LECTURE NOTES

ON

WATER RESOURCES ENGINEERING– II (A70133)

IV B. Tech I Semester (JNTUH-R15)

By

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UNIT-I
STORAGE WORKS-RESERVOIRS

Instructional objectives

On completion of this lesson, the student shall learn:

1. The usual classification of the zones of a reservoir
2. The primary types of reservoirs and their functions
3. The steps for planning reservoirs
4. Effect of sedimentation in reservoirs
5. What are the geological explorations required to be carried out for reservoirs
6. How to determine the capacities of reservoirs
7. How to determine the dead, live and flood storages of reservoirs
8. How to reduce the loss of water from reservoirs
9. How to control sedimentation of reservoirs
10. The principles to be followed for reservoir operations

Introduction

Water storage reservoirs may be created by constructing a dam across a river, along with suitable appurtenant structures. However, in that lesson not much was discussed about fixing the size of reservoir based on the demand for which it is being constructed. Further, reservoirs are also meant to absorb a part of flood water and the excess is discharged through a spillway. It is also essential to study the relation between flood discharge, reservoirs capacity and spillway size in order to propose an economic solution to the whole project. These and topics on reservoir sedimentation have been discussed in this lesson which shall give an idea as to how a reservoir should be built and optimally operated.

Fundamentally, a reservoir serves to store water and the size of the reservoir is governed by the volume of the water that must be stored, which in turn is affected by the variability of the inflow available for the reservoir. Reservoirs are of two main categories: (a) Impounding reservoirs into which a river flows naturally, and (b) Service or balancing reservoirs receiving supplies that are pumped or channeled into them artificially. In general, service or balancing reservoirs are required to balance supply with demand. Reservoirs of the second type are relatively small in volume because the storage required by them is to balance flows for a few hours or a few days at the most. Impounding or storage reservoirs are intended to accumulate a part of the flood flow of the river for use during the non-flood months. In this lesson, our discussions would be centered on these types of reservoirs.

Reservoir storage zone and uses of reservoir

The storage capacity in a reservoir is nationally divided into three or four parts (Figure 1) distinguished by corresponding levels.

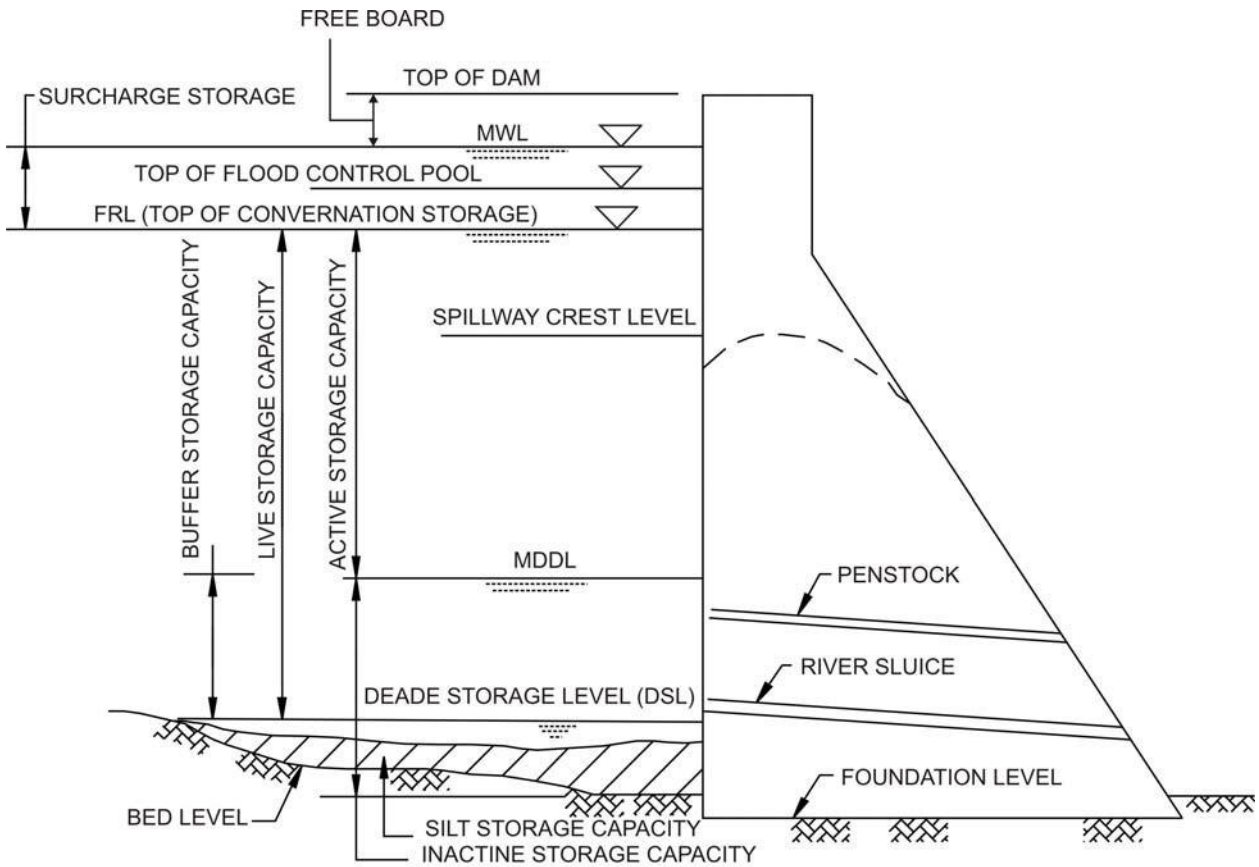


FIGURE 1. SCHEMATIC DIAGRAM SHOWING STORAGE ZONES (OF CAPACITY) NOMENCLATURE

These specific levels and parts are generally defined as follows:

Full Reservoir Level (FRL)**:** It is the level corresponding to the storage which includes both inactive and active storages and also the flood storage, if provided for. In fact, this is the highest reservoir level that can be maintained without spillway discharge or without passing water downstream through sluice ways.

Minimum Drawdown Level (MDDL)**:** It is the level below which the reservoir will not be drawn down so as to maintain a minimum head required in power projects.

Dead Storage Level (DSL)**:** Below the level, there are no outlets to drain the water in the reservoir by gravity.

Maximum Water Level (MWL)**:** This id the water level that is ever likely to be attained during the passage of the design flood. It depends upon the specified initial reservoir level and the spillway gate operation rule. This level is also called sometimes as the ***Highest Reservoir Level*** or the ***Highest Flood Level***.

Live storage**:** This is the storage available for the intended purpose between Full Supply Level and the Invert Level of the lowest discharge outlet. The Full Supply Level is normally that level above which over spill to waste would take place. The minimum operating level must be sufficiently above the lowest discharge outlet to avoid vortex formation and air entrainment. This may also be termed as the volume of water actually available at any time between the Dead Storage Level and the lower of the actual water level and Full Reservoir Level.

***Dead storage*:** It is the total storage below the invert level of the lowest discharge outlet from the reservoir. It may be available to contain sedimentation, provided the sediment does not adversely affect the lowest discharge.

***Outlet Surcharge or Flood storage*:** This is required as a reserve between Full Reservoir Level and the Maximum Water level to contain the peaks of floods that might occur when there is insufficient storage capacity for them below Full Reservoir Level.

Some other terms related to reservoirs are defined as follows:

***Buffer Storage*:** This is the space located just above the Dead Storage Level up to Minimum Drawdown Level. As the name implies, this zone is a buffer between the active and dead storage zones and releases from this zone are made in dry situations to cater for essential requirements only. Dead Storage and Buffer Storage together is called Interactive Storage.

***Within-the-Year Storage*:** This term is used to denote the storage of a reservoir meant for meeting the demands of a specific hydrologic year used for planning the project.

***Carry-Over Storage*:** When the entire water stored in a reservoir is not used up in a year, the unused water is stored as carry-over storage for use in subsequent years.

***Silt / Sedimentation zones*:** The space occupied by the sediment in the reservoir can be divided into separate zones. A schematic diagram showing these zones is illustrated in Figure 2 (as defined in IS: 5477).

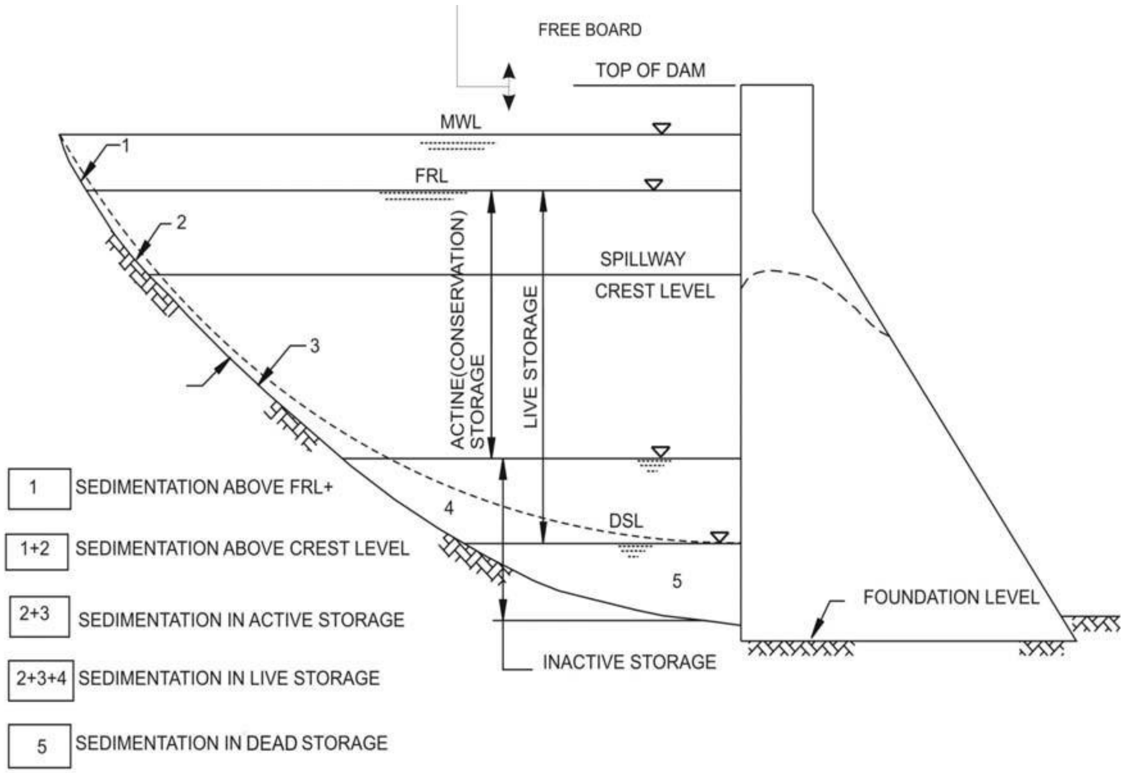


FIGURE 2. SCHEMATIC DIAGRAM SHOWING ZONES OF RESEERVOIR SEDIMENTION

***Freeboard*:** It is the margin kept for safety between the level at which the dam would be overtopped and the maximum still water level. This is required to allow for settlement of the dam, for wave run up above still water level and for unforeseen rises in water level, because of surges resulting from landslides into the reservoir from the peripheral hills, earthquakes or unforeseen floods or operational deficiencies.

The functions of reservoirs are to provide water for one or more of the following purposes. Reservoirs that provide water for a combination of these purpose, are termed as 'Multi Purpose' reservoirs.

- ***Human consumption*** and/or ***industrial use:***
- ***Irrigation:*** usually to supplement insufficient rainfall.
- ***Hydropower:*** to generate power and energy whenever water is available or to provide reliable supplies of power and energy at all times when needed to meet demand.
- ***Pumped storage hydropower schemes*:** in which the water flows from an upper to a lower reservoir, generating power and energy at times of high demand through turbines, which may be reversible, and the water is pumped back to the upper

reservoir when surplus energy is available. The cycle is usually daily or twice daily to meet peak demands. Inflow to such a reservoir is not essential, provided it is required to replace water losses through leakage and evaporation or to generate additional electricity. In such facilities, the power stations, conduits and either or both of the reservoirs could be constructed underground if it was found to do so.

- ***Flood control:*** storage capacity is required to be maintained to absorb foreseeable flood inflows to the reservoirs, so far as they would cause excess of acceptable discharge spillway opening. Storage allows future use of the flood water retained.

- ***Amenity use:*** this may include provision for boating, water sports, fishing, sight seeing.

Formally, the Bureau of Indian Standards code IS: 4410 (part 6)1983 “Glossary of terms relating to river valley projects -Reservoirs" defines the following types of reservoirs:

- ***Auxiliary or Compensatory Reservoir:*** A reservoir which supplements and absorbed the spill of a main reservoir.

- ***Balancing Reservoirs:*** A reservoir downstream of the main reservoir for holding water let down from the main reservoir in excess of that required for irrigation, power generation or other purposes.

- ***Conservation Reservoir:*** A reservoir impounding water for useful purposes, such as irrigation, power generation, recreation, domestic, industrial and municipal supply etc.

- ***Detention Reservoir:*** A reservoir where in water is stored for a relatively brief period of time, past of it being retained until the stream can safely carry the ordinary flow plus the released water. Such reservoirs usually have outlets without control gates and are used for flood regulation. These reservoirs are also called as the ***Flood Control Reservoir*** or ***RetardingReservoir***.

- ***Distribution Reservoir:*** A reservoir connected with distribution system a water supply project, used primarily to care for fluctuations in demand which occur over short periods and as local storage in case of emergency such as a break in a main supply line failure of a pumping plant.

- ***Impounding or Storage Reservoir:*** A reservoir with gate-controlled outlets wherein surface water may be retained for a considerable period of time and released for use at a time when the normal flow of the stream is in sufficient to satisfy requirements.

- ***Multipurpose Reservoir:*** A reservoir constructed and equipped to provide storage and release of water for two or more purposes such as irrigation, flood control, power generation, navigation, pollution abatement, domestic and industrial water supply, fish culture, recreation, etc.

It may be observed that some of these objectives may be incompatible in combination. For example, water may has to be released for irrigation to suit crop growing seasons, while water releases for hydropower are required to suit the time of public and industrial demands. The latter will be affected not only by variations in economic conditions but also by variations over a day and night cycle.

