pumped storage plant is a

(a) high head plant (b) runoff river plant

(c) peak load plant (d) base load plant.

ANSWERS

1. (*d*) **2.** (*a*) **3.** (*c*) **4.** (*d*) **5.** (*c*).

Year 1996 (*Objective Paper II of Engineering Services*)

Q. 1. The percentage standard error of precipitation averages is often expressed functionally or graphically in terms of (*i*) precipitation gauge network density expressed as area per gauge ; and (*ii*) total area of catchment. The percentage standard error

(*a*) increases with area per gauge as well as with total area
(*b*) decreases with area per gauge as well as with total area.
(*c*) increases with area per gauge but decreases with total area.
(*d*) decreases with area per gauge but increases with total area.

Q. 2. If '*p*' the precipitation, '*a*' is the area represented by a raingauge, and *n* is the number of rainguages in the catchment area, then the weighted mean rainfall is :

(*a*) $\frac{\Sigma ap^3}{\Sigma a^2}$ (*b*) $\frac{\Sigma ap}{n}$

(*c*) $\frac{\Sigma ap}{\Sigma a}$ (*d*) $\frac{\Sigma ap^5}{\Sigma a^3}$

Q. 3. Depth-Area-Duration curves of precipitation are drawn as

(*a*) Minimising envelopes through the appropriate data points.
(*b*) Maximising envelopes through the appropriate data points.
(*c*) best fit mean curves through the appropriate data points.
(*d*) best fit mean straight lines through the appropriate data points.

Q. 4. Match the list I with list II and select the correct answer using the codes given below the lists :

List I	*List II*
A. Conservation reservoirs	1. Uncontrolled outlets
B. Retarding basins	2. Flood-fighting
C. Flood plains	3. Temporary storage of flood water
D. Flood walls	4. Controlled outlets

Codes :

(*a*)	A	B	C	D
	1	4	3	2
(*b*)	A	B	C	D
	1	4	2	3
(*c*)	A	B	C	D
	4	1	3	2
(*d*)	A	B	C	D
	4	1	2	3

Q. 5. The following four hydrological features have to be estimated or taken as inputs before one can compute the flood hydrograph at any catchment outlet.

1. Unit hydrograph 2. Rainfall hydrograph
3. Iufiltration index 4. Base flow.

The correct order in which they have to be employed in the computations is

(*a*) 1, 2, 3, 4 (*b*) 2, 1, 4, 3
(*c*) 2, 3, 1, 4 (*c*) 4, 3, 2, 1

Q.6. In the alignment of an irrigation channel, wherefrom off-takes have to be provided at regular intervals, changes in the given channel parameters are made use of. The correct sequence of the decreasing order of preference of these parameters is

(*a*) width, slope, depth (*b*) width, depth, slope
(*c*) depth, slope, width (*d*) depth, width, slope.

Q.7. Match List I will List II and select the correct answer using the codes given below the lists

List I (*Indications of terms*)		*List II* (*Terms*)
A.	Tail water curve much above jump height curve	1. Basin at bed level
B.	Tailwater curve slightly above jump height curve	2. Sunk basin
C.	Tailwater curve coinciding with jump height curve	3. roller bucket
D.	Tailwater curve below the jump height curve	4. Sloping apron

Codes :

	A	B	C	D
(*a*)	4	3	2	1
(*b*)	3	4	1	2
(*c*)	4	3	1	2
(*d*)	3	4	2	1

Q. 8. The yield of a well depends upon

(*a*) permeability of soil
(*b*) area of aquifer opening into the wells
(*c*) actual flow velocity
(*d*) all of the above

Q. 9. The construction of impounding reservoir is required when

(*a*) average annual flow in the stream is lower than average demand
(*b*) the rate of flow in the stream in dry season is more than the demand
(*c*) the rate of flow in the stream in dry season is less than the demand
(*d*) the rate of flow in the stream is equal to the demand.

ANSWERS

1. (*a*)	**2.** (*c*)	**3.** (*c*)	**4.** (*c*)	**5.** (*c*)
6. (*b*)	**7.** (*c*)	**8.** (*a*)	**9.** (*c*)	

Year 1997 (*Objective Paper II of Engineering Services*)

Q. 1*. A channel of bed slope 0.0009 carries a discharge of 30 m^3/s when the depth of flow is 1.0 m. What is the discharge carried by an exactly similar channel at the same depth of flow if the slope is increased to 0.0001 ?

(*a*) 10 m^3/s (*b*) 15 m^3/s
(*c*) 60 m^3/s (*d*) 90 m^3/s

Q. 2*. At a hydraulic jump, the depths at the two sides are 0.4 m and 1.4 m. The head loss in the jump is nearly

(*a*) 1.0 m (*b*) 0.9 m
(*c*) 0.7 m (*d*) 0.45 m

Q. 3. Which one of the following pairs relating to flumes carrying open channel flow is correctly matched ?

(*a*) Non-modular flume ... Flow is unaffected by drowning
(*b*) Venturi flume ... Standing wave forms at the throat.
(*c*) Venturi flume ... Flow at the throat is less than critical velocity.
(*d*) Standing wave flume ... Hump is not provided at the throat.

Q. 4*. Mean precipitation over an area is best obtained from gauged amounts by

(*a*) arithmetic mean method
(*b*) Thiessen method
(*c*) linearly interpolated isohyetal method
(*d*) Orographically weighed isohyetal method.

Q. 5. The following steps are involved in arriving at a unit hydrograph.

1. Estimating the surface runoff in depth
2. Estimating the surface runoff in volume
3. Separation of base flow
4. Dividing surface runoff ordinates by depth of runoff

The correct sequence of these steps is

(*a*) 3, 2, 1, 4 (*b*) 2, 3, 4, 1
(*c*) 3, 1, 2, 4 (*d*) 4, 3, 2, 1

Q. 6*. Probability of a 10 year flood to occur at least once in the next 4 years is

(*a*) 25% (*b*) 35%
(*c*) 50% (*d*) 65%

Q. 7. The standard Project flood is

(*a*) derived from the probable maximum precipitation in the region
(*b*) derived from the severemost meteorological conditions any where in the country.
(*c*) the flood with return period of 100 years.
(*d*) the same as the probable maximum flood.

Q. 8. Match List I (control structures) with List II (Functions of the control structures) and select the correct answer using the codes given below the Lists.

List I	*List II*
A. Canal drop	1. Control of flow depth
B. Canal Escape	2. Control of bed grade
C. Canal cross-regulator	3. Control of full supply level
D. Canal outlets.	4. Control of discharge

Codes :

	A	B	C	D
(*a*)	2	3	4	1
(*b*)	2	3	1	4
(*c*)	3	2	1	4
(*d*)	3	2	4	1

Q. 9*. For medium silt whose average grain size is 0.16 mm, Lacey's silt factor is likely to be

(*a*) 0.30 (*b*) 0.45

(*c*) 0.70 (*d*) 1.32

Q. 10. Match List I (Main provision) with List II (surplussing arrangement) and select the correct answer using the codes given below the Lists.

List I	*List II*
A. Minor Irrigation work	1. Saddle spillway
B. Medium irrigation project in interior area	2. Syphon spillway
C. Earth dam across main river	3. Ogee spillway
D. Masonry dam on good rock	4. Surplus weir.

Codes

	A	B	C	D
(*a*)	4	2	1	3
(*b*)	4	2	3	1
(*c*)	2	4	3	1
(*d*)	2	4	1	3

Q. 11. While considering weir designs on permeable soils, the correction for mutual interference of sheet piles is NOT applicable on an intermediate pile if the outer pile :

(*a*) goes deeper than the intermediate pile and is farther from the intermediate pile by more than twice its own length

(*b*) goes only just as deep as the intermediate pile and is within a distance of one and half times its own length

(*c*) does not go as deep as the intermediate pile, no matter what the horizontal distance between them is

(*d*) is safe against deleterious exit gradient

Q. 12*. The following data were recorded from an irrigation field ;

1. Field capacity = 20%
2. Permanent wilting point = 10%
3. Permissible depletion of available soil moisture = 50%
4. Dry unit weight of soil = 1500 kgf/m^3
5. Effective rainfall = 25 mm

Based on these data, the net irrigation requirement per metre depth of soil will be

(*a*) 75 mm (*b*) 125 mm
(*c*) 50 mm (*d*) 25 mm

Q. 13. The correct sequence, in the direction of the flow of water for installations in a hydropower plant is

(*a*) reservoir, surgetank, turbine, penstock
(*b*) reservoir, penstock, surge tank, turbine
(*c*) reservoir, penstock, turbine, surge tank
(*d*) reservoir, surge tank, penstock, turbine

ANSWERS

1. (*a*)	**2.** (*d*)	**3.** (*b*)	**4.** (*d*)	**5.** (*a*)	**6.** (*b*)
7. (*b*)	**8.** (*b*)	**9.** (*c*)	**10.** (*a*)	**11.** (*b*)	**12.** (*c*)
13. (*b*)					

Hints for Solving Starred Questions of Year 1997

Q. 1. $Q = \frac{1}{n} \cdot A.R^{2/3} \cdot \sqrt{S}$

$$\therefore \quad Q_2 = \sqrt{\frac{0.0001}{0.0009}} \cdot Q_1 = \frac{1}{3} \times 30 \text{ m}^3/\text{s} = 10 \text{ m}^3/\text{s} \text{ Ans.}$$

Q. 2. $H_L = \frac{(y_2 - y_1)^3}{4 \cdot y_1 y_2} = \frac{(1.4 - 0.4)^3}{4 \times 0.4 \times 1.4} = 0.45 \text{ m}$ Ans.

Q. 4. The Isohyetal method for determining mean rainfall in a catchment is superior to the arithmetic mean method as well as 'Thiessans' Polygon method, especially when the stations are large in number and the catchment is large (more than 5000 sqkm or so). The isohyetal method also permits consideration of topographic effects, and is therefore more accurate in mountainous areas. In this method, the average value of rainfall indicated by two isohyets is assumed to be acting over the inter isohyet area. The isohyetal method thus considers different rainfalls for different part areas, which amounts to orographical weighing of the part areas.

Q. 5. When the river runoff graph is known, the base flow has to be first of all subtracted from it to obtain S.R.O. graph. The volume of S.R.O. is then computed from the S.R.O. graph. The S.R.O. volume is then divided by catchment area to obtain S.R.O. depth. The S.R.O. ordinates are finally

divided by S.R.O. depth (in cm) to obtain ordinates of unit hydrograph. Hence, the sequence 3, 2, 1, 4 is correct. **Ans.**

Q. 6. $T = 10$ yrs

$$p = \frac{1}{T} = \frac{1}{10} = 0.1$$

$$R = \left[1 - (1 - 0.1)^4\right]$$

$$= 1 - (0.9)^4 = 0.3439 \approx 0.35 \text{ or } 35\% \quad \textbf{Ans.}$$

Q. 9. $f = 1.75 \cdot \sqrt{dmm}$

$$= 1.75\sqrt{0.16} = 1.75 \times 0.4 = 0.70 \textbf{ Ans.}$$

Q. 12. Available m.c. = Field capacity m.c. – wilting Pt. m.c.

$$= 20\% - 10\% = 10\%$$

Readily available m.c. = 50% × Available m.c.

$$= 50\% \times 10\% = 5\%$$

Optimum m.c. = 20% – 5% = 15%

Irrigation water is to be supplied when m.c. falls to 15% and is to be recouped upto 20%

$$= \frac{\gamma_d}{w}\left[\text{Field m.c.} - \text{Optimum m.c.}\right]$$

For 1 m depth of soil, the water reqd. to recoup m.c. from 15% to 20%

$$= \frac{1500}{1000} \times 1 \text{ m } [20\% - 15\%]$$

$$= 1.5 \times 5\% \text{ m} = 1.5 \times 0.05 = 0.075 \text{ m} = 75 \text{ mm}$$

Irrigation water reqd. = Total water reqd. – Effective Rainfall

$$= 75 \text{ mm} - 25 \text{ mm} = 50 \text{ mm} \quad \textbf{Ans.}$$

Year 1998 (*Engineering Services*)

Q. 1. Depth-area-duration curves would seem to resemble

(*a*) arcs of a circle concave upwards with duration increasing outward

(*b*) first quadrant limbs of hyperbolae with duration increasing outward

(*c*) third quadrant limbs of hyperbolae with duration decreasing outward

(*d*) first quadrant limbs of hyperbolae with duration decreasing outward

Q. 2. When a river starts meandering, the sediment carrying capacity

(*a*) first decreases and ultimately increases

(*b*) first increases and ultimately decreases

(*c*) remains unaffected as the plan changes continue

(*d*) changes erratically all the time leaving permanent braids

Q. 3. Asubmerged pipe outlet from a minor feeds into a well chamber across the bank. Water from the well chamber overflows a weir- like face into the field channel. The flow into the field channel

(*a*) will increase with any drop in the water level in the minor

(*b*) will not be much affected by any change in the water level in the minor
(*c*) will decrease with any drop in the water level in the minor
(*d*) will be proportional to the flow in the minor when the minor flows between set ranges

Q. 4. Match List-I (Types of turbines) with List-II (Ranges of specific speeds in MKS units) and select the correct answer using the codes given below the lists :

List I	*List II*
A. Francis	1. 10 – 35
B. Kaplan	2. 35 – 60
C. Pelton with one jet	3. 60 – 300
D. Pelton with two jets	4. 300 – 1000

Codes :

	A	B	C	D
(*a*)	3	4	2	1
(*b*)	4	3	2	1
(*c*)	3	4	1	2
(*d*)	4	3	1	2

ANSWERS

1. (*a*) **2.** (*c*) **3.** (*c*) **4.** (*c*)

Year 1999 (*Engineering Services*)

Q. 1.* To generate 10,000 hp under a head of 81 m while working at a speed of 500 rpm, the turbine of choice would be :

(*a*) Pelton (*b*) Kaplan
(*c*) Bulb (*d*) Francis.

Q. 2. Consider the following statements :

1. Run of river plants can be located on any river
2. Runaway speed of a turbine is generally 180% of normal speed.
3. Underground power stations are suited to areas susceptible to land slides.
4. Higher the specific speed, higher will be the discharge and head.

Which of these statements are correct ?

(*a*) 1 and 2 (*b*) 2 and 3
(*c*) 1 and 4 (*d*) 2 and 4

Q. 3. Match List I and II and select the correct answer using the codes given below the lists

List I (Turbine)		*List II (Specific speed)*	
A	Pelton	1	25
B	Propeller	2	75
C	Kaplan	3	500
D	Francis	4	800
		5	900

Codes

	A	*B*	*C*	*D*
(*a*)	1	4	5	2
(*b*)	3	4	2	5
(*c*)	4	1	5	3
(*d*)	1	5	4	2

Q. 4.* An aquifer confined at top and bottom by impermeable layers is stratified into three layers as follows.

Layer	*Thickness (m)*	*Permeabilaty m/day*
Top layer	4	30
Middle layer	2	10
Bottom layer	6	20

The transmissivity (m^2/day) of the aquifer is

(*a*) 260 (*b*) 227
(*c*) 80 (*d*) 23

Q. 5. The spacing of tile drains to relieve water-logged land is directly proportional to the

(*a*) depth of drain below the ground surface
(*b*) depth of impervious strata from the drain
(*c*) depth of drain below the water level.
(*d*) coefficient of permeability of the soil to be drained.

Q. 6.* The maximum permissible suction lift for centrifugal pump *in practice* (at sea level and at 30°C) is :

(*a*) 12 m (*b*) 10 m
(*c*) 6 m (*d*) 3 m.

Q. 7.* Consider the following statements :

1. Pumps in series operation allow the head to increase.
2. Pumps in series operation increase the flow rate.
3. Pumps in parallel operation increase the flow rate.
4. Pumps in parallel operation allow the head to increase.

Which of these statements are correct ?

(*a*) 1 and 3 (*b*) 1 and 4
(*c*) 2 and 4 (*d*) 3 and 4.

Q. 8. In a flow-mass curve study, the demand line drawn from a ridge does not intersect the mass curve again. This implies that

(*a*) the reservoir is not full at the beginning
(*b*) the storage is not adequate.
(*c*) the demand cannot be met by the inflow, as the reservoir will not refill
(*d*) the reservoir is wasting water by spill.

Q. 9.* The following rainfall data refers to stations 'A' and 'B' which are equidistant from station 'X'.

	Station 'A'	*Station 'X'*	*Station 'B'*
Long term normal annual rainfall in mm	200	250	300
Annual rainfall in mm for the year 1940	140	P	270

The value of P will be :

(*a*) 250 (*b*) 220

(*c*) 205 (*d*) 200

Q. 10. Consider the following statements :

1. A 100-year flood discharge is greater than a 50 year flood discharge.

2. 90% dependable flow is greater than 50% dependable flow

3. Evaporation from salt-water surface is less than that from fresh water surface.

Which of these statements are correct ?

(*a*) 1 and 2 (*b*) 2 and 3

(*c*) 1 and 3 (*d*) 1, 2 and 3

Q. 11. The trap efficiency of a reservoir is a function of :

(*a*) inflow in the reservoir (*b*) ratio of inflow to storage capacity

(*c*) ratio of reservoir capacity to inflow. (*d*) reservoir capacity.

Q. 12. If a 4-hour unit hydrograph of a certain basin has a peak ordinate of 80 m^3/s, the peak ordinate of a 2-hour unit hydrograph for the same basin will be :

(*a*) equal to 80 m^3/s (*b*) greater then 80 m^3/s

(*c*) less than 80 m^3/s (*d*) between 40 m^3/s to 80 m^3/s

Q. 13. Match List I (Name of scientist) with List II (Contribution to field of hydrology) and select the correct answer using the Codes given below the lists :

List I	*List II*
A. Dalton	1. Unit hydrograph
B. Snyder	2. Evaporation
C. Blaney Criddle	3. Empirical flood formula
D. Sherman	4. Synthetic unit hydrograph
	5. Consumptive use equation

Codes

	A	*B*	*C*	*D*
(*a*)	2	3	5	1
(*b*)	1	4	3	2
(*c*)	2	4	5	1
(*d*)	1	3	5	4

Q. 14.* In a linear reservoir, the

(*a*) volume varies linearly with elevation

(*b*) outflow rate varies linearly with storage

(*c*) storage varies linearly with time

(*d*) storage varies linearly with inflow rate

Q. 15. A culvert is designed for a peak flow Q_p on the basis of rational formula. If a storm of the same intensity as used in the design and twice the duration occurs, then the resulting peak discharge will be :

(*a*) Q_p (*b*) $\frac{Q_p}{2}$

(*c*) $\sqrt{2}\,Q_p$ (*d*) $2\,Q_p$.

Q. 16. Match List I with List II and select the correct answer using the codes given below the list

List I	*List II*
A. Specific yield	1. Volume of water retained per unit volume of aquifer
B. Specific capacity	2. Volume of water drained by gravity per unit volume of aquifer
C. Specific retention	3. Difference of porosity and specific storage.
D. Specific storage	4. Well yield per unit drawdown
	5. Volume of water released from unit volume of aquifer for unit decline in piezometric head.

Codes

	A	*B*	*C*	*D*
(*a*)	2	4	1	5
(*b*)	4	2	3	5
(*c*)	2	5	1	4
(*d*)	4	2	3	1

Q. 17. The best design of an arch dam is when :

(*a*) All horizontal water loads are transferred horizontally to the abutments.

(*b*) The dam is safe against sliding at various levels

(*c*) The load is divided between the arches and cantilevers, and the deflections at the conjugal points being equal.

(*d*) The deflections of the cantilevers are equal at different points.

ANSWERS

1. (*d*)	**2.** (*b*)	**3.** (*a*)	**4.** (*b*)	**5.** (*d*)	**6.** (*c*)
7. (*a*)	**8.** (*c*)	**9.** (*d*)	**10.** (*c*)	**11**(*c*).	**12.** (*b*)
13. (*c*)	**14.** (*b*)	**15.** (*a*)	**16.** (*a*)	**17.** (*c*)	

Hints for Solving Starred Questions of the Year 1999

Q. 1. From Eq. (24.1), we have

$$N_s = \frac{N.\sqrt{P_t}}{H^{5/4}}$$

where N = Normal speed of turbine in RPM = 500

P_t = Turbine output in metric horse power = 10,000 hp.

H = Effective head in m = 81 m

$$\therefore \quad N_s = \frac{500\sqrt{10{,}000}}{(81)^{5/4}} = 205.76$$

From Table 24.1, the turbine which is suitable for a head between 25–450 m and N_s of 60 – 300 is Francis turbine; and hence the recommended turbine in this case is Francis type (Reaction type). Hence, choice (*d*) is correct. **Ans.**

Q. 4. For non-homogeneous layered aquifers, we have to determine effective permeability coefficient by using equation (5.20) given in "Soil Mechanics and Foundation Engineering" by the same author, for vertical flow, as :

$$K_v = \frac{d_1 + d_2 + d_3}{\frac{d_1}{K_1} + \frac{d_2}{K_2} + \frac{d_3}{K_3}}$$

$$= \frac{4+2+6}{\frac{4}{30} + \frac{2}{10} + \frac{6}{20}} = 12 \times \frac{60}{38} \text{ m/day}$$

$$T = K.d = 12 \times \frac{60}{38} \times (4+2+6)$$

$$= 12 \times \frac{60}{38} \times 12 \text{ m}^2/\text{day} = 227 \text{ m}^2/\text{day}.$$

Hence, choice (*b*) is correct **Ans.**

Q. 6. The max. permissible suction lift for a centrifugal pump is given by the equation :

$$H_s = H_a - H_f - e_s - NPSH - F_s \qquad ...(32.1)$$

where, H_s = Max. Practical suction lift, *m*

H_a = Atmospheric pressure at the water surface = 10.33 m at sea level, with reduction of 0.36 m for each 300 m rise in altitude above sea level.

e_a = saturated vapour pressure of water = 4.609 kN/m^2 at 30° C(from Standard Physical tables — Appendix Table A–1)

$NPSH$ = Net positive suction head of the pump, including losses in the impeller& vel. head. It is calculated from the characteristic curves of the given pump, supplied by the manufacturer.

F_s = Factor of safety, which is usually taken as 0.6 m.

From the above eqn., we can easily infer that $H_s < H_a$, while $H_a < 10$ m. For this reason, for all practical proposes, the suction lift for centrifugal pumps is usually limited to 6 – 8 m, as stated in our text. In the given question, the choice below 10m is given to be 6m. Hence, we mark this choice at (*c*) as the correct choice. **Ans.**

Q. 7. When two or more pumps are connected in series, such as in multi-stage pumping operations, the discharge from the first stage is pumped to the second stage, and so on. The same discharge passes through all the stages. This arrangement becomes essential when the reqd. head is more than that can be produced with a single pump. Each stage in such pumping operations, adds an additional head to the flow. Thus, the combined head of two pumps operating in series is equal to the sum of the individual heads, for a specific discharge.

Sometimes, water pumps are installed in parallel as in the case of pumping water from a sump well using more than one pump. The head remains constant. The combined discharge of the two pumps operating in parallel is equal to the sum of the individual discharges, for a specific head.

From the above discussion, it can be very clearly inferred that the statement 1 and 3 are only true, and hence choice (*a*) is correct. **Ans.**

Q. 9.

$$N_x = 250$$

$$10\% N_x = 25$$

$$N_x + 10\% N_X = 275 < 300 \ (N_B)$$

Hence, use weighted mean method to compute the missing rainfall data P_X.

$$\therefore \quad P_x = \frac{1}{2}\left[\frac{N_X}{N_A} \cdot P_A + \frac{N_X}{N_B} \cdot P_B\right]$$

$$= \frac{1}{2}\left[\frac{250}{200} \times 140 + \frac{250}{300} \times 270\right] = 200$$

Hence, choice (*d*) is correct. **Ans.**

Q. 14. Please see chapter on 'Flood Forecasting' in "*Hydrology and Water Resources Engg.*" by the same author, Page 418, 11th edition, to tick the correct choice, which is given at (*b*) **Ans.**

Year 2000 (*Engineering Services*)

Q. 1. If the base period of a 6 hour unit hydrograph of a basin is 84 hours, then a 12 hour unit hydrograph derived from this 6 hour unit hydrograph will have a base period of

(*a*) 72 hours (*b*) 78 hours

(*c*) 84 hours (*d*) 90 hours.

Q. 2*. The stage in a river is 4.8 m; the water surface slope is 1 in 10,000 and the discharge in the stream is 600 m^3/s. If the stage remains the same and the water surface slope is 1 in 14,400, then the discharge in the stream will be :

(*a*) 300 m^3/s (*b*) 400 m^3/s

(*c*) 600 m^3/s (*d*) 500 m^3/s

Q. 3. If a tangent drawn parallel to the demand line from a ridge point of a mass curve dose not intersect the mass curve again, it can be inferred that the :

(*a*)frequency of the flood entering into the reservoir is less.
(*b*) inflow into the reservoir cannot meet the demand.
(*c*) reservoir is overflowing resulting in wastage.
(*d*) reservoir can meet higher demand.

Q. 4*. In order to ensure that no scouring takes place in the bed of a channel of bed slope '*S*' constructed on alluvial soil of particle size '*d*' cm, the flow velocity should be restricted to :

(*a*) $4.85\, d^{\frac{1}{2}} . S^{-\frac{1}{6}}$ (*b*) $4.85\, d^{-\frac{1}{2}} . S^{\frac{1}{6}}$
(*c*) $0.48\, d^{\frac{1}{2}} . S^{\frac{1}{6}}$ (*d*) $0.48\, d^{\frac{1}{2}} . S^{-\frac{1}{6}}$

Q. 5. Which one of the following sets is used to control the seepage through the foundations of an earth dam ?

(*a*) Chimney drain, upstream blanket and cut off trench
(*b*) Cut off sheet piles, upstream blanket and cut off trench.
(*c*) Upstream blanket, cut off sheet piles and chimney drains
(*d*) Relief wells, upstream blanket and chimney drain.

Q. 6. Consider the following statements :

1. In designing the hydraulic structure in permeable foundation by Khosla's theory, the slope correction is applicable to piles located at the ends of the sloped floor only in a structure with one or more sloped floors.
2. Khosla's theory can be correctly applied in alluvium of finite depth.
3. Length of flow has lesser effect on exit gradient than the depth of piles.
4. Intercepts between hydraulic gradient line and free water surface on the glacis and horizontal floor downstream is the unbalanced uplift in a structure.

Which of these statements related to the design of hydraulic structure are correct ?