Compatibility between irrigation demand and flood control strategy in operating a reservoir is even more difficult for a reservoir which intends to serve both, like the Hirakud Dam reservoir on the river Mahanadi. Flood wave moderation requires that the reservoir be emptied as much as possible so that it may absorb any incoming flood peak. However, this decision means reducing the water stored for irrigation. Usually, such a reservoir would be gradually emptied just before the arrival of monsoon rains, anticipating a certain flood and hoping that the reservoir would be filled to the brim at the end of the flood season. However, this anticipation may not hold good for all years and the reservoir does not get filled up to the optimal height. On the other hand, if the reservoir is not depleted sufficiently well, and actually a flood of high magnitude arrives, then the situation may lead to the flood inundations on the downstream.

Planning of reservoirs

The first step in planning the construction of a reservoir with the help of a dam is for the decision makers to be sure of the needs and purposes for which the reservoir is going to be built together with the known constraints (including financial), desired benefits. There may be social constraints, for examples people's activism may not allow a reservoir to be built up to the desired level or even the submergence of good agricultural level may be a constraint. Some times, the construction of a dam may be done that is labour intensive and using local materials, which helps the community for whom the dam is being built. This sort of work is quite common in the minor irrigation departments of various steps, especially in the drought prone areas. The Food-for-Work schemes can be utilised in creating small reservoirs that helps to serve the community. In a larger scale, similar strategy was adopted for the construction of the Nagarjuna Sagar Dam on the River Krishna, which was built entirely of coursed rubble masonry and using manual labour in thousands.

The second step is the assembly of all relevant existing information, which includes the following:

- Reports of any previous investigations and studies, if any.
- Reports on projects similar to that proposed which have already been constructed in the region.
- A geographical information system (GIS) for the area of interest may be created

using a base survey map of the region.

- Topographical data in the form of maps and satellite pictures, which may be integrated within the GIS.
- Geological data in the form of maps and borehole logs, along with the values of relevant parameters.
- Seismic activity data of the region that includes recorded peak accelerations or ground motion record.
- Meteorological and hydrological data - of available parameters like rainfall, atmospheric and water temperatures, evaporation, humidity, wind speed, hours of sunshine, river flows, river levels, sediment concentration in rivers, etc.
- For water supply projects, data on population and future population growth based on some acceptable forecast method, industrial water requirement and probable future industrial development.
- For irrigation projects, data on soils in the project area and on the crops already grown, including water requirement for the crops.
- For hydropower projects, data on past demand and forecasts of future public and industrial demand for power and energy; data on existing transmission systems, including transmission voltage and capacity.
- Data on flora and fauna in the project and on the fish in the rivers and lakes, including data on their migratory and breeding habits.
- Data on tourism and recreational use of rivers and lakes and how this may be encouraged on completion of the proposed reservoir.

As may be noted, some of the data mentioned above would be needed to design and construct the dam and its appurtenant structures which would help to store water behind the reservoir. However, there are other data that decides the following:

- How large the reservoir should be and, consequently, what should be the dam height?
- What should be the size of the spillway and at what elevation the crest level of the spillway be located?
- How many and at what levels sluices be provided and they should be of what sizes?

Two important aspects of reservoirs planning: Sedimentation Studies and Geological Explorations are described in detail in the following section.

Effect of sedimentation in planning of reservoirs

It is important to note that storage reservoirs built across rivers and streams loose their capacity on account of deposition of sediment. This deposition which takes place progressively in time reduces the active capacity of the reservoir to provide the outputs of water through passage of time. Accumulation of sediment at or near the dam may interfere with the future functioning of water intakes and hence affects decisions regarding location and height of various outlets. It may also result in greater inflow of into canals / water conveyance systems drawing water from the reservoir. Problems of rise in flood levels in the head reaches and unsightly deposition of sediment from recreation point of may also crop up in course of time.

In this regard, the Bureau of Indian Standard code IS: 12182 - 1987 “Guidelines for determination of effects of sedimentation in planning and performance of reservoir” is an important document which discusses some of the aspects of sedimentation that have to be considered while planning reservoirs. Some of the important points from the code are as follows:

While planning a reservoir, the degree of seriousness and the effect of sedimentation at the proposed location has to be judged from studies, which normally combination consists of:

1. Performance Assessment (Simulation) Studies with varying rate of sedimentation.
2. Likely effects of sedimentation at dam face.

In special cases, where the effects of sedimentation on backwater levels are likely to be significant, backwater studies would be useful to understand the size of river water levels. Similarly, special studies to bring out delta formation region changes may be of interest. The steps to be followed for performance assessment studies with varying rates of sedimentation are as follows:

a. Estimation of annual sediment yields into the reservoir or the average annual sediment yield and of trap efficiency expected.
b. Distribution of sediment within reservoir to obtain a sediment elevation and capacity curve at any appropriate time.
c. Simulation studies with varying rates of sedimentation.
d. Assessment of effect of sedimentation.

In general, the performance assessment of reservoir projects has to be done for varying hydrologic inputs to meet varying demands. Although analytical probability based methods are available to some extent, simulation of the reservoir system is the standard method. The method is also known as the working tables or sequential routing. In this method, the water balance of the reservoir s and of other specific locations of water use and constraints in the systems are considered. All inflows to and outflows from the reservoirs are worked out to decide the changed storage during the period. In simulation studies, the inflows to be used may be either historical inflow series, adjusted for future up stream water use changes or an adjusted synthetically generated series.

Procedure for planning a new reservoir

The standard procedure that needs to be carried out for planned storages requires an assessment of the importance of the problem to classify the reservoir sedimentation problem as insignificant, significant, or serious. Assessment of reservoir sedimentation problem, in a particular case may be made by comparing the expected average annual volume of sediment deposition with the gross capacity of the reservoir planned. If the ratio is more than 0.5 percent per year, the problem is usually said to be serious and special care is required in estimating the sediment yields from the catchment. If it is less than 0.1 percent per year, the problem of siltation may be insignificant and changes in reservoir performance. For cases falling between these two limits, the sedimentation problem is considered significant and requires further studies.

The following studies are required if the problem is insignificant:

1. No simulation studies with sediment correlation is necessary.
2. The feasible service time for the project may be decided. Sediment distribution studies to ensure that the new zero-elevation does not exceed the dead storage level may be made.

In the above, the following terms have been used, which are explained below:

- **Feasible Service Time**: For a special purpose, the period or notional period for which a reservoir is expected to provide a part of the planned benefit in respect of storage in the reservoirs being impaired by sedimentation. Customarily, it is estimated as the time after which the new zero elevation of the reservoir would equal the sill of the outlet relevant for the purpose.

- **New Zero Elevation**: The level up to which all the available capacity of the reservoir is expected to be lost due to progressive sedimentation of the reservoir up to the specified time. The specified time should be any length of time such as Full Service Time, Feasible Service time, etc.

- **Full Service Time**: For a specified purpose, the period or notional period for which the reservoir provided is expected to provide, a part of the full planned benefit inspite of sedimentation.

The following studies are required if the problem of sedimentation in the reservoir is assessed to be significant, but not serious.

1. Both the full service time and feasible service time for the reservoir may be decided.
2. Simulation studies for conditions expected at the end of full service time may be made to ensure that firm outputs with required depend ability are obtained. The studies used also assess non-dependable secondary outputs, if relevant, available at the end of this period. Studies without sedimentation, with the same firm outputs should bring out the additional potential secondary outputs which may be used, if required in economic analysis, using a linear decrease of these additional benefits over the full service time.
3. No simulation studies beyond full service time, is required.
4. Sediment distribution studies required for feasible service time are essential.

The following studies are required if the problem of sedimentation is serious.

1. All studies described for the 'Significant' case have to be made.
2. The secondary benefits available in the initial years should be more in such cases. If they are being utilized, for a proper assessment of the change of these, a simulation at half of full service time should be required.
3. In these cases, the drop of benefits after the full service time may be sharper. To bring out these effects, a simulation of the project at the end of the feasible service time is required to be done.

Life of reservoir and design criteria

A reservoir exists for a long time and the period of its operation should normally check large technological and socio-economic changes. The planning assumptions about the exact socio-economic outputs are, therefore, likely to be changed during operation, and similarly, the implication of socio-economic differences in the output due to sedimentation are difficult to access**.** The ever increasing demands due to both increase of population and increases in per capita needs are of a larger magnitude than the reductions in outputs, if any, of existing reservoirs. Thus effects of sedimentation, obsolescence, structural deterioration, etc. of reservoirs may require adjustments in future developmental plans and not simply replacement projects to bring back the lost potential. On a regional or national scale, it is the sufficiency of the total economic outputs, and not outputs of a particular project which is relevant. However, from local considerations, the reduction of outputs of reservoir like irrigation and flood control may cause a much greater degree of distress to the population which has got used to better socio-economic conditions because of the reservoir.

'Life' strictly is a term which may be used for system having two functional states 'ON' and 'OFF'. Systems showing gradual degradation of performance and not showing any sudden non-functional stage have no specific life period. Reservoirs fall in the later category.

The term 'life of reservoir' as loosely used denotes the period during which whole or a specified fraction of its total or active capacity is lost. In calculating this life, the progressive changes in trap efficiency towards the end of the period are commonly not considered. In some of the earlier projects, it has been assumed that all the sedimentation would occur only in the dead storage pocket and the number of years in which the pocket should be filled under this assumption was also sometimes termed as the life of reservoir. This concept was in fact used to decide the minimum size of the pocket. Under this concept, no effect of sedimentation should be felt within the live storage of the reservoir. It has subsequently been established that the silt occupies the space in the live storage of reservoir as well as the dead storage.

If the operation of the reservoir becomes impossible due to any structural defects, foundation defects, accidental damages, etc., this situation should also signify the end of the feasible service time. Before the expiry of this feasible service time, it may be possible to make large changes in the reservoir (for example, new higher level outlets, structural strengthening, etc.) or other measures, if it is economically feasible to do so. If these studies are done, the feasible service time may be extended.

Geological explorations for reservoir sites

In Lesson 4.4, geological exploration procedures for constructions of dams were discussed in detail. Though a dam is constructed to build a reservoir, a reservoir has a large area of spread and contained in a big chunk of the river valley upstream of the dam. Hence, while identifying a suitable site for a proposed dam, it is of paramount importance that the proposed reservoir site is also thoroughly investigated and explored. The basis of planning for such explorations is to have a rapid economical and dependable pre-investment evaluation of subsurface conditions. It is also necessary that a degree of uniformity be followed while carrying out subsurface explorations so that the frame of reference of the investigation covers all requisite aspects. In view of above, the Bureau of Indian Standards has brought out a code IS: 13216 - 1991 "Code of practice for geological exploration for reservoir sites", that discusses the relevant aspects. According to the code since reservoir projects in river valleys are meant to hold water; therefore, the following aspects of the reservoirs have to be properly investigated

(a) Water tightness of the basins
(b) Stability of the reservoir rim
(c) Availability of construction material in the reservoir area
(d) Silting
(e) Direct and indirect submergence of economic mineral wealth
(f) Seismo-techtonics

These aspects are determined through investigations carried out by surface and sub- surface exploration of proposed basin during the reconnaissance, preliminary investigation, detailed investigation, construction and post-construction stages of the project. The two basic stages of investigation: reconnaissance and preliminary investigations are explained below:

Reconnaissance

In the reconnaissance stage, the objective of investigation is to bring out the overall geological features of the reservoir and the adjacent area to enable the designers, construction engineers and geologists to pinpoint the geotechnical and ecological problems which have to be tackled. The scale of geological mapping for this stage of work need not be very large and the available geological maps on 1:50,000 or 1: 250,000 scales may be made use of. It is advantageous to carry out photo geological interpretation of aerial photographs of the area, if available. If a geological map of the area is not available, a traverse geological map should be prepared at this stage preferably using the aerial photos as base maps on which the engineering evaluation of the various geotechnical features exposed in the area should be depicted.

A topographical index map on 1: 50 000 scales should be used at this stage to delineate the areas which would require detailed study, subsequently.

To prevent an undesirable amount of leakage from the reservoir, the likely zones of such leakage, such as major dislocations and pervious or cavernous formations running across the divide of the reservoir should be identified at this stage of investigation for further detailed investigations.

Major unstable zones, particularly in the vicinity of the dam in tight gorges, should be identified at this stage for carrying out detailed investigations for the stability of the reservoir rim.

The locations for suitable construction material available in the reservoir area should be pin pointed at this stage so that after detailed surveys such materials can be exploited for proper utilisation during the construction stage prior to impounding of reservoir.

The rate of silting of the reservoir is vital for planning the height of the dam and working out the economic life of the project. Since the rate of silting, in addition to other factors, is dependent on the type of terrain in the catchment area of the reservoir, the major geological formations and the ecological set up should be recognized at this stage to enable a more accurate estimation of the rate of silting of the reservoir. For example, it should be possible to estimate at this stage that forty percent of the catchment of a storage dam project is covered by Quaternary sediment and that this is a condition which is likely to a yield a high silt rate or that ninety percent of the catchment of another storage dam project is composed of igneous and metamorphic rocks and is likely to yield a relatively low sediment rate. This information will also be useful in examining whether or not tributaries flowing for long distances through soft or unconsolidated formations, prior to forming the proposed reservoir, can be avoided and if not, what remedial measures can be taken to control the silt load brought by these tributaries.

The impounding of a reservoir may submerge economic/strategic mineral deposits occurring within the reservoir area or the resultant rise in the water table around the reservoir may cause flooding, increased seepage in quarries and mines located in the area and water logging in other areas. It is, therefore, necessary that the economic mineral deposits, which are likely to be adversely affected by the reservoir area, are identified at this stage of the investigation. For example, if an underground working is located close to a proposed storage reservoir area, it should be identified for regular systematic geo-hydrological studies subsequently. These studies would establish whether the impoundment of the water in the reservoir had adversely affected the underground working or not. References should also be made to various agencies dealing with the economic minerals likely to be affected by the impoundment in the reservoir for proper evaluation of the problem and suitable necessary action.

A dam and its reservoir are affected by the environment in which they are located and in turn they also change the environment. Impoundment of a reservoir sometimes results in an increase of seismic activity at, or near the reservoir. The seismic activity may lead to microtremors and in some cases lead to earthquakes of high magnitude. It is, therefore, necessary to undertake the regional seismotechtonic study of the project area. The faults having active seismic status should be delineated at this stage.

Simultaneous action to plan and install a network of seismological observatories encompassing the reservoir area should also be taken.

Preliminary Investigation

The object of preliminary investigation of the reservoir area is to collect further details of the surface and subsurface geological conditions, with reference to the likely problems identified during the reconnaissance stage of investigation by means of surface mapping supplemented by photo geological interpretation of aerial photographs, hydro geological investigations, geophysical investigations, preliminary subsurface exploration and by conducting geo-seismological studies of the area.