(*a*) 1, 2 and 3 (*b*) 1, 3 and 4
(*c*) 2, 3 and 4 (*d*) 1, 2 and 4

Q. 7. Match *List-I* (Energy dissipation) with *List-II* (Water level and slope condition) and select the correct answer using the codes given below the lists :

List I	*List II*
A. Roller bucket	1. TWL (tail channel water level) is slightly above JWL (Jump height water level) and the slope of the channel is mild.
B. Ski-jump bucket	2. TWL is considerably above JWL and the slope of the channel is mild.
C. Standing wave basin with depressed floor	3. TWL is slightly below JWL and the slope of the channel is mild
D. Standing wave basin with raised floor	4. TWL is considerably less than JWL and the slope of the channel is steep.

Codes

	A	B	C	D
(a)	4	2	1	3
(b)	2	4	1	3
(c)	2	4	3	1
(d)	4	2	3	1

Q. 8. If the depth is 8.64 cm on a field over a base period of 10 days, then the duty is :

(a) 10 hectares per cum/s (b) 100 hectares per cum/s
(c) 864 hectares per cum/s (d) 1000 hectares per cum/s

Q. 9. Specific capacity of a well is the :

(a) volume of water that can be extracted by the force of gravity from a unit volume of aquifer.
(b) discharge per unit drawdown of the well.
(c) drawdown per unit discharge of the well.
(d) rate of flow through a unit width and entire thickness of aquifer.

ANSWERS

1. (d) **2.** (d) **3.** (b) **4.** (a) **5.** (b)
6. (b) **7.** (c) **8.** (d) **9.** (b)

Hints for Solving Starred Questions of the year 2000

Q. 2. $Q_1 = 600 = K \cdot \sqrt{S_{f_1}} = \dfrac{K}{\sqrt{10000}}$; $Q_2 = ? = K \cdot \sqrt{S_{f_2}} = \dfrac{k}{\sqrt{14400}}$

$$\frac{Q_1}{Q_2} = \frac{600}{Q_2} = \frac{\sqrt{14400}}{\sqrt{10000}} = \frac{1}{1.2}; \; Q_2 = 500 \text{ m}^3/\text{sec}$$

Hence, choice (d) is correct. **Ans.**

Q. 4. For designing a non scouring alluvial channel, we know from eqn. (4.13) that

$$d \le 11 \text{ RS}$$

or $$R \le \frac{d}{11\,S} \quad \text{or} \quad R_{max} = \frac{d}{11\,S}$$

Also, $$V = \frac{1}{n} R^{2/3} \cdot \sqrt{S}$$

or $$V_{max} = \frac{1}{n} \cdot R_{max}^{2/3} \cdot \sqrt{S}$$

But $$= n = \frac{1}{24} \cdot d^{1/6} \text{ (Strickler's formula)}$$

$$\therefore \quad V_{max} = \frac{24}{d^{1/6}} \cdot \left(\frac{d}{11\,S}\right)^{2/3} \cdot \sqrt{S} = \frac{24}{(11)^{2/3}} \cdot d^{2/3 - 1/6} \cdot S^{1/2 - 2/3}$$

or $$\boxed{V_{max} = 4.85 \cdot d^{1/2} \cdot S^{-\frac{1}{6}}}$$

Hence, velocity in a non scouring channel should be restricted to above eqn., which makes choice (a) as the correct choice. **Ans.**

Year 2001 (*Engineering Services*)

Q. 1. Match *List I* (Surface) with *List II* (Approximate range of Manning's *n*) and select the correct answer using the codes given below the lists :

List 1	*List 2*
A. Gravel river bed with 50 mm diameter bed	1. 0.02 to 0.022
B. Badly maintained unlined irrigation channel with weed growth	2. 0.025 to 0.04
C. Concrete lined channel	3. 0.013 to 0.017
D. Channel with brick lining	4. 0.04 to 0.08

Codes :

	A	B	C	D
(*a*)	4	3	2	1
(*b*)	4	2	3	1
(*c*)	1	3	2	4
(*d*)	1	2	3	4

Q. 2. Which of the following principles relate to a unit hydrograph ?

1. The hydrographs of direct run off due to effective rainfall of equal duration have the same time base.
2. Effective rainfall is not uniformly distributed within its duration.
3. Effective rainfall is uniformly distributed throughout the whole area of drainage basin.
4. Hydrograph of direct run off from a basin due to a given period of effective rainfall reflects the combination of all the physical characteristics of the basin.

Select the correct answer using the codes given below :

Codes :

(*a*) 1, 2 and 3 (*b*) 1, 2 and 4

(*c*) 2, 3 and 4 (*d*) 1, 3 and 4.

Q. 3. Match *List I* with *List II* and select the correct answer using the codes given below the lists :

List I	*List II*
A. Anemometer	1. Humidity
B. Rain simulator	2. Evapotranspiration
C. Lysimeter	3. Infiltration
D. Hygrometer	4. Wind speed

Codes :

	A	B	C	D
(*a*)	4	3	1	2
(*b*)	3	4	1	2
(*c*)	4	3	2	1
(*d*)	3	4	2	1

Q. 4. If the life of a reservoir is determined by its capacity (C), volume of annual inflow into the reservoir (I) and concentration of sediment in the incoming flow (C_s). Life will be more if :

(*a*) C, I and C_s are high

(*b*) C and I are high but C_s is low

(*c*) C is high but I and C_s are low

(*d*) C, I and C_s are low.

Q. 5*. The probability that a 100-year flood is equalled or exceeded at least once in 100 years is :

(*a*) 99% (*b*) 64%

(*c*) 36% (*d*) 1%

Q. 6. A 3-hour storm on a small drainage basin produced rainfall intensities of 3.5 cm/hr, 4.2 cm/hr and 2.9 cm/hr in successive hours. If the surface runoff due to storm is 3 cm, then the value of ϕ-index will be :

(*a*) 2.212 cm/hr
(*b*) 2.331 cm/hr
(*c*) 2.412 cm/hr
(*d*) 2.533 cm/hr.

Q. 7. As a flood wave passes a given section of a river, the time of occurrence of the maximum stage and that of the maximum discharge will be such that :

(*a*) the maximum discharge passes down before the maximum stage is attained

(*b*) the maximum stage is attained before the maximum discharge passes down

(*c*) the two events occur simultaneously

(*d*) no specific sequence would be universally assignable.

Q. 8. In a curved reach of a meandering river, both deposition of sediments and erosion of bank occur. Which one of the following statements is true in this regard ?

(*a*) Deposition of sediments occurs in the inner bank while the outer bank is subjected to erosion

(*b*) Deposition of sediments occur in the outer bank while the inner bank is subjected to erosion

(*c*) In the direction of flow, the outer bank undergoes erosion first and the sediment is later deposited on the outer bank downstream

(*d*) In the direction of flow, the inner bank undergoes erosion first and the sediments are subsequently deposited on the inner bank downstream.

Q. 9. Gibb's module is a type of outlet which ensures :

(*a*) Constant discharge even if the water levels in the supply channel and water course fluctuate

(*b*) Variable discharge as per the need

(*c*) Constant discharge into the water course when the water levels in the supply channel vary

(*d*) Constant discharge for varying water levels in the water course for a given water level in the supply channel.

Q. 10. The Lacey's silt factor for a particular alluvium is 2.0. This alluvium would comprise :

(*a*) medium sand of size 0.5 mm (*b*) coarse sand of size 0.75 mm

(*c*) medium bajri of size 1.3 mm (*d*) coarse bajri of size 2.4 mm.

Q. 11. In a barrage on pervious foundation, sheet piles are provided both upstream and downstream of the barrage to reduce uplift pressure and to prevent piping. Which one of the following statements is true in this regard ?

(*a*) Compared to downstream sheet pile, the upstream sheet pile is more effective in reducing uplift and piping

(*b*) Compared to upstream sheet pile, the downstream sheet pile is more effective in reducing uplift and piping

(*c*) Downstream sheet pile is more effective in reducing uplift while the upstream sheet pile is more effective in reducing piping

(*d*) Upstream sheet pile is more effective in reducing uplift while the downstream sheet pile is more effective in reducing piping.

Q. 12. The following data pertains to a natural drain crossing an irrigation canal :

Item	*Canal data*	*Drainage data*
Flow (m^3/s)	5	500
Bed level (m)	120	116
Depth of flow (m)	0.8	10

Which one of the following types of cross-drainage should be recommended in this case ?

(*a*) Aqueduct (*b*) Syphon aqueduct

(*c*) Syphon (*d*) Super passage

Q. 13. Consider the following statements :

The downstream impervious floor of concrete for a barrage has ruptured. This can be due to :

1. insufficient length of upstream impervious floor
2. insufficient length of downstream impervious floor
3. insufficient depth of downstream pile
4. choking of inverted filter.

Which of these statements are correct ?

(*a*) 1 and 2 (*b*) 1 and 4

(*c*) 2, 3 and 4 (*d*) 1, 2, 3 and 4.

Q. 14*. Ten m^3/s of water is diverted to a 32 hectare field for 4 hours. Soil proving after irrigation showed that 0.3 m of water has been stored in the root zone. Water application efficiency in this case would be :

(*a*) 96% (*b*) 66.67%

(*c*) 48% (*d*) 24%.

Q. 15. Which one of the following statements is correct ?
In a river, silt excludes and silt ejector are constructed
(*a*) at a location after the head regulator and at the head of the canal, respectively
(*b*) at the head of the canal and at a location after the head regulator, respectively
(*c*) at the same location
(*d*) at specific locations depending upon diverse factors and their locations do not follow a set pattern.

ANSWERS

1. (*b*)	**2.** (*d*)	**3.** (*c*)	**4.** (*b*)	**5.** (*b*)
6. (*d*)	**7.** (*c*)	**8.** (*a*)	**9.** (*a*)	**10.** (*c*)
11. (*d*)	**12.** (*b*)	**13.** (*d*)	**14.** (*b*)	**15.** (*b*)

Hints for Solving Starred Questions of the year 2001

Q. 5. See Eqn. (7.135) and compute :

$$p = \frac{1}{T} = \frac{1}{100} = 0.01$$

Probability of non-occurrence of this flood in next 100 yrs

$$= p_{o,n} = q^n = (1-p)^n$$

$$= (1-0.01)^{100} = 0.36$$

Probability of occurrence of this flood at least once in 100 successive yrs

$$= R = 1 - q^n$$

$$= 1 - 0.36 = 0.64 \qquad i.e.\ 64\% \quad \textbf{Ans.}$$

Q. 14. Water depth applied to fields

$$= \frac{10\,\frac{\text{m}^3}{\text{s}} \times (4 \times 60 \times 60\ \text{s})}{(32 \times 10^4\ \text{m}^2)} = 0.45\ \text{m}$$

Water depth actually stored in root zone. = 0.3 m

$$\eta_a = \frac{0.3\ \text{m}}{0.45\ \text{m}} = \frac{2}{3} = \mathbf{66.67\%} \quad \textbf{Ans.}$$

Year 2002 (*Engineering Services*)

Q1. The following items relate to hydrologic requirements in a hydropower system. Match List I with List II and select the correct answer using the codes given below the lists :

	List I		*List II*
A	Stream flow in order of magnitude is plotted as ordinate and percent of time as abscissa	1	Mass curve
B	Cumulative value of stream flow is plotted against time for whole period record	2	Flow duration curve
C	Peat of available power against percent of time	3	Hydrograph
D	Plot of flow in stream against time at specific interval	4	Power duration curve

Codes :

	A	B	C	D
(a)	2	3	4	1
(b)	4	1	2	3
(c)	2	1	4	3
(d)	4	3	2	1

Q2. Consider the following statements :

1. An ephemeral stream is one which has a base flow contribution.
2. Flow characteristics of a stream depend upon rainfall and catchment characteristics, and also the climatic factors which influence evapotranspiration.
3. Sequent Peak Algorithm is used for estimating run off from rainfall.

Which of these statements is/are correct ?

(a) 1, 2 and 3 (b) 1 and 3

(c) 2 and 3 (d) 2 clone.

Q3. Match List I with List II and select the correct answer using the codes given below the lists :

List I	*List II*
A ϕ index	1. Used for measurement of evapo-transpiration for given vegetation
B Lysimeter	2. Used for flow measurement
C Dilution technique	3. Average rainfall above which the rainfall volume is equal to the run-off volume
D Snydel's equation	4. Relates the basin lag to the basin characteristics

Codes :

	A	B	C	D
(a)	3	1	2	4
(b)	4	2	1	3
(c)	3	2	1	4
(d)	4	1	2	3

Q4. A 6-hour rainstorm with hourly intensities of 7, 18, 25, 17, 11 and 3 mm/hour produced a run off of 39 mm. Then, the ϕ_{index} is

(a) 3 mm/hour

(b) 7 mm/hour

(c) 8 mm/hour

(d) 10 mm/hour.

Q5.* In a water-shed, four rain gauges I, II, III and IV are installed. The depths of normal annual rainfall at these stations are 60, 75, 80 and 100 cm, respectively. The rain gauge at station II went out of order during a particular year. The annual rainfall for this

year, recorded at the remaining three stations was 90, 60 and 70 cm. The rainfall at station III can be considered as

(*a*) 60 cm (*b*) 70 cm

(*c*) 80 cm (*d*) 120 cm

Q6. The unit hydrograph theory is based on the assumption of

(*a*) Nonlinear response and time invariance

(*b*) Linear response and non-linear time variance

(*c*) Time invariance and linear response

(*d*) Non-linear response and non-linear time variance.

Q7. An effective storage of a flood control reservoir is

(*a*) the amount of water which can be supplied from it in a particular interval of time

(*b*) the storage between the minimum and maximum reservoir levels under ordinary operating conditions

(*c*) the useful storage plus the surcharge storage less the valley storage

(*d*) the storage volume of flood water above the maximum reservoir level.

Q8. The lenman's evapo-transpiration equation is based on

(*a*) water budget method

(*b*) energy balance method

(*c*) mass transfer method

(*d*) energy balance and mass transfer approach.

Q9. The performance of a well is measured by its :

(*a*) specific capacity (*b*) specific yield

(*c*) storage coefficient (*d*) permeability coefficient.

Q10. Regional hydrological cycle is shown in the figure below (Fig. 32.2)

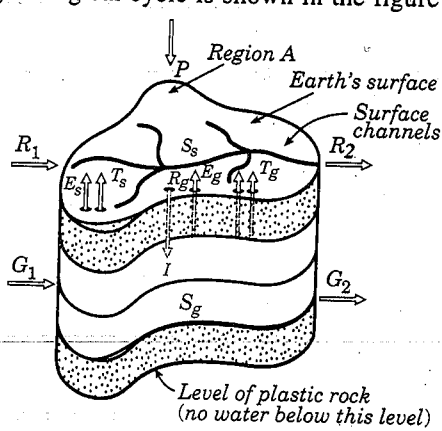

Fig. 32.2.

Identify the correct hydrologic budget equation

(*a*) $P + R_1 - R_2 + R_g - E_s - T_s - I = \Delta S_s$

(*b*) $I + G_1 - G_2 - R_g - E_g - T_g = \Delta S_g$

(*c*) $P-(R_2-R_1)-(E_s+E_g)-(T_s+T_g)-(G_2-G_1)=\Delta(S_s+S_g)$

(*d*) $P-R-G-E-T=\Delta S_s$.

Q11. A two-hour storm hydrograph has 5 units of direct run-off. The two-hour unit hydrograph for this storm can be obtained by dividing the ordinates of the storm hydrograph by

(*a*) 2

(*b*) $\frac{2}{5}$

(*c*) 5

(*d*) $\frac{5}{2}$.

Q12. The slope area method is extensively used in

(*a*) development of rating curve

(*b*) estimation of flood discharge based on high water marks

(*c*) cases where shifting control exists

(*d*) cases where back-water effect is present.

Q13. Consider the following :

Garret's diagram for the design of irrigation channel is based on

1. Kennedy's theory
2. Lacey's theory
3. Kutter's formula
4. Manning's formula.

Which one of these are correct ?

(*a*) 1 and 3 (*b*) 1 and 4

(*c*) 2 and 3 (*d*) 2 and 4.

Q14. The total number of dependent equations that form the Lacey's regime theory is

(*a*) 2 (*b*) 3

(*c*) 4 (*d*) 6.

Q15. Balanced depth of cutting of canal is

(*a*) half the total depth of a canal

(*b*) half of full supply depth

(*c*) the maximum cut that an excavator can take

(*d*) where volume of cutting is equal to volume of filling.

Q16. At a certain point in the floor of weir, the uplift pressure head due to seepage in 4.5 m. If the relative density of concrete is 2.5, the minimum thickness of floor required at this point to counteract the uplift pressure is

(*a*) 1 m (*b*) 2 m

(*c*) 3 m (*d*) 4 m

Q17. A constant angle arch dam when compared to a constant radius arch dam utilizes concrete quantity of about

(*a*) 33% (*b*) 43%

(*c*) 73% (*d*) 143%

Q18. Drowning ratio of canal outlet is the ratio of

(*a*) rate of change of discharge of outlet to that of distributary

(*b*) water depth above outlet crest to the full supply depth of the channel

(*c*) water depth above the crest on downstream to that on upstream of outlet

(*d*) rate of change of discharge of an outlet to the rate of change of water level of the channel.

Q19. A canal was designed to supply the irrigation needs of 1000 *ha* of land growing rice of 140 days base period and having a delta of 130 cm. If the canal water is used to irrigate wheat of base period 119 days and having a delta of of 50 cm, the area that can be irrigated is ;

(*a*) 452 ha (*b*) 904 ha

(*c*) 1105 ha (*d*) 2210 ha

Q20. A canal fall is a control structure

(*a*) located at a place where the country slope is flatter than the canal bed slope

(*b*) located most economically where the depth of cutting is less than the balancing depth

(*c*) the location of which is independent of the command to be served

(*d*) designed to secure raising of water surface on its upstream.

Q21. Match List I (Equation) with List II (Applicability or principle of equation) and select the correct answer using the codes given below the lists :

List I	*List II*
A Theim's equation	1 is based on energy conservation principle
B Dupuit's assumption	2 is based on mass conservation principle
C Bernoullis equation	3 is applicable to steady flow towards a well in confined aquifer
D Continuity equation	4 is applicable to steady flow in an unconfined aquifer

Codes :

	A	B	C	D
(*a*)	4	3	2	1
(*b*)	3	4	2	1
(*c*)	4	3	1	2
(*d*)	3	4	1	2

ANSWERS

1. (*c*)	**2.** (*d*)	**3.** (*a*)	**4.** (*c*)	**5.** (*b*)	**6.** (*c*)	**7.** (*c*)
8. (*d*)	**9.** (*a*)	**10.** (*c*)	**11.** (*c*)	**12.** (*b*)	**13.** (*a*)	**14.** (*d*)
15. (*d*)	**16.** (*c*)	**17.** (*b*)	**18.** (*c*)	**19.** (*d*)	**20.** (*b*)	**21.** (*c*)

Hints for Solving Sttared Questions of the Year 2002

Q5. Station I, N_1 = Annual Av. rainfall = 60 cm

Station II, N_2 (N_x) = Annual Av. rainfall = 75 cm

Station III, N_3 = Annual Av. rainfall = 80 cm.

Station IV, N_4 = Annual Av. rainfall = 100 cm

$P_1 = 90$ cm $\qquad P_3 = 60$ cm

$P_4 = 70$ cm $\qquad P_2 = ?$

Since the value of anyone av. annuai ppt. of known stations, *i.e* N_1, N_3 and N_4 or differs from N_2 (*i.e.* N_x) by more than 10%, is applicables as :

$$\therefore \qquad P_x = P_2 = \frac{N_x}{3}\left[\frac{P_1}{N_1} + \frac{P_3}{N_3} + \frac{P_4}{N_4}\right]$$

$$= \frac{75}{3}\left[\frac{90}{60} + \frac{60}{80} + \frac{70}{100}\right] = 25\,[1.5 + 0.75 + 0.7]$$

$$= 25 \times 2.95 = 73.75 \text{ cm} \quad \textbf{Ans.}$$

The nearest choice available in question is 70 cm, at choice (*b*), which is marked as the correct choice. **Ans.**

Q19. Design discharge for canal

Rice Area to be irrigated = 1000 ha, B for Rice = 140 days and Δ for rice = 130 cm

Duty (D) for rice

$$= \frac{864\,B}{\Delta} = \frac{864 \times 140}{130}$$

$$= 930.46 \text{ ha/cumec}$$

Discharge reqd for above duty

$$= \frac{1000}{930.46} \text{ cumec} = \mathbf{1.0747\ cumec}$$

When the above discharge of canal is used for irrigating wheat crop of

$B = 119$ days $\qquad \Delta = 50$ cm.

$$\text{Duty for wheat } (D) = \frac{864\,B}{\Delta} = \frac{864 \times 119}{50} = 2056.32 \text{ ha/cumec.}$$

The ha which can be irrigated by 1.0747 cumec.

$$= 2056.32 \times 1.0747 \text{ ha} = \mathbf{2210\ ha}$$

Hence, choice at (*d*) is correct. **Ans.**

Year 2003 (*Engineering Services*)

Q1. The worst condition of uplift on the floor of a siphon aqueduct occurs when there is

(*a*) high flood flow in the drainage with canal dry

(*b*) full supply flow in the canal with drainage dry

(*c*) high flood flow in the drainage with canal running full

(*d*) water is at drainage bed and canal is dry.

Q2. A check dam is a

(*a*) flood control structure (*b*) soil conservation structure

(*c*) river training structure (*d*) water storage structure.

Q3. Match List I (Well Hydraulics Parameters) with List II (Definition) and select the correct answer using the codes given below the Lists :

List I (Well Hydraulics Parameters)	*List II (Definition)*
A Specific yield.	1 Discharge per unit draw down of well.
B Safe yield	2 Same as specific retention.
C Specific capacity	3 Measure of water that can be removed by pumping.
D Field capacity	4 Limit of withdrawal from well without depletion of the aquifier
	5 Water bearing capacity of aquifer.

Codes :

	A	B	C	D
(*a*)	4	3	2	2
(*b*)	3	4	1	2
(*c*)	4	3	1	2
(*d*)	3	4	2	5

Q4. Consider the following statements :

A well development

1. involves reversal of flow through the well screen
2. increases permeability towards the well
3. decreases permeability towards the well
4. is continued till sand/silt free water is pumped out

Which of these statements is/are correct ?

(*a*) 1, 3 and 4 (*b*) 1, 2 and 4

(*c*) 3 only (*d*) 1 and 4.

Q5. Leaching is a process

(*a*) by which alkali salts present in the soil are dissolved and drained away

(*b*) by which alkali salts in soil come up with water

(*c*) of draining excess water of irrigation

(*d*) which controls water-logging.

Owner Deepika Titan un 09114404476

Q6. Directions

The following items consist of two statements, one labelled as the 'Assertion A' and the other as 'Reason (R)'. You are to examine these two statements carefully and select the answers to these items using the codes given below :

Codes :

(*a*) Both A and R are individually true and R is the correct explanation of A

(*b*) Both A and R are individually true but R is not the correct explaination of A

(*c*) A is true but R is false

(*d*) A is false but R is true.

(i) *Assertion (A)* : With tile lining of canals, permissible velocity of flow is lower than that with concrete lining.

Reason (R) : The surface of the lining becomes rough due to loss of surface material with high velocity.

(ii) *Assertion (A)* : Canal escape serves as a safety valve for a canal.

Reason (R) : Canal escape discharges the excess water in the parent canal due to sudden closure of outlets by farmers.

Q7. Which one of the following phenomena in a pipe flow is termed as water hammer ?

(*a*) The sudden rise of pressure in a long pipe due to sudden closure of valve

(*b*) the rise of a pressure in a pipe flow due to gradual closure of valve

(*c*) The rise of negative pressure

(*d*) The zero pressure in a pipe flow.

Q8. Flow duration curve is a plot of

(*a*) Flow against its time of occurrence in chronological order

(*b*) Flow in ascending order against percentage time in chronological order

(*c*) Flow that equalled or exceeded against percentage time

(*d*) Flow against duration of time for which it is sustained.

Q9*. Pondage in a hydropower station is defined as :

(*a*) impounding of considerable amount of excess water during seasons of surplus flow

(*b*) A regulating body of water in the form of relatively small amount of run-off to regulat flow variation in daily or weekly power requirements

(*c*) Excess run off to last for years

(*d*) Excess run off for a few hours only.

Q10. The specific speed of a turbine under a head of 150 m to develop 2000 HP while running at 300 rpm is

(*a*) 10—35 (*b*) 35—60

(*c*) 60—300 (*d*) 300—1000.

Q11. The moving average of annual precipitation record is carried out to determine

(*a*) Trend (*b*) Annual mean

(*c*) Extreme annual variation (*d*) Extreme seasonal variation.

Q12. Match List I (Hydrological Terms) with List II (Relationship/Nature of Curve) and select the correct answer using the codes given below the Lists :

List I (Hydrological Terms)	*List II (Relationship/Nature of Curve)*
A Theissen Polygon	1 Average depth of rainfall over an area
B Mass Curve	2 Relationship of rainfall intensity and time
C Hyetograph	3 Relationship of accumulated rainfall and time
D DAD curve	4 Relationship of river run-off and time
	5 Always a falling curve.

Codes :

	A	B	C	D
(*a*)	1	3	2	5
(*b*)	1	5	3	2
(*c*)	4	3	2	5
(*d*)	4	5	3	2

Q13. Match List I (Type of Precipitation) with List II (Principal Causes) and select the correct answer using the codes given below the Lists :

List I (Type of Precipation)	*List II (Principal Causes)*
A Convective	1 Atmospheric disturbance
B Cyclonic	2 Mountain barrier
C Frontal	3 Pressure difference
D Orographic	4 Temperature difference
	5 Warm and cold air masses

Codes :

	A	B	C	D
(*a*)	1	4	5	2
(*b*)	4	3	5	2
(*c*)	1	4	2	5
(*d*)	4	3	2	5

Q14. Which one of the following characteristics describes a watershed system in system's parlance ?

(*a*) Linear (*b*) Non-linear

(*c*) Linear and time-invariant (*d*) Non-linear and time-variant.

Q15. A 6-hour storm with hourly intensities of 7, 18, 25, 12, 10 and 3 mm per hour produced a run-off of 33 mm. Then the ϕ_{index} is

(*a*) 7 mm/h (*b*) 3 mm/h (*c*) 10 mm/h (*d*) 8 mm/h.

Q16. A catchment area of 90 hectares has a run-off coefficient of 0.4. A storm of duration larger than the time of concentration of the catchment and of intensity 4.5 cm/hr creates a peak discharge rate of

(*a*) 11.3 m^3/s (*b*) 0.45 m^3/s (*c*) 450 m^3/s (*d*) 4.5 m^3/s.

Q17. Consider the following statements :

1. Time-area histogram method aims at developing an IUH
2. Isochrone is a line joining equal rainfall on a map
3. Linear reservoir is a reservoir having straight boundaries
4. Linear channel is a fictitious channel in which an inflow hydrograph passes through, with only translation and no attenuation.

Which of these statements are correct ?

(*a*) 1, 2 and 3 (*b*) 1 and 4

(*c*) 2, 3 and 4 (*d*) 1, 2, 3 and 4.

Q18. Match List I (Floods) with List II (Parameters) and select the correct answer using the codes given below the Lists :

List I (Floods)		*List II (Parameters)*	
A	Standard Project Flood (SPF)	1	Includes catastrophic floods
B	Maximum Probable Flood (MPF)	2	Includes floods of severe conditions
C	Design Flood	3	Peak flow obtained from observed data
D	Maximum Flood	4	Flood of desired recurrence interval

Codes :

	A	B	C	D
(*a*)	2	1	4	3
(*b*)	1	2	3	4
(*c*)	2	1	3	4
(*d*)	1	2	4	3

Q19. Which of the following pairs are correctly matched ?

1. Silt factor : Average size of silt particle.
2. Silt load : Volume of suspended sediments flowing with water in unit time.
3. Silt charge : Weight of silt per unit volume of water
4. Silt grade : Gradation between different silt particles

Select the correct answer using the codes given below :

Codes

(*a*) 1 and 4 (*b*) 3 and 4 (*c*) 2 and 3 (*d*) 1 and 2.

Q20. Consider the following statements :

The function of a cut-off in an earth dam is to

1. Reduce uplift pressures on the dam
2. Prevent undermining of foundation
3. Reduce loss of stored water
4. Support the dam

Which of these statements are correct ?

(*a*) 1 and 3 (*b*) 2 and 4 (*c*) 2 and 3 (*d*) 3 and 4.

Q21. Match List I (Theory) with List II (Propounded By) and select the correct answer using the codes given below the Lists :

List I (Theory)	*List II (Propounded By)*
A Exit gradient	1 G. Lacey
B Alluvial canal	2 L.K. Sherman
C Unit hydrograph	3 A.N. Khosla
D Boundary layer	4 C. Inglis
	5 T.V. Karman
	6 L. Prandtl

Codes :

	A	B	C	D
(*a*)	1	3	2	6
(*b*)	6	2	3	5
(*c*)	3	1	2	6
(*d*)	3	1	4	2

Q22. The ideal condition for energy dissipation in the design of spillways is the one when the tail water rating curve

(*a*) Lies above jump curve at all discharges

(*b*) Coincides with the jump rating curve at all discharges

(*c*) Lies below jump rating curve at all discharges

(*d*) Lies either above or below the jump rating curve depending upon discharge.

ANSWERS

1. (*d*)	**2.** (*b*)	**3.** (*b*)	**4.** (*b*)	**5.** (*a*)
6. (*i*)-(*a*), (*ii*)-(*a*)		**7.** (*a*)	**8.** (*c*)	**9.** (*b*)
10. (*a*)	**11.** (*a*)	**12.** (*a*)	**13.** (*b*)	**14.** (*c*)
15. (*d*)	**16.** (*d*)	**17.** (*b*)	**18.** (*a*)	**19.** (*c*)
20. (*c*)	**21.** (*c*)	**22.** (*b*)		

Hints for Solving Starred Quetions of the Year 2003

Q9. As pointed out in article 24.2.1, run-off river plants installed on main rivers or on diversion canals are usually provided with a small storage to meet the short term fluctuations of the water requirement of the plant. A balancing reservoir to store excess stream flow during periods of low consumption in the power house during off peak hours or days, and for its release during excess load periods, is essentially required in all such low head power development schemes. Such a balancing reservoir of small capacity, called a *pond*, is hence required near the power house.

In case of a power plant located on the river, the main weir itself provides on its upstream side the required pondage ; while in the case of a plant located on a diversion canal (power canal) the pond is required to be created at the end of the canal in the form of a **foreway reservoir**, from which the penstock pipes shall lead the water into the power house, as shown in Fig. 24.2 (*b*). No matter where the pond is located, its capacity has to be worked out considering the inflow and outflow patterns, just as in the case of

storage reservoirs. As a matter of fact, the storage reservoirs provide storage of water for long term requirements, whereas, the *pondage* takes care of the hour to hour fluctuations over a day, or occassionally, day to day fluctuations over a weekly cycle. Provision of pondage ensures that the desired instantaneous rate of flow into the turbines under day to day changing conditions of power load as well as of inflow.

Short term fluctuations may arise due to the following reasons :

(*i*) The power consumption and hence the load on turbines may suddenly increase or decrease, needing in a few seconds either excess or lesser water, respectively. This change in water demand of the power house can be immediately met by the stored water available in the pond created for this purpose. This pond would provide extra water when needed, and retain excess flow when not needed in the plant.

(*ii*) The power load and, thus, the water demand of the power house, may be steady, but the supply of water may undergo a change. In natural streams, where some other power house is located on the upstream, the available flow may change depending upon the quantity of water released from the upstream power house.

Similarly, if the power house in question is located on a diversion channel, the inflow into the power house may get affected due to some breach(es) that may occur in the said diversion canal.

(*iii*) The water demand of turbines and the natural flow (supply) may change from time to time.

The pondage, in some cases, may also be provided for weekly demands. In most of the advanced countries, the free weekends result in decreased demand of power, reducing the water need of the given power house. The inflow during this period could be ponded for use during week days.

Year 2004 (*Engineering Services*)

Q1.* Which one of the following defines aridity index (*AI*) ?

(*a*) $AI = \frac{PET - AET}{PET} \times 100$ (*b*) $AI = \frac{PET}{AET} \times 100$

(*c*) $AI = \frac{AET}{PET} \times 100$ (*d*) $AI = \frac{AET - PET}{AET} \times 100$

Q2. In Snyder's method of synthetic unit hydrograph development, basin lag is taken as

(*a*) the time interval between centroid of the rainfall excess and surface runoff
(*b*) the time interval from mid point of the unit rainfall excess to the peak of the unit hydrograph
(*c*) independent of rainfall duration
(*d*) independent of catchment characteristics.

Q3. Which one of the following is the correct sequence in the increasing order of the Froude number of flow assumed by the bed form of an alluvial stream with movable bed material ?

(*a*) Ripple — Plane bed — Dune — Plane bed — Antidune
(*b*) Dune — Ripple — Plane bed — Antidune — Plane bed
(*c*) Plane bed — Ripple — Dune — Plane bed — Antidune
(*d*) Plane bed — Ripple — Antidune — Dune — Plane bed.

Q4. A bridge has an expected life of 50 years and is designed for a flood magnitude of return period 100 years. What is the risk associated with this hydrologic design ?

(*a*) $1 - (0.99)^{50}$ (*b*) $(0.5)^{50}$ (*c*) $(0.99)^{50}$ (*d*) $(0.99)^{100}$

Q5. Match List-I with List-II and select the correct answer using the codes given below the lists :

List I		*List II*	
A	Rising limb of a hydrograph.	1	Depends on intensity of rainfall
B	Falling limb of a hydrgraph	2	Function of total channel length
C	Peak rate of flow	3	Function of catchment slope
D	Drainage density	4	Function of storage characteristics

Codes :

	A	B	C	D
(*a*)	3	4	1	2
(*b*)	1	4	3	2
(*c*)	3	2	1	4
(*d*)	1	2	3	4

Q6. Match List-I with List-II and select the correct answer using the codes given below the lists :

List I		*List II*	
A	Unit hydrograph	1	Design flood
B	Synthetic unit hydrograph	2	Pemeability
C	Darcy's law	3	Ungauged basin
D	Rational method	4	1 cm runoff

Codes :

	A	B	C	D
(*a*)	2	3	4	1
(*b*)	2	1	4	3
(*c*)	4	3	2	1
(*d*)	4	1	2	3

Q7. Which one of the following statements is correct in respect of the two important aspects of flood forecast — (1) reliability of the forecast, and (2) the time available in between the forecast and the occurrence of flood ?

(*a*) Meteorological forecast is least reliable and time available is also the least

(*b*) Hydrological forecast is most reliable but the time available is the least

(*c*) River forecast is least reliable and the time available is the maximum

(*d*) River forecast is most reliable but the time available is the least.

Q8. The delta for a crop having base period 120 days is 70 cm. What is the duty ?

(*a*) 2480 hectare/cumec (*b*) 1481 hectare/cumec

(*c*) 148 hectare/cumec (*d*) 1.481 hectare/cumec.

Q9. For a culturable command area of 1000 hectare with intensity of irrigation of

50%, the duty on field for a certain crop is 2000 hectare/cumec. What is the discharge required at head of water course with 25% losses of water ?

(*a*) 3/16 cumec (*b*) 1/4 cumec
(*c*) 1/3 cumec (*d*) 1/2 cumec.

Q10. What is the moisture depth available for evapotranspiration in root zone of 1 m depth soil, if dry weight of soil is 1.5 gm/cc, field capacity is 30% and permanent wilting point is 10% ?

(*a*) 450 mm (*b*) 300 mm
(*c*) 200 mm (*d*) 150 mm.

Q11. Which one of the following correctly defines aquiclude ?

(*a*) A saturated formation of earth material which not only stores water but also yields it in sufficient quantity
(*b*) A formation through which only seepage is possible and thus the yield is insignificant compared to an aquifer
(*c*) A geological formation which is neither porous nor permeable
(*d*) A geological formation which is essentially impermeable to the flow of water.

Q12. What is the regime scour depth for a channel in soil with silt factor of unity and carrying 8 m^2/s of discharge intensity in accordance with Lacey's regime theory ?

(*a*) 3.6 m (*b*) 4 m
(*c*) 5.4 m (*d*) 25.6 m.

Q13. Which one of the following equations represents the downstream curve of the "Ogee" spillway (where x and y are the co-ordinates of the crest profile measured from the apex of the crest and H is the design head) ?

(*a*) $x^{1.85} = 2H^{0.85}y$ (*b*) $x = 2H^{1.85}\,y^{0.85}$
(*c*) $x^{0.85} = 2H^{1.85}y$ (*d*) $x = 2H^{0.85}y^{1.85}$.

Q14.* Clark's method aims at which one of the following :

(*a*) Developing an IUH due to an instantaneous rainfall excess over a catchment
(*b*) Developing stage-discharge relationship
(*c*) Measurement of infiltration
(*d*) Flood routing through channels.

Q15.* For a saddle siphon, the maximum operative head is 6.25 m. The width and height of the throat of the siphon are 4 m and 2 m respectively. The coefficient of discharge is 0.90. How many units are required to pass a flood of 300 cumec ? (Take g = 10 m/s^2)

(*a*) One (*b*) Two
(*c*) Three (*d*) Four.

ANSWERS

1. (*a*)	**2.** (*b*)	**3.** (*c*)	**4.** (*a*)	**5.** (*a*)
6. (*c*)	**7.** (*d*)	**8.** (*b*)	**9.** (*c*)	**10.** (*b*)
11. (*d*)	**12.** (*c*)	**13.** (*a*)	**14.** (*a*)	**15.** (*d*).

Hints for Solving Sterred Questions of the Year 2004

Q1. Aridity Index, as its name implies, is an indicator of the dryness (aridness) of a soil/crop, due to which the plants do not get their full required quantity of water upto the field capacity. The max. available moisture [*i.e.* Field capacity *mc* – Wilting point *mc*], thus, does not become available to the crops, which reduces their actual evapotranspiration (*AET*) in comparison to the potential (*i.e.* maximum) evapotranspiration (*PAT*). The deficiency in evapotranspiration [*i.e.* (*PAT* – *AET*)] when expressed as the ratio with *PAT* is, hence, defined as the *Aridity Index*. Thus,

$$\text{Aridity Index} = AI = \frac{PAT - AET}{PAT} \ (\%)$$

Thus, if *PAT* is 80 mm and *AET* is 20 mm, then, naturally, the crops will be facing $\frac{80-20}{80} = 75\%$ dry conditions, since the aridity index is 75%.

Hence, choice (*a*) is correct. **Ans.**

Q14. Clark's method is one of those methods, which are used for computing an *Instantaneous Unit Hydrograph (IUH)*. Since an *IUH* is a concept of advanced Hydrology, we have not discussed it in our text. However, an IUH is explained below in brief.

Instantaneous Unit Hydrograph (IUH). The ordinary *UH*, as you know by now, is obtained from an excess rainfall (*ER*) of 1 cm magnitude uniformally falling during a time period of *T* hour, with an intensity equal to $\frac{1}{T}$ cm/h. If this rain duration (*T*) is progressively decreased, its intensity will go on increasing, and the resulting unit hydrograph will become more skewed, as shown in Fig. 32.3 by hydrograph marked 2. The hydrographs will go on becoming more skewed with progressively reduced value of *T*, as shown by unit graphs marked 3 and 4. A more skewed but certainly of finite shape unit hydrograph will finally be obtained, when the rainfall duration *T* is made infinitely small ($T \to 0$). This limiting case of a unit hydrograph produced by a rainfall of 1 cm in zero hr time, is called an *instantaneous unit hydrograph (IUH)*. Of course, this is only a theoretical concept, because 1 cm of rain cannot fall in zero hr time on any catchment, *but is useful because such a unit graph represents the watershed response to any amount of rainfall excess without reference to the duration of the*

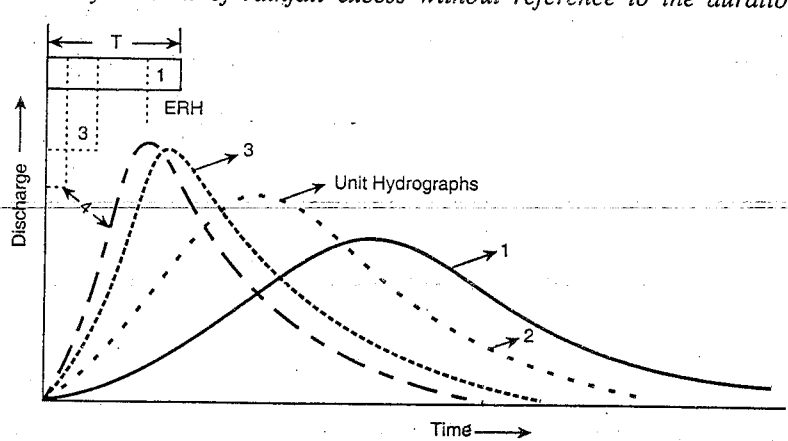

Fig. 32.3. Some Typical Unit Hydrographs.

rainfall. An *IUH*, being independent of rainfall characteristics, is indicative of the storage characteristics of a catchment, and is eminently suitable for theoretical analysis of effective rainfall-runoff relationship of a catchment.

The ideal shape of an IUH resembles that of a single-peaked direct runoff hydrograph (*DRH*) with a finite base width ; yet however, an *IUH* can have negative and undulating ordinates.

The *IUH* for a catchment can be derived from a given *ERH* (Excess rain hydrograph) and *DRH* (Direct runff hydrograph) by several methods. Say for example, the ordinate of an *IUH* at any time *t* has been shown to be equal to the slope at time *t* of the *S*-curve caused by an excess rain of intensity 1 cm/h (*i.e.* *S*-curve derived from a *UH* of 1-h duration). The *IUH* so obtained is, infact, an approximate one, because the slope of *S*-curve is difficult to be measured accurately.

Mathematical solutions evolved for determining *IUH* include. Laplace transform (chow 1964), Fourier transform (Black and others, 1971) and *Z*-transform (Bree 1978). These methods involve mathematical modelling related to watershed geomorphology, harmonic analysis, etc. which are beyond the scope of this book.

A comparatively easier method for developing an *IUH* has been evolved by Clark, and is hence known as the *Clark's method* or **Time-area histogram method.** This method aims at developing an *IUH* due to an instantaneous excess rainfall over a catchment. It is assumed that the rainfall excess first undergoes pure *translation* (channel movement) in the catchment, and then *attenuation* (reduced peak and elongated time base) as in reservoir routing. The translation is computed upto the catchment outlet by a travel time-area histogram (by ignoring the storage of the catchment) ; while the attenuation is computed by routing the result of the above, through a hypothetically available linear reservoir at the catchment outlet, to account for the storage in the catchment. A detailed description of the computations is beyond the scope of this book.

Q15. From Eq. (21.13), discharge through a saddle syphon is given as ;

$$Q = C_d \cdot A \cdot \sqrt{2gH_1}$$

where $C_d = 0.9$

A = Area of flow width × Ht. of Throat

$= 4 \text{ m} \times 2 \text{ m} = 8 \text{ m}^2$ (given)

$g = 10 \text{ m}^3/\text{s}$ (given)

$H_1 = 6.25$ m (given)

$$\therefore \quad Q = 0.9 \times 8\sqrt{2 \times 10 \times 6.25} = \mathbf{80.5\ m^3/s.}$$

Total discharge = 300 cumec (given)

No. of units of saddle syphon $= \dfrac{300}{80.5} = 3.73$; Say 4.

Hence, choice (*d*) is correct. **Ans.**

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Objective Questions of AMIE Exams (New Scheme)

Year 1993 (*Summer*) Exam

Q. Complete the sentences

(*i*) If the duty of water for a crop of base period 120 days is 1400 hec/cumec, then the delta (Δ) for the crop is

(*a*) 741 mm ; (*b*) 843 mm ;

(*c*) 949 mm ; (*d*) 1056 mm.

(*ii*) According to Khosla's theory the exit gradient is given by

(*A*) $\frac{d}{H}\frac{1}{\pi\lambda}$; (*B*) $\frac{H}{d}\frac{1}{\pi\sqrt{\lambda}}$;

(*C*) $\frac{H}{d}\pi\sqrt{\lambda}$ (*D*) $\frac{d}{H}\pi\lambda$.

(*iii*) If the dominant discharge of an alluvial river is 1800 cumecs, the waterway for a bridge on the river should be

(*A*) 101.5 m ; (*B*) 151.5 m ;

(*C*) 201.5 m ; (*D*) 251.5 m.

(*iv*) Slite excluder is provided

(*A*) in the canal on the downstream side of the head regulator

(*B*) in the river on the downstream side of the barrage ;

(*C*) in the river far away from the weir on the downstream side

(*D*) in the river adjacent to the head regulator.

(*v*) According to Dicken's formula, the peak flood discharge is proportional to

(*A*) area ; (*B*) $(\text{area})^{1/2}$;

(*C*) $(\text{area})^{2/3}$; (*D*) $(\text{area})^{3/4}$.

(*vi*) The head due to sudden expansion is expressed by

(*A*) $\frac{v_1^2 - v_2^2}{2g}$; (*B*) $\frac{(v_1 - v_2)^2}{g}$;

(*C*) $\frac{(v_1 - v_2)^2}{2g}$; (*D*) $\left(\frac{v_1 - v_2}{2g}\right)^2$.

(*vii*)* A rectangular channel of 4 m width carries water at the rate of 8 cumecs under critical conditions. Specific energy for the flow is...

(*viii*)* In a syphon aqueduct the most severe condition of uplift on the floor occurs when

(*A*) Canal runs full drain is dry but the water table is at stream bed ;

(*B*) Canal is dry and drain is at high flood level ;

(*C*) Canal runs dry and the drain also runs dry

(*D*) Both the canal and drain run full.

(*ix*)* If the conjugate depths for a horizontal rectangular channel 4 m wide are 0.2 m and 1 m the discharge in the channel is — m^3/s.

(*x*) If y_1 and y_2 are the depths of flow before and after the hydraulic jump in a rectangular channel then the loss of energy is given by

(*A*) $\dfrac{(y_2-y_1)^3}{4y_1 y_2}$; (*B*) $\dfrac{(y_2-y_1)^2}{4y_1 y_2}$;

(*C*) $\dfrac{(y_2-y_1)^3}{y_1 y_2}$; (*D*) $\dfrac{(y_2-y_1)^2}{y_1 y_2}$.

Answers to Questions of 1993 (*Summer*) Exam

(*i*)–(*a*)	(*ii*)–*B*	(*iii*)–C
(*iv*)–*D*	(*v*)–*D*	(*vi*)–*A*
(*vii*)–1.11	(*viii*)–*B*	(*ix*)–4.34
(*x*)–*A*		

Hints for Solving Starred Questions of 1993 (*Summer*) Exam.

Q. (*vii*) $B = 4$ m, $Q = 8$ cumecs at critical depth

q at critical depth $= \dfrac{8}{4} = 2\ m^3/s$

But $$y_c = \sqrt[3]{\frac{q^2}{g}} = \sqrt[3]{\frac{2^2}{9.81}} = 0.74\ m$$

Also, $$\frac{Q}{A} = v_c = \frac{8}{4 \times 0.74} = 2.7\ m/s$$

$$E_{fc} = y_c + \frac{v_c^2}{2g} = 0.74 + \frac{(2.7)^2}{2 \times 9.81} = \mathbf{1.11\ m} \quad \textbf{Ans.}$$

Q. (*ix*) $y_1 = 0.2$ m ; $y_2 = 1$ m

using eq. (10.6), we have,

$$y_2 = -\frac{y_1}{2} + \sqrt{\frac{y_1^2}{4} + \frac{2q^2}{g \cdot y_1}}$$

or $$1 = -\frac{0.2}{2} + \sqrt{\frac{0.2^2}{4} + \frac{2.q^2}{9.81 \times 0.2}}$$

or $$1+0.1=\sqrt{0.01+\frac{q^2}{0.981}}$$

$$1.1^2=1.21=0.01+\frac{q^2}{0.981}$$

or $$q=\sqrt{1.20\times 0.981}=1.085$$

$$Q=q.B=1.085\times 4=\mathbf{4.34\ m^3/s}$$ **Ans.**

Year 1993 (*Winter*) **Exam**

Q. (*A*) Fill in the blanks :

(*i*) Where water must be raised by pumps or other means to irrigate crops is called ..... irrigation.

(*ii*) Soil in which the groundwater table rises so high as to affect the productivity of the soil due to air circulation to the plant root zone is called .....

(*iii*) An irrigation outlet whose discharge is independent of water levels in the distributary and the water course and which ensures constant supply within certain working limits is called a .....

(*iv*) The scour depth for a channel in fine silt is ..... than in coarse silt.

(*v*) Excess flood over a concrete dam is released through the .....

(*vi*) The loss of water from plant surfaces and utilised in its growth is known as .....

(*B*) Choose the correct statement for the following :

(*i*) The bed slope of a canal off taking from a barrage is kept

(*a*) steeper than the bed slope of the river

(*b*) milder than the bed slope of the river

(*c*) same as that of the river

(*d*) none of the above

(*ii*) For the same height and top width, the width at the bottom of the earth dam is

(*a*) less than that of a concrete dam

(*b*) same as that of a concrete dam

(*c*) more than that of a concrete dam

(*d*) none of the above

(*iii*) For a given discharge, the afflux for a pipe culvert will be

(*a*) the same as for a large opening

(*b*) independent of the size of the opening

(*c*) more than that for a large opening

(*d*) none of the above

(*iv*) Lining of a canal is required when

(*a*) the water table is above the full supply level of canal

(*b*) the water table is below the bed of the canal

(*c*) there is a natural impervious stratum below the bed

(*d*) none of the above

(*v*) In a deflecting spur, the angle between the axis of the spur and the bank on the upstream side is

(*a*) acute

(*b*) obtuse

(*c*) same as on the downstream side

(*d*) one of these

(*vi*) Khosla's theory of indpendent variables is applicable to structure founded on

(*a*) rock (*b*) clay

(*c*) sands (*d*) concrete

(*vii*) A cross regulator is provided on a main canal

(*a*) to minimise the amount of silt entering the branch canal

(*b*) to let maximum silt be carried into the branch canal

(*c*) for no specific purpose

(*d*) to carry the canal across the drain

(*C*) Match the sentences of Column A to that in Column -B :

	Column-A	*Column-B,*
(*i*)	(*a*) In storage irrigation	(*A*) a barrage or a weir is constructed to divert water directly into the canal
	(*b*) In direct irrigation	(*B*) water is supplied to individual trees as in orchards
	(*c*) In sprinkler irrigation	(*C*) a dam is constructed to store water during the monsoons and supply water to canals during non-monsoon season
	(*d*) In basin irrigation	(*D*) water is applied to the surface of the soil in the form of a spray.
(*ii*)	(*a*) In an aqueduct	(*A*) the drainage water is carried in a trough supported on piers while their irrigation canal passes below with its FSL below the slab of the trough above.
	(*b*) In a level crossing	(*B*) the drainage water is carried in a trough supported on piers while the irrigation canal passes below with its FSL above the slab of the trough above
	(*c*) In a super-passage	(*C*) the canal and the drain approach each other practically at the same level
	(*d*) In a syphon	(*D*) the irrigation canal is carried across the drain in a trough supported on piers with the canal bed level well above the HFL of the drain

(*iii*)	(*a*) Surface run-off is that form of	(*A*) precipitation in which the drops of water are under 0.5 mm in diameter and intensity is usually less than 1 mm per hour
	(*b*) Infiltration is that form of	(*B*) the hydrologic cycle in which water moves from the surface of the ground into the soil
	(*c*) Drizzle is that form of	(*C*) precipitation in which the water drops are greater than 0.5 mm diameter
	(*d*) Rain is that form of	(*D*) precipitation which travels over land surface or just below it
(*iv*)	(*a*) For an earthen dam	(*A*) the rock in the abutment must be stronger than in the foundation
	(*b*) For an arch dam	(*B*) the foundation may be rock or soil
	(*c*) For a solid gravity dam	(*C*) the foundation should consist of pervious material
	(*d*) For a barrage	(*D*) the foundation rock should be stronger than in the abutments
(*v*)	(*a*) Rabi season	(*A*) last from Ist April to 30th September
	(*b*) Kharif season	(*B*) is the total quantity of water needed for growth of crop to maturity
	(*c*) Delta	(*C*) is the relation between the volume of water and the area of the crop which it matures
	(*d*) Duty	(*D*) starts on Ist October and ends on 31st March
(*vi*)	(*a*) Undersluices	(*A*) regulates the supply of water in the canal
	(*b*) Canal head regulator	(*B*) enable the fish to migrate to their spawning grounds upstream of the barrage
	(*c*) Barrage bays	(*C*) preserve a clear and defined river channel approaching the canal regulator
	(*d*) Fish ladder	(*D*) are operated to release the excess water during floods
(*vii*)	(*a*) Cross drainage works include	(*A*) notches, escapes, sluices and syphon well drops
	(*b*) Works for regulation of water levels include	(*B*) bridges and culverts
	(*c*) Communication works include	(*C*) spurs, groynes, bunds, mattersses and aprons
	(*d*) River training works include	(*D*) aqueducts, syphons, syphon, aqueducts, superpassages and level crossings

Answers to Questions of 1993 (*Winter*) Exam.