On the basis of studies carried out during the reconnaissance stage it should be possible to estimate the extent of exploration that may be required during the preliminary stage of investigation including the total number of holes required to be drilled and the total number and depth of pits, trenches and drifts as also the extent of geophysical surveys which may be necessary. For exploration by pits, trenches, drifts and shafts guidelines laid down in IS 4453: 1980 Name of IS code should be followed.

The potential zones of leakage from the reservoir and the lateral extent of various features, such as extent of aeolian sand deposits, glacial till, land slides, major dislocations or pervious and cavernous formations running across the divide, should be delineated on a scale of 1: 50000.

The geo-hydrological conditions of the reservoir rim should be established by surface and sub-surface investigation as well as inventory, as a free ground water divide rising above the proposed level of the reservoir is a favourable condition against leakage from the reservoir. The level of water in a bore hole should be determined as given in IS 6935: 1973.

The extension of various features at depth, wherever necessary, is investigated by geophysical exploration and by means of pits, trenches, drifts and drill holes. For example, the resistivity survey should be able to identify water saturated zones. The nature of the material is investigated by means of laboratory and in situ tests, to determine permeability and assess the quantum of leakage which may take place through these zones on impoundment of the reservoir. Moreover, permeability of rocks/overburden in the reservoir area is determined from water table fluctuations and pumping tests in wells. For determining in situ permeability in overburden and rock, reference should be made to IS 5529 (Part I): 1985 and IS: 5529 (Part II): 1985 respectively. The information about permeability would enable the designers to estimate the treatment cost for controlling leakage/seepage from the reservoir and to decide whether it would be desirable to change the location of height of the dam to avoid these zones.

Major unstable zones along the reservoir identified during the reconnaissance stage and which are of consequence to the storage scheme should be investigated in detail at this stage by means of surface and sub-surface exploration.

The areas should be geologically mapped in detail on a scale of 1: 2000. The suspect planes/zones of failure should be identified and explored by means of drifts, trenches and pits. Disturbed and undisturbed samples of the plastic material should be tested for cohesion (c) and angle of internal friction (ϕ) as well as for other relevant properties The stability of slopes should also be evaluated considering the reservoirs operational conditions. These studies should provide the designers with an idea of the magnitude of the problems that may be encountered, so that they may be able to take remedial measures to stabilize zones or to abandon the site altogether, if the situation demands.

The areas having potential economic mineral wealth and which are likely to be adversely affected by the impoundment of the reservoir should be explored by means of surface and sub-surface investigation to establish their importance both in terms of their value as well as strategic importance. This information would be necessary for arriving at a decision regarding the submergence, or otherwise, of the mineral deposit. The nature and amount of the existing seepage, if any, in the existing mines and quarries in the adjacent areas of the reservoir should be recorded and monitored regularly. This data is necessary, to ascertain whether or not there has been any change in the quantum of seepage in the mines and quarries due to the impoundment of water in the reservoir, directly or indirectly.

Large scale geological mapping and terrace matching across the faults with seismically active status, delineated during the reconnaissance stage, should be carried out on a scale of 1 : 2000 and the trend, and behaviour of the fault plane should be investigated in detail by means of surface studies and sub-surface exploration by pits, trenches and drifts etc. A network of geodetic survey points should be established on either side of the suspected faults to study micro-movements along these suspected faults, if any, both prior to and after impoundment of the reservoir. Micro earthquake studies should be carried out using portable 3-station or 4-station networks in areas with proven seismically active fault features.

On the basis of the studies carried out during the preliminary stage it should be possible to estimate the quantum of exploration which may be required during the detailed stage of investigation including the total number of holes required to be drilled and the total number and depth of pits, trenches and drifts as also the extent of geophysical survey which may be necessary.

Detailed surface and sub-surface investigation of all features connected with the reservoir should be carried out to provide information on leakage of water through the periphery and/or basin of the reservoir area.

Based on these investigations and analysis of data it should be possible to decide as to whether the reservoir area in question would hold water without undue leakage. If, not, the dam site may have to be abandoned in favour of suitable alternative site.

The zones, which on preliminary investigation are found to be potential zones of leakage/seepage from the reservoir, and which due to other considerations cannot be avoided are geologically mapped on a scale of 1 : 2 000 and investigated in detail at this stage by means of a close spaced sub-surface exploration programme. The purpose of this stage of investigation is to provide the designers sufficient data to enable them to plan the programme of remedial treatment. The sub-surface explorations are carried out by means of pits and trenches, if the depth to be explored is shallow, say up to 5 meters, and by drill holes and drifts, if the depth to be explored is greater than 5 meters.

The unstable zones around the reservoir rim, specially those close to the dam sites in tight gorges, should be explored in detail by means of drifts, pits and trenches so that the likely planes of failures are located with precision. The physical properties including angle of internal friction and cohesion of representative samples of the material along which movement is anticipated should be determined. The above information would enable the designers to work out details for preventive measures, for example, it may be possible to unload the top of the slide area or to load the toe of the slide with well drained material, within economic limits.

Sub-surface explorations by drill holes, drifts, pits and trenches should be carried out at possible locations of check dams and at the locations of other preventive structures proposed to restrict the flow of silt into the reservoir. These studies would enable the designers to assess the feasibility of such proposals.

Detailed plans, regarding the economic mineral deposits within the zones of influence of the reservoir should be finalized during this stage by the concerned agencies. The seepage investigations in the quarries and mines within the zone of influence of the reservoir should be continued

Fixing the capacity of the reservoirs

Once it is decided to build a reservoir on a river by constructing a dam across it, it is necessary to arrive at a suitable design capacity of the reservoir. As has been discussed in section 4.5.1, the reservoir storage generally consists of there main parts which may be broadly classified as:

1. Inactive storage including dead storage
2. Active or conservation storage, and
3. Flood and surcharge storage.

In general, these storage capacities have to be designed based on certain specified considerations, which have been discussed separately in the following Bureau of Indian Standard codes:

IS: 5477 Fixing the capacities of reservoirs- Methods
(Part 1): 1999 General requirements
(Part 2): 1994 Dead storage
(Part 3): 1969 Line storage
(Part 4): 1911 Flood storage

The data and information required for fixing the various components of the design capacity of a reservoir are as follows:

a) Precipitation, run-off and silt records available in the region;
b) Erodibility of catchment upstream of reservoir for estimating sediment yield;
c) Area capacity curves at the proposed location;
d) Trap efficiency;
e) Losses in the reservoir;
f) Water demand from the reservoir for different uses;

g) Committed and future upstream uses;
h) Criteria for assessing the success of the project;
i) Density current aspects and location of outlets;
j) Data required for economic analysis; and
k) Data on engineering and geological aspects.

These aspects are explained in detail in the following sections.

Precipitation, Run-Off and Silt Record

The network of precipitation and discharge measuring stations in the catchment upstream and near the project needs to be considered to assess the capacity of the same to adequately sample both spatially and temporally the precipitation and the stream flows.

The measurement procedures and gap filling procedures in respect of missing data as also any rating tables or curves need to be critically examined so that they are according to guidelines of World Meteorological Organization (WMO). Long-term data has to be checked for internal consistency between rainfall and discharges, as also between data sets by double mass analysis to highlight any changes in the test data for detection of any long term trends as also for stationarity. It is only after such testing that the data should be used for generating the long term inflows of water (volumes in 10 days, 15 days, monthly or yearly inflow series) into the reservoir.

Sufficiently long term precipitation and run-off records are required for preparing the water inflow series. For working out the catchment average sediment yield, long-term data of silt measurement records from existing reservoirs are essential. These are pre- requisites for fixing the storage capacity of reservoirs.

If long term run-off records are not available, concurrent rainfall and run-off data may be used to convert long term rainfall data (which is generally available in many cases) into long-term run-off series adopting appropriate statistical/conceptual models. In some cases regression analysis may also be resorted to for data extension.

Estimation of average Sediment Yield from the catchment area above the reservoir

It is usually attempted using river sediment observation data or more commonly from the experience of sedimentation of existing reservoirs with similar characteristics. Where observations of stage/flow data is available for only short periods, these have to be suitably extended with the help of longer data on rainfall to estimate as far as possible sampling errors due to scanty records. Sediment discharge rating curve may also be prepared from hydraulic considerations using any of the standard sediment load formulae, such as, Modified Einstein's procedure, Young's stream power, etc. It is also necessary to account for the bed load which may not have been measured. Bed load measurement is preferable and when it is not possible, it is often estimated as a percentage generally ranging from 5 to 20 percent of the suspended sediment load. However, actual measurement of bed load needs to be undertaken particularly in cases where high bed loads are anticipated. To assess the volume of sediment that would deposit in the reservoir, it is further necessary to make estimates of average trap efficiency of the reservoir and the likely unit weight of sediment deposits, along with time average over the period selected. The trap efficiency would depend on the capacity inflow ratio but would also vary with the locations of controlling outlets and reservoir operating procedures. Computations of reservoir trap efficiency may be made using the trap efficiency curves such as those developed by Brune and by Churchill (see IS: 12182-1987).

Elevation Area Capacity Curves

Topographic survey of the reservoir area should form the basis for obtaining these curves, which are respectively the plots of elevation of the reservoir versus surface area and elevation of the reservoir versus volume. For preliminary studies, in case suitable topographic map with contours, say at intervals less than 2.5 m is not available, stream profile and valley cross sections taken at suitable intervals may form the basis for computing the volume. Aerial survey may also be adopted when facilities are available.

Trap Efficiency

Trap efficiency of reservoir, over a period, is the ratio of total deposited sediment to the total sediment inflow. Figures 1 and 2 given in Annex A of IS 12182 cover relationship between sedimentation index of the reservoir and percentage of incoming sediment and these curves may be used for calculation of trap efficiency.

Losses in Reservoir

Water losses mainly of evaporation and seepage occur under pre-project conditions and are reflected in the stream flow records used for estimating water yield. The construction of new reservoirs and canals is often accompanied by additional evaporation and infiltration. Estimation of these losses may be based on measurements at existing reservoirs and canals. The measured inflows and outflows and the rate of change of storage are balanced by computed total loss rate.

The depth of water evaporated per year from the reservoir surface may vary from about 400 mm in cool and humid climate to more than 2500 mm in hot and arid regions. Therefore, evaporation is an important consideration in many projects and deserves careful attention. Various methods like water budget method, energy budget method, etc may be applied for estimating the evaporation from reservoir. However, to be more accurate, evaporation from reservoir is estimated by using data from pan-evaporimeters or pans exposed to atmosphere with or without meshing in or near the reservoir site and suitably adjusted.

Seepage losses from reservoirs and irrigation canals may be significant if these facilities are located in an area underlain by permeable strata. Avoidance in full or in part of seepage losses may be very expensive and technical difficulties involved may render a project unfeasible. These are generally covered under the conveyance losses in canals projected on the demand side of simulation studies.

Demand, Supply and Storage

The demand should be compared with supplies available year by year. If the demand is limited and less than the available run-off, storage may be fixed to cater to that particular demand which is in excess of the run-off. The rough and ready method is the mass curve method for initial sizing.

Even while doing the above exercise, water use data are needed to assess the impact of human activities on the natural hydrological cycle. Sufficient water use information would assist in implementing water supply projects, namely, evaluating the effectiveness of options for demand management and in resolving problems inherent in competing uses of water, shortages caused by excessive withdrawal, etc. Water demands existing prior to construction of a water resource project should be considered in the design of project as failure to do so may result in losses apart from legal and social problems at the operation stage.

Committed and future upstream uses

The reservoir to be planned should serve not only the present day requirements but also the anticipated future needs. The social, economic and technological developments may bring in considerable difference in the future needs/growth rate as compared to the present day need/growth rate. Committed and upstream future uses should also be assessed in the same perspective.

Criteria for assessing the success of the project

Water Resources Projects are to be designed for achieving specified success. Irrigation projects are to be successful for 75 percent period of simulation. Likewise power projects and water supply projects are to be successful for 90 percent and nearly 100 percent period of simulation respectively.

Density Current aspects and location of outlets

Density current is defined as the gravitational flow of one fluid under another having slightly different density. The water stored in reservoir is generally free from silt but the inflow during floods is generally muddy. There are, thus two layers having different densities resulting in the formation of density currents. The density currents separate the water from the clearer water and make the turbid water flow along the river bottom. The reservoir silting rate can be reduced by venting the density currents by properly locating and operating the outlets and sluice ways.

Data Required for Economic Analysis

Economic Analysis is carried out to indicate the economic desirability of the project. Benefit cost ratio, Net benefit, Internal Rate of Return are the parameters in this direction. It is desirable to have the benefit cost ratio in the case of irrigation projects and flood mitigation projects to be above 1.5 and 1.1 respectively. Benefit functions for reservoir and water utilisation for irrigation, power, water supply etc., are also to be determined judiciously. Cost benefit functions are obtained as continuous functions using variable cost/benefit against reservoir storage/net utilisation of water and from benefit functions the benefit from unit utilisation of water can be determined. The spillway capacity has to be adequate to pass the inflow design flood using moderation possible with surcharge storage or any other unobstructed capacity in the reservoir without endangering the structural safety as provided elsewhere in the standard. In the event of the inflow design flood passing the reservoir, the design needs to ensure that dam break situation does not develop or induce incremental damage downstream.

Data on Engineering and Geological Aspects

Under engineering and geological aspects the following items of work shall invariably be carried out:

a) Engineering

1) Preliminary surveys to assess the catchment and reservoir,
2) Control surveys like topographical surveys,
3) Location of nearest Railway lines/Roads and possible access, and
4) Detailed survey for making area capacity curves for use in reservoir flood routing.

b) Geology

1) General formations and foundation suitability;
2) Factors relating to reservoir particularly with reference to water tightness;
3) Contributory springs;
4) Deleterious mineral and salt deposits; and
5) Location of quarry sites, etc.

Important aspects of the methods are briefly described in the subsequent sections.

Fixing of Inactive Storage including Dead Storage

Inactive storage including dead storage pertains to storage at the lowest level up to which the reservoir can be depleted. This part of the storage is set apart at the design stage for anticipated filling, partly or fully, by sediment accumulations during the economic life of the reservoir and with sluices/outlets so located that it is not susceptible to full depletion. In case power facility is provided, it is also the storage below the minimum draw down level (MDDL).

Sill level of lowest outlets for any reservoir is fixed from command considerations in case of irrigation purposes and minimum draw down level on considerations of efficient turbine operation in the case of power generation purpose. The lowest sill level should be kept above the new zero elevation expected after the feasible service period according to IS 12182 which is generally taken as 100 years for irrigation projects and 70 years for power projects supplying power to a grid.

By providing extra storage volume in the reservoir for sediment accumulation, in addition to live storage, it is ensured that the live storage although it contains sediment, will function at full efficiency for an assigned number of years. The distribution pattern of sediments in the entire depth of a reservoir depends upon many factors, such as slope of the valley, length of reservoir, constriction in the reservoir, particle size of the suspended sediment and capacity inflow ratio, but the reservoir operation has an important control over the factors. However, the knowledge of the pattern is essential, especially, in developing areas, in order to have an idea about the formation of delta and recreational spots.

The dead storage of a reservoir depends upon the sediment yield of the catchment. The measurement of sediment yield is done as follows:

Measurement of sediment yields

The sediment yield in a reservoir may be estimated by any one of the following two methods:

a) Sedimentation surveys of reservoirs with similar catchment characteristics, or
b) Sediment load measurements of the stream.

Reservoir Sedimentation Survey

The sediment yield from the catchment is determined by measuring the accumulated sediment in a reservoir for a known period, by means of echo sounders and other electronic devices since the normal sounding operations give erroneous results in large depths. The volume of sediment accumulated in a reservoir is computed as the difference between the present reservoir capacity and the original capacity after the completion of the dam. The unit weight of deposit is determined in the laboratory front the representative undisturbed samples or by field determination using a calibrated density probe developed for this purpose. The total sediment volume is then converted to dry- weight of sediment on the basis of average unit weight of deposits. The total sediment yield for the period of record covered by the survey will then be equal to the total weight of the sediment deposited in the reservoir plus that which has passed out of the reservoir based on the trap efficiency. In this way, reliable records may be readily and economically obtained on long-term basis.

The density of deposited sediment varies with the composition of the deposits, location of the deposit within the reservoir, the flocculation characteristics of clay content and water, the age of deposit, etc. For coarse material (0.0625 mm and above) variation of density with location and age may be unimportant.

Normally a time and space average density of deposited materials applicable for the period under study is required for finding the overall volume of deposits. For this purpose the trapped sediment for the period under study would have to be classified in different fractions. Most of the sediment escape front getting deposited into the reservoir should be front the silt and clay fractions. In some special cases local estimates of densities at points in the reservoir may be required instead of average density over the whole reservoir.

The trap efficiency mainly depends upon the capacity-in-flow ratio but may vary with location of outlets and reservoir operating procedure. Computation of reservoir trap efficiency may be made using trap efficiency curves, such as those developed by Brune and by Churchill (see IS: 12182-1987).

Sediment Load Measurements

Periodic samples front the stream should be taken at various discharges along with the stream gauging observations and the suspended sediment concentration should be measured as detailed in IS 4890: 1968. A sediment rating curve which is a plot of sediment concentration against the discharge is then prepared and is used in conjunction with stage duration curve (or flow duration) based on uniformly spaced daily or shorter time units data in case of smaller river basins to assess sediment load. For convenience, the correlation between sediment concentration against discharge, may be altered to the relation of sediment load against run-off for calculating sediment yield. Where observed stage/flow data is available for only shorter periods, these have to be suitably extended with the help of longer data on rainfall. The sediment discharge rating curves may also be prepared from hydraulic considerations using sediment load formula, that is, modified Einstein's procedure.

The bed load measurement is preferable. How- ever, where it is not possible, it may be estimated using analytical methods based on sampled data or as a percentage of suspended load (generally ranging from 10 to 20 percent). This should be added to the suspended load to get the total sediment load.

Fixing of Live Storage

The storage of reservoir includes the Active Storage (or Conservation Storage) and the Buffer Storage.

Active or conservation storage assures the supply of water from the reservoir to meet the actual demand of the project whether it is for power, irrigation, or any other demand water supply.

The active or conservation storage in a project should be sufficient to ensure success in demand satisfaction, say 75 percent of the simulation period for irrigation projects, whereas for power and water supply projects success rates should be 90 percent and 100 percent respectively. These percentages may be relaxed in case of projects in drought prone areas. The simulation period is the feasible service period, but in no case be less than 40 years. Storage is also provided to satisfy demands for maintaining draft for navigation and also maintaining water quality for recreation purpose as envisaged in design.

Live storage capacity of a reservoir is provided to impound excess waters during periods of high flow, for use during periods of low flow. It helps the usage of water at uniform or nearly uniform rate which is greater than the minimum flow live storage has to guarantee a certain quantity of water usually called safe (or firm) yield with a predetermined reliability. Though sediment is distributed to some extent in the space for live storage, the capacity of live storage is generally taken as the useful storage between the full reservoir level and the minimum draw-down level in the case of power projects and dead storage in the case of irrigation projects.

The design of the line storage include certain factors, of which the most important in the availability of flow, since, without an adequate flow, it is not possible to cope up with the demand at all periods and seasons throughout the year. When adequate flow is available, there may still be certain problems like the possible maximum reservoir capacity from physical considerations may be limited and then this becomes the governing criteria. Even if an adequate reservoir capacity may be possible to be built, the governing factor may have to be based on the demand.

For fixing the live storage capacity, the following data should be made use of:

a) Stream flow data for a sufficiently long period at the site;
b) Evaporation losses from the water-spread area of the reservoir and seepage losses and also recharge into reservoir when the reservoir is depleting;
c) The contemplated irrigation, power or water supply demand;
d) The storage capacity curve at the site.

Stream flow records are required at proposed reservoir site. In the absence of such records the records from a station located upstream or downstream of the site on the stream or .on a nearby stream should be adjusted to the reservoir site. The run off records are often too short to include a critical drought period. In such a case the records should be extended by comparison with longer stream flow records in the vicinity or by the use of rainfall run off relationship.

The total evaporation losses during a period are generally worked out roughly as the reduction in the depth of storage multiplied by the mean water-spread area between the full reservoir level and the minimum draw-down level. For accurate estimation, monthly working tables should be prepared and the mean exposed area during the month is found out and the losses should be then worked out on the basis of this mean exposed area, and the evaporation data from pan evaporimeter at the reservoir site. The details are expected to be covered in the draft ‘Indian Standard criteria for determination of seepage and evaporation losses including the code for minimizing them. In the absence of’ actual data these may be estimated from the records of an existing reservoir with similar characteristics, like elevation, size, etc, in the neighbourhood.

Of the various methods available for fixing the live storage capacity, the Working Table method may be used which is prepared on the basis of preceding long term data on discharge observation at the site of the proposed reservoir, inclusive of at least one drought period. A typical format for carrying out the working table computation, is given in the following table:

The working table calculations may be represented graphically by plotting the cumulative net reservoir inflow exclusive of upstream abstraction as ordinate against time as abscissa. This procedure is commonly called the Mass Curve Technique, where the ordinate may be denoted by depth in centimeters or in hectare meters or in any other unit of volume. Discharge, with units of 10 days or a month may be used culmination in the mass curve. A segment of the mass curve is shown in Figure 3.

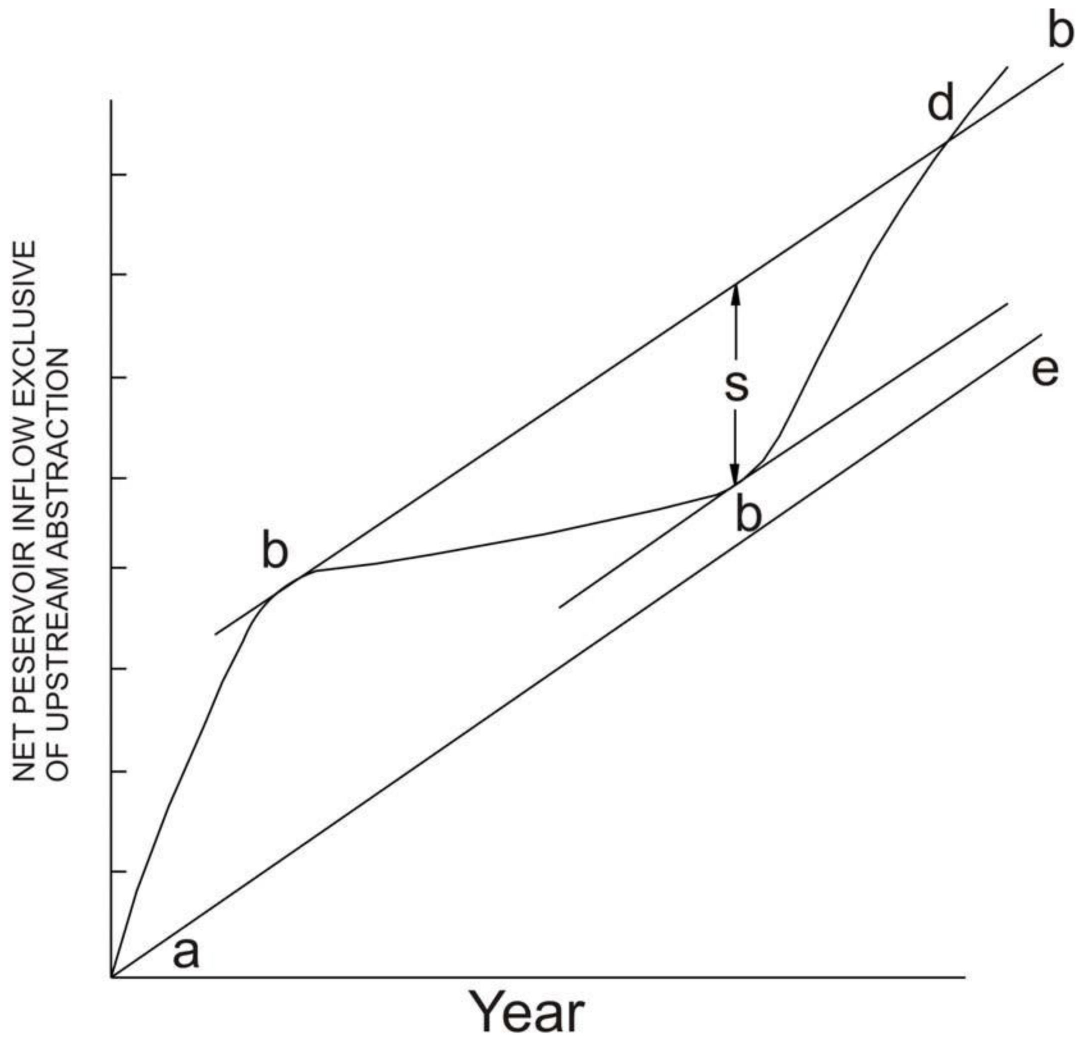


FIGURE3.SEGMENT OF NET INFLOW MASS CURVE

The difference in the ordinate at the end of a segment of the mass curve gives the inflow volume during that time interval. Lines parallel to the lines of uniform rate of demand are drawn at the points ***b*** and ***c*** of the mass curve. At ***d***, the following inferences can be made:

- The inflow rate between ***a*** to ***b*** is more than the demand rate and the reservoir is full.
- Reservoir is just full as the inflow rate is equal to the demand rate.
- Reservoir storage is being drawn down between ***b*** and ***c*** since the demand rate exceeds the inflow rate.
- Draw down, S, is maximum at ***c*** due to demand rate being equal to inflow rate.
- Reservoir is filling or in other words draw down is decreasing from ***c*** to ***d*** as the inflow rate is more than the demand rate.
- Reservoir is full at ***d*** and from ***d*** to *b* again the reservoir is over flowing because the inflow rate exceeds the demand rate. The greatest vertical distance, S at ***c*** is the storage required to make up the proposed demand.

The withdrawals from the reservoir to meet the irrigation demand are generally variable and in such cases the demand line becomes a curve instead of a straight line. The demand mass curve should be super-imposed on the inflow mass curve on the same time scale. When the inflow and demand mass curves intersect, the reservoir may be assumed to be full. For emptying conditions of the reservoir the demand curve would be above the inflow curve and the maximum ordinate between the two would indicate the live storage capacity required.

Fixation of Live Storage Capacity for a Given Demand

Lines parallel to the demand lines are drawn at all the peak points of the mass inflow curve exclusive of upstream abstraction obtained from a long run off record on 10-day (or monthly) basis as shown in Figure 4. When the demand line cuts the mass curve the reservoir may be assumed to be full. The maximum ordinate between the demand line and the mass curve will give the live storage to meet the required demand. The vertical distance between the successive lines parallel to the demand line represents the surplus water from the reservoir.

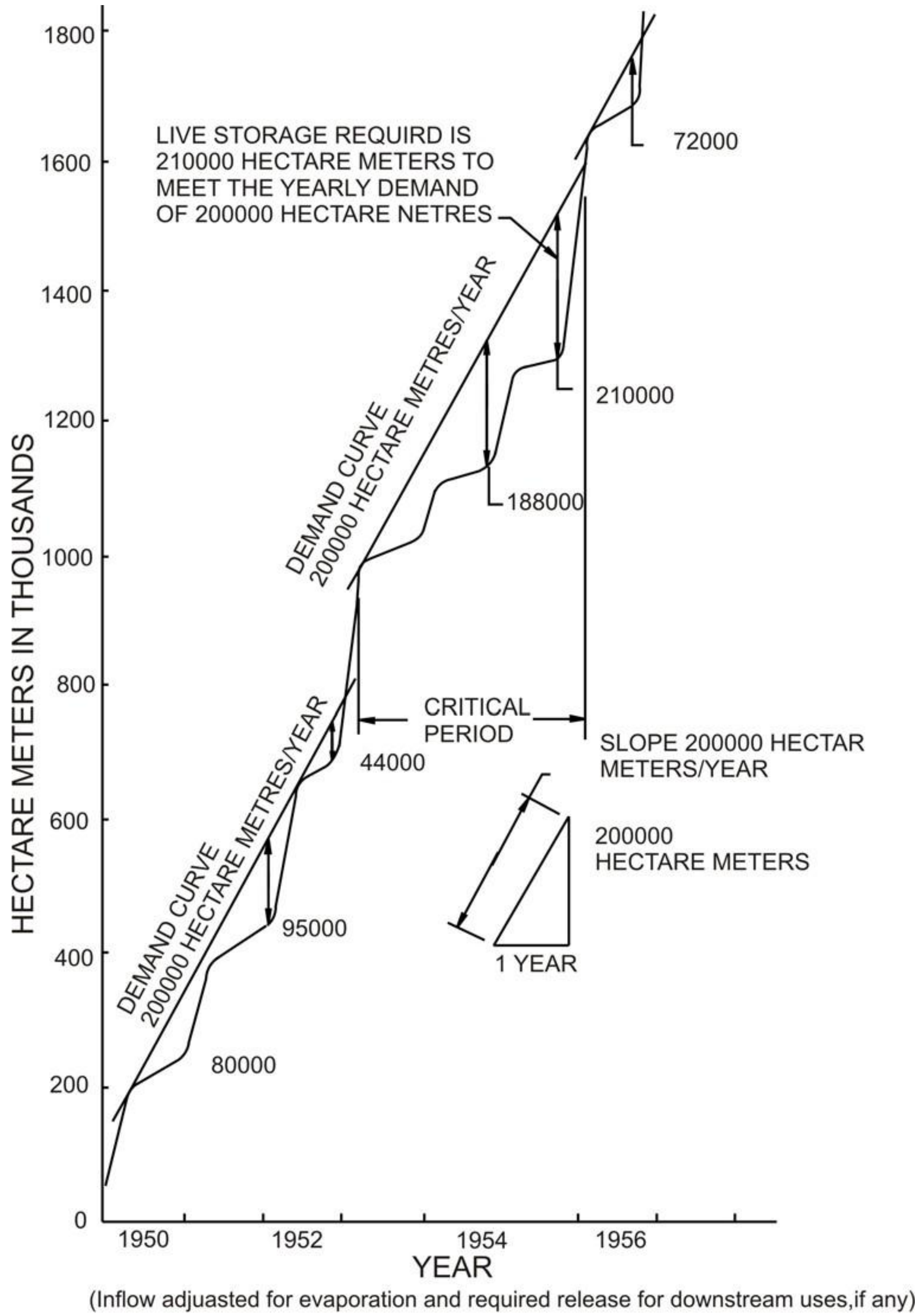


FIGURE 4. A TYPICAL MASS CURVE

Estimation of a Demand from a Given Live Storage Capacity

The net inflow mass curve is plotted from the available records. The demand lines arc, drawn at peak points of the mass curve in such a way that the maximum ordinate between the demand line and the mass curve is equal to the specified live storage. The demand lines shall intersect the mass curve when extended forward. The slop of the flattest line indicates the film demand that could he met by the given live storage rapacity.