A (*i*) – Lift A (*ii*) – saline A (*iii*) – rigid module or modular outlet

A (*iv*) – more A (*v*) – spillway A (*vi*) – transpiration loss

B (*i*)– (*d*) B (*ii*) – (*c*) B (*iii*) – (*c*)

B (*iv*) – (*d*) B (*v*) – (*a*) B (*vi*) –(*c*)

B (*vii*)– (*a*)

C (i) : (*a*)–(*C*), (*b*)–(*A*)
(*c*)–(*D*), (*d*)–(*B*)

C (ii) : (*a*)–(*D*), (*b*)–(*C*)
(*c*)–(*A*), (*d*)–(*B*)

C (iii) : (*a*)–(*D*)
(*b*)–(*B*)
(*c*)–(*A*)
(*d*)–(*C*)

C (iv) : (*a*)–(*B*)
(*b*)–(*A*)
(*c*)–(*D*)
(*d*)–(*C*)

C (v) : (*a*)–(*D*)
(*b*)–(*A*)
(*c*)–(*B*)
(*d*)–(*C*)

C (vi) : (*a*)–(*C*)
(*b*)–(*A*)
(*c*)–(*D*)
(*d*)–(*B*)

Year 1994 (*Summer*) Exam.

Q. Choose the correct answer for the following :

(*i*) Evaporation is measured by using
- (*a*) an open pan
- (*b*) lysimeter
- (*c*) infiltrometer
- (*d*) a neutron tube

(*ii*) The Standard Symon's Type Rain Gauge has a collecting a diameter
- (*a*) 10 cm
- (*b*) 12.7 cm
- (*c*) 5.08 cm
- (*d*) 25.4 cm

(*iii*) The average annual rainfall over whole of India is estimated as :
- (*a*) 189 cm
- (*b*) 319 cm
- (*c*) 89 cm
- (*d*) 117 cm

(*iv*) An Isohyet is a line joining points having
- (*a*) equal evaporation value
- (*b*) equal barometric pressure
- (*c*) equal height above MSL
- (*d*) equal rainfall depth in a given duration

(*v*) A coffer dam is
- (*a*) kind of gravity dam
- (*b*) an earthen dam of small height built to protect important areas
- (*c*) a temporary structure constructed to exclude water from the work area during construction.
- (*d*) an embankment built along a river to regulate the river

(*vi*) In an Inundation Irrigation System there will
- (*a*) be neither a diversion work nor a regulator
- (*b*) be only a diversion structure across the river
- (*c*) be both a regulator on the canal as well as a diversion structure across the river

(*vii*) In well drained soil, the useful soil moisture for plant growth essentially comes from
- (*a*) gravity water
- (*b*) capillary water
- (*c*) hygroscopic water
- (*d*) water of adhestion

(*viii*) A land is known as waterlogged when
- (*a*) the *p*H of soil reaches a value of 8.5
- (*b*) the soil in the root zone has high salinity
- (*c*) the soil pores in the root are saturated with water
- (*d*) there is water flowing on the surface of the land

(*ix*) Irrigation water having an SAR value of 20 is called as
- (*a*) very high sodium water
- (*b*) high sodium water
- (*c*) medium sodium water
- (*d*) low sodium water

(*x*) Alkali soils are reclaimed by
- (*a*) leaching only
- (*b*) addition of gypsum and leaching
- (*c*) addition of gypum only
- (*d*) provision of drainage

(*xi*) A groyne pointing upstream in a river is known as
- (*a*) attracting groyne
- (*b*) repelling groyne
- (*c*) fending groyne
- (*d*) Denehy's grone

(*xii*) The chemical suitable for use as a water evaporation retarding and reducing agent is
- (*a*) butyl alcohol
- (*b*) methyl alcohol
- (*c*) cetyl alcohol
- (*d*) ethyl alcohol

(*xiii*) The chemical dilution method of stream gauging is ideally suited for measuring discharges in
- (*a*) a large alluvial river
- (*b*) steady flow in a small turbulent stream
- (*c*) a stretch of a river having heavy pollution
- (*d*) a tidal estuary

(*xiv*) Consumptive use is
- (*a*) the water used up in plant metabolism
- (*b*) the sum of evapotranspiration and amount used up in plant metabolism
- (*c*) the sum of evapotranspiration and infiltration losses
- (*d*) the combined use of surface and ground water resources

(*xv*) Conjunctive use of water in a basin means
- (*a*) combined use of water for irrigation and for hydro-power generation
- (*b*) combined use of surface and groundwater resources
- (*c*) use of irrigation water for both Rabi and Kharif crops
- (*d*) use of irrigation water by a cooperative of farmers

(*xvi*)* Permanent wilting point is
- (*a*) a characteristic of the plant
- (*b*) a soil characteristic
- (*c*) a soil characteristic modified by the crop
- (*d*) dependent on soil water plant fertiliser interaction

(*xvii*) A hytograph is a plot of
- (*a*) cumulative rainfall versus time
- (*b*) rainfall depth versus duration
- (*c*) evaporation rate versus time
- (*d*) rainfall intensity versus time

(*xviii*) The doubles mass analysis is adopted to
- (*a*) estimate the missing rainfall data
- (*b*) obtain intensities of rainfall at various duration
- (*c*) check the consistency of data
- (*d*) obtain the amount of storage needed to maintain a demand pattern

(*xix*) There is minimum wastage of water in
- (*a*) furrow irrigation
- (*b*) check basin irrigation
- (*c*) sprinkler irrigation
- (*d*) border method of irrigation

(*xx*) The coefficient of roughness for an earthen canal in excellent condition is
- (*a*) 0.01
- (*b*) 0.0225
- (*c*) 0.025
- (*d*) 0.7.

Answers to Questions of 1994 (*Summer*) Exam

(*i*)–(*a*)	(*ii*)–(*b*)	(*iii*)–(*d*)
(*iv*)–(*c*)	(*v*)–(*c*)	(*vi*)–(*c*)
(*vii*)–(*b*)	(*viii*)–(*c*)	(*ix*)–(*b*)
(*x*)–(*b*)	(*xi*)–(*b*)	(*xii*)–(*c*)
(*xii*)–(*b*)	(*xiv*)–(*b*)	(*xv*)–(*b*)
(*xvi*)–(*b*)	(*xvii*)–(*d*)	(*xviii*)–(*c*)
(*xix*)–(*c*)	(*xx*)–(*b*)	

Hints for Solving Starred Questions of Year 1994 (*Summer*) Exam

Q. (*xvi*) *Permanent wilting percentage or permanent wilting point or wilting coefficient is a soil characteristic, since all plants whose root systems thoroughly permeats the soil wilt at nearly the same soil moisture content ; when grown in a particular soil in a humid atmosphere.*

Year 1994 (*Winter*) Exam.

Q. (*A*) Fill in the blanks :

(*i*) A 6-hours storm had 6 cm of rainfall and the resulting runoff was 3 cm. If the ϕ-index remains at the same value, the runoff due to 12 cm of rainfall in 9 hours in the catchment is .... cm.

(*ii*) The duty for a crop with a base period of 140 days is 3456 hectare per cumec. The delta for the crop will be ..... m.

(*iii*) A one-day rainfall of 10.0 cm at a place was found to have return period of 100 years. The probability that a one day rainfall of this or larger magnitude will occur in the next year is .....

(*B*) Choose the correct answer for the following :

(*i*) Retrogression of downstream levels, generally considered in the design of weirs or barrages, is

(*a*) higher at high flood stage than at low water levels
(*b*) same at high flood stage and at low water levels
(*c*) higher at low water levels stage than at high flood stage
(*d*) independent of the stage of flow.

(*ii*) The average pan coefficient for the standard US Weather Bureau class A pan is

(*a*) 0.55 (*b*) 0.70
(*c*) 0.85 (*d*) 0.95.

(*iii*) To determine discharge of a stream from rating curve, at a section, one needs

(*a*) current meter readings at various verticals at the section
(*b*) slope of the water surface at the section
(*c*) stage at the section
(*d*) surface velocity at various sections.

(*iv*) The useful moisture of a soil is equal to its

(*a*) field capacity
(*b*) saturation capacity
(*c*) moisture content at permanent wilting point
(*d*) difference between field capacity and permanent wilting point within the root zone of plants.

(*v*) Distribution outlet is said to be proportional, if its flexibility is

(*a*) equal to unity (*b*) less than unity
(*c*) slightly greater than unity (*d*) equal to two.

(*vi*) If average particle size of silt in millimetres is m, then Lacey's silt factor (f) is proportional to

(*a*) m^2 (*b*) $m\ m^{1/2}$
(*c*) $m^{\frac{1}{2}}$ (*d*) $m^{1/3}$.

(*vii*) The basic assumptions of unit hydrograph theory are
- (*a*) linear response and non-linear time variance
- (*b*) linear response and time invariance
- (*c*) non-linear response and time invariance
- (*d*) linear response and linear time variance.

(*viii*) The monthly rainfall at a place is generally indicated as above or below *normal*. The term *normal* means
- (*a*) the rainfall in the same month of the previous year
- (*b*) the rainfall expected based on previous month's data
- (*c*) the average rainfall of the previous twelve months
- (*d*) the average monthly rainfall for the same month computed from 30 years of past record.

(*ix*) The optimum depth of kor-watering for rice crop, is
- (*a*) 22 cm (*b*) 19 cm
- (*c*) 16 cm (*d*) 13 cm.

(*x*) The main function of division headworks provided at the off-take of a canal from a river, is
- (*a*) to raise the water level in the river
- (*b*) to control the floods
- (*c*) to store water
- (*d*) to control entry of silt into the canal.

(*xi*)* The downstream expansion head of a guide bank is extended to subtend an angle at the centre between
- (*a*) 30° and 45° (*b*) 45° and 60°
- (*c*) 60°and 75° (*d*) 75° and 90°.

(*xii*)* Direct runoff is made up of
- (*a*) surface runoff, prompt interflow and channel precipitation
- (*b*) surface runoff, infiltration and evapotranspiration
- (*c*) overland flow and quick return flow
- (*d*) infiltration, rainfall and evaporation.

(*xiii*)* The velocity of drainage water in the barrels of syphon aqueduct is normally limited to
- (*a*) 1 to 2 m per second (*b*) 2 to 3 m per second
- (*c*) 3 to 4 m per second (*d*) 4 to 5 m per second.

(*xiv*) Cross-regulators in main canals are provided to
- (*a*) regulate water supply in the off-taking channel
- (*b*) regulate water supply in the main channel
- (*c*) regulate excessive flood water
- (*d*) head up water for adequate supply into the off-taking channel.

Answers to Question of 1994 (*Winter*) Exam.

A (*i*) – 7.5	*A* (*ii*) – 0.35	*A* (*iii*) – 0.01
B (*i*) – (*c*)	*B* (*ii*) – (*b*)	*B* (*iii*) – (*c*)
B (*iv*) – (*d*)	*B* (*v*) – (*a*)	*B* (*vi*) – (*c*)
B (*vii*) – (*b*)	*B* (*viii*)– (*d*)	*B* (*ix*) – (*b*)
B (*x*) – (*c*)	*B* (*xi*)– (*b*)	*B* (*xii*)– (*c*)
B (*xiii*) – (*b*)	*B* (*xiv*)– (*d*)	

Hints for Solving Starred Questions of 1994 (*Winter*) Exam

(*xi*) Please refer Fig. 8.19 on page 583
(*xii*) D.R.O = S.R.O = True S.R.O. + Sub surface storm flow or quick return flow
(*xiii*) Please refer articles 14.5.2 and 14.5.3 on page 850

Year 1995 (*Winter*) Exam

Q. (*A*) Choose the correct answer for the following :

(*i*) Conjuctive use of water in a basin means ;
- (*a*) Combined use of water for irrigation and for hydropower generation ;
- (*b*) Combined use of surface and ground water resources ;
- (*c*) Use of irrigation water for both rabi and kharif season ;
- (*d*) Use of irrigation water by cooperative of farmers.

(*ii*) Consumptive use is
- (*a*) the water used up in plant metabolism
- (*b*) the sum of evapotranspiration and the amount used up in plant metabolism;
- (*c*) the sum of evapotranspiration and infiltration losses ;
- (*d*) the combined use of surface and ground water resources.

(*iii*) A plot between rainfall intensity versus time is called
- (*a*) mass curve ; (*b*) hyetograph ;
- (*c*) isohyet ; (*d*) hydrograph.

(*iv*) The volume of water that can be extracted by force of gravity from a unit volume of aquifer material is known as
- (*a*) specific yield ; (*b*) safe yield ;
- (*c*) specific retention (*d*) specific storage.

(*v*) For plant's growth useful soil moisture is
- (*a*) capillary water ; (*b*) gravity water ;
- (*c*) hygroscopic water ; (*d*) chemical water.

(*vi*) The normal average annual rainfall over the whose of India is estimated as
- (*a*) 88 cm ; (*b*) 217 cm ;
- (*c*) 119 cm ; (*d*) 290 cm.

(*vii*)* The minimum free board recommended for lined canals carrying a discharge greater than 10 m^3/s is

(*a*) 0.15 m ; (*b*) 0.60 m ;
(*c*) 1.25 m ; (*d*) 0.75 m.

(*viii*) A land is known as water logged when

(*a*) the pH of the soil reaches a value of 8.5 ;
(*b*) the soil in the root zone has high salinity
(*c*) the soil pores in the root zone are saturated with water ;
(*d*) there is water flowing on the surface of the land.

(*ix*) A characteristic feature of a barrage is

(*a*) the provision of a raised crest ;
(*b*) the creation of a storage resevoir on the upstream side ;
(*c*) the provision of a series of gates across the river for flow regulation ;
(*d*) that it is built in delta areas only.

(*x*) The use of rational formula for estimating floods is limited to catchments of sizes less than

(*a*) 5000 km^2 ; (*b*) 500 km^2 ;
(*c*) 50 km^2 ; (*d*) 5 km^2.

(*xi*) The average pan coefficient for the standard U.S. Weather Bureau class *A* pan is

(*a*) 0.60 ; (*b*) 0.65 ;
(*c*) 0.70 ; (*d*) 0.75.

(*xii*) The standard Symon's type Rain Gauge has a collecting area of diameter

(*a*) 5.08 cm ; (*b*) 10 cm ;
(*c*) 12.7 cm ; (*d*) 25.4 cm.

(*xiii*) A medium head Hydroelectric Scheme is one where

(*a*) Gross head is between 15 m to 60 m ;
(*b*) Net head operating on turbine is between 15 m to 60 m ;
(*c*) Total loss of head in the penstocks is between 15 m to 60 m.
(*d*) The difference in the level of tail water and centre line of turbine is between 15 m to 60 m.

(*xiv*) Dicken's formula is the form of

(*a*) $Q = CM^{1/3}$; (*b*) $Q = CM^{2/3}$;
(*c*) $Q = CM^{1/4}$; (*d*) $Q = CM^{3/4}$.

where *M* is the catchment Area

(*B*) Fill in the blanks :

(*i*)* A 2 hour unit hydrograph of a catchment is triangular in shape with a time base of 60 hour and peak ordinate of 30 m^3/s. The area of the catchment is ..... km^2

(*ii*) The delta for a crop with a base period of 150 days is 50 cm. The duty for the crop will be ..... hectares per cumec.

(*iii*)* A mean annual runoff of 1 m^3/s from a catchment of area 10 km^2 represents an effective rainfall ..... cm.

Answers to Questions o 1995 (*Winter*) Exam

A (*i*) –	(*b*)	*A* (*ii*)–	(*b*)	*A* (*iii*)–	(*b*)
A (*iv*) –	(*a*)	*A* (*v*)–	(*a*)	*A* (*vi*)–	(*c*)
A (*vii*) –	(*d*)	*A* (*viii*)–	(*c*)	*A* (*ix*)–	(*c*)
A (*x*) –	(*c*)	*A* (*xi*)–	(*c*)	*A* (*xii*)–	(*c*)
A (*xiii*)–	(*b*)	*A* (*xiv*)–	(*d*)	*B* (*i*) –	648
B (*ii*)–	2592	*B* (*iii*)–	315.36		

Hints for Solving Starred Questions of 1995 (*Winter*) Exam

Q. A (*vii*) Please refer Table 4.11

Q. B (*i*) $$\frac{\frac{30\,m^3}{s} \times (60 \times 60 \times 60s) \times 100}{\text{Catchment area } A \text{ in m}^2} = 1 \text{ cm}$$

$$\therefore A = 648 \times 10^6 \text{ m}^2 = 648 \text{ km}^2$$

Q. B (*iii*) Vol. of water in 1 yr $= \frac{1\,m^3}{s} \times (365 \times 24 \times 60 \times 60$ secs in a yr

$$= 315.36 \times 10^5 \text{ m}^3$$

$$\text{Eff. rainfall depth} = \frac{\text{Volume of water}}{\text{Area}} = \frac{315.36 \times 10^5\,m^3}{10 \times 10^6 \text{ m}^2}$$

$$= \frac{315.36}{10^2} \text{ m} = 315.36 \text{ cm.}$$ **Ans.**

Year 1996 (*Summer*) Exam

Q. (*A*) Fill in the blanks :

(*i*)* A canal is 80 km long and has an average surface width of 15 m. If the average evaporation measured in a standard U.S. Weather Bureau Class, a pan is 0.5 cm/day, the volume of water evaporation in the month of September is ..... cubic metres.

(*ii*)* If the wind velocity at a height of 2 m above ground is 5.0 km per hour, its value at a height of 10 m above ground can be expected to be about ..... km per hour.

(*iii*)* In a triangular channel the top width and depth of flow were 2.0 m and 0.9 m respectively. Velocity measurements on the centre line at 18 cm and 72 cm below the free water surface indicated velocities of 0.6 m/s and 0.4 m/s respectively. The discharge in the channel is ..... cumec.

(*iv*)* During a flood in a wide rectangular channel it is found that at a section the depth of flow increases by 50 per cent and at this depth the water surface slope is half its original value in a given interval of time. This marks an approximate change in the discharges of ..... per cent.

(*v*)* A triangular direct runoff hydrograph due to a storm has a time base of 80 hours and a peak flow of 50 m^3/s occurring at 20 hours from the start. If the catchment area is 144 km^2, the rainfall excess in the strom was ..... cm.

(*B*) Choose the correct answer for the following :

(*i*) Cold front precipitation is :
- (*a*) very intense and of short duration
- (*b*) very intense and of considerable duration
- (*c*) light and of short duration
- (*d*) Showery and of considerable duration.

(*ii*) A hyetograph is a plot against time of :
- (*a*) cumulative rainfall (*b*) Rainfall intensity
- (*c*) discharge (*d*) stage.

(*iii*) The mass curve of rainfall of a storm is plot of :
- (*a*) rainfall depths for various equal durations plotted in decreasing order
- (*b*) rainfall depths for various equal durations plotted in increasing order
- (*c*) rainfall intensity *vs.* time in chronological order
- (*d*) accumulated precipitation *vs.* time in chronological order.

(*iv*) The slope-area method is extensively used in :
- (*a*) development of rating curve
- (*b*) estimation of flood discharge based on high-water marks
- (*c*) cases where shifting control exist
- (*d*) cases where backwater effect is present.

(*v*) The term base flow denotes delayed flow reaching a stream from :
- (*a*) groundwater only
- (*b*) ground water and snow melt
- (*c*) groundwater and inter flow
- (*d*) groundwater, snow melt and inteflow.

(*vi*) A module is said to be 'proportional' when 'flexibility' equals :
- (*a*) 0.8 (*b*) 0.9
- (*c*) 1.0 (*d*) 1.1.

(*vii*)* For consolidation grouting, depth of *B*-holes drilled through the foundation rock generally varies between :
- (*a*) 5 and 10 m (*b*) 10 and 15 m
- (*c*) 15 and 20 m (*c*) 20 and 25 m.

(*viii*)* According to Gales the ratio of upstream length of guide bank to the length of structure between the aboutments for flood discharges more than 20000 cumecs shall be :
- (*a*) 1.3 (*b*) 1.4
- (*c*) 1.5 (*d*) 1.6.

(*ix*) For row crops, e.g., potatoes, sugarbeets, etc. the most commonly adopted method of surface irrigation is :
- (*a*) check flooding (*b*) free flooding
- (*c*) basin flooding (*d*) furrow and corrugation irrigation.

(*x*) According to Khosla, to keep the structure safe against piping, exit gradient to be provided lies between :

(*a*) 0.10 and 0.15 (*b*) 0.15 and 0.20

(*c*) 0.20 and 0.26 (*d*) 0.25 and 0.30.

Answers to Questions of 1996 (*Summer*) Exam

A (*i*)– 126000	*A* (*ii*)– 6.29	*A* (*iii*)– 0.45
A (*iv*)– 39	*A* (*v*)– 5	
B (*i*)– (*a*)	*B* (*ii*)– (*b*)	*B* (*iii*)– (*d*)
B (*iv*)– (*b*)	*B* (*v*)– (*a*)	*B* (*vi*)– (*c*)
B (*vii*)– (*b*)	*B* (*viii*)– (*c*)	*B* (*ix*)– (*d*)
B (*x*)– (*c*)		

Hints for Solving Starred Questions of 1996 (*Summer*) Exam

Q. A (*i*) Actual Evaporation from canal during September month

$$= (\text{Area of Canal}) \times \text{Evaporation rate} \times \text{No of days} \times \text{Pan coeff.}$$

$$= (80 \text{ km} \times 15 \text{ m})\left(\frac{0.5 \text{ m}}{100 \text{ day}} \times 0.7\right) \times 30 \text{ days}$$

$$= \left[80000 \times 15 \times \frac{0.35}{100} \times 30 \text{ m}^3\right] = \mathbf{126000\ m^3}\ \textbf{Ans.}$$

Q. A (*ii*) Using Eqn. (7.43 a), we have

$$\frac{V_{z1}}{V_{z2}} = \left(\frac{z_1}{z_2}\right)^{0.143}$$

where $V_{z1} = V_{2m} = 5.0$ km/hr

$z_1 = 2$ m

$z_2 = 10$ m

$$\therefore \frac{V_{2m}}{V_{10m}} = \left(\frac{2}{10}\right)^{0.143} = \frac{5.0 \text{ km/h}}{V_{10}} m$$

or $$V_{10m} = \frac{5 \text{ km/h}}{0.794} = \mathbf{6.29\ km/h}\ \textbf{Ans.}$$

Q. A (*iii*) Vel. at 0.2y depth =- vel at 0.18 m depth = 0.6 m/s

Vel. at 0.8y depth = vel. at 0.72m depth = 0.4 m/s

$$\text{Av. mean velocity} = \frac{0.6 + 0.4}{2} = 0.5 \text{ m/s}$$

$$\text{Area of channel}(A) = \frac{1}{2} \times (2.0 \text{ m} \times 0.9 \text{ m}) = 0.9 \text{ m}^2$$

Discharge $Q = V.A = 0.5 \text{ m/s} \times 0.9 \text{ m}^2$

$= 0.45 \text{ m}^3/\text{s} =$ **0.45 cumecs** **Ans.**

Q. A (*iv*) For wide rectangular channel.

$$Q = \frac{1}{n} \cdot A.R^{2/3} \sqrt{S_f}$$

$$= \frac{1}{n} \cdot (B \cdot y) \cdot (y)^{2/3} \cdot \sqrt{S_f}$$

or $$Q = \frac{1}{n} \cdot y^{5/3} \cdot \sqrt{S_f}$$

$$\therefore \quad \frac{Q_2}{Q_1} = \frac{y_2^{5/3} \cdot \sqrt{S_{f_2}}}{y_1^{5/3} \cdot \sqrt{S_{f_1}}}$$

But $$y_2 = 1.5\, y_1$$

$$S_{f_2} = \frac{1}{2} S_{f_1} = 0.5\, S_{f_1}$$

$$\therefore \quad \frac{Q_2}{Q_1} = \frac{(1.5)^{5/3} \cdot y_1^{5/3} \cdot \sqrt{0.5} \cdot \sqrt{S_{f_1}}}{y_1^{5/3} \cdot \sqrt{S_{f_1}}} = (1.5)^{5/3} \cdot \sqrt{0.5} = 1.39$$

%age change in discharge $= \dfrac{Q_2 - Q_1}{Q_1} \times 100$

$$= \left(\frac{Q_2}{Q_1} - 1\right) 100 = (1.39 - 1)\, 100 = \mathbf{39\%} \quad \textbf{Ans.}$$

Q. A (*v*) Vol. of water contained in the U.H =

$$= \frac{1}{2} \times 50 \text{ m}^3/\text{s} \times 80 \text{ h}$$

$$= \frac{2000\, m^3 \cdot h}{s} = 2000\, m^3 \times 3600 \frac{s}{s}$$

$$= 7.2 \times 10^6\, m^3.$$

$$\text{Depth of water} = \frac{\text{Volume of water}}{\text{C.A}} = \frac{7.2 \times 10^6\, m^3}{144 \times 10^6\, m^3}$$

$= 0.05 \text{ m} = 5 \text{ cm}$ **Ans.**

Q. B (*vii*) Please refer article 19.15 (2).

Q. B. (*viii*) Please refer article 8.5.3 (2) (*iii*).

Year 1996 (*Winter*) Exam

Q. (*A*) Choose the correct answer in the following

(*i*) The interval of supply of irrigation water depends on the type of

(*a*) crop only (*b*) soil and crop

(*c*) crop and climate (*d*) soil, crop and climate.

(*ii*) Increase in the duration of the unit hydrograph causes an increase in the

(*a*) peak discharge (*d*) rainfall intensity

(*c*) equilibrium discharge (*d*) time base.

(*iii*) Fish ladder is provided on the side of

(*a*) divide wall (*b*) diaphragm wall

(*c*) core wall (*d*) wing wall.

(*iv*) The volume of water stored between normal reservoir level and the maximum reservoir level is known as

(*a*) live storage (*b*) surcharge storage

(*c*) valley storage (*d*) bank storage.

(*v*) Consumptive use is the sum of

(*a*) evapotranspiration and infiltration losses

(*b*) evapotranspiration plus application losses

(*c*) evapotranspiration plus the amount of water used up in plant metabolism

(*d*) evapotranspiration plus the amount of water used up in plant metabolism plus percolation losses.

(*vi*) Other factors remaining same, the open water evaporation rate increases as

(*a*) sunshine hours increase

(*b*) altitude decreases

(*c*) barometric pressure increases

(*d*) salt content of water decreases.

(*vii*) For a barrage, the exit gradient is independent of

(*a*) the applied head of water

(*b*) the horizontal length of floor

(*c*) the depth of upstream cut-off

(*d*) the depth of downstream cut-off.

(*viii*) A syphon aqueduct is provided when

(*a*) the canal bed level is well above the HFL of the drain

(*b*) the canal bed level lies between the bed level and HFL of the drain

(*c*) the drain bed level is well above the FSL of the canal

(*d*) the drain bed level lies between the bed level and FSL of the canal.

(*ix*) An inundation irrigation system requires the construction of

(*a*) only a diversion structure

(*b*) only a canal regulator

(*c*) neither a diversion structure nor a canal regulator

(*d*) both a diversion structure and a canal regulator.