Before fixing the reservoir capacity, it would be desirable to plot a curve between the net annual drafts and the required live storage capacities for these drafts. This curve will give an indication of the required live storage capacity. However, the economics of the capacity will have to be considered before deciding final capacity.

Fixing of flood and surcharge storage

In case of reservoirs having flood control as one of the purposes, a separate flood control storage is to be set apart above the storage meant for power, irrigation and water supply. Flood control storage is meant for storing flood waters above a particular return period temporarily and to attenuate discharges up to that flood magnitude to minimise effects on downstream areas from flooding. Flood and surcharge storage between the full reservoir level (FRL), and maximum water level (MWL) attainable even with full surplussing by the spillway takes care of high floods and moderates them.

Flood Control Storage

Storage space is provided in the reservoir for storing flood water temporarily in order to reduce peak discharge of a specified return period flood and to minimize flooding of downstream areas for all floods IS: 5477 (Part 1) : 1999 equal to or lower than the return period flood considered. In the case of reservoirs envisaging flood moderation as a purpose and having separate flood control storage, the flood storage is provided above the top of conservation pool.

Surcharge Storage

Surcharge storage is the storage between the full reservoir level (FRL) and the maximum water level (MWL) of a reservoir which may be attained with capacity exceeding the reservoir at FRL to start with. The spillway capacity has to be adequate to pass the inflow design flood making moderation possible with surcharge storage.

The methods that are generally used for estimate of the Design Flood for computing the Flood Storage are broadly classified as under:

1. Application of a suitable factor of safety to maximum observed flood or maximum historical flood.
2. Empirical flood formulae.
3. Envelope curves.
4. Frequency analysis.
5. Rating method of derivation of design flood from storm studies and application of the Unit Hydrograph principle.

The important methods amongst the above have been explained in module 2. Nevertheless, these methods are briefly reiterated below:

Application of a Suitable Factor of Safety to Maximum Observed Flood or Maximum Historical Flood

The design flood is obtained by applying a safety factor which depends upon the judgement of the designer to the observed or estimated maximum historical flood at the project site or nearby site on the same stream. This method is limited by the highly subjective selection of a safety factor and the length of available stream flow record which may give an inadequate sample of flood magnitudes likely to occur over a long period of time.

Empirical Flood Formulae : The empirical formulae commonly used in the country are the Dicken's formula, Ryve's formula and Inglis' formula in which the peak flow is given as a function of the catchment area and a coefficient. The values of the coefficient vary within rather wide, limits and have to be selected on the basis of judgement. They have limited regional application, should be used with caution and only, when a more accurate method cannot be applied for lack of data.

Envelope Curves: In the envelope curve method maximum flood is obtained from the envelope curve of all the observed maximum floods for a number of catchments in a homogeneous meteorological region plotted against drainage area. This method, although useful for generalizing the limits of floods actually experienced in the region under consideration, cannot be relied upon for estimating maximum probable floods for the determination of spillway capacity except as an aid to judgement.

Frequency Analysis: The frequency method involves the statistical analysis of observed data of a fairly long (at least 25 years) period. It is a purely statistical approach and when applied to derive design floods for long recurrence intervals, several times larger than the data, has many limitations. Hence this method has to be used with caution.

Rational Method of Derivation of Design Flood from Storm Studies and Application of Unit Hydrograph Principle

The steps involved, in brief, are:

a. Analysis of rainfall and run-off data for derivation of loss rates under critical conditions;
b. Derivation of unit hydrograph by analysis (or by synthesis, in cases where data are not available);
c. Derivation of the design storm; and
d. Derivation of design flood from the design storm by the application of the rainfall excess increments to the unit hydrograph.

The Maximum Water Level of a reservoir is obtained by routing the design flood through the reservoir and the spillway. This process of computing the reservoir storages, storage volumes and outflow rates corresponding to a particular hydrograph of inflow is commonly referred to as flood routing. The routing is carried out with the help of the following data,

1. Initial reservoir stage
2. The design flood hydrograph
3. Rate of outflow including the flow over the crest, through sluices or outlets and through power units, and
4. Incremental storage capacity of the reservoir.

Typical values of the last two types of data, is shown in Figure 5.

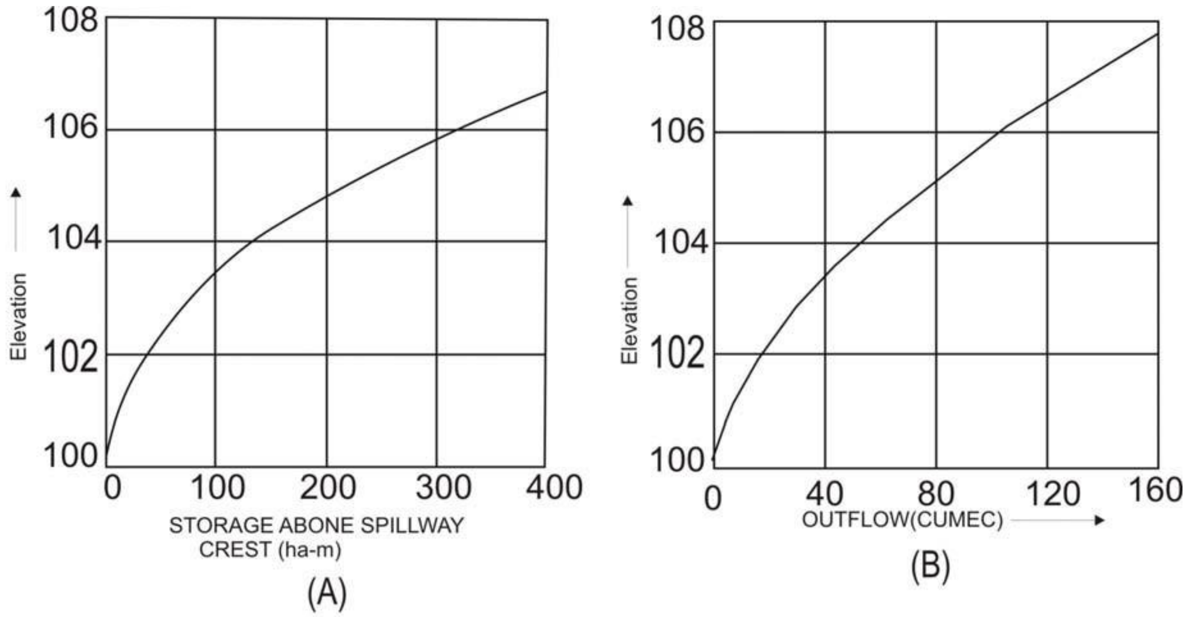


FIGURE 5. TYPICAL CURVES FOR (A) STORAGE Vs ELEVATION
(B) OUTFLOW Vs ELEVATION

The routing of flood through the reservoir and the spillway is done by solving the continuity of flow within reservoir, which may simply be stated as:
Inflow to reservoir - Outflow to reservoir = Rise is water surface of the reservoir, that is an increase in the storage of the reservoir. That is,

$$(I - O) * \Delta t = \Delta S \qquad (1)$$

Where, I is the inflow discharge (m^3/s), O is the Outflow discharge (m^3/s), Δ s is the in storage volume (m^3/s) in time interval Δ t (h).
If the inflow hydrograph is known, then we may read out the inflow ordinates at every time interval (t. Suppose, the following values are known:

I1 = Inflow (m3/s) at the beginning of a time interval I2
= Inflow (m3/s) at the end of the time interval

O1= Outflow (m3/s) at the beginning of the time interval
S1= Total storage volume of the reservoir (m3/s) at the beginning of the time interval

And the unknown values are

S2= Total storage volume of the reservoir (m3/s) at the end of the interval O2=
Outflow (m3/s) at the end of the time interval

Reservoir losses and their minimization

Loss of reservoir water would mainly take place due to evaporation and a number of methods have been suggest for controlling such loss. The Bureau of Indian Standard code IS: 14654 - 1999 “Minimizing evaporation losses from reservoirs- guidelines” describes the cause of evaporation reduction methods in detail, some important aspects of which are described in the subsequent paragraphs. As such, percolation or seepage loss is small for most of the reservoirs and progressively gets lowered with the passage of time since the sediment getting deposited at the reservoir bottom helps to reduce percolation losses. Of course, in some hills and valleys forming the reservoir, there may be continuous seams of porous rock strata or limestone caverns which cause huge amount of water to get drained out of the reservoirs. The reservoir of the Kopili Hydroelectric Project in Assam-Meghalaya border had faced similar problems due to the presence of large caverns which had to be sealed later at quite large cost at a later stage.

A number of factors affect the evaporation from open water surface, of which, the major factors are water spread area and frequent change of speed and direction of wind over the water body. Other meteorological factors like.

a) Vapour pressure difference_ between water surface and the layer of air above;
b) Temperature of water and air;
c) Atmospheric pressure;
d) Radiation;
e) Heat storage in water body; and
f) Quality of water,

have direct influence on the rate of evaporation.

Since the meteorological factors affecting evaporation cannot be controlled under normal conditions, efforts are made for inhibition of evaporation by control of flow of wind over water surface or by protection of the water surface area by physical or chemical methods. The methods generally used are as follows:

a) Wind breakers,
b) Covering the water surface,
c) Reduction of exposed water surface,
d) Integrated operation of reservoirs, and
e) Treatment with chemical water evaporetardants (WERs).

Wind Breakers

Wind is one of the most important factors which affect rate of evaporation loss from water surface. The greater the movement of air over the water surface, greater is the evaporation loss. Planting of trees normal to windward direction is found to be an effective measure for checking of evaporation loss. Plants (trees, shrubs or grass) should be grown around the rim of tanks in a row or rows to act as wind breaker.

These wind breakers are found to influence the temperature, atmospheric humidity, soil moisture, evaporation and transpiration of the area protected.

Plants to act as wind breakers are usually arranged in rows, with tallest plants in the middle and the smallest along the end rows, so that more or less conical formation is formed.

Covering the Water Surface

Covering the surface of water bodies with fixed or floating covers considerably retards evaporation loss. These covers reflect energy inputs from atmosphere, as a result of which evaporation loss is reduced. The covers literally trap the air and prevent transfer of water vapour to outer atmosphere.

Fixed covers are suitable only for relatively small storages. For large storages, floating covers or mat or spheres may be useful and effective. However, for large water surfaces the cost of covering the surface with floats is prohibitive, Further in case of reservoirs with flood outlets, there is also the danger of floats being lost over spillway or through outlets. The floating covers are thus of limited utility to larger water bodies.

Reduction of Exposed Water Surface

In this method shallow portions of the reservoirs are isolated or curtailed by construction of dykes or bunds at suitable locations. Water accumulated during the monsoon season in such shallow portions IS diverted or pumped to appropriate deeper pockets in summer months, so that the shallow water surface area exposed to evaporation is effectively reduced.

Control of sedimentation in reservoirs

Sedimentation of a reservoir is a natural phenomenon and is a matter of vital concern for storage projects in meeting various demands, like irrigation, hydroelectric power, flood control, etc. Since it affects the useful capacity of the reservoir based on which projects are expected to be productive for a design period. Further, the deposited sediment adds to the forces on structures in dams, spillways, etc.

The rate of sedimentation will depend largely on the annual sediment load carried by the stream and the extent to which the same will be retained in the reservoir. This, in turn, depends upon a number of factors such as the area and nature of the catchment, level use pattern (cultivation practices, grazing, logging, construction activities and conservation practices), rainfall pattern, storage capacity, period of storage in relation to the sediment load of the stream, particle size distribution in the suspended sediment, channel hydraulics, location and size of sluices, outlet works, configuration of the reservoir, and the method and purpose of releases through the dam. Therefore, attention is required to each one of these factors for the efficient control of sedimentation of reservoirs with a view to enhancing their useful life and some of these methods are discussed in the Bureau of Indian Standard code IS: 6518-1992 “Code of practice for control of sediment in reservoirs”. In this section, these factors are briefly discussed.

There are different techniques of controlling sedimentation in reservoirs which may broadly be classified as follows:

- Adequate design of reservoir
- Control of sediment inflow
- Control of sediment deposition
- Removal of deposited sediment.

Each of these methods is briefly described as follows:

Design of reservoirs

The capacity of reservoirs is governed by a number of factors which are covered in IS: 5477 (Parts 1 to 4). From the point of view of sediment deposition, the following points may be given due consideration:

a) The sediment yield which depends on the topographical, geological and geo-morphological set up,meteorological factors, land use/land cover, intercepting tanks, etc;
b) Sediment delivery characteristics of the channel system;
c) The efficiency of the reservoir as sediment trap;
d) The ratio of capacity of reservoir to the inflow;
e) Configuration of reservoir;
f) Method of operation of reservoir;
g) Provisions for silt exclusion.