(*x*) In order to reduce uplift on a gravity dam, the type of grouting done is

(*a*) curtain grouting near the heel

(*b*) consolidation grouting near the heel

(*c*) curtain grouting near the toe

(*d*) consolidation grouting near the toe.

(*xi*) In order to keep the saturation line in a canal embankment well within the toe, it is necessary to provide

(*a*) spoil bank (*b*) counterberm

(*c*) free board (*d*) berm.

(*xii*) The term piping used in connection with diversion structures on permeable soil is associated with

(*a*) drainage of seepage water

(*b*) measurement of uplift pressure

(*c*) failure initiated by boiling

(*d*) consolidation of foundation.

(*xiii*) The upstream face of an earth dam is

(*a*) an equipotential line (*b*) a streamline

(*c*) a phreatic line (*d*) a streak line

(*xiv*) Silt ejector is provided on

(*a*) the river, just upstream of the undersluices

(*b*) the canal, just downstream of the canal head regulator

(*c*) the river, some distance downstream of the undersluices

(*d*) the canal, some distance downstream of the canal head regulator

(*B*) Fill in the blanks :

(*i*)* The area of a drainage basin is twice that on an adjoining one. Excepting area, other characteristics of the two basins are identical. Q1 and Q2 are the peak discharges, computed by Dicken's formula, for the smaller and larger basin respectively. Then $Q_2 = Q_1 \times$ .....

(*ii*)* In a field measuring 30 hectares, 40 cm of water was stored in the root zone when 6 cumecs of water was supplied for 8 hours. The field application efficiency is .....

(*iii*)* he dominant discharge of a river in alluvial plain is 3600 cumecs. The waterway on this river, according to Lacey's formula, should be ..... m.

Answers to Questions of 1996 (*Winter*) Exam.

A (*i*) – (*d*)	*A* (*ii*) – (*d*)	*A* (*iii*) – (*a*)
A (*iv*) – (*b*)	*A* (*v*) – (*c*)	*A* (*vi*) – (*d*)
A (*vii*)– (*c*)	*A* (*viii*)– (*b*)	*A* (*ix*) – (*d*)
A (*x*) – (*a*)	*A* (*xi*) – (*b*)	*A* (*xii*)– (*c*)
A (*xiii*)– (*a*)	*A* (*xiv*)– (*d*)	
B (*i*) – 1.6818	*B* (*ii*) – 69.44%	*B* (*iii*)– 285

Hints for Solving Questions of 1996 (*Winter*) Exam.

Q. A (*ix*) Please refer article 19.15 (2) (*b*) at page 1167

Q. B (*i*) $Q_1 = C \cdot A_1^{3/4}$

where Q_1 is the discharge of smaller basin

$$Q_2 = C \cdot A_2^{3/4} = C \cdot (2A_1)^{3/4}$$

$$\frac{Q_2}{Q_1} = (2)^{3/4} = 1.6818$$

or $Q_2 = Q_1 \times 1.6818$ **Ans.**

Q. B. (*ii*) Vol. of water actually stored in root zone

$$= (30 \times 10^4 \text{ m}^2)\,(0.4\ m) = 12 \times 10^4 \text{ m}^3$$

Vol. of water actually supplied

$$= 6 \text{ m}^3/s\ (8 \times 60 \times 60s)$$

$$= 48 \times 3600 = 172800 \text{ m}^3 = 17.28 \times 10^4 \text{ m}^3$$

$$\text{Field application efficiency} = \frac{\text{Vol. of water actually stored}}{\text{Vol. of water supplied}}$$

$$= \frac{12 \times 10^4}{17.28 \times 10^4} = 0.6944 = \mathbf{69.44\%}$$ **Ans.**

Q. B. (*iii*) From Eqn. (15.1),

$$L = 4.75\sqrt{Q} = 4.75 \times \sqrt{3600} = 4.75 \times 60 = \mathbf{285\ m.}$$ **Ans.**

Year 1997 (*Summer*) Exam

(*i*) An arch dam is constructed with

(*a*) Earth (*b*) Concrete
(*c*) Boulders (*d*) None of the above

(*ii*) A core wall is provided in

(*a*) An earth dam (*b*) A canal regulator
(*c*) An aqueduct (*d*) A gravity dam

(*iii*) A hydrograph is a relation between time in hours and

(*a*) Rainfall in mm (*b*) Percolation in cm
(*c*) Surface runoff in cumec (*d*) Evaporation in cm

(*iv*) The self weight of a gravity dam

(*a*) Favours stability (*b*) Causes overturning
(*c*) Causes sliding (*d*) Causes uplift

(*v*) A level crossing is a cross-drainage work where

(*a*) Canal flows under a specified level above the drain
(*b*) Canal flows under pressure
(*c*) Canal has a free flow
(*d*) None of the above

(*vi*) Evaporation is maximum

(*a*) When the temperature is very low
(*b*) When it is raining
(*c*) When the temperature is high
(*d*) During night

(*vii*) Other conditions remaining the same, runoff is maximum from precipitation occurring over

(*a*) Sandy strata

(*b*) Rocky area

(*c*) Alluvial tracts after a dry spell

(*d*) Boulders

(*viii*) Waterlogging is eliminated by

(*a*) Deep ploughing (*b*) Shallow ploughing

(*c*) Irrigation (*d*) Providing tile drains

(*ix*) Lacey's theory is applicable to flow

(*a*) Through pipes

(*b*) Over spillways

(*c*) In alluvial rivers and canals

(*d*) In concrete lined channels

(*x*) Lift irrigation is flow

(*a*) By gravity (*b*) From lower level to higher level

(*c*) By percolation (*d*) Through sprinkler heads

(*xi*) Critical velocity ratio is used in connection with the design of

(*a*) Alluvial canals (*b*) Earth dams

(*c*) Concrete lined canals (*d*) Barrages

(*xii*) Base flow separation is used in connection with

(*a*) Seepage flow (*b*) Infiltration

(*c*) Evaporation (*d*) Stream flow

(*xiii*) The time base of a hydrograph increases with

(*a*) Increase in intensity of the storm

(*b*) Decrease in duration of the storm

(*c*) Increase in time of concentration

(*d*) Decrease in infiltration capacity of the storm

(*xiv*) Thiessen polygon is used in connection with

(*a*) Average rainfall determination

(*b*) Catchment area computation

(*c*) Drawing hydrographs

(*d*) Estimating infiltration

(*B*) Fill in the blanks in the following statements :

(*i*) The depth of scour for a discharge intensity of 5 cumec/m and a silt factor of 1 is ____ m.

(*ii*) Friction loss through a pipe 100 m long, 1.5 m diameter, carrying a discharge of 6 cumec and rugosity coefficient of 0.014 is ____ m.

Answers to Questions of 1997 (*Summer*) Exam

A (*i*)–(*b*)	*A* (*ii*)–(*a*)	*A* (*iii*)–(*c*)	*A* (*iv*)–(*a*)
A (*v*)–(*d*)	*A* (*vi*)–(*c*)	*A* (*vii*)–(*b*)	*A* (*viii*)–(*d*)
A (*ix*)–(*c*)	*A* (*x*)–(*b*)	*A* (*xi*)–(*a*)	*A* (*xii*)–(*d*)
A (*xiii*)–(*c*)	*A* (*xiv*)–(*a*)	*B* (*i*)–3.95	*B* (*ii*)–1.52

Hints for Solving Starred Questions 1997 (*Summer*) Exam

Q. B (*i*) $R = \text{Scour depth} = 1.35\left(\frac{q^2}{f}\right)^{1/3} = 1.35\left(\frac{5^2}{1}\right)^{1/3} = \mathbf{3.95\ m}$ **Ans.**

Q. B (*ii*) From Manning's formula, we have

$$Q = \frac{1}{n} \cdot A \cdot R^{2/3} \cdot \sqrt{S}$$

or
$$Q = \frac{1}{n} \cdot A \cdot R^{2/3} \cdot \sqrt{\frac{H_L}{L}}$$

or
$$\frac{n^2 Q^2 L}{A^2 \cdot R^{4/3}} = H_L$$

or
$$H_L = \frac{n^2 \cdot Q^2 \cdot L}{\left(\frac{\pi}{4} d^2\right) \cdot \left(\frac{d}{4}\right)^{4/3}} = \frac{(0.014)^2 \cdot (6)^2 \times 100}{\left(\frac{\pi}{4} \times 1.5^2\right)^2 \left(\frac{1.5}{4}\right)^{4/3}} = \mathbf{1.52\ m}$$ **Ans.**

Year 1997 (*Winter*) Exam

Q. A. *Fill in the blanks*

(*i*) Field capacity is defined as _____.

(*ii*) The delta for the crop with duty 864 hectares/cumec on the field and base period of 120 days is _____ cm.

(*iii*) Infiltration capacity is more than infiltration rate when _____.

(*iv*) A hyetograph is the plot of _____.

(*v*) In Kennedy's equation for non-silting non-scouring velocity, the critical velocity ratio was introduced to take into consideration the effect of _____.

(*vi*) Syphon aqueduct is the cross-drainage work provided to carry canal over natural drain when _____.

(*vii*) The ϕ_{index} is defined as the _____.

Q. B. *Choose the correct statement for the following :*

(*i*) Ravi crops are sown and harvested in the month of :

(*a*) April and September (*b*) October to March

(*c*) February and June (*d*) January and May

(*ii*) Mixed cropping is defined as :

(*a*) two or more crops grown during any year
(*b*) two or more crops grown during the same crop season in different fields
(*c*) two or more crop seasons of the year
(*d*) growing of two ore more crops together in the same field during the same crop season

(*iii*) For the upstream slope of an earth dam, the most critical condition is :

(*a*) sudden drawdown condition
(*b*) steady seepage condition
(*c*) neither sudden drawdown nor steady seepage condition
(*d*) during construction when the reservoir is allowed to be filled

(*iv*) Seepage endangers the stability of an earth dam built on pervious foundation because of piping which depends on :

(*a*) height of the dam (*b*) quantity of seepage flow
(*c*) value of exist gradient (*d*) total storage capacity of reservoir

(*v*)* If a river in alluvial plain has a dominant discharge of 1600 cumecs, the waterway for a bridge on the river should be approximately

(*a*) 250 m (*b*) 240 m
(*c*) 190 m (*d*) 180 m

(*vi*) For no tension at any point on a gravity dam, the criterion to be satisfied is :

(*a*) the resultant of all the forces must always pass through the mid point of the base of the dam
(*b*) the resultant force for all conditions of loading must pass through the middle third of the base
(*c*) the resultant of all forces must pass through the upstream extremity of the middle third of the base
(*d*) the resultant of all forces must always pass through the downstream extremity of the middle third of the base

(*vii*) Due to valley storage, the peak discharge of an outflow hydrograph is :

(*a*) increased (*b*) reduced
(*c*) not affected (*d*) sometimes increased and sometimes reduced

(*viii*) According to Dicken's formula for estimating floods, the peak discharge is proportional to :

(*a*) A (*b*) $A^{1/2}$
(*c*) $A^{2/3}$ (*d*) $A^{5/3}$

where A is the catchment area in km^2

Q. C. *Match the sentences of column A to that of column B*

(*i*)

Column A	*Column B*
(*a*) Ryve's formula	(A) to determine average depth of rainfall
(*b*) Theissen's polygon	(B) for estimation of peak flood
(*c*) Lacey's theory	(C) design of alluvial channels

(*ii*)

Column A	*Column B*
(*a*) Isohyet	(A) line joining points on catchment having equal time of travel
(*b*) Isochrone	(B) lines having equal depth of evaporation
(*c*) Isopleths	(C) line joining points of equal rainfall magnitude

(*iii*)

Column A	*Column B*
(*a*) Lysimeter	(A) measurement of stage
(*b*) Current meter	(B) measurement of evapotranspiration
(*c*) Bubble gauge	(C) measurement of velocity of streamflow

(*iv*)

Column A	*Column B*
(*a*) Hydraulic jump	(A) Spillway
(*b*) Seepage	(B) Hydroelectric power project
(*c*) Penstock	(C) Earth dam

Answers to Questions of 1997 (*Winter*) Exam.

A (*i*) the quantity of water retrained by a given soil against the pull of gravity, when expressed as depth of water spread over the basin

A (*ii*) 120

A (*iii*) the rainfall rate is less than the infiltration capacity

A (*iv*) rainfall rate Vs. time

A (*v*) silt grade

A (*vi*) HFL of drain is a little bit higher than the canal bed level

A (*vii*) rate of rainfall above which the rainfall volume becomes equal to runoff volume (Note. See Eqn. 7.102)

B (*i*) –(*b*) **B (*ii*)** –(*d*) **B (*iii*)** – (a) **B (*iv*)** – (*c*)

B (*v*) – (*a*) **B (*vi*)** – (b) **B (*vii*)** –(*b*) **B (*viii*)**– (*d*)

C (*i*) – (*a*)-(*B*), (*b*)-(*A*), (*c*)-(*C*) **C (*ii*)** – (*a*)-(*C*), (*b*)-(*A*), (c)-(*B*)

C (*iii*) – (*a*)-(*B*), (*b*)-(*C*), (*c*)-(*A*) **C (*iv*)** – (a)-(*A*), (*b*)-(*C*), (*c*)-(*B*)

Hints for Solving Starred Questions of 1997 (*Winter*) Exam

Q. B (*v*) Dominant discharge = 1600 cumecs

$$\text{Max. or Peak discharge} = \frac{16}{9} \times 1600 \text{ cumecs}$$

Waterway for a bridge = Lacey's Regime Perimeter = $4.75\sqrt{Q}$

where Q is the peak design discharge

$$\therefore \text{Waterway} = 4.75\sqrt{\frac{16}{9} \times 1600} = 4.75 \times \frac{4}{3} \times 40$$

$$= 253.3 \text{ m} \cong \mathbf{250\ m} \quad \textbf{Ans.}$$

Year 1998 (*Summer*) Exam

Q. (A) *Choose the correct answer from the various alternatives for the following :* (1 × 14)

(*i*) For a crop, the consumptive use of water is equal to the depth of water :

(*a*) transpired

(*b*) evaporated

(*c*) transpired and evaporated

(*d*) used in transpiration, evaporation and evaporated from adjacent soil

(*ii*) Effective precipitation is defined as :

(*a*) Precipitation minus the loss due to evaporation

(*b*) Precipitation minus the loss due to infiltration

(*c*) Precipitation occurred during the crop period

(*d*) available water stored within the root zone

(*iii*) Irrigation canals are generally aligned along :

(*a*) ridge line (*b*) contour line

(*c*) valley line (*d*) straight line

(*iv*) The difference in elevation between top of dam and full reservoir level is called:

(*a*) berm (*b*) free board

(*c*) height of bank (*d*) wave height

(*v*) The land is said to be waterlogged, if the soil pores within :

(*a*) a depth of 40 cm is saturated

(*b*) a depth of 60 cm is saturated

(*c*) root zone of crops is saturated

(*d*) soil up to ground watertable is saturated

(*vi*) An outlet maintaining a constant discharge, irrespective of fluctuation in water levels of supplying canal and water course, is known as

(*a*) non-modular outlet (*b*) semimodular outlet

(*c*) flexible modular outlet (*d*) rigid modular outlet

(*vii*) A drop in canal bed is generally provided, if :

(*a*) ground slope exceeds designed bed slope

(*b*) design bed slope exceeds ground slope

(*c*) ground slope is same as designed bed slope

(*d*) none of the above.

(*vii*) Bligh's Creep theory of seepage assumes :

(*a*) equal weightage to horizontal and vertical creep

(*b*) more weightage to horizontal creep than vertical creep

(*c*) less weightage to horizontal creep than vertical creep

(*d*) loss of head follows a sine curve.

(*ix*) The structure constructed to allow drainage water to flow under pressure through an inverted syphon below a canal is called :

(*a*) syphon (*b*) superpassage
(*c*) syphon aqueduct (*d*) super aqueduct

(*x*) The meandering of river is due to :

(*a*) sediment load of streams
(*b*) discharge and hydraulic properties of stream
(*c*) erodibility of the bed and banks of streams
(*d*) all of the above.

(*xi*) According to thin cylinder theory, the volume of concrete required for an arch dam would be minimum if the central angle is

(*a*) 130°34′ (*b*) 133°34′
(*c*) 136°34′ (*d*) 139°34′

(*xii*) According to Ryve's formula for estimating floods, the peak discharge is proportional to :

(*a*) A (*b*) $A^{3/4}$
(*c*) $A^{2/3}$ (*d*) $A^{1/2}$

(*xiii*) Levees are constructed :

(*a*) parallel to river flow
(*b*) transverse to the river flow
(*c*) perpendicular to the river flow
(*d*) all the above.

(*xiv*) Hydrograph is a plot of :

(*a*) rainfall intensity against time
(*b*) discharge against time
(*c*) cummulative rainfall against time
(*d*) cummulative discharge against time

(B) Fill in the blanks in the following statements :

(*i*) If the duty for a base period of 120 days is 1500 hectares/cumec, then the delta for crop is ____ m.

(*ii*) If a river in alluvial plain has dominant charge of 1600 cumecs, the waterway for a bridge on this river should be ____ m (appx.)

(*iii*) For aone metre depth of flow in a canal, with CVR equal to 0.90, the velocity of flow according to Kennedy's theory will be ____ m/s.

Answers to Questions of 1998 (*Summer*) Exam

A (*i*)–(*d*)	***A*** (*ii*)–(*b*)	***A*** (*iii*)–(*a*)	***A*** (*iv*)–(*b*)
A (*v*)–(*c*)	***A*** (*vi*)–(*d*)	***A*** (*vii*)–(*a*)	***A*** (*viii*)–(*a*)
A (*ix*)–(*c*)	***A*** (*x*)–(*d*)	***A*** (*xi*)–(*b*)	***A*** (*xii*)–(*c*)
A (*xiii*)–(*a*)	***A*** (*xiv*)–(*b*)		
B (*i*)–0.69 m	***B*** (*ii*)–250 m	***B*** (*iii*)–0.495	

Year 1998 (*Winter*) Exam.

Q. (A) *Fill in the blanks :* (1 × 10)

(*i*) Consumptive use is defined as .....

(*ii*) The relationship between duty and delta is .....

(*iii*) Base period of an irrigated crop is the period from ..... to .....

(*iv*) Hydraulic mean depth is the ratio between ..... and .....

(*v*) Fan shaped catchment gives ..... run off than Fern leaf shaped catchment.

(*vi*) Tortuosity of a meandering river is the ratio between ..... and .....

(*vii*) Middle third rule is used in .....

(*viii*) ..... is used commonly to measure actual evapotranspiration.

(*ix*) The sequent peak algorithm is a technique used in the estimation of .....

(*x*) The surge tank is provided in the water supply line to reduce the effect of .....

Q. (B). *Choose the correct statement for the following* (1 × 10)

(*i*)* A rain-fall with an intensity of 5 mm/h is classified as

(*a*) trace (*b*) light rain

(*c*) moderate rain (*d*) heavy rain.

(*ii*) Double mass curve technique is adopted to :

(*a*) estimate the missing data.

(*b*) obtain intensities of rainfall at various durations

(*c*) check the consistency of data.

(*d*) obtain the storage capacity needed for a reservoir.

(*iii*) The drainage density is the :

(*a*) average length of streams per unit drainage are within the basin.

(*b*) stream discharge per unit drainage area

(*c*) drainage area per unit stream length

(*d*) number of streams per unit drainage area.

(*iv*) Streams that contribute to the ground water are called :

(*a*) effluent streams (*b*) ground water streams

(*c*) influent streams (*d*) perennial streams.

(*v*) The basic assumptions of unit hydrograph theory are :

(*a*) linear response and time variance

(*b*) linear response and time invariance

(*c*) non-linear response and time variance

(*d*) non-linear response and time invariance

(*vi*) The probable maximum flood is :

(*a*) an impossibly large flood discharge

(*b*) a flood discharge that is maximum over the past years.

(*c*) an extremely large but physically possible flood at the site.

(*d*) a flood with maximum probability of occurrence.

(*vii*) If γ' is the submerged unit weight of soil, ϕ is the angle of internal friction and h is the height to which the silt is deposited, they the pressure on the dam due to silt is given by :

(*a*) $P_s = \gamma' \cdot h^2 \left(\frac{1 - \sin\phi}{1 + \sin\phi}\right)$ (*b*) $P_s = \gamma' \cdot h^2 \left(\frac{1 + \sin\phi}{1 - \sin\phi}\right)$

(*c*) $P_s = \frac{1}{2} \gamma' h^2 \left(\frac{1 - \sin\phi}{1 + \sin\phi}\right)$ (*d*) $P_s = \frac{1}{2} \gamma' \cdot h^2 \left(\frac{1 + \sin\phi}{1 - \sin\phi}\right)$

(*viii*) In cross-drainage works, the function of an inlet is to admit drain water into a :

(*a*) canal (*b*) field

(*c*) drain (*d*) none of the above

(*ix*)* The points connecting equal evapotranspiration is known as :

(*a*) isobar (*b*) isopleth

(*c*) isohyet (*d*) isovel.

(*x*) Leaching is the process by which :

(*a*) water is excessively pumped from a water-logged area.

(*b*) excess salt in the soil is pushed down by standing water above the soil.

(*c*) improving the soil structure end texture by adding suitable chemicals.

(*d*) sodium ions are removed from irrigation water.

Answers to Questions of 1998 (*Winter*) Exam

A (*i*). the amount of water used in evapotranspiration from an area under vegetation including the water used by the plants in their metabolic process for building of plant tissues.

A (*ii*).

$$\Delta = \frac{8.64\,B}{D}\ \text{m}$$

where D = duty in ha/cumec

B = Base period of crop in days

Δ = Delta in m.

A (*iii*)–the time between the first watering of a crop at the time of its sowing to its last watering before harvesting.

A (*iv*)–area of canal; Wetted perimeter of canal.

A (*v*)–greater

A (*vi*)–arcual length along the river reach; direct axial length of the river reach.

A (*vii*)–the design of concrete gravity dams.

A (*viii*)–A lysimeter

A (*ix*)–minimum storage capacity of a reservoir.

A (*x*)–water hammer -

B (*i*)–(*c*) **B** (*ii*)–(*c*) **B** (*iii*)–(*a*) **B** (*iv*)–(*b*)

B (*v*)–(*b*) **B** (*vi*)–(*c*) **B** (*vii*)–(*c*) **B** (*viii*)–(*a*)

B (*ix*)–(*b*) **B** (*x*)–(*b*)

Hints for Solving Starred Questions of 1998 (*Summer*) Exam

Q. B (*i*). On the basis of its intensity, the rainfall is classified as light, moderate or heavy, as shown in table 33.1.

Table 33.1. Types of Rainfalls

S. No.	*Type of rainfall*	*Intersity of rainfall*
1	Light rain	trace to 2.5 mm/h
2	Moderate rain	> 2.5 mm/h to 7.5 mm/h
3	Heavy rain	> 7.5 mm/h

Q.B. (*ix*). As defined in the text, an *isobar* is a curve which joins the points of equal atmospheric pressure; while an *isohyet* is the one which joins the points of equal rainfall in a catchment plan.

An **isovel** on the other hand, is a contour (a curve) connecting points of equal velocities is a river cross-section; while an **isopleth** is a curve which connects the points of equal evapotranspiration. IMD has provided isopleths-map for the country, showing isopleths of annual PET (cm), wherein Delhi is found to be on the isopleth curve of 160 cm.

Year 1999 (*Summer*) Exam.

Q. (A). *Fill in the blanks of the following statements* (1 × 10)

(*i*) Percolation losses are more from a canal that is .....

(*ii*) Lacey's theory is applicable only to ..... rivers and not to .....

(*iii*) Gross command area is the sum of ..... and .....

(*iv*) Piping under an earthen dam starts developing from the ..... side of the structure.

(*v*) The loss of water during the growth of a crop is termed as .....

(*vi*) The flow occurring in a tunnel with water level above the roof at the exit end is ..... flow.

(*vii*) The drainage holes provided inside the body of a concrete gravity dam are located on the ..... side of the grout curtain.

(*viii*) The type of hydraulic structure to raise or lower the vessel travelling in a navigation canal is called a .....

(*ix*) A syphon is a cross-drainage work in which the canal is taken ..... the river.

(*x*) Repelling spurs project from the bank and form an acute angle on the ..... side.

Q. B. *Pick up the right alternative from the various choices provided in the following questions :* (1 × 10)

(*i*) For conveying the same discharge, a lined canal can have a bed slope :

(*a*) steeper than an unlined canal

(*b*) milder than an unlined canal

(*c*) same as an unlined canal

(*d*) none of the above.

(*ii*) An arch dam requires that :

(*a*) the foundation be stronger than the abutments

(*b*) the abutments be stronger than the foundation

(*c*) the foundation consists of gravel and sand.

(*d*) None of the above.

(*iii*) By increasing the length of the floor on the upstream of the weir axis :

(*a*) the uplift pressure will increase below the floor downstream of the weir.

(*b*) the uplift pressure will decrease below the floor downstream of the weir.

(*c*) the uplift pressure will remain the same in both the cases

(*d*) none of the above.

(*iv*) The nose of the divide wall of a barrage is founded on :

(*a*) firm rock which may be at any depth

(*b*) well foundation

(*c*) pile foundation

(*d*) none of the above.

(*v*) The scour depth as measured below the highest flood level of a river :

(*a*) will be more in a bouldery river than in an alluvial river

(*b*) will be less in a bouldery river than in an alluvial river

(*c*) will be the same in both cases

(*d*) none of the above.

(*vi*) In a level crossing type of cross-drainage works :

(*a*) the canal is above the drain

(*b*) the canal is below the drain

(*c*) the canal and the drain are at the same level

(*d*) none of the above.

(*vii*) The tunnels of a sediment excluder are so designed that :

(*a*) the longest tunnel is farthest from the canal head regulator.

(*b*) the longest tunnel is nearest to the canal head regulator.

(*c*) all the tunnels are of the same length

(*d*) none of the above.

(*viii*) Khosla's theory is applicable to structures founded on :

(*a*) rock

(*b*) any type of material

(*c*) permeable material only

(*d*) none of the above.

(*ix*) To minimise the head losses at the entry of a branch canal taking off from the main canal, the centre line of the branch canal should make an angle with the centre line of the main canal that is :

(*a*) obtuse in the direction of flow

(*b*) acute in the direction of flow

(*c*) obtuse or acute

(*d*) none of the above.

(*x*) For a metering flume, the crest over which the head is measured should be :

(*a*) sharp (*b*) broad

(*c*) either sharp or broad (*d*) none of the above.

Answers to Questions of 1999 (*Summer*) Exam.

A (*i*)–Unlined

A (*ii*)–stable alluvial ; rocky or boulder river stages and unstable (aggrading or degrading) alluvial rivers.

A (*iii*)–cultivable ; non-cultivable area which can be economically irrigated without considering the limitation of the quantity of available irrigation water.

A (*iv*)–downstream **A** (*v*)–evapotranspiration **A** (*vi*)–submerged

A (*vii*)–upstream **A** (*viii*)–navigation lock **A** (*ix*)–below

A (*x*)–upstream

B (*i*)–(*b*) **B** (*ii*)–(*b*) **B** (*iii*)–(*b*) **B** (*iv*)–(*b*)

B (*v*)–(*b*) **B** (*vi*)–(*c*) **B** (*vii*)–(*b*) **B** (*viii*)–(*c*)

B (*ix*)–(*b*) **B** (*x*)–(*c*)

Year 1999 (*Winter*) Exam

Q. A. *Fill in the blanks :* (1 × 10)

(*i*) If the duty for a crop with base period of 120 days is 1500 ha/cumec, then its delta is ..... .

(*ii*) An isohyet is a line joining points of ..... .

(*iii*) The infiltration capacity of a soil ..... with time.

(*iv*) Canals taken directly from the rivers with or without head regulators are called ..... .

(*v*) When the canal with bed level higher than the high flood level in drain cross each other, then the cross drainage structure is called ..... .

(*vi*) The suction pressure on the ogee spillway is caused, when the head on spillway is ..... the design head.

(*vii*) An example of vertical lift gate is ..... .

(*viii*) Cross drainage works are not needed when canal is aligned as ..... .

(*ix*) Movement of water under or around an impervious structure is called ..... .

(*x*) The type of dam preferred for a narrow gorge with strong abutments is ..... .

Q. B. *Choose the correct statement for the following :* (1 × 10)

(*i*) Enveloping curve is a method to :

(*a*) determine the peak flood (*b*) determine the infiltration capacity

(*c*) estimate interception (*d*) route the flood through reservoir.

(*ii*) The maximum water application efficiency is in :

(*a*) surface irrigation (*b*) lift irrigation

(*c*) sprinkler irrigation (*d*) furrow irrigation.

(*iii*) Evaporation depends on :

(*a*) temperature (*b*) humidity
(*c*) wind velocity (*d*) all the above.

(*iv*) Canals meant for the purpose of draining off water from water-logged areas are called :

(*a*) seepage canals (*b*) laterals
(*c*) drains (*d*) water courses.

(*v*) Cross drainage work, where the bed levels of canal and drain are equal is called :

(*a*) aqueduct (*b*) level crossing
(*c*) super passage (*d*) siphon aqueduct.

(*vi*) The most economical central angle of an arch dam is :

(*a*) 93° (*b*) 123°
(*c*) 133° (*d*) 183°.

(*vii*) Radial gate is also known as :

(*a*) tainter gate (*b*) stoney gate
(*c*) drum gate (*d*) rolling gate.

(*viii*) Guide banks are provided to :

(*a*) decrease depth of flow in river (*b*) confine the width of river
(*c*) reduce the flood peak (*d*) maintain minimum depth in river.

(*ix*) River intake is usually situated on :

(*a*) convex side of curve (*b*) concave side of curve
(*c*) straight reach (*d*) downstream side of sewer outfall.

(*x*) Stability of the upstream slope of an earthen dam becomes critical for the condition of :

(*a*) reservoir full (*b*) reservoir empty
(*c*) sudden drawdown (*d*) slow filling of reservoir.

Answers to Questions of 1999 (*Winter*) Exam.

A (*i*)–69 cm ***A*** (*ii*)–equal rainfall ***A*** (*iii*)–varies
A (*iv*)–main canals ***A*** (*v*)–aqueduct ***A*** (*vi*)–more than
A (*vii*)–stoney gate ***A*** (*viii*)–water-shed canal ***A*** (*ix*)–seepage flow
A (*x*) an arch dam

B (*i*)–(*a*) ***B*** (*ii*)–(*c*) ***B*** (*iii*)–(*d*) ***B*** (*iv*)–(*c*)
B (*v*)–(*b*) ***B*** (*vi*)–(*c*) ***B*** (*vii*)–(*a*) ***B*** (*viii*)–(*b*)
B (*ix*)–(*c*) ***B*** (*x*)–(*c*)

Year 2000 (*Summer*) Exam.

Q. A. *Fill in the blanks :* (1 × 10)

(*i*) Thiessen polygon method is used to find ..... .
(*ii*) Sugar-cane, according to crop period is ..... crop.
(*iii*) Base period is ..... than the crop period.

(*iv*) Weed growth in unlined canal is ....., if the velocity of water is less than 60 cm/sec.

(*v*) Fish ladder is a component part in ..... .

(*vi*) Passage provided in gravity dam to drain seepage water is called ..... .

(*vii*) If bed level of canal is at higher level than that of drain, then the cross-drainage work is called ..... .

(*viii*) Gibb's module is an example for ..... outlet.

(*ix*) Deflecting groynes are constructed on ..... bank of river.

(*x*) Water-logging, takes place due to ..... in ground water table.

Q. B. *Choose the correct statement for the following :* (1 × 10)

(*i*) Kharif crops are sown and harvested in the months of :

(*a*) January and May (*b*) July and January
(*c*) April and September (*d*) October and March.

(*ii*) If the pan evaporation indicates a total yearly evaporation of 1.5 m, then the annual water loss by evaporation from reservoir of water surface area of 100 km^2 will be :

(*a*) 1.5×10^3 ha-m (*b*) 1.5×10^4 ha-m
(*c*) 1.5×10^5 ha-m (*d*) 1.5×10^6 ha-m.

(*iii*) Water enters the soil at capacity rate when intensity of rainfall is

(*a*) more than the infiltration capacity of the soil
(*b*) less than the infiltration capacity of the soil
(*c*) equal to the infiltration capacity of the soil
(*d*) slightly less than the infiltration capacity of the soil.

(*iv*) With the increase in quantity of water supplied, the yield of most crops :

(*a*) continuously increases
(*b*) continuously decreases
(*c*) increases up to certain limit and then decreases
(*d*) decreases up to certain limit and then increases.

(*v*) The ordinates of unit hydrograph may be obtained by dividing the ordinates of the direct runoff hydrograph of a storm by :

(*a*) duration of unit hydrograph (*b*) storm duration
(*c*) base flow volume (*d*) direct runoff volume.

(*vi*) For triangular dam section of height 'H', for just no tension, under action of water pressure, self weight and uplift pressure, the minimum base width required with usual notation is :

(*a*) $\frac{H}{S-1}$ (*b*) $\frac{H}{(S-1)^{-1}}$

(*c*) $\frac{H}{\sqrt{S}}$ (*d*) $\frac{H}{\sqrt{S-1}}$.

(*vii*) Due to valley storage, the peak discharge of an outflow hydrograph :

(*a*) increases (*b*) reduces
(*c*) is not affected (*d*) not any of the above.

(*viii*)* The upstream face of the earth dam is considered as :

(*a*) equipotential line (*b*) stream line

(*c*) streak line (*d*) path line.

(*ix*) A super passage is a cross-drainage work provided when bed of natural drain is :

(*a*) at the same level as the canal bed

(*b*) well above the full supply level in canal

(*c*) below the full supply level of canal

(*d*) below the bed level of canal.

(*x*) River training works are seldom required in

(*a*) rocky stage of a river (*b*) meandering river

(*c*) trough stage of a river (*d*) deltaic stage of river.

Answers to Questions of 2000 (*Summer*) Exam.

A (*i*)–mean rainfall over a drainage area *A* (*ii*)–an overlapping

A (*iii*)–slightly less *A* (*iv*)–more *A* (*v*)–canal head works

A (*vi*)–drainage gallery *A* (*vii*)–an aqueduct or a syphon aqueduct

A (*viii*)–modular or rigid *A* (*ix*)–concave *A* (*x*)–rise

B (*i*)–(*c*) *B* (*ii*)–(*b*) *B* (*iii*)–(*a*) *B* (*iv*)–(*c*)

B (*v*)–(*d*) *B* (*vi*)–(*d*) *B* (*vii*)–(*b*) *B* (*viii*)–(*a*)

B (*ix*)–(*b*) *B* (*x*)–(*a*)

Hints for Solving Starred Questions

B (*viii*) Equipotential lines are the lines of equal energy, since every point on the upstream face of an earth dam will be under equal total energy (pressure head + datum head + vel. head), this line acts as an equipotential line. Similarly, the upstream floor of a weir acts as an equipotential line.

Year 2000 (*Winter*) Exam.

Q. A. Fill in the blanks in the following statements : (1 × 10)

(*i*) A groyne projecting from the river bank and with its nose pointing upstream is a ..... groyne.

(*ii*) A canal in which the amount of excavation is same as in the fill portion is called a ..... section.

(*iii*) A lined canal leading water to a power house has a slope ..... than an unlined canal for irrigation.

(*iv*) An arrangement for removal of silt that has already entered a channel is called a silt ..... .

(*v*) A cistern in a canal fall serves as a ..... .

(*vi*) The structure provided in a concrete dam to release excess water during floods is called a ..... .

(*vii*) Failure of an earth dam due to flow of water under the foundation and emerging on the downstream side is termed as a failure due to ..... .

(*viii*) A cross dreinage work in which the canal water and the drainage meet at the same level is called a ..... .

(*ix*) Crops grown during winter months and harvested in March-April are called ..... crops.

(*x*) A device to release water from a canal to the water course is called an ..... .

Q. B. *Choose the right answer from the list given below each question :* (1 × 10)

(*i*) Lacey's regime theory was developed after extensive research in :

(*a*) Israel (*b*) Indonesia
(*c*) India (*d*) Iraq.

(*ii*) A chute spillway is generally provided with :

(*a*) a weir (*b*) a barrage
(*c*) a concrete gravity dam (*d*) an earth dam.

(*iii*) A siphon spillway in a concrete gravity dam is in the shape of :

(*a*) latter U
(*b*) an inverted U
(*c*) horizontal bend through the abutment
(*d*) any straight line.

(*iv*) Energy will be required to irrigate land :

(*a*) under gravity (*b*) much above the river bed level
(*c*) in delta region (*d*) in all the above cases.

(*v*) On a meandering river, the current is strongest on :

(*a*) the concave bend (*b*) the convex bend
(*c*) both banks (*d*) the middle of the stream.

(*vi*) A barrage is a hydraulic structure provided on :

(*a*) deep narrow valleys (*b*) wide shallow valley
(*c*) rock foundation (*d*) silts, clays and gravels.

(*vii*) Conjunctive use of water means :

(*a*) use of surface water alone
(*b*) use of ground water alone
(*c*) use of pumped water alone
(*d*) combined use of surface water and ground water.

(*viii*) A straight concrete gravity dam should have :

(*a*) strong abutments (*b*) clay foundations
(*c*) strong foundations (*d*) None of the above.

(*ix*) Flexibility of an outlet is the ratio of :

(*a*) rate of change of discharge in an outlet to the normal depth in the channel
(*b*) rate of change of discharge in an outlet to the rate of change of discharge in the parent channel.
(*c*) rate of depth of crest level of outlet and FSL of channel to its full supply depth.
(*d*) head recovered to head put in.

(*x*) A canal on a watershed requires :

(*a*) a level crossing (*b*) a syphon

(*c*) a super passage (*d*) None of the above.

Answers to Questions of 2000 (*Winter*) Exam.

A (*i*)–repelling (or deflecting) *A* (*ii*)–balance

A (*iii*)–steeper *A* (*iv*)–ejector *A* (*v*)–device for energy dissipation

A (*vi*)–spillway *A* (*vii*)–piping *A* (*viii*)–level crossing

A (*ix*)–Rabi *A* (*x*)–Outlet

B (*i*)–(*c*) *B* (*ii*)–(*d*) *B* (*iii*)–(*b*) *B* (*iv*)–(*b*)

B (*v*)–(*a*) *B* (*vi*)–(*b*) *B* (*vii*)–(*d*) *B* (*viii*)–(*c*)

B (*ix*)–(*b*) *B* (*x*)–(*d*)

Year 2001 (*Summer*) Exam.

Q. A. *Fill in the blanks :* (2 × 5)

(*i*) In a hydraulic structure, the position of hydraulic gradient line is 2.0 m from the top of the impervious floor. The relative density of the material of impervious floor is 2.40. From uplift consideration, floor thickness at this location should be ..... .

(*ii*) A launching apron in a hydraulic structure provides safety against ..... .

(*iii*) In a hydro-power scheme, the pipe between surge tank and power house is known as ..... .

(*iv*) The difference in water level u/s and d/s of a bridge is known as ..... .

(*v*) Equation ..... given by Laccy is used for fixing the waterway of bridges and barrages in alluvial streams.

Q. B. *Choose the correct statement for the following :* (1 × 10)

(*i*) In a design of canal, falls are provided :

(*a*) to dissipate the energy

(*b*) when ground slope is steeper than the canal bed slope

(*c*) when the canal bed slope is steeper than the ground slope

(*d*) to reduce hydraulic losses.

(*ii*) Permeable spurs are used to :

(*a*) promote siltation along the banks of the river

(*b*) deflect the flow

(*c*) attract the flow

(*d*) contract the width of the river for navigation purposes.

(*iii*) Lacey's silt factor *f* for the average size of the river bed material of 0.30 mm is :

(*a*) 0.96 (*b*) 1.0

(*c*) 1.2 (*d*) 0.3.

(*iv*) For the estimation of flood peak, the value of Q in Dicken's formula is given by :

(*a*) $CM^{3/4}$ (*b*) $CM^{2/3}$

(*c*) $CM^{3/2}$ (*d*) $CM^{4/3}$.

(*v*) Seepage losses in a canal are more when it is :

(*a*) completely in filling (*b*) partly in filling and partly in cutting

(*c*) fully in cutting (*d*) lined.

(*vi*) Current meters are used for measurement of :

(*a*) velocity (*b*) discharge

(*c*) depth of flow (*d*) scour depth.

(*vii*) In a diversion headworks, the crest of the undersluice portion is kept :

(*a*) at the minimum river bed level

(*b*) higher than the crest level of canal head regulator

(*c*) at the same level as that of canal head regulator

(*d*) higher than the minimum bed level of the river.

(*viii*) In a syphon aqueduct, the maximum uplift pressure on the floor occurs when :

(*a*) canal is dry and drain is running full

(*b*) canal is running full and drain is dry

(*c*) both canal and drain are running

(*d*) both are dry.

(*ix*) Phreatic line in seepage analysis is defined as the line on which pressure is :

(*a*) equal to the atmosphere (*b*) greater than atmosphere

(*c*) lower than atmospheric (*d*) varying.

(*x*) Hydraulic jump occurs when flow changes from :

(*a*) super critical to sub-critical (*b*) subcritical to supercritical

(*c*) critical to turbulent (*d*) laminar to turbulent.

Answers to Questions of 2001 (*Summer*) Exam.

A (*i*)–min. of 1.43 m	***A*** (*ii*)–*scouring* of river bed		
A (*iii*)–penstock	***A*** (*iv*)–afflux	***A*** (*v*)–$P = 4.75\sqrt{Q}$	
B (*i*)–(*b*)	***B*** (*ii*)–(*a*)	***B*** (*iii*)–(*a*)	***B*** (*iv*)–(*a*)
B (*v*)–(*a*)	***B*** (*vi*)–(*b*)	***B*** (*vii*)–(*a*)	***B*** (*viii*)–(*a*)
B (*ix*)–(*a*)	***B*** (*x*)–(*a*)		

Year 2001 (*Winter*) Exam

Q. A. *Fill in the blanks :* (1 × 10)

(*i*) Lysimeter method is used for direct measurement of ..... .

(*ii*) The commonly used rain gauge in India is ..... .

(*iii*) According to Kennedy, non silting and non scouring velocity is called ..... .

(*iv*) Escope provided across the channel at its fag end to maintain required full supply level is called ..... .

(*v*) Canal should be ..... in order to reduce the chances of water-logging.

(*vi*) Arch dam transfers major port of hydrostatic load to ..... .

(*vii*) The commonly used earth dam is ..... .

(*viii*) Outlet, whose discharge depends on the difference in water levels in distributary and water course is called ..... .

(*ix*) Earthen embankments provided to confine flood water of alluvial river, without allowing to change its course is called ..... .

(*x*) Overflow spillway is also called ..... .

Q. B. *Choose the correct answer for the following :* (1 × 10)

(*i*) The best method of applying irrigation water to sandy undulating area is :

(*a*) free flooding (*b*) furrow method

(*c*) sub-irrigation (*d*) sprinkler irrigation.

(*ii*) The evaporation from an open water surface :

(*a*) depends upon the difference of saturation vapour pressure and actual vapour pressure in air

(*b*) is independent of wind velocity

(*c*) is independent of the area of water surface

(*d*) cannot be altered by any means.

(*iii*) Unit hydrograph resulting from a storm of specified unit duration and areal distribution gives :

(*a*) 1 mm direct run off (*b*) 1 cm direct run off

(*c*) 1 ha-m of direct run off (*d*) 1 million m^3 of direct run off.

(*iv*) The crest level of under sluices is :

(*a*) higher than the crest level of head regulator

(*b*) equal to the crest level of the weir

(*c*) equal to the bed level of the river channel

(*d*) equal to the crest level of head regulator.

(*v*) By providing seepage drains, the chances of water-logging :

(*a*) decrease (*b*) increase

(*c*) will not alter (*d*) None of the above.

(*vi*) Bhakra dam is a :

(*a*) non rigid gravity dam (*b*) rigid earth dam

(*c*) rigid gravity dam (*d*) rock fill dam.

(*vii*) In gorges with steep and strong side slopes, the most suitable type of dam is :

(*a*) earth dam (*b*) gravity dam

(*c*) buttress dam (*d*) arch dam.

(*viii*) The upstream face of an earth dam is :

(*a*) a flow line (*b*) an equipotential line

(*c*) a phreatic line (*d*) a stream line.

(*ix*) If the eccentricity of the resultant falls outside the middle third, the ultimate failure of dam occurs by :

(*a*) tension (*b*) crushing

(*c*) sliding (*d*) overturning.

(*x*) Spurs provided for the river training works are :

(*a*) parallel to the river banks (*b*) transverse to the river flow

(*c*) usually made of earth bends (*d*) provided to guide the flow to a bridge.

Answers to Questions of 2001 (*Winter*) Exam.

A (*i*)–Actual evapotranspiration

A (*ii*)–Simon's type of standard non-recording rain gauge

A (*iii*)–critical velocity ***A*** (*iv*)–tail escape ***A*** (*v*)–lined

A (*vi*)–abutments ***A*** (*vii*)–of homogenous type

A (*viii*)–non-modular ***A*** (*ix*)–dykes (or levees) ***A*** (*x*)–ogee spillway

B (*i*)–(*d*) ***B*** (*ii*)–(*a*) ***B*** (*iii*)–(*b*) ***B*** (*iv*)–(*c*)

B (*v*)–(*a*) ***B*** (*vi*)–(*c*) ***B*** (*vii*)–(*d*) ***B*** (*viii*)–(*b*)

B (*ix*)–(*a*) ***B*** (*x*)–(*b*)

Year 2002 (*Summer*) Exam

Q. A. *Choose the correct answer for the following :* (1 × 10)

(*i*) Float type rain gauge belongs to the type :

(*a*) recording rain gauge (*b*) Non-recording rain gauge

(*c*) May be both of the above (*d*) None of the above.

(*ii*) The length of stream per unit area is known as :

(*a*) Overflow density (*b*) Drainage density

(*c*) Stream density (*d*) Average density.

(*iii*) Which of the following is a costlier system ?

(*a*) Gravity irrigation (*b*) Lift irrigation

(*c*) Drip irrigation (*d*) Combination of (*a*) and (*b*) above.

(*iv*) Water supply system may be managed by artificial recharge of :

(*a*) Water-logged area (*b*) Aquifers

(*c*) Pond (*d*) All of the above.

(*v*) An earthen embankment running parallel to the river to protect the area behind it is called :

(*a*) Dam (*b*) Gully

(*c*) Dyke (*d*) Flood wall.

(*vi*) For medium silt whose average grain size is 0.16 mm, Lacey's silt factor is likely to be :

(*a*) 0.30 (*b*) 0.45

(*c*) 0.70 (*d*) 1.32.

(*vii*)* The discharge per metre width at the foot of a spillway is 10 m^3/s at a velocity of 20 m/s. A perfect free hydraulic jump will occur at the foot of the spillway when the tail water depth is approximately equal to

(*a*) 4.50 m (*b*) 5.00 m
(*c*) 5.50 m (*d*) 6.50 m.

(*viii*) The foundation soil under toe of a dam has a void ratio *e*. The specific gravity of the soil solids is *G*. Factor of safety against piping is to be taken as 2.5. The maximum permissible upward exit gradient is given by

(*a*) $i = 2.5\left(\frac{G-1}{1+e}\right)$ (*b*) $i = 2.5\left(\frac{1+e}{G-1}\right)$
(*c*) $i = 0.4\left(\frac{1+e}{G-1}\right)$ (*d*) $i = 0.4\left(\frac{G-1}{1+e}\right)$

(*ix*) According to Khosla's theory, if downstream sheet file is not provided, the exit gradient will be

(*a*) 0 (*b*) 1
(*c*) ∞ (*d*) 2.

(*x*) An attractive groyne is one which is

(*a*) Inclined upstream (*b*) Inclined downstream
(*c*) Normal to bank (*d*) Same as a repelling groyne.

Q. B. *Fill in the blanks*

(*i*) A hyetograph is a graph representing rainfall intensity ..... .
(*ii*) During seepage through an earthen mass, the direction of seepage ..... to the equipotential lines.
(*iii*) When a river starts meandering, the sediment carrying capacity of the river first ..... and ultimately ..... .
(*iv*) The agriculture product ..... has the maximum demand of water.
(*v*) Facility of irrigation is preferable in the region which is ..... .
(*vi*) Electrical logging method of groundwater investigations belongs to ..... method.
(*vii*) Cheapest mode of irrigation is ..... .
(*viii*) The volume of water taken into the storage of aquifer per unit surface per unit slope of head is known as ..... coefficient.
(*ix*) Empirical formulas to estimate evaporation is ..... .
(*x*) 'Afforestation' is a method of flood control by ..... .

Answers To Questions Of 2002 (*Summer*) Exam

A (*i*)–(*a*) ***A*** (*ii*)–(*b*) ***A*** (*iii*)–(*c*) ***A*** (*iv*)–(*b*)
A (*v*)—(*c*) ***A*** (*vi*)–(*a*) ***A*** (*vii*)–(*d*) ***A*** (*viii*)–(*d*)
A (*ix*)–(*c*) ***A*** (*x*)–(*b*) ***B*** (*i*)–(V_s time) ***B*** (*ii*)–(*is* ⊥)
B (*iii*)–decreases ; becomes constant ***B*** (*iv*)–Sugarcane
B (*v*)–Tropical ***B*** (*vi*)—resistivity surveys ***B*** (*vii*)–flow irrigation
B (*viii*)–storage ***B*** (*ix*)–Meyer formula and Rowher's formula
B (*x*)–soil and water conservation methods of flood control

Hints for Solving Storred Questions of 2002 (*Summer*) Exam.

Q .A (*vii*) $y_1 = \frac{q}{v_1} = \frac{10}{20} = 0.5$

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

or $$0.5\, y_2 (0.5 + y_2) = \frac{2 \times 10^2}{9.81} = 20.387$$

or $$y_2 (0.5 + y_2) = 40.77$$

or $$0.5\, y_2 + y_2^2 - 40.77 = 0$$

or $$y_2^2 + 0.5\, y_2 - 40.77 = 0$$

or $$y_2 = \frac{-0.5 \pm \sqrt{0.25 + 4 \times 40.77}}{2}$$

or $$y_2 = \frac{-0.5 \pm 12.78}{2}$$

or $$y_2 = \frac{12.28}{2} = 6.14 \text{ m}$$ (ignoring the un feasible –*ve* sign)

Nearest given choice to this value of 6.14m is at choice (*d*); and hence choice (*d*) is to be marked as the correct answer.

Year 2002 (*Winter*) Exam

Q. A. *Match List P with that of List Q* (1 × 10)

List P	*List Q*
(*i*) Leguminous crops	(*a*) Average rainfall
(*ii*) Thiessen polygon	(*b*) Average infiltration
(*iii*) S-curve hydrograph	(*c*) Increase nitrogen content of soil
(*iv*) Ridge canals	(*d*) Summation of unit hydrograph
(*v*) Canal lining	(*e*) Non rigid dam
(*vi*) Rock fill dam	(*f*) Relieves pore pressure in earth dam
(*vii*) Toe drain	(*g*) Energy dissipator
(*viii*) Chute-spillway	(*h*) Diversion head works
(*ix*) Fish ladder	(*i*) Have no cross-drainage works
(*x*) River intake	(*j*) Reduces water-logging
	(*k*) Flow is super critical
	(*l*) Helps to draw water in controlled manner.

Q. 3. (*Choose the correct answer for the following* (1 × 10)

(*i*) Average annual rainfal at any station is mean of average annual rainfalls over a period of

(*a*) 14 years (*b*) 21 years

(*c*) 28 years (*d*) 35 years.

(*ii*) For a crop, consumptive use of water is equal to the depth of water

(*a*) transpired by the crop

(*b*) evaporated by the crop

(*c*) used in transpiration, evaporation and quantity evaporated from adjacent soil.

(*d*) transpired and evaporated by the crop.

(*iii*) Channel is said to be in regime condition, if

(*a*) discharge is constant

(*b*) silt grade and silt charge are constant

(*c*) flows uniformly in incoherent alluvium, as that transported in suspension

(*d*) All of the above.

(*iv*) Canal fall is provided, if

(*a*) ground slope exceeds the designed bed slope

(*b*) designed bed slope exceeds the ground slope

(*c*) ground slope and bed slope are almost same

(*d*) None of the above.

(*v*) When a canal is carried over a natural drain, at crossing, the structure provided is called

(*a*) syphon (*b*) aqueduct

(*c*) super passage (*d*) level crossing.

(*vi*) Silt storage in the reservoir is same as

(*a*) dead storage (*b*) live storage

(*c*) effective storage (*d*) total storage.

(*vii*) Economic height of a dam is that height corresponding to which

(*a*) amount of silt collected is minimum

(*b*) cost of dam per unit of storage is minimum

(*c*) cost of dam per unit of storage is maximum

(*d*) free board provided is maximum.

(*viii*) Outlet is said to be proportional, if its flexibility is

(*a*) equal to zero (*b*) less than unity

(*c*) equal to unity (*d*) more than unity.

(*ix*) In a barrage, crust level is kept

(*a*) low with large gates (*b*) high with large gates

(*c*) high with no gates (*d*) low with no gates.

(*x*) River training work is required, when the river is

(*a*) aggrading (*b*) degrading

(*c*) meandering (*d*) in super-critical flow.