The rate of sediment delivery increases with the volume of discharge. The percentage of sediment trapped by a reservoir with a given drainage area increases with the capacity. In some cases an increased capacity will however, result in greater loss of water due to evaporation. However, with the progress of sedimentation, there is decrease of storage capacity which in turn lowers the trap efficiency of the reservoir.

The capacity of the reservoir and the size and characteristics of the reservoir and its drainage area are the most important factors governing the annual rate of accumulation of sediment. Periodical reservoir sedimentation surveys provide guidance on the rate of sedimentation. In the absence of observed data for the reservoir concerned, data from other reservoirs of similar capacity and catchment characteristics may be adopted.

Silting takes place not only in the dead storage but also in the live storage space in the reservoir. The practice for design of reservoir is to use the observed suspended sediment data available from key hydroIogica1 networks and also the data available from hydrographic surveys of other reservoirs in the same region. This data be used to simulate sedimentation status over a period of reservoir life as mentioned in IS 12182: 1987.

Control of sediment inflow

There are many methods for controlling sediment inflows and they can be divided as under:

a) Watershed management/soil conservation measures to check production and transport of sediment in the catchment area.

b) Preventive measures to check inflow of sediment into the reservoir. The

soil conservation measures are further sub-divided as:

a) Engineering,

b) Agronomy, and
c) Forestry.

The engineering methods include:

a) Use of check dams formed by building small barriers or dykes across stream channels.
b) Contour bounding and trenching;
c) Gully plugging;
d) Bank protection.

The agronomic measures include establishment of vegetative screen, contour farming, strip cropping and crop rotation.

Forestry measures include forest conservancy, control on grazing, lumbering, operations and forest fires along with management and protection of forest plantations.

Preventive measures to check inflow of sediment into the reservoir include construction of by-pass channels or conduits.

Check Dams

Check dams are helpful for the following reasons:

a) They help arrest degradation of stream bed thereby arresting the slope failure;
b) They reduce the veIocity of stream flow, thereby causing the deposition of the sediment load.

Check dams become necessary, where the channel gradients are steep and there is a heavy inflow of sediment from the watershed. They are constructed of local material like earth, rock, timber, etc. These are suitable for small catchment varying in size from 40 to 400 hectares. It is necessary to provide small check dams on the subsidary streams flowing into the main streams besides the check dams in the main stream. Proper consideration should be given to the number and location of check dams required. It is preferable to minimize the height of the check dams. If the stream ha, a very-steep slope, it is desirable to start with a smaller height for the check dams than may ultimately be necessary.

Check dams may generally cost more per unit of storage than the reservoirs they protect. Therefore, it may not always be possible to adopt them as a primary method of sediment control in new reservoirs. However, feasibility of providing check dams at a later date should not be overlooked while planning the protection of-a new reservoir.

Contour Bunding and Trenching

These are important methods of controlling soil erosion on the hills and sloping lands, where gradients of cultivated fields or terraces are flatter, say up to 10 percent. By these methods the hill side is split up into small compartments on which the rain is retained and surface run-off is modified with prevention of soil erosion. In addition to contour bunding, side trenching is also provided sometimes.

Gully Plugging

This is done by small rock fill dams. These dams will be effective in filling up the gullies with sediment coming from the upstream of the catchment and also prevent further widening of the gully.

Control of sediment deposition

The deposition of sediment in a reservoir may be controlled to a certain extent by designing and operating gates or other outlets in the dam in such a manner as to permit selective withdrawals of water having a higher than average sediment content. The suspended sediment content of the water in reservoirs is higher during and just after flood flow. Thus, more the water wasted at such times, the smaller will be the percentage of the total sediment load to settle into permanent deposits. There are generally two methods: (a) density currents, and (b) waste-water release, for controlling the deposition and both will necessarily result in loss of water.

Density Current

Water at various levels of a reservoir often contains radically different concentrations of suspended sediment particularly during and after flood flows and if all waste-water could be withdrawn at those levels where the concentration is highest, a significant amount of sediment might be removed from the reservoir. Because a submerged outlet draws water towards it from all directions, the vertical dimension of the opening should be small with respect to the thickness of the layer and the rate of withdrawal also should be low. With a view to passing the density current by sluices that might be existed, it is necessary to trace the movement of density currents and observation stations (consisting of permanently anchored rafts from which measurements could be made of temperature and conductivity gradient from the surface of the lake to the bottom, besides collecting water samples at various depths) at least one just above the dam and two or more additional stations in the upstream (one in the inlet and one in the middle) should be located.

Waste-Water Release

Controlling the sedimentation by controlling waste-water release is obviously possible only when water can be or should be wasted. This method is applicable only when a reservoir is of such size that a small part of large flood flows will fill it.

In the design of the dam, sediment may be passed through or over it as an effective method of silt control by placing a series of outlets at various elevations. The percentage of total sediment load that might be ejected from the reservoir through proper gate control will differ greatly with different locations. It is probable that as much as 20 percent of the sediment inflow could be passed through many reservoirs by venting through outlets designed and con-trolled.

Scouring Sluicing

This method is somewhat similar to both the control of waste-water release and the draining and flushing methods. The distinction amongst them care the following:

1) The waste-water release method ejects sediment laden flood flows through deep spillway gates or large under sluices at the rate of discharge that prevents sedimentation.

2) Drainage and flushing method involves the slow release of stored water from the reservoir through small gates or valves making use of normal or low flow to entrain and carry the sediment, and

3) Scouring sluicing depends for its efficiency on either the scouring action exerted by the sudden rush of impounded water under a high head through under sluices or on the scouring action of high flood discharge coming into the reservoir.

Scouring sluicing method can be used in the following:

a) Small power dams that depend to a great extent on pondage but not on storage;
b) Small irrigation reservoirs, where only a small fraction of the total annual flow can be stored;
c) Any reservoir in narrow channels, gorges, etc, where water wastage can be afforded; and
d) When the particular reservoir under treatment is a unit in an interconnected system so that the other reservoirs can supply the water needed.

Removal of deposited sediment

The most practical means of maintaining the storage capacity are those designed to prevent accumulation of permanent deposits as the removal operations are extremely expensive, unless the material removed is usable. Therefore, the redemption of lost storage by removal should be adopted as a last resort. The removal of sediment deposit implies in general, that the deposits are sufficiently compacted or consolidated to act as a solid and, therefore, are unable to flow along with the water. The removal of sediment deposits may be accomplished by a variety of mechanical and hydraulic or methods, such as excavation, dredging, siphoning, draining, flushing, flood sluicing, and sluicing aided by such measures as hydraulic or mechanical agitation or blasting of the sediment. The excavated sediments may be suitably disposed off so that, these do not find the way again in the reservoir.

Excavation

The method involves draining most of or all the water in the basin and removing the sediment by hand or power operated shovel, dragline scraper or other mechanical means. The excavation of silt and clay which constitute most of the material in larger reservoirs is more difficult than the excavation of sand and gravel. Fine-textured sediment cannot be excavated easily from larger reservoirs unless it is relatively fluid or relatively compact.

Dredging

This involves the removal of deposits from the bottom of a reservoir and their conveyance to some other point by mechanical or hydraulic means, while water storage is being maintained.

Dredging practices are grouped as:

a) Mechanical dredging by bucket, ladder, etc;

b) Suction dredging with floating pipeline and a pump usually mounted on a barrage; and

c) Siphon dredging with a floating pipe extending over the dam or connected to an opening in the dam and usually with a pump on a barrage.

Draining and Flushing

The method involves relatively slow release of all stored water in a reservoir through gates or valves located near bottom of the dam and the maintenance thereafter of open outlets for a shorter or longer period during which normal stream flow cuts into or directed against the sediment deposits. Therefore, this method may be adopted in flood control reservoirs.

Sluicing with Controlled Water

This method differs from the flood sluicing in that the controlled water supply permits choosing the time of sluicing more advantageously and that the water may be directed more effectively against the sediment deposits. While the flood sluicing depends either on the occurrence of flood or on being able to release rapidly all of a full or nearly full supply of water in the main reservoir is empty. The advantage of this method is that generally more sediment can be removed per unit of water used than in flood scouring or draining and flushing.

Sluicing with Hydraulics and Mechanical Agitation

Methods that stir up, break up or move deposits of a sediment into a stream current moving through a drained reservoir basin or into a full reservoir will tend to make the removal of sediment from the reservoir more complete. Wherever draining, flushing or sluicing appear to be warranted, the additional use of hydraulic means for stirring up the sediment deposits, or sloughing them off, into a stream flowing through the reservoir basin should be considered. It has, however, limited application.

Reservoir operation

The flow in the river changes seasonally and from year to year, due to temporal and spatial variation in precipitation. Thus, the water available abundantly during monsoon season becomes scarce during the non-monsoon season, when it is most needed. The traditional method followed commonly for meeting the needs of water during the scarce period is construction of storage reservoir on the river course. The excess water during the monsoon season is stored in such reservoirs for eventual use in lean period. Construction of storages will also help in control of flood, as well as generation of electricity power. To meet the objective set forth in planning a reservoir or a group of reservoirs and to achieve maximum benefits out of the storage created, it is imperative to evolve guidelines for operation of reservoirs. Without proper regulation schedules, the reservoir may not meet the full objective for which it was planned and may also pose danger to the structure itself.

Control of flood is better achieved if the reservoir level is kept low in the early stages of the monsoon season. However, at a later stage, if the anticipated inflows do not result the reservoir may not get filled up to FRL in the early stages of monsoon, to avoid the risk of reservoir remaining unfilled at later stage, there may be problem of accommodating high floods occurring at later stage. In some cases while planning reservoirs, social and other considerations occasionally result in adoption of a plan that may not be economically the best.

Operation of Single Purpose Reservoirs

The common principles of single purpose reservoir operation are given below:

a) ***Flood control-*** Operation of flood control reservoirs is primarily governed by the available flood storage capacity of damage centers to be protected, flood characteristics, ability and accuracy of flood/ storm forecast and size of the uncontrolled drainage area. A regulation plan to cover all the complicated situations may be difficult to evolve, but generally it should be possible according to one of the following principles:

1) *Effective use of available flood control storage:* Operation under this principle aims at reducing flood damages of the locations to be protected to the maximum extent possible, by effective use of flood event. Since the release under this plan would obviously be lower than those required for controlling the reservoir design flood, there is distinct possibility of having a portion of the flood control space occupied during the occurrence of a subsequent heavy flood. In order to reduce this element of risk, maintenance of an adequate network of flood forecasting stations both in the upstream and down stream areas would be absolutely necessary.

2) *Control of reservoir design flood:* According to this principle, releases from flood control reservoirs operated on this concept are made on the same hypothesis as adopted for controlling the reservoir design flood, that is the full storage capacity would be utilized only when the flood develops into the reservoir design flood. However, as the design flood is usually an extreme event, regulation of minor and major floods, which occur more often, is less satisfactory when this method is applied.

3) *Combination of principle (1) and (2):* In this method, a combination of the principles (1) and (2) is followed. The principle (1) is followed for the lower portion of the flood reserve to achieve the maximum benefits by controlling the earlier part of the flood. Thereafter releases are made as scheduled for the reservoir design flood as in principle (2). In most cases this plan will result in the best overall regulation, as it combines the good points of both the methods.

4) *Flood control in emergencies:* It is advisable to prepare an emergency release schedule that uses information on reservoir data immediately available to the operator. Such schedule should be available with the operator to enable him to comply with necessary precautions under extreme flood conditions.

b) ***Conservation:*** Reservoirs meant for augmentation of supplies during lean period should usually be operated to fill as early as possible during filling period, while meeting the requirements. All water in excess of the requirements of the filling period shall be impounded. No spilling of water over the spillway will normally be permitted until the FRL is reached. Should any flood occur when the reservoir is at or near the FRL, release of flood waters should be affected, so as not to exceed the discharge that would have occurred had there been no reservoir. In case the year happens to be dry, the draft for filling period should be curtailed by applying suitable factors. The depletion period should begin thereafter. However, in case the reservoir is planned with carry- over capacity, it is necessary to ensure that the regulation will provide the required carry-over capacity at the end of the depletion period.

Operation of multi purpose reservoirs: The general principles of operation of reservoirs with these multiple storage spaces are described below:

1. Separate allocation of capacities- When separate allocations of capacity have been made for each of the conservational uses, in addition to that required for flood control, operation for each of the function shall follow the principles of respective functions. The storage available for flood control could, however be utilized for generation of secondary power to the extent possible. Allocation of specific storage space to several purposes with the conservation zone may some times be impossible or very costly to provide water for the various purposes in the quantities needed and at the time they are needed.

2. Joint use of storage spac*e*- In multi-purpose reservoir where joint use of some of the storage space or storage water has been envisaged, operation becomes complicated due to competing and conflicting demands. While flood control requires low reservoir level, conservation interests require as high a level as is attainable. Thus, the objectives of these functions are not compatible and a compromise will have to be effected in flood control operations by sacrificing the requirements of these functions. In some cases parts of the conservational storage space is utilized for flood moderation, during the earlier stages of the monsoon. This space has to be filled up for conservation purpose towards the end of monsoon progressively, as it might not be possible to fill up this space during the post-monsoon periods, when the flows are insufficient even to meet the current requirements. This will naturally involve some sacrifice of the flood control interests towards the end of the monsoon.

Operation of system of reservoirs

It is not very uncommon to find a group or 'system' of reservoirs either in a single river or in a river and its tributaries. An example of the former are the dams proposed on the river Narmada (Figure 7) and an example of the latter are the dams of the Damodar Valley project (Figure 8).

In case of system of reservoirs, it is necessary to adopt a strategy for integrated operated of reservoirs to achieve optimum utilization of the water resources available and to benefit the best out of the reservoir system.

In the preparation of regulation plans for an integrated operation of system of reservoirs, principles applicable to separate units are first applied to the individual reservoirs. Modifications of schedule so developed should then be considered by working out several alternative plans. In these studies optimization and simulation techniques may be extensively used with the application of computers in water resources development.

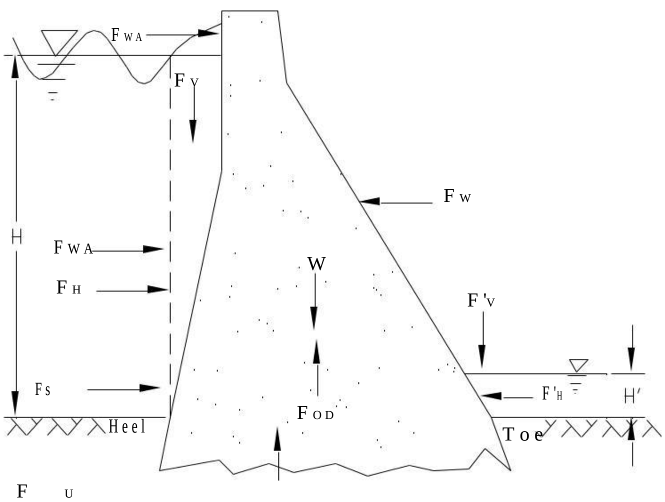
UNIT III
Concrete Gravity Dam

3.1 General

Basically, gravity dams are solid concrete structures that maintain their stability against design loads from the geometric shape and the mass and strength of the concrete. Generally, they are constructed on a straight axis, but may be slightly curved or angled to accommodate the specific site conditions. Gravity dams typically consist of a non-overflow section(s) and an overflow section or spillway.

3.2 Forces acting on gravity dams

The structural integrity of a dam must be maintained across the range of circumstances or events likely to arise in service. The design is therefore determined through consideration of the corresponding spectrum of loading conditions. In all foreseeable circumstances the stability of the dam and foundation must be ensured, with stresses contained at acceptable levels and watertight integrity essentially unimpaired.



Where:

H = Head water depth H' = Tail Water depth F_{WA} = Wave pressure force

F_H = Horizontal hydrostatic force

F_S = Silt/sediment pressure force

F_{EQ} = Earthquake/Seismic force

F_W = Wind pressure force

F_H' = Tail water hydrostatic force

W = Weight of dam

F_{OD} = Internal pore water pressure

F_U = Uplift pressure force [base of dam]

F_V = Weight of water above dam [u/s]

F_V' = Weight of water above dam [d/s]

Figure 3-1 Representation of typical loads acting on Gravity dam

3.2.1 Water pressure

Water pressure is the force exerted by the water stored in the reservoir on the upstream and the water depth at the tail of the dam.

i. External water pressure load

External water pressure can be calculated by the law of hydrostatics according to which in a static mass of liquid the pressure intensity varies linearly with the depth of liquid and it acts normal to the surface in contact with the liquid. For the non-overflow section of the dam water pressure may be calculated as follows and for the overflow portion the loading will be discussed in section six of the course.

F_H = horizontal component of hydrostatic force, acting along a line 1/3 H above the base = ½ $\gamma_w H^2$

γ_w = Unit weight of water (=10 kN/m^3)

Fv = Vertical component of hydrostatic pressure

= Weight of fluid mass vertically above the upstream face acting through the center of gravity of the mass.

ii. Internal water pressure (Uplift Pressure)

Internal water pressure is the force exerted by water penetrating through the pores, cracks and seams with in the body of the dam, at contact surface between the dam and its foundation, and with in the foundation. It acts vertically upward at any horizontal section of the dam as well as its foundation and hence it causes a reduction in the effective weight of the portion of the structure lying above this section.

The computation of internal pressure involves the consideration of two constituent elements, i.e,

- Hydrostatic pressure of water at a point
- The percentage C, area factor, of the area on which the hydrostatic pressure acts Both these elements are discussed below.

Hydrostatic pressure

In practice dams are usually provided with cut-off walls or grout curtains to reduce seepage and drain to relieve pressure downstream from the cutoff. Actually cutoff and grout curtains may not be perfectly tight and hence fail to dissipate the head ($h_1 – h_2$)

Usually a distribution like 1-2-3-4 is used with 3-4 a straight line as shown in Figure 3-3.
Opinions about the value of uplift reduction factor, □ (Zeta), are varied, the tendency is to take:

- □ = 0.85 (for normal loading cases)
- □ = 1.00 (for exceptional loading cases like earthquake)

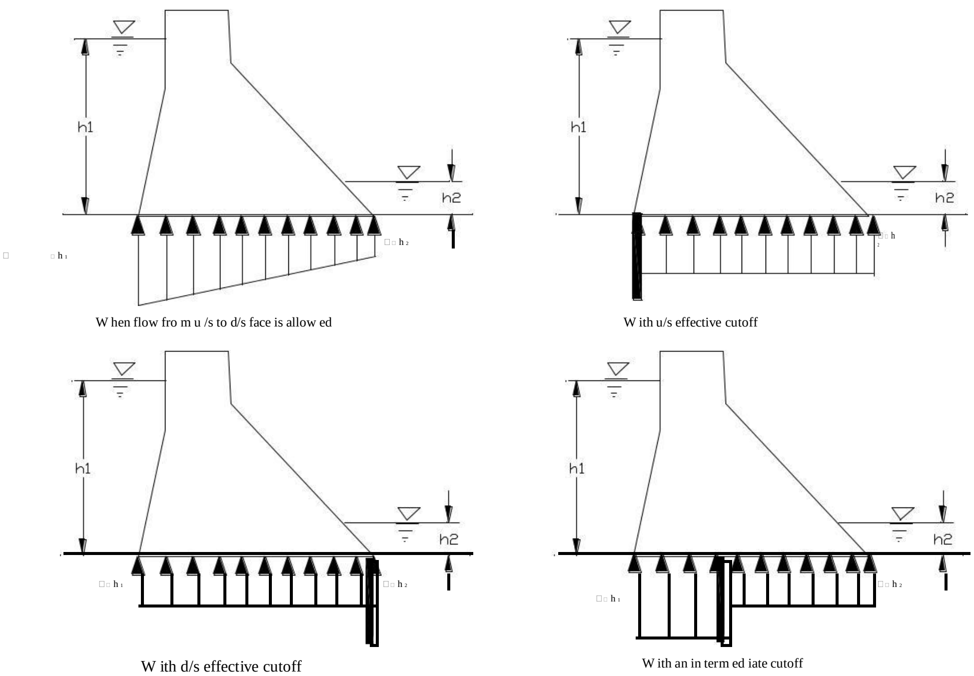


Figure 3-2 Uplift pressure distribution for perfectly tight cutoff walls.

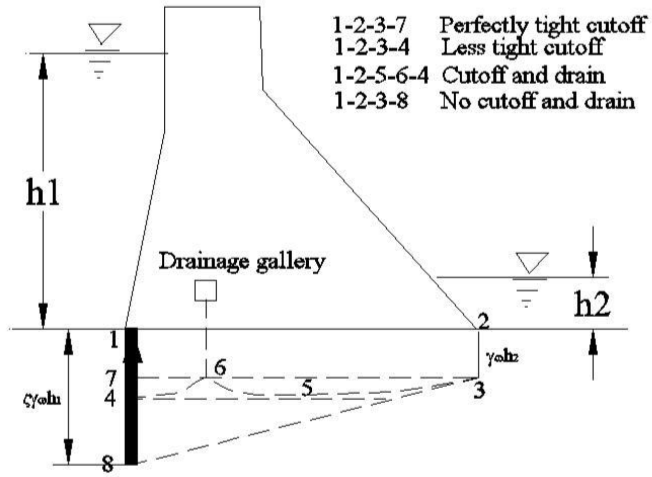


Figure 3-3. Uplift pressure distribution for less tight cutoff.

Uplift area factor, C

The value of area factor for concrete has been determined experimentally by several investigators. However, for the foundation rock the value of area factor is not determinable experimentally and hence the same has been estimated on the basis of theoretical considerations.

Some of the earliest investigators recommended, for both concrete and rock, a value of area factor ranging from one third to two-thirds of the area to be considered as effective area over which the uplift pressure acts. However, Harza, Terzaghi and Lelivakey have indicated that, for both concrete and rock, the value of area factor is nearly equal to unity.

Table 3-1Values suggested for uplift area factor are

Value of C	Suggested by
0.25 to 0.40	Henry
1.00	Maurice Levy
0.95 to 1.00	Terzaghi

As such the present practice followed in the design of dams is that the uplift pressure is assumed to act over 100 percent of the area with in the body of the dam as well as its foundation. Hence, under all conditions, the value C = 1.00 is recommended.

3.2.2 Wight of Structure

For a gravity dam the weight of the structure is the main stabilizing force, and hence the construction material should be as heavy as possible.

Structure self weight is accounted for in terms of the resultant, W, which acts through the centroid (center of gravity) of the cress-sectional area. The weight of the structure per unit length is

W = □c * A

Where: □c is the unit weight of concrete

A is the cross-sectional area of the structure

The unit weight of concrete may be assumed to be 24 kN/m^3 in the absence specific data from laboratory test trials. For final designs the specific weights shall be based on actual test data. Where crest gates and other ancillary structures or equipments of significant weigh are present they must also be accounted for in determining the weight of the structure.

It is essential to make sure that the actual specific weight obtained for the construction material is more than or at least equal to that assumed in the design.

3.2.3 Earth and silt pressure

The gradual accumulation of significant deposits of fine sediment, notably silt, against the face of the dam generates a resultant horizontal force, F_s. The magnitude of this force in additional to water load, F_{WH}, is a function of the sediment depth, h_s, the submerged unit weight, γ_{ss}, and the active pressure coefficient, K_a, and is determined according to Rankine"s formula.

$$Fs = ½ K_a \gamma_{ss} h_s^2$$

Where $Ka = (1-\sin\phi) / (1+\sin\phi)$

ϕ = angle of internal friction of material.

3.2.4 Wind pressure

When the dam is full, wind will act only on the downstream face, thus contributing to stability. When the dam is empty, wind can act on the upstream face, but the pressure is small compared to the hydraulic pressure of the water. Hence for gravity dams wind is not considered. For buttress dams, wind load on the exposed buttresses has to be considered.

3.2.5 Wave pressure and wave height

Wave exerts pressure on the upstream face. This pressure force, F_{wv} depends on fetch (extent of the water surface on which the water blows) and wind velocity. It is of relatively small magnitude and, by its nature, random and local in its influence. An empirical allowance for wave load may be made by adjusting the static reservoir level used in determining F_{WV}. According to Molitor the following formula could be used to determine the rise in water level, h_w

$$h_w = 0.763 + 0.032\sqrt{vf} - 0.271 f^{1/4} \quad \text{for } f < 32\ km$$

$$h_w = 0.032\sqrt{vf} \quad \text{for } f > 32\ km$$

$$F_{wz} = 2.0\gamma_w h_w^2$$

where: h_w in meters
v wind velocity in km/hr and
f fetch in km

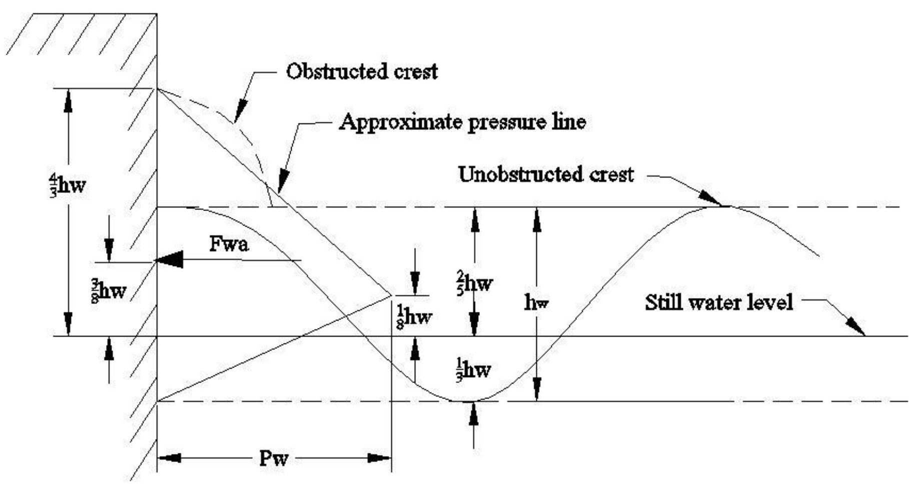


Figure 3-4 Wave configuration and wave pressure on a gravity dam

For high dams the wave pressure is small compared to other forces.

The point of application of F_{wv} can be taken as 3/8hw from the still water level.

The wave rides up higher on inclined dam faces as compared to the vertical one.

3.2.6 Earthquake forces

Dynamic loads generated by seismic disturbances must be considered in the design of all major dams situated in recognized seismic “high risk” regions. The possibility of seismic activity should also be considered for dams located outside those regions, particularly where sites in close proximity to potentially active geological fault complexes.

Seismic activity is associated with complex oscillating patterns of accelerations and ground motions, which generated transient dynamic loads due to the inertia of the dam and the retained body of water. For design purposes both should be considered operative in the sense least favorable to stability of the dam. Horizontal accelerations are therefore assumed to operate normal to the axis of the dam. Under reservoir full conditions the most adverse seismic loading will then occur when a ground shock is associated with:

1. horizontal foundation acceleration operating upstream, and
2. vertical foundation acceleration operating downward

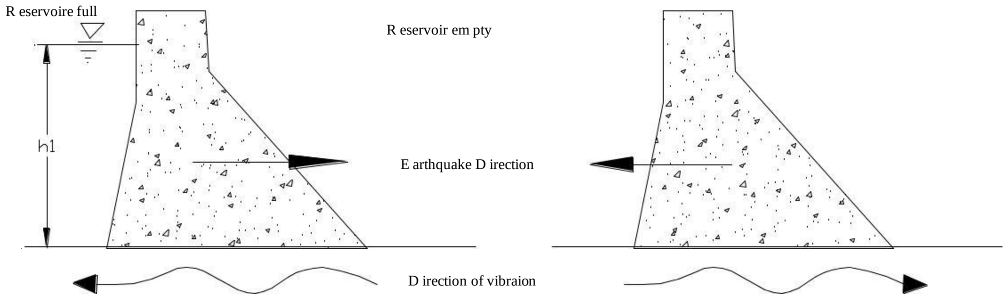


Figure 3-5 Direction of ground acceleration and the respective horizontal earthquake force on gravity dam

As a result of 1, inertia effects will generate an additional hydrodynamic water load acting downstream, plus a further inertia load attributable to the mass of the dam and also acting in a downstream sense. Foundation acceleration downwards, 2 above, will effectively reduce the mass of the structure. The more important recurring seismic shock waves have a frequency in the range 1-10Hz. Seismic loads consequently oscillate very rapidly and are transient in their effect. The strength of seismic event can be characterized by its magnitude and its intensity.

Ground motions associated with earthquakes can be characterized in terms of acceleration, velocity or displacement. Only peak ground acceleration, *pga*, generally expresses as a portion of gravitational acceleration, g, is considered in this course. It has been suggested that in general seismic events with a high *pga* of short duration are less destructive than seismic events of lower pga and greater duration.

The natural frequency of vibration, f_n, for a triangular gravity profile of height H (m) and base thickness B(m) constructed in concrete with an effective modulus of elasticity E=14GPa can be approximated as

$$f_n = 600\ B/H^2 \quad \text{(Hz)}$$

For a dam of H = 500m and B = 375m, f_n = 0.9 Hz. But the most important recurring seismic shock waves are in the order of magnitude of 1-10Hz. Hence resonance (the frequency of vibration of the structure and earthquake are equal) of an entire dam is unlikely and is not a series concern in design. But vulnerable portion of the dam should be detailed.