Answers to Questions of 2002 (*Winter*) Exam

A (*i*)—(*a*)	(*ii*)—(*a*)	(*iii*)—(*d*)	(*iv*)—(*i*)
(*v*)—(*j*)	(*vi*)—(*e*)	(*vii*)—(*f*)	(*viii*)—(*k*)
(*ix*)—(*h*)	(*x*)—(*l*)		
B (*i*)—(*d*)	*B* (*ii*)—(*c*)	*B* (*iii*)—(*d*)	*B* (*iv*)—(*a*)
B (*v*)—(*b*)	*B* (*vi*)—(*a*)	*B* (*vii*)—(*b*)	*B* (*viii*)—(*c*)
B (*ix*)—(*a*)	*B* (*x*)—(*c*)		

Years 2003 (*Summer*) Exam

Q. A. *Choose the correct answer for the following [use the codes given at the end of questions to indicate your answer]* (10 × 2)

(*i*) Match the following two lists :

	List I		*List II*
(*a*)	Maxmum quantity of water that a soil can retain against gravity	1	Specific yield
(*b*)	Moisture content of a soil at which the water is no longer available in sufficient quantity to sustain growth	2	Infiltration capacity
(*c*)	Volume of water that can be extracted by gravity from a unit volume of saturated earth material	3	Field capacity
(*d*)	Maximum rate at which a soil can absorb water through the surface.	4	Permanent wilting point

Codes :

	A	B	C	D
(*a*)	3	1	4	2
(*b*)	4	3	1	2
(*c*)	3	4	1	2
(*d*)	3	4	2	1

(*ii*)* Which of the following instruments are concerned with streamflow measurements ?

(1) Sounding weight (2) Echo-depth recorder
(3) Hydrometer (4) Bubble gauge
(5) Stevenson Box

Codes

(*a*) 1, 2 and 5 (*b*) 3, 4 and 5
(*c*) 1, 2 and 4 (*d*) 2 and 5.

(*iii*) Match the following two lists :

	List I (Plots of)		*List II (Known as)*
(*a*)	Accumulated precipitation Vs time in chronological order	1	Hydrograph
(*b*)	Streamflow Vs time in chronological order	2	Hyetograph
(*c*)	Rainfall intensity Vs time	3	Flow duration curve
(*d*)	Stream discharge Vs Percentage time the flow is equalled to or exceeded	4	Flow mass curve
(*e*)	Cumulative discharge Vs time in chloronological order	5	Mass curve of rainfall

Codes :

	A	B	C	D
(*a*)	2	1	5	3
(*b*)	5	1	2	3
(*c*)	5	4	2	1
(*d*)	5	1	2	4

(*iv*) A 596 km^2 catchment has a 12 hour unit hydrograph which can be approximated as a triangle. If its time base is 144 hours, its peak ordinate is

(*a*) 23 m^3/s (*b*) 2 m^3/s

(*c*) 50 m^3/s (*d*) 13 m^3/s.

(*v*) Identify the correct statement in the following :

In the analysis of seepage through an earthen dam

(1) the upstream face is an equipotential line

(2) the upstream face is a stream line

(3) The phreatic line is a stream line

(4) the phreatic line is an equipotential line

(5) the upstream face is neither a streamline nor an equipotential line.

Code

The statements

(*a*) 4 and 5 are correct (*b*) 1 and 2 are correct

(*c*) 2 and 3 are correct (*d*) 1 and 3 are correct.

(*vi*) A coffer dam is

(*a*) a kind of gravity dam

(*b*) an earthen dam of small height built to protect important areas like townships from floods

(*c*) a temporary structure constructed to exclude water from work area during construction

(*d*) an embankment built along a river to regulate the river for navigation.

(*vii*) A non-cohesive soil has a porosity of 30% and the relative density of soil particles is 2.70. The value of critical exit gradient for this soil is

(*a*) 0.81 (*b*) 1.19

(*c*) 1.00 (*d*) 1.89.

(*viii*)* Which of the following pairs are matched correctly ?

(1) Total time from sowing of crop to its harvesting ..... base period

(2) Winter crops ..... Rabi

(3) Monsoon crop ..... Kharif

(4) Total time between the first watering for preparation of land to last watering before harvesting ..... crop period.

Codes

(*a*) 1, 2, 3 and 4 are correct

(*b*) 1, 2 and 3 are correct

(*c*) 2 and 3 are correct

(*d*) 1 and 4 are correct.

(*ix*) Water-table drops by 3 m in an irrigable area of 50 hectares. If porosity and specific retention are 0.30 and 0.10, respectively, the change in storage in hectare metres is

(*a*) 15 (*b*) 30

(*c*) 45 (*d*) 60.

(*x*) Match the following two lists relating to irrigation outlets :

List I (Name of outlet)		*List II (Type)*	
(*a*)	Kennedy gauge outlet	1	Modular
(*b*)	Gibb's outlet	2	Non-Modular
(*c*)	Submerged pipe outlet	3	Semi-modular
(*d*)	Pipe outlet discharging free		
(*e*)	Open flume outlet.		

Answers to questions of 2003 (*Summer*) Exam

A (*i*)—(*c*) A (*ii*)—(*a*) A (*iii*)—(*d*) A (*iv*)—(*a*)

A (*v*)—(*d*) A (*vi*)—(*c*) A (*vii*)—(*d*) A (*viii*)—(*c*)

A (*ix*)—(*c*) A (*x*)—(A)—(III), (B)—(I), (C)—(II), (D)—(III), (E)—(III)

Hints for Solving Starred Questions of 2003 (*Summer*) Exam

Q. A. (*ii*) **Sounding weights.** Current meters, which are basically used to measure velocity of streams, are weighted down by *lead weights*, called **balancing weights** or *sounding weights*, to enable them to be positioned in a stable manner at the required

location in flowing water. These weights are of streamlined shape with a fin in the rear, as shown in Fig. 7.70, and are connected to the current meter by a hanger bar and pin assmebly. Sounding weights come in different sizes and the minimum weight in N is estimated as equal to $50\bar{v}.y$, where $\bar{v}$ is the average stream velocity in the vertical in m/s and y = depth of flow at the vertical in m. These wts are hence essentially required in stream flow measurements

Echo depth recorder is an automatic recording device used in current meter measuring operations to mark the various vertical sections and to know the depths at these points. This instrument can hence be useful for quick and automatic depth reading, and is essential for current meter measurements in rivers passing heavy floods, where moving boat method is used for streamflow measurements.

Stevenson box is a water-depth recorder which may be used for measuring water depth in stream flow measurements. This instrument is hence very much concerned with streamflow measurements.

A **bubble gauge** is a permanent installation made on the bank of a stream to record water depths in the river (upto the reference level at the bed of the stream). Such an installation is hence not connected with streamflow measurements, though may be useful in computing Gauge-Discharge curves.

A (*viii*) Time taken from sowing of a crop to its harvesting is actually known as the crop period, while time between first watering and last watering before harvesting is the base period. The choices given at (1) and (4) are reverse, and hence do not match.

Kharif crops include all those crops which are grown during the summer (Kharif) season between April to September. Kharif crops, therefore includes **monsoon crops**, which are grown during the period 16th june to 15th october.

Years 2003 (*Winter*) Exam

Q. 1. *Match the List P with the List Q* (1 × 10)

List P	*List Q*
(*i*) Flow net	(*a*) Piping failure
(*ii*) Irrigation capacity	(*b*) Flood estimation
(*iii*) Isohyetal method	(*c*) Duty
(*iv*) Exit gradient	(*d*) River training
(*v*) Pan method	(*e*) Seepage control
(*vi*) Spurs	(*f*) Water-logging
(*vii*) Stilling basin	(*g*) Evaporation
(*viii*) Salinity problems	(*h*) Seepage characteristic
(*ix*) Rock toes	(*i*) Average rainfall
(*x*) Unit hydrograph	(*j*) Hydraulic energy

Q. B. *Select the correct answer for the following* (1 × 10)

(*i*) The ordinate of mass curve at any time is

(*a*) the ordinate of flood hydrograph at that time

(*b*) the area under flood hydrograph upto that time

(*c*) sum of the ordinates of flood hydrograph upto that time

(*d*) average of the area under flood hydrograph upto that time.

(*ii*) Base flow is

(*a*) the flow that seeps below the base of a dam

(*b*) the ground water flow that enters the aquifer

(*c*) the ground water flow that enters the river

(*d*) seepage flow across the earth dams.

(*iii*) Levees in the river training works are constructed

(*a*) parallel to the rive flow

(*b*) across the river flow

(*c*) inclined to the bank in U/S direction

(*d*) inclined to the bank in D/S direction.

(*iv*) Upstream and downstream cut-off at the weir base are meant to control

(*a*) seepage and uplift respectively

(*b*) uplift and seepage respectively

(*c*) exit gradient and uplift respectively

(*d*) uplift and exit gradient respectively.

(*v*) Sodium-Absorption Ratio (SAR) is expressed as

(*a*) $\dfrac{Na^+}{\left[\frac{1}{2}(Ca^{++}+Mg^{++})^2\right]}$ (*b*) $\dfrac{Na^+}{\left[\frac{1}{2}(Ca^{++}+Mg^{++})^{1/2}\right]}$

(*c*) $\dfrac{Na^+}{[(Ca^{++}+Mg^{++})^{1/2}]}$ (*d*) $\dfrac{Na^+}{[2(Ca^{++}+Mg^{++})^{1/2}]}$.

(*vi*) Balancing depth of a canal is achieved when

(*a*) the depth of cutting and height of embankment are equal

(*b*) the discharge in cutting section and between the embankment section are equal

(*c*) the volume of excavated portion and the banks is the same

(*d*) the discharge carried by the canal is maximum.

(*vii*) A land is said to be water-logged

(*a*) when the moisture content of the root zones rises to the field capacity of soil

(*b*) wen water-table touches the surface

(*c*) when water ponding occurs

(*d*) when pores of the root zone get saturated to cut-off air circulation.

(*viii*) Canal bed level is + 100 m and FSL is 104.5 m. Bed level of a natural drain crossing canal is 102.0 m and HFL is 106.0 m. Suitable cross-drainage work is :

(*a*) aqueduct (*b*) super-passage

(*c*) canal syphon (*d*) level crossing.

(*ix*) The upward acceleration of dam due to seismic activity will

(*a*) decrease the base pressure

(*b*) increase the base pressure

(*c*) not affect the effective weight of dam

(*d*) increase the horizontal dynamic force.

(*x*) A canal aligned at right angles to the contours is called

(*a*) ridge canal (*b*) contour canal

(*c*) branch canal (*d*) side slope canal.

Answers to Questions of 2003 (*Winter*) Exam

A (*i*)—(*h*)	(*ii*)—(*c*)	(*iii*)—(*i*)	(*iv*)—(*a*)
(*v*)—(*g*)	(*vi*)—(*d*)	(*vii*)—(*j*)	(*viii*)—(*f*)
(*ix*)—(*e*)	(*x*)—(*b*)		
B (*i*)—(*b*)	*B* (*ii*)—(*c*)	*B* (*iii*)—(*a*)	*B* (*iv*)—(*d*)
B (*v*)—(*b*)	*B* (*vi*)—(*c*)	*B* (*vii*)—(*d*)	*B* (*viii*)—(*c*)
B (*ix*)—(*b*)	*B* (*x*)—(*d*)		

Year 2004 (*Summer*) Exam

Q1. (*i*) Match the following two lists.

List I	*List II*
A Sandy soil	1 Venturi Flume
B Hilly region	2 Drip irrigation
C Consumptive use	3 Contour farming
D Water flow measurement	4 Blaney-Criddle equation
	5 Storage capacity

Codes :

	A	B	C	D
(*a*)	1	2	3	4
(*b*)	2	3	5	1
(*c*)	3	2	5	4
(*d*)	2	3	4	1

Q2. Match the following two lists :

List I	*List II*
A Arch dam	1 Rock foundation
B Gravity dam	2 Strong abutment
C Earth dam	3 Any type of foundation
D Buttress dam	4 Multiple arch
	5 Sandy soils

Codes :

	A	B	C	D
(*a*)	2	1	3	4
(*b*)	2	3	4	5
(*c*)	1	2	4	5
(*d*)	4	2	3	1

Q3. Match the following two lists :

List I	*List II*
A Bandhara	1 Spurs
B Groynes	2 Anicut
C Weir	3 Maharastra
D Spillway	4 Ogee
	5 Punjab

Codes :

	A	B	C	D
(*a*)	2	5	4	3
(*b*)	3	4	5	1
(*c*)	3	1	2	4
(*d*)	1	2	4	3

Q4. *Which of the following pairs are correctly matched.*

1. C.D. structure, where canal bed level and drainage bed level are same – level crossing
2. C.D. structure, when it receive and discharge drainage water from a canal-Inlet and Outlet
3. C.D. structure when canal bed level is lower than drainage bed level – Aqueduct
4. C.D. structure when canal bed level is higher than drainage bed level-Super passage.

Codes :

(*a*) 1, 2, 3 ; (*b*) 1, 2, 4 ; (*c*) 1, 2, 3, 4 ; (*d*) 1, 2.

Q5. Assertion (A) : Thickness of u/s floor of weir is nominal

Reason (R) : Thickness of impervious floor is governed by net uplift pressure.

Codes :

(*a*) Both *A* and *R* are true but *R* is not correct explanation of *A*

(*b*) Both *A* and *R* are true but *R* is correct explanation of *A*

(*c*) *A* is true but *R* is false

(*d*) *A* is false but *R* is true

(*e*) Both are false.

Q6. The following structure serves as safety valve for the canal :

(*a*) head regulator (*b*) cross regulator
(*c*) escape (*d*) canal outlet
(*d*) module.

Q7. Base period of a particular crop is 120 days. If duty is 1500 ha/cumec, the delta is :

(*a*) 69 cm (*b*) 138 cm
(*c*) 35 cm (*d*) 54 cm
(*e*) 86.4 cm.

Q8. The following is *not* the basis for the design of an arch dam :

(*a*) thin cylinder theory (*b*) elastic theory
(*c*) unit column theory (*d*) trial load theory
(*e*) creep theory.

Q9. In the empirical formula, for estimating peak flow

$Q = CA^n$ match the value of n in Lists I and II

Dicken, Ryves, Inglis, Rational	List I
1, 1/2, 2/3, 3/4, 4/5	List II

Q10. The following type of lining material of canal is *not* exposed to atmosphere :

(*a*) cement concrete (*b*) tiles
(*c*) asphaltic concrete ; (*d*) bentonite clay membrane
(*e*) reinforced concrete.

Answers to Questions of 2004 (*Summer*) Exam

1. (*d*) **2.** (*a*) **3.** (*c*) **4.** (*d*)
5. (*b*) **6.** (*c*) **7.** (*a*) **8.** (*e*)
9. Dicken-3/4, Ryves-2/3, Inglis-1/2, Rational-1 **10.** (*e*).

Appendix Table A-I : Some Important Properties of Water

Temp. °C	Sp. gravity	Unit. wt. in kN/m^3 i.e. Sp. gravity × 9.807 ($S_s \cdot \rho_w$)	Vapour pressure kN/m^2	Viscosity	
				Dynamic or Absolute $kN\text{-}sec/m^2$	Kinematic $\left(\nu = \frac{\mu}{\rho}\right)$ $m^2/sec.$
0	0.99987	9.806	0.611	1.797×10^{-6}	1.78×10^{-6}
5	0.99999	9.807	0.872	1.517×10^{-6}	1.518×10^{-6}
10	0.99975	9.805	1.226	1.306×10^{-6}	1.310×10^{-6}
15	0.99907	9.800	1.703	1.126×10^{-6}	1.124×10^{-6}
20	0.99810	9.788	2.334	1.003×10^{-6}	1.009×10^{-6}
30	0.99574	9.765	4.609	0.802×10^{-6}	0.804×10^{-6}
40	0.99228	9.731	7.60	0.652×10^{-6}	0.654×10^{-6}
60	0.98338	9.644	19.34	0.470×10^{-6}	0.478×10^{-6}
80	0.97196	9.532	46.583	0.356×10^{-6}	0.366×10^{-6}
100	0.95865	9.401	101.50	0.282×10^{-6}	0.295×10^{-6}

Appended Table A-II : Other Important Properties of Water

Standard Atmospheric Pressure

$= P_{atm} = 760$ mm of Mercury

$= 1$ bar (b)

$= 1000$ m.b.

$= 101.325$ kPa*

$= 10.33$ m of water head**

Density of water (ρ) *at 4°C*

in MKS Units $= 1000\ kg/m^3$

in SI units $= 1000\ kg/m^3$

$$^{*}P_{atm} = 760 \text{ mm of Hg} = \frac{760}{1000} (\text{m of Hg}) \times 13.595 \times 9.807\ kN/m^3$$

$\because$ Unit wt of Hg $= S_s \cdot \rho_w = 13.595 \times 9.807\ kN/m^3$

$= 101.325\ kN/m^2$

$= \mathbf{101.325\ kPa}$

$^{**}P_{atm} =$ m of water height (h_w) × unit wt of water

$= h_w\ (m) \times 9.807\ kN/m^3 = 9.807\ h_w$

$\therefore \quad 101.325 = 9.987\ h_w$

$\therefore\ h_w = \mathbf{10.33\ m.}$

Unit weight of water (γ or w) at 4°C

in MKS units = 1000 kgf/m^3, usually written as 1000 kg/m^3

in SI Units = 9.807 kN/m^3

Dynamic Viscosity of water at 4°C

in MKS Units = 1.598×10^{-4} kgf. sec/m^2

in SI Units = 1.567×10^{-3} Ns/m^2

= 1.567×10^{-2} Poise (P)

Kinematic Viscosity of water (ν) at 4°C

in MKS Units = 1.567×10^{-6} m^2/s

in SI Units = 1.567×10^{-6} m^2/s

= 1.567×10^{-6} Stoke (St)

Appendix Table A-III. Water Resources Potentials of River Basins of India

Sl. No.	River Basin	Average Annual Surface Water Availability (50% dependability)		Utilisable Surface Water (at 75% dependability) excluding ground water	Utilisable or Replenishable Ground Water	Present Exploitation (storage)	
		Water Reesources Potential b.cum (BCM)	Per Capita (m^3)	b.cum (BCM)	b.cum	Surface water	Ground water
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
	(A) Major River Basins						
1.	Indus	73.30	1757	46.0	26.49	16.29	21.26
2.	Ganga-Brahmaputra-Meghna						
	(*a*) Ganga	525.02	1473	250.0	170.99	39.44	74.62
	(*b*) Brahmaputra	537.2	18417	24.0	26.55	2.33	4.74
	(*c*) Barak	48.4	7646				
		585.60					
3.	Godavari	110.54	2026	76.3	46.65	25.12	15.71
4.	Krishna	78.12	1312	58.0	26.41	41.80	11.91
5.	Cauvery	21.36	666	19.0	12.30	8.60	7.62
6.	Brahmani-Baitarani	28.48	2696	18.3	4.05	4.65	0.90
7.	Mahanadi	66.88	2546	50.0	16.46	12.33	3.44
8.	Pennar	6.31	648	6.9	4.93	2.65	2.27
9.	Mahi	11.02	1057	3.1	7.19	4.72	3.53
10.	Sabarmati	3.81	421	1.9	—	1.31	
11.	Narmada	45.64	2855	34.5	10.93	7.23	3.64
12.	Tapi	14.88	1091	14.5	8.27	9.41	4.30
	(B) Composite River Basins						
1.	Subernarekha	12.37	1392	6.8	1.82	0.67	0.42
2.	West Flowing rivers from Tapi to Tadri	87.41	3194	11.9	8.27	11.27	6.51
3.	West Flowing rivers from Tadri to Kanyakumari	113.53	3539	24.3	8.52	10.24	
4.	East Flowing rivers between Mahanadi and Pennar	22.52	919	13.1	18.22	1.60	11.66
5.	East Flowing rivers between Pennar and Kanyakumari	16.46	383	16.7	18.84	1.84	5.58
6.	West Flowing rivers of Kutch and Saurashtra including Luni	15.10	631	15.0	11.23	4.73	6.59
7.	Area of inland drainage in Rajasthan desert	negl.	—	—	—	—	—
8.	Minor rivers draining into Myanmar (Burma) and Bangladesh)	31.0	14616	—	—	0.31	—
	Total	1869.35	2214	690.3 Say 690	431.42 Say 432	**206-54***	184.70

* An additional live storage of 6.24 b.cum has been created through medium projects each having a capacity less than 10 Mcum (not included in above data) ; thus making a total storage of 212.78 b.cum ; say **213 b.cum**

Appendix Table A-IV : Fresh Water Supplies and their Storages on Different Rivers in India

Sl. No.	Name of the River Basin	Catchment area in Mha	Average annual flow in the river basins in b.cum	Live Storage Created/Likely to be Created in b.cum		
			50% Dependability	Through Completed projects (upto 2004)	Through Ongoing projects	Through projects under consideration
1	2		4	5	6	7
1.	Indus (Up to Border)	32.13	73.30	16.29	0.28	2.58
2.	(a) Ganga	86.15	525.02	39.46	21.22	30.08
	(b) Brahmaputra Barak and Others	23.61	585.60	2.33	9.35	41.26
3.	Godavari	31.28	110.54	25.12	6.21	5.84
4.	Krishna	25.89	78.12	41.80	7.74	1.31
5.	Cauvery	8.12	21.36	8.60	0.27	0.26
6.	Pennar	5.52	6.31	2.65	2.17	—
7.	East Flowing Rivers from Mahandi to Godavari and Krishna to Pennar	8.66	22.52	1.60	1.42	0.95
8.	East Flowing Rivers Between Pennar and Kanyakumari	10.01	16.46	1.84	0.07	—
9.	Mahanadi	14.16	66.88	12.33	1.87	10.09
10.	Brahamani and Baitarni	5.18	28.48	4.65	0.88	8.72
11.	Subernarekha	—	12.37	0.67	1.65	1.38
12.	Sabarmati	2.17	3.81	1.31	0.66	0.10
13.	Mahi	3.48	11.02	4.72	0.26	0.01
14.	West Flowing Rivers of Kutch, Saurashtra including Luni	32.19	15.10	4.73	0.80	2.85
15.	Narmada	9.88	45.64	7.23	16.38	0.47
16.	Tapi	6.51	14.88	9.41	0.85	0.29
17.	West Flowing Rivers from Tapi to Tadri	5.29	87.41	11.27	3.46	0.08
18.	West Flowing Rivers from Tadri to Kanyakumari	5.62	113.53	10.24	1.32	1.45
19.	Area of Inland Drainage in Rajasthan desert	6.0	Negligible	—	—	—
20.	Minor River Basins draining into Bangladesh and Burma	3.63	31.00	0.31	—	Negligible
	Total		**1869.35**	206.54	76.26	107.54

* Additional 6.24 b.cum storage is created through medium projects having live storage of less then 0.01 b.cum which have not been accounted here, thus creating a total storage of 212.78; say **213 b.cum**

Appendix Table A-V : Statewise Position of Live Storages Created and Likely to be Created in b.cum (2004)

S. No.	State	Live storage Created in in b. cum through completed projects	Live storages to be created in b.cum through ongoing projects	Live storage to be created through projects under consideration
1.	Andhra Pradesh	27.305	6.148	1.611
2.	Arunachal Pradesh	Nil	0.241	37.934
3.	Assam	0.012	Nil	0.726
4.	Bihar	1.842	0.675	5.822
5.	Chatisgarh	6.217	0.788	0.819
6.	Goa	0.044	0.217	Nil
7.	Gujarat	16.138	7.407	3.446
8.	Haryana	Nil	Nil	0.258
9.	Himachal Pradesh	13.917	0.189	0.986
10.	Jammu & Kashmir	Nil	0.094	1.832
11.	Jharkhand	2.472	6.878	0.476
12.	Karnataka	33.631	1.413	0.066
13.	Kerala	5.384	1.336	1.686
14.	Madhya Pradesh i/c Chhatisgarh	17.156	16.776	7.335
15.	Maharashtra	25.523	13.243	0.763
16.	Manipur	0.397	8.450	Nil
17.	Meghalaya	0.698	Nil	0.516
18.	Mizoram	Nil	0.663	1.561
19.	Nagaland	1.220	Nil	0.526
20.	Orissa	17.225	1.998	21.100
21.	Punjab	2.369	Nil	Nil
22.	Rajasthan	8.285	1.426	1.807
23.	Sikkim	Nil	Nil	0.001
24.	Tamil Nadu	6.501	0.068	0.013
25.	Tripura	0.312	Nil	Nil
26.	Uttranchal	3.056	5.341	154.00
27.	Uttar Pradesh	15.345	2.712	18.407
28.	West Bangal	1.475	0.184	Nil
29.	Andaman & Nicobar island (U.T.)	Nil	Nil	Nil
30.	Chandigarh (U.T.)	Nil	Nil	Nil
31.	Dadar Nagar Haveli (U.T.)	Nil	Nil	Nil
32.	Daman & Diu (U.T.)	Nil	Nil	Nil
33.	N.C.T of Delhi	Nil	Nil	Nil
34.	Lakshdeep (U.T.)	Nil	Nil	Nil
35.	Pondicherry	0.014	Nil	Nil
	Total	206.548 Say 206.54 b.cum*	76.257 Say 76.26 b.cum	107.545 Say 107.54 b.cum

* An additional live storage capacity of 6.24 b.cum is estimated to have been created through medium projects, each having capacity of less than 0.010 b.cum (which are not included here, thus making total storage capacity of **212.78 ; say 213 b.cum** in completed projects.

Appendix Table A-VI : Levels/Capacities of Important Dam Reservoirs of India

S. No.	Name of Reservoir	Location	Dead Storage level in m	Full reservoir level in m	Capacity at Dead Storage level in M.cum	Capacity at Full Reservoir level in M. cum
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1.	Nagarjunasagar	A.P.	149.05	179.83	4719	11561
2.	Sriram Sagar	A.P.	324.31 (MDDL)	332.54	849	3454
3.	Srisailum	A.P.	214.88	269.75	434	8722
4.	Ukai	Gujarat	82.30	105.16	1411	8511
5.	Sabarmati	Gujarat	170.90	189.58	54	908
6.	Kadana	Gujarat	99.08	127.71	340	1542
7.	Krishnarajasagar	Karnataka	18.29	38.04	125	1400
8.	Linganamakki	Karnataka	527.30	554.43	295	4497
9.	Tungabhadra	Karnataka	472. 44	49.74	2	3429
10.	Ghataprabha (Hidkal)	Karnataka	629.11	662.95	60	1440
11.	Malaprabha	Karnataka	619.35	633.83	96	1068
12.	Kabini	Karnataka	674.57	694.33	100	553
13.	Bhadra	Karnataka	30.48	56.70	241	26
14.	Idukki	Kerala	681.23	732.43	292	997
15.	Gandhi Sagar	M.P.	381.00	399.90	586	7413
16.	Tawa	M.P.	334.24	355.40	263	2312
17.	Mahandadi	M.P.	336.21	348.70	144	811
18.	Jayal wadi	Maharashtra	452.17	463.91	352	2909
19.	Khadakvasla	Maharashtra	574. 24	582.47	0	86
20.	Koyna	Maharashtra	609.60	657.91	120	2797
21.	Hirakud	Orissa	179.83	192.02	2318	8146
22.	Machkund	Orissa	818.35	838.16	78	971
23.	Balimala	Orissa	438.91	462.08	934	3610
24.	Salandi	Orissa	50.30	80.30	8	566
25.	Gobind Sagar	H.P.	445.62	515.11	95	9351
26.	Pong Dam	H.P.	384.00	436.70	1288	8579
27.	Rana Pratap Sagar	Rajasthan	342.90	352.91	1332	2905
28.	Lower Bhawani	Tamil Nadu	248.41	280.22	0	929
29.	Mettur	Tamil Nadu	204.22	240.79	62	2647
30.	Vaigai	Tamil Nadu	257.56	279.20	1	193
31.	Parambikulam	Tamil Nadu	534.31	556.26	125	505
32.	Adiyar	Tamil Nadu	283.46	320.04	0	109
33.	Matatila	U.P.	295.65	308.46	50	883
34.	Ramganga	U.P.	323.00	365.30	395	2448
35.	Rihand	U.P.	236.22	268.22	1640	10607
36.	Mayurkshi	West Bengal	106.38	121.31	68	617
37.	Kangsabati	West Bengal	120.43	134.14	138	52
38.	Maithon	DVC	132.59	146.30* 152.40	165	1275
39.	Panchet Hill	DVC	119.48	120.47*	170	1475
40.	Konar DVC	DVC	410.57	427.94	61	336
41.	Tilaiya	DVC	363.32	372.47	75	394

*In case of Maithon and Panchet reservoirs, F.R. levels are given without and with flood storage, both.

Appendix Table A-VII : The Highest Dams of the World

S. No.	Name of the Dam	Country where situated	Type of Dam	Height in metres	Year in which completed
1	Jafar Masadi dam (Sahid Yaqobi)	Iran	Earth dam	611	1996
2	Rogun dam	Russia	—do—	335	
3	Nurek dam	Russia	—do—	300	1961
4	**Grand Dixence dam**	Switzerland	Gravity dam	285*	1961
5	Inguri dam	Russia	Arch dam	272	1980
6	Vaiont dam	Italy	Arch dam	262	1961
7	Chicoasen dam	Mexico	Earth/Rockfill dam	261	1980
8	Kambaratinish dam	Russia	—do—	255	—
9	Mauvoisin dam	Switzerland	Erch dam	250	1990
10	Guavio dam	Colombia	Earth/Rockfill	250	1990
11	Sayano- Shushensk dam	Russia	Arch/gravity	245	1989
12	Mica dam	Canada	Earth/Rockfll	242	1973
13	Chivor dam (La Esmeraisa)	Colombia	Rockfill	237	1975
14	Oroviolle dam	USA	Earth dam	235	1963
15	Chirkey dam	Russia	Arch dam	233	1975
16	**Bhakra dam**	India	Gravity dam	226**	1963
17	Luzzone dam	Switzerland	Arch dam	225	1998
18	Hoover dam	USA	Arch/gravity dam	223	1936
19	Contra dam	Switzerland	Arch dam	220	1965
20	Mrantinje dam	Yogoslavia	Arch dam	220	1976
21	Dworshak dam	USA	Gravity dam	219	1973
22	Glen Canyan dam	USA	Arch dam	216	1964
23	Toktogul dam	Russia	Gravity dam	215	1978
24	Daniel Johnson dam	Canada	Multi arch dam	214	1968
25	Manicouagan 5PA dam	Canada	Multi arch dam	214	1989

* Grand Dixence is at present, the highest existing concrete gravity dam of the world.

** Bhakra dam is the highest concrete gravity dam of India, and is the second highest such dam in the world.

Appendix Table A-VIII : Largest Dams of the World

S. No.	Name of the Dam	Type of Dam	Country where Situated	App. Vol. of Fill Material in M. cum.	Year in which completed
1	Katum dam	Earth/Rock fill	Russia	4742	—
2	635 dam	Earthfill	China	458	2000
3	Ali E-delvari (Jarreh) dam	Arch	Iran	395	—
4	Sihwaho dam	Barrage Mobile (Moveable dam)	Korea (Republic)	342	1996
5	Talequan dam	Earth	Iran	329	—
6	Chapeton dam	Gravity	Argentina	296	—
7	Youngamho dam	Gravity	Korea (Republic)	245	1993
8	Penzadahe Khurded dam	Earth	Iran	175	—
9	Gumganho	Gravity	Korea (Republic)	138	1990
10	Tarbela dam	Earth/Rockfill	Pakistan	129	1976
11	Kambaratunsk	Gravity	Russia	112	—
12	Fortpeck dam	Rockfill	USA	96	1957
13	Cipasang dam	Earth/Rockfill	Indonesia	90	—
14	Ataturk dam	Rockfill	Turkey	85	1992
15	Yacyreta Apipe dam	Gravity	Argentina	68	—
16	Lee dam	Earthfill	Zimbabwe	66	—
17	Gardiner dam	Earthfill	Canada	65	1968
18	Afslulidijk	Earth	Netherland	63	1932
19	San Luis dam	Rockfill	USA	59	1967
20	Nurek dam	Earthfill	Russia	58	1980
21	**Nagarjuna Sagar dam**	Masonry gravity	India	56**	1976
22	Xiaolangdi dam	Rockfill	China	52	2002
23	Garrison dam	Rockfill	USA	51	1956
24	Oostersch-idekerng dam	Gravity	Netherland	50	1986
25	Cochiti dam	Rockfill	USA	50	1975

*Maximum Volume

**Nagarjuna Sagar dam across Krishna river is the largest dam of India, and is the highest masonry dam of the world (124.7 m high). As against this, Hirakud dam across Mahanadi river is the longest earthen dam of the world (25.3 km long).

Appendix Table A-IX : Useful Conversions Between Different Units

Length

1 metre	= 3.281 ft ;	1 ft	= 0.3048 m
1 km	= 0.622 mi ;	1 mi	= 1.6093 km
1 cm	= 0.394 in ;	1 in	= 2.54 cm
1 mm	= 1000 μ (micron)	1 micron (μ) = 10^{-6} m	

Area

1 m^2	= 10.761 ft^2 ;	1 ft^2	= 0.0929 m^2
1 km^2	= 10^6 m^2 = 100 ha	1 mi^2	= 2.59 km^2
	= 0.386 mi^2 = 247 acres		= 251 ha
			= 640 acres
1 ha	= 10^4 m^2	1 acre	= 4840 sq. yards
	= 2.47 acres ;		= 0.4047 ha

Volume and Discharge

1 m^3	= 35.307 ft^3 ;	1 ft^3	= 0.02832 m^3
1 ha.m	= 8.13 acre ft.	**1 acre ft**	= 43560 ft^3
			= 1233.6 m^3
			= 0. 123 ham
			= $\frac{1}{2}$ **cusec day**
1 m^3	= 1000 litres (*L*)	**1 gallon** (*i.e.* imp gallon)	= 4.546 litres
1 *L*	= 1.76 pints (imp)		
	= 0.22 gallon (imp)	1 U.S. gallon	= 3.785 litres.
1 cumec	= 35.307 cusec ;	1 cusec	= 0.02832 cumec

or 1 cusec = 28.32 L/s

or 1 cusec = 2.447 MLD (million litre per day)

or 1 cusec = 0.538 MGD (million gallon per day)

1 MGD = 1.86 cusec.

Force

1 kg wt	= 9.807 N (Newton) :	1 N = 0.102 kg wt
1 gm wt	= 980.7 dynes	
1 lb wt	= 32.2 poundals	
1 kg wt	= 2.204 lb wt.	

$$\boxed{1 \text{ kg (w)} = 9.807 \text{ N} = 2.204 \text{ lb (wt)} = 9.807 \times 10^5 \text{ dynes}}$$

or

$$\boxed{1 \text{ N} = 0.102 \text{ kg (wt)} = 0.2247 \text{ lb (wt)} = 10^5 \text{ dynes}}$$

1 poundal = 138.26 dynes = 14.102 gm (wt).

Pressure

Units : N/m^2 = Pascal (*Pa*) ; kg/m^2 (*i.e.* kgf/m^2) : lb wt/ft^2

1 kg/m^2	= 9.807 N/m^2	1 P_a	= 1 N/m^2 = 0.102 kg/m^2
	= 9.807 P_a	1 kP_a	= 102 kg/cm^2

1 kg/cm^2 = 10^4 kg/m^2 100 kp_a = 10,200 kg/m^2
= 1.02 kg/cm^2

1 Atmosphere
= 760 mm of Hg
= 1 bar = 1000 mb
= 101.325 kPa = 1.033 kg/cm^2
= 10.33 m water head

1 kg/cm^2 = 98.07 kP_a = 10 m of water	100 kP_a = 1.02 kg/cm^2 = 10.2 m of water

1 lb/ft^2 = 47.88 k.P_a
= 16.02 ft of water
1 psi = 1 lb/in^2 = 0.0703 kg/cm^2
1 lb/ft^2 = 4.882 kg/m^2

100 k.P_a = 2.089 lb/ft^2
= 33.45 ft of water
1 kg/cm^2 = 14.22 psi
1 kg/m^2 = 0.205 lb/ft^2

Density (ρ)

1 kg/m^3 = 0.0624 lb/cft ; 1 lb/cft = 16.02 kg/m^3

Unit wt. (γ or w = ρ. g.)

Units. N/m^3, kg/m^3 (*i.e.* kg-f/m^3) ; lb wt/ft^3

1 kg/m^3 (*i.e.* kg wt/m^3) = 9.807 N/m^3 1 N/m^3 = 0.102 kg/m^3

Engery

Units : Joule (N.m) ; kg (wt.) m; lb (wt). m

1 Joule (j) = 1 Nm
1 kg (wt) m = 9.807 N.m
i.e. 1 kg (wt) m = 9.807 J ; 1 J = 0.102 kg (wt) m
1 ft lb (wt) = 0.138 kg (wt) m = 1.356 J
1 kg (wt) m = 7.23 ft lb (wt) = 9.807 J
1 J = 0.102 kg (wt) m = 0.737 ft lb (wt)

Power

Units : J/s ; kgm/sec ; ft lb/sec

1 J/s = 1 watt.
1 J/s = 1 N.m/s = 1 watt = 0.102 kg (wt) m/sec = 0.737 ft lb (wt)/sec
1 HP (fps) = 550 ft lbs/sec = 746 J/s = 746 watts.
1 HP (metric) = 75 kg (wt) m/sec = 75 × 9.807 = 735 J/s = 735 watts.

Dynamic Viscosity (μ or η)

Units : $N.s/m^2$; $kgf.sec/m^2$; dyne sec/cm^2 (Poise)

1 $N{\cdot}s/m^2$ = 1 Pa.s = 10 Poise = 0.102 kgf sec/m^2

1 dyne. sec/cm^2 (Poise) = 0.0102 kgf sec/m^2

Kinematic Viscosity (v)

Units : m^2/s ; m^2/sec ; cm^2/sec (Stoke)

1 $m^2/s = 10^4$ $cm^2/sec = 10^4$ Stoke (S)

1 centi stoke (CS) $= \frac{1}{100} S$

Bibliography

General References :

1. *Irrigation Principles and Practices* : *O. W. Isrelsen and V.E. Hansen* (John Wiley & Sons, New York)
2. *Irrigation Engineering* : *I.E. Houk* (John Wiley and Sons, New York)
3. *Fundamentals of Irrigation Engineering* : *Dr. Bharat Singh* (Nem Chand and Bros Roorkee U.P., India)
4. *Water Resources Engineering* : *R.K. Linsley and J.B. Faranzini* (McGraw Hill Book Co. New York)
5. *Theory and Design and Irrigation Structures* : : *Varshney, Gupta & Gupta* (Nem chand and Bros, Roorkee, U.P. India)
6. *Irrigation* : *J. D. Zimmerman* (John Wiley and Sons, New York)

Special References

Chapter I

7. *Design and Evaluation of Irrigation Methods* : : *A.M. Michael, Shri Mohan and K.R. Swaminathan* (Water Technology Centre, IARI, New Delhi,)
8. *Elementary Soil and Water Engineering* : *Glann O, Schwab, K. Barnes, Richard K, Frevert, and Talcott W Edminister* (John Wiley and Sons, New York)
9. *Report of the Irrigation Commission 1972, Vol. I* : : *Publication of* (Ministry of Irrigation and Agriculture, New Delhi)
11. *Engineering Management of Water Quality* : : *P.H. McGauhey* (McGraw Hill Book Co., New Delhi)
12. *Proceedings of Work Shop on Sprinkler and Drip Irrigation Systems* : *Publication of* (Central Board of Irrigation and Power, New Delhi).

Chapter 2

13. *A Guide for Estimating Water Requirements of Crops* : *Publication of* (Water Management Division of Department of Agriculture, New Delhi)
14. *Symposium on "Optimum Requirements and Utilisation of water for Irrigated crops"* : *Publication No. 93* (Central Board of Irrigation and Power, New Delhi, 1963)
15. *Moisture Requirements in Agriculture* : : *B.H. Roe* (Mc Graw Hill Book Co., New Delhi)

16. *Review of Work done on Water Requirements of Crops* : *Dastane Singh, Hukkeri and Vamdav* (Navbharat Prakashan, Poona)

17. *Water Resources of India and Their Utilisation in Agriculture* : *C. Dakshinamurthy, A.M. Michael* and *Sri Mohan*

Chapter 3

18. *Irrigation Engineering Vol. I (Canals and Barrages)* : *S. Leliavsky* (Chapman and Hall, London)

19. *Symposium on Irrigation Water Management* : *Publication No. 90.* (Central Board of Irrigation and Power, New Delhi, 1967)

20. *The Fundamental Principles of Irrigation and Water Power* : *V.B. Priyani* (Charotar Book Stall, Anand, India 1967)

Chapter 4

21. *Hydraulics of Sediment Transport* : *Graf* (McGraw Hill Book Company, New York)

22. *Regime Flow in Incoherent Alluviums* : *Publication No. 20* by *Gerald Lacey (Central Board of Irrigation and Power, New Delhi, 1939)*

23. *"Symposium on Sediment Problems in Irrigation and Drainage Channels"* : *Publication No. 91* (Central Board of Irrigation and Power, New Delhi, 1965)

24. *Engineeering Hydraulics* : *Hunter Rouse* (John Wiley and Sons, New York)

25. *Irrigation Engineering Vol. IV (River and Canal Hydraulics)* : *S. Laliavsky* (Chapmann and Hall, London)

26. *Open Channel Flow* : *F.M. Handerson* (McMillian and Co.)

Chapter 5

27. *I.S. 3860-1966 : Specifications for Precast Cement Concrete Canal Linings* : *Publication of* (Bureau of Indian Standards, New Delhi)

28. *IS 3872-1966 : Code of Practice for Lining of Canals with Burnt Clay Tiles* : *Publication of* (Bureau of Indian Standards New Delhi)

29. *IS : 3873-1966 : Code of Practice for Laying insitu Cement Concrete Lining on Canals* : *Publication of* (Bureau of Indian Standards, New Delhi)

30. *IS 4515-1976 : Code of Practice for Boulder Lining for Canals* : *Publication of* (Bureau of Indian Standards, New Delhi)

31. *IS 4558-1968 Code of Practice for Under-Drainage of Lined Canals* : *Publication of* (Bureau of Indian Standards, Manak Bhavan, New Delhi)

32. IS 4701-1968 : *Code of Practice for Earthquakes on Canals* : *Publication of* (Bureau of Indian Standards, Manak Bhavan, New Delhi)

33. IS 4745-1968 : *Code of practice for Design for Cross-section of Lined Canals* : *Publication of* (Bureau of Indian Standards, Manak Bhavan, New Delhi)

34. *Lower Cost Canal Linings* : *Publication of* (Bureau of Reclamation U.S. Deptt. Interior, Denver, Colorado, June 1948)

35. *Canal Lining in India* : *Kanwar Sain* (Proceedings of International Commission on Irrigation and Drainage, Third congress San Francisco 1957, pp 145-175

36. *Investigation of Necessity for Drainage behind Canal Lining* : *A.S. Kalha and I.P. Kapila (Report* Central Design office, Punjab Irrigation Branch, Simla, 1952)

Chapter 6

37. *Drainage Engineering* : *J.N. Luthin* (John Wiley and Sons, New York)

38. *Soil Conditions and Plant Growth* : *E.W. Russell* (Longman Green and Co., London)

39. *Symposium on "Water Logging — Causes and Measures for its Prevention, Vol. I and II* : *Publication No. 118* (Central Board of Irrigation and Power, New Delhi, 1972

Chapter 7

40. *Hydrology* : *C.O. Wisler and E.F. Brater* (John Wiley and Sons, New York, 1959)

41. *Hydrology* : *O.E. Meinzer* (McGraw Hill Book Co., New York)

42. *Engineering for Dams, Vol. I (General Design)* : *Creager, Justin and Hinds* (John Wiley and Sons, New York)

43. *A Hand book of Hydrology* : *Publication of* (Govt. of India, Ministry of Agriculture, 1972)

44. *Applied Hydrology* : *Linsley Kohlar and Paulhus* (McGraw Hill Book Co., New York)

45. *Hydrology for Engineers* : *Linsley, Kohlar and Paulhus* (McGraw Hill Book Co., New York)

Chapter 8

46. *Manual on River Behaviour Control and Training* : *Publication No. 60* (Central Board of Irrigation and Power, New Delhi, 1971)

47. Drainage and Flood control Engineering (Chapter XI) : *G.W. Pickels* (McGraw Hill Book Company, New York, 1941)

Chapter 9

48. *Irrigation Engineering, Vol. II (Canals and Barrages)* : *S. Leliavsky* (Chapmann and Hall, London)

49. *Irrigation Engineering, Vol. V (Weirs)* : *S. Leliavsky* (Chapman and Hall, London)

Chapter 10

50. *Open Channel Hydraulics* : *Ven Te Chow* (McGraw Hill Book Co. International Students Edition)

51. The Standing Wave or Hydraulic Jump : *Publication No. 7 with supplement from Sh. R.M. Advani 1964* (Central Board of Irrigation and Power, New Delhi)

52. *Design of Weirs on Permeable Foundations* : *Publication No. 12 by A.N. Khosla, Dr. N.K. Bose, and Dr. E. Machenizie Taylor* (Central Board of Irrigation and Power, 1936 reprinted 1962, 1981)

Chapter 12

53. *Irrigation Canal Falls* : *Publication No. 10 — Compiled by Mr. A.M.R. Montague* (Central Board of Irrigation and Power, 1935)

54. Draft Manual on Falls : *Publication of* (Central Water Commission, Delhi)

Chapter 13

55. Irrigation (Vol. IV) : *K.B. Khushalani and Sh. Manohar Khushalani* (Oxford and IBH Publishing Co. Pvt, New Delhi)

Chapter 14

56. *Design of Syphons* : *Publication No. 59 by Prof. N.S. Govinda Rao* (Central Board of Irrigation and Power, New Delhi, 1956)

57. *Irrigation Engineering, Vol. II (Syphons, Weirs and Locks)* : *S. Leliavsky* (Chapman and Hall, London)

Chapter 15

58. *Design of small Bridges and Culverts* : *Goverdhan Lal* (CWC Publication)

59. *Guidelines for the Design of Small Bridges & Culverts* : Publication of (Indian Roads Congress, Jamnagar House, New Delhi-11)

60. *California Culvert Practice* : *Publication of* (State of California, Division of Highways, Publication Sacrmento California)

Chapter 16

61. *Ground Water, its Development, Uses and Conservation* ; *E.W. Bennisen* (E.F. Johnson Inc. St. Paul, Minn)

62. *Ground Water Hydrology* : *D.K. Todd* (John Wiley and Sons, New York)

63. *Tubewell and Ground Water Resources* : *Publication No. 54* (Central Board of Irrigation and Power, 1965)

64. *Irrigation (Theory and Practice)* : *A.M. Michael* (Vikas Publishing House Pvt. Ltd., New Delhi, 1973)

65. *Handbook on Boring and Deepening of Wells* : *Publication of* (Ministry of Food and Agriculture, Govt. of India, 1963)

66. *Design of Tubewells* : *A. Sanghi* (Institution of Engineers, India)

67. *Water Wealth of India* : *Dr. K.L. Rao* (Longman Publication)

68. *Water Well and Pump Engineering* : *A.M. Michael and S.D. Khepar* (Tata McGraw Hill Publishing Co. Ltd., New Delhi, 1985)

Chapter 17

69. *Rivers in Harness (The Story of Dams)* : *Allan H. Culler* (A division of Chilton Company, Publishers, New York)

70. *Silting of Reservoirs* : *A.N. Khosla* (Central Board of Irrigation and Power, New Delhi)

Chapter 18

71. *Investigation Manual for Storage Reservoirs* : *Publication No. P-58* (Central Board of Irrigation and Power, New Delhi)

72. *A Handbook of Applied Hydraulics* : *Publication of* (Central Board of Irrigation and Power, New Delhi)

73. *Manual on Capacity Survey of Reservoirs* : *Sh. B.N. Murthy*

Chapter 19

74. *Engineering for Dams (Vol. II), (Concrete Dams)* : *Creager, Justin and Hinds* (John Wiley and Sons, New York)

75. *Design of Small Dams* : *A Technical Publication of* (Govt. of U.S. Deptt. of Interior-Washington, and the Bureau of Reclaimation, Colorado, 1960, 1965)

76. *Treatise of Dams* : *USBR Manual* (The Bureau of Reclaimation Colorado, United States)

77. *Design and Construction of Dams* : *Edward Wegmann* (John Wiley and Sons, New York)

78. *IS-1893-1962 Recommendations for Earthquake Resistant Design of Structures* : *Publication of :* (Bureau of Indian Standards, New Delhi)

79. *Irrigation Engineering Vol. VI (Dams)* : *S. Lieliavsky* (Chapman and Hall, London)

Chapter 20

80. *Fundamentals of Soil Mechanics* : *D.W. Taylor* (Asia Publishing House, New Delhi)

81. *Geotech Engineering – Soil Mechanics and Foundation Engineering* : *Santosh Kumar Garg* (Khanna Publishers, New Delhi, 1998 ed.)

82. *Engineering for Dams, Vol. III (Earth, Rockfill, Steel and Timber Dams)* : *Creager, Justin and Hinds* (John Wiley and Sons, New York)

83. *Earth and Rock Dams* : Sheard, L. James and Others

Chapter 21 and 22

84. *Symposium on : "Spillway Capacity of Dams", and (ii) Various Methods of Designing Spillway Capacity Discharges"* : *Publication No. 65* (Central Board of Irrigation and Power, New Delhi
85. *Hydraulic Design of Spillways* : *Manual of* (U.S. Army Engineers EM 1102-2-1603)
86. *Dams and Control Works* : *USBR Publication* (Washington, DC, 1954)
87. *Hydraulic Energy Dissipators* : *I.A. Edwerd Elevatoroki* (Mc Graw Hill Book Co., New York)
88. *Symposium on Energy Dissipators* : *Publication No. 70* (Central Board of Irrigation & Power, New Delhi, 1955)

Chapter 23

89. *Corrosion and Corrosion Control* : *H.H. Uhlig* (John Wiley and Sons, New York)
90. *Fluid Mechanics with Engineering Applications* : *R.L. Daugherty and J.B. Franzini* (McGraw Hill Book Co., New York)

Chapter 24

91. *Hydropower Engineering* : *James Doland* (Ronald Publication, New York)
92. *Hydroelectric Engineering Practice, Vol. I to Vol. III* : *J. Gutherie Brown* (Blackie and Sons Ltd., London, 1958)
93. *Irrigation Engineering, Vol. III (Hydroelectric Engneering for Civil Engineers)* : *S. Leliavsky* (Chapman and Hall, London)
94. *Water Power Engineering* : *H.K. Barrows* (McGraw Hill Book Co., New York, 1943)
95. *Hydroelectric Hand Book* : *W.P. Creager and J.D. Justin,* (John Wiley and Sons, New York, 1963)

Chapter 25

96. *An introduction to Fluvial Hydraulics* : *S. Leliavsky* (Constable, London, 1955)
97. *Hand Book of Applied Hydraulics* : *Rich George* (McGraw Hill Book Co., New York)

Chapter 26

98. *College of Engineering Manual on Irrigation* : *Cl. W.M. Ellis* (Govt. Press, Madras, 1950)
99. *Irrigation Planning for Intensive Cultivation* : *H.L. Sally* (Asia Publishing House, Bombay)

Chapter 27

100. *Irrigation (Vol. III)-Arch, Buttress, Earth and Rockfill Dams* : *K.B. Khushalani and Manohar Khushalani* (Oxford and IBH Publishing Co. Pvt. Ltd., New Delhi)

Chapter 28

101. *Report of the Committee on pricing of Irrigation Water* : *Publication of* (Planning Commission, Govt. of India, New Delhi, 1992)

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IRRIGATION ENGINEERING AND HYDRAULIC STRUCTURES

Irrigation is the science of providing water to the lands in accordance with the requirements of the crops grown therein. It has been found to increase the productivity of rain-fed lands by as much as five times in a tropical country like India.

The annual fresh water supply (runoff) in India is 1869 b.cum. However, the present utilisation (1997) is only 605 b.cum for all its uses. Out of this, about 500 b.cum, *i.e.* about 83%, is being used for irrigation alone. Even with such a huge percentage of utilisation of water for irrigation, we are just able to irrigate only about 40% of our gross cropped area.

Additional waters are, thus, required to be developed for bringing additional cropped area under irrigation, to increase our food and fibre production. *The techniques which are employed for development and management of water resources become vitally important in this context, and have been lucidly explained in this popular book.*

The prestigious UPSC competitions continue to carry questions from this book year after year.

The present edition has further been revised and updated with rewriting of certain articles and with addition of certain new topics and numerical examples. The question bank containing questions of Engineering Services and Civil Services Exams. has also been updated.

AUTHOR'S PROFILE

The author, Mr. Santosh Kumar Garg, is a veteran in the field of civil engineering and has authored eight widely acclaimed books. His books on **Environmental Engineering Vol. I & II** as well as on **Irrigation Engineering and Hydraulic Structures** are being followed in almost all the engineering colleges in India for the last more than 18-20 years. His three new publications titled **Geotech Engineering, International and Interstate River Water Disputes,** and **Physical and Engineering Geology** have also gained a lot of popularity within a short span of their publications.