There are two methods to determine the seismic load on a dam

Pseudostaic (equivalent static load) method: inertia forces are calculated based on the acceleration maxima selected for design and considered as equivalent to additional static loads. This method generally is conservative and is applied to small and less vulnerable dams.

The acceleration intensities are expressed by acceleration coefficients α_h (Horizontal) and α_v(vertical) each representing the ratio of peak ground acceleration. Horizontal and vertical accelerations are not equal, the former being of greater intensity ($\alpha_h = (1.5 - 2.0\alpha_v)$).

Based on the vertical and horizontal acceleration, the inertial force will be

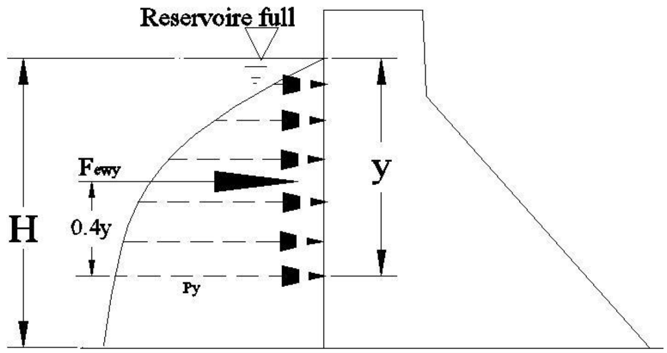
Horizontal force = $\pm\alpha_h$ * (static mass)

Vertical force = $\pm\alpha_v$ * (static mass)

Three loading cases can be used for the assessment of seismic load combination:

7.1 Peak horizontal ground acceleration with zero vertical ground acceleration
7.2 Peak vertical ground acceleration with zero horizontal acceleration
7.3 Appropriate combination of both (eg. Peak of the horizontal and 40-50% of the vertical)

Inertia forces



1. Mass of dam
   Horizontal $F_{eqh} = \pm\alpha_h W$
   Vertical $F_{eqv} = \pm\alpha_v W$
2. Water body

As analyzed by Westerguard(1993)

$$P_y = k''\alpha_h \gamma_w \sqrt{H.y}$$

$$F_{ewy} = \frac{2}{3}\alpha_h \gamma_w k'' y \sqrt{H.y}$$

where k” = earthquake factor for the water body

$$k'' = \frac{0.816}{\sqrt{1 - 7.75\left(\frac{H}{1000\,T}\right)^2}}$$

Where: T = period of earthquake
γ_w = in tone/m^3
H, y in meters

The force acts at 0.4y from the dam joint being considered.
For inclined upstream face of dam

$$P_y = k''\alpha_h \gamma_w \sqrt{H.y}\cos\theta$$

where θ is the angle the face makes with the vertical.

The resultant vertical hydrodynamic load, F_{ewv}, effective above an upstream face batter or flare may be accounted for by application of the appropriate seismic coefficient to vertical water load. It is considered to act through the centroid of the area.

$$F_{ewv} = \pm\alpha_v F_v$$

Uplift load is normally assumed to be unaltered by seismic shock.

Dynamic analysis: the dam is idealized as a two dimensional plane-strain or plane-stress finite element system, the reservoir being regarded as a continuum. The foundation zone is generally idealized as a finite element system equivalent to a visco-elastic half space. The complexities of such an approach are evident, and take it outside the scope of this course.

3.3 Load combination for Design

The design of a gravity dam is based on the most adverse combination of the loads/forces acting on it, which includes only those loads having a reasonable probability of simultaneous occurrence. The combination of transient loads such as those due to maximum flood and earthquake are not considered because the probability of occurrence of each of these phenomena is quite low and hence the probability of their simultaneous occurrence is almost negligible. Thus for the design of gravity dams according to Indian Standard is specified as the following load combination:

I. *Load combination A (construction condition or empty reservoir condition):* Dam completed but no water in the reservoir and no tail water.

II. *Load combination B (Normal operating condition):* Full reservoir elevation (or top of gates at crest), normal dry weather tail water, normal uplift, ice and uplift (if applicable)

III. *Load combination C (Flood Discharge condition):* Reservoir at maximum flood pool elevation, all gates open, tail water at flood elevation, normal uplift, and silt (if applicable)

IV. *Load combination D* - Combination A, with earthquake.

V. *Load combination E* - Combination A, with earthquake but no ice

VI. *Load Combination F* - Combination C, but with extreme uplift (drain inoperative)

VII. *Load Combination G* - Combination E, but with extreme uplift (drain inoperative)

3.4 Reaction of the foundation

The foundation should provide the required reaction to the resultant force for the dam to be stable.

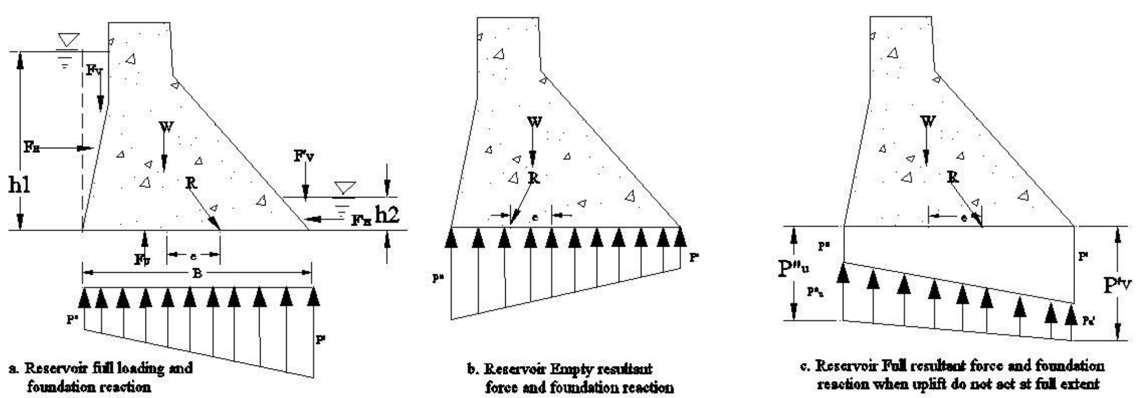


Figure 3-6 Foundation reaction for reservoir full and empty loading

For full reservoir

$$P', P'' = \frac{\Sigma W}{A} \pm \frac{Mc}{I} = \frac{\Sigma W}{B} \pm \frac{6\,\Sigma W . e}{B^2} = \frac{\Sigma W}{B}\left(1 \pm \frac{6e}{B}\right)$$

For empty reservoir

$$P', P'' = \frac{\Sigma W}{B}\left(1 \pm \frac{6e}{B}\right)$$

where: B = Width of the base of the dam section

e = eccentricity

ΣW = Sum of vertical forces including uplift

$$e = \frac{\Sigma M_{VCG} \pm \Sigma M_{HCG}}{\Sigma W}$$

Requirements for stability

A masonry of plain concrete dam must be free from tensile stress, i.e. neither *P'* nor *P''* shall be negative, or

$e \leq B/6$ (law of the middle third)

To limit compressive stress with in the dam body use:

P', P'' if uplift always acts to the fullest extent.

P'_V, P''_V if uplift does not act always.

Horizontal forces must be resisted both by shear and friction in the dam joint or in the foundation.

3.5 Rules Governing the Design of Gravity Dams

The following are basic assumptions that should be considered relative to the design of important masonry/concrete dams.

1. The rock that constitutes the foundation and abutments at the site is strong enough to carry the forces imposed by the dam with stresses well below the elastic limit at all places along the contact planes.
2. The bearing power of the geologic structure along the foundation and abutments is great enough to carry the total loads imposed by the dam without rock movements of detrimental magnitude.
3. The rock formations are homogeneous and uniformly elastic in all directions, so that their deformations may be predicted satisfactorily by calculations based on the theory of elasticity, by laboratory measurements on models constructed of elastic materials, or by combinations of both methods.
4. The flow of the foundation rock under the sustained loads that result from the construction of the dam and the filling of the reservoir may be adequately allowed for by using a somewhat lower modulus of elasticity than would otherwise be adopted for use in the technical analyses.
5. The base of the dam is thoroughly keyed into the rock formations along the foundations and abutments.
6. Construction operations are conducted so as to secure a satisfactory bond between the concrete and rock materials at all areas of contact along the foundation and abutments.
7. The concrete in the dam is homogeneous in all parts of the structure.
8. The concrete is uniformly elastic in all parts of the structure, so that deformations due to applied loads may be calculated by formulae derived on the basis of the theory of

elasticity or may be estimated from laboratory measurements on models constructed of elastic materials.

9. Effects of flow of concrete may be adequately allowed for by using a somewhat lower modulus of elasticity under sustained loads than would otherwise be adopted for use in technical analyses.
10. Contraction joints are properly grouted under adequate pressures, or open slots are properly filled with concrete, so that the dam may be considered to act as a monolith.
11. Sufficient drains are installed in the dam to reduce such uplift pressures as may develop along areas of contact between the concrete and rock materials.
12. Effects of increases in horizontal pressures caused by silt contents of flood waters usually may be ignored in designing high storage dams, but may require consideration in designing relatively low diversion structures.
13. Uplift forces adequate for analyzing conditions at the base of the dam are adequate for analyzing conditions at horizontal concrete cross sections above the base.
14. Internal stresses caused by natural shrinkage and by artificial cooling operations may be adequately controlled by proper spacing of contraction joints.
15. Internal stresses caused by increases in concrete temperature after grouting are beneficial.
16. Maximum pressures used in contraction joint grouting operations should be limited to such values as may be shown to the safe by appropriate stress analyses.
17. No section of the Ethiopia may be assumed to be entirely free from the occurrence of earthquake shocks.
18. Assumptions of maximum earthquake accelerations equal to one tenth of gravity are adequate for the design of important masonry dams without including additional allowances for resonance effects.
19. Vertical as well as horizontal accelerations should be considered, especially in designing gravity dams.
20. During the occurrence of temporary abnormal loads, such as those produced by earthquake shocks, some increases in stress magnitudes and some encroachments on usual factors of safety are permissible.
21. Effects of foundation and abutment deformations should be included in the technical analyses.
22. In monolithic straight gravity dams, some proportions of the loads may be carried by twist action and beam action at locations along the sloping abutments, as well as by the more usually considered gravity action.
23. Detrimental effects of twist and beam action in straight gravity dams, such as cracking caused by the development of tension stresses, may be prevented by suitable construction procedure.
24. In monolithic curved gravity and arch dams, some proportions of the loads may be carried by tangential shear and twist effects, as well as by the more usually considered arch and cantilever actions.
25. The distribution of loads in masonry dams may be determined by bringing the calculated deflections of the different systems of load transference into agreement at all conjugate points in the structure.

The aforementioned assumptions are rephrased as rule/guideline for design of concrete gravity dam as described below:

Rule1: Location of the resultant: No tension in any joint of the dam under all loading conditions (i.e. for full and empty reservoir). Thus, resultant of all forces (including uplift) must intersect the joint within the middle third.

Rule2a: Resistance to sliding when shear is neglected: the tangent of the angle between the vertical and the resultant (including uplift) above horizontal plane shall be less than the allowable coefficient of frictional force „f‟. If empirical values are taken, factor of safety, $S_f = 2$.

Table 3-2 Some values of Coefficient of friction *f*

Surface	***f***
Masonry on masonry or masonry on good rock or concrete on concrete	0.75
Concrete or masonry on gravel	0.50
Concrete or masonry on sand	0.40
Concrete or masonry on clay	0.30

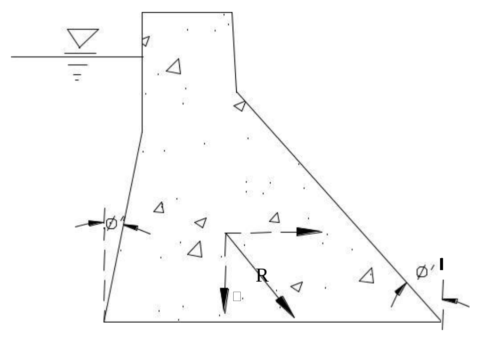
However, the value of *f* for specific cases should be obtained by test

For foundation on earth, $\frac{\Sigma P}{\Sigma W} = \tan\theta \leq \frac{f}{S_f}$ S_f is taken as 3

Rule 2b: Resistance to sliding when shear is considered

The total friction resistance to sliding on any joint plus the ultimate shearing strength of the joint, must exceed the total horizontal force above the joint by a safe margin, i.e.

$$\Sigma P \leq \frac{f\Sigma W + r.S_n.A}{S_{sf}}$$



Where: S_n – ultimate shearing strength of material

S_{sf} = shear friction factor of safety

A = cross sectional area of joints

r = ratio of average to maximum shearing strength

Recommended values $S_{sf} = 5$, $r = 0.5$

$rS_{sf} = 200$ to $500 t/m^2$

While analyzing resistance to sliding, first compute $\tan\theta$ and if $\tan\theta > f$ apply Rule 2b. In that case, S_{sf} should equal or exceed the allowable value.

Rule 3: Governing compressive stresses: P'_v, or P''_v (maximum vertical stresses) are not the maximum stresses in the structure. The maximum stresses occur at the end joints, or inclined planes, normal to the face of the dam.

Maximum stress for downstream face, reservoir full:

$$P_i{}' = P_v{}'(1 + \tan^2\phi')$$

Maximum stress for upstream face, reservoir full

$$P_i{}'' = P_v{}''(1 + \tan^2\phi'')$$

The inclined compressive stresses in the dam and foundation shall not exceed the allowable values.

Ultimate stress, $\square''_c$ = 14 to 31 MPa (after 28 days curing)

Working stress $\square_c = \square''_c/6$

For foundation materials some indications for allowable stress are:

Limestone -------------200 to 350 t/m3

Granite -------------250 to 300t/m3

Rule 4: Governing internal tension: The dam shall be designed and constructed in such a manner as to avoid or adequately provide for tension on interior planes, inclined, vertical or horizontal.

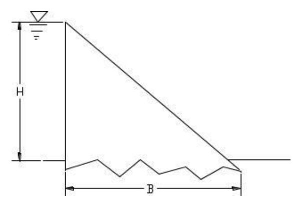
Rule 5: Margin of safety: all assumptions of forces acting on the dam shall be unquestionably on the safe side, all unit stresses adopted in design should provide an ample margin of safety against rupture and the shear-factors shall be considered.

Rule 6: Detail of design and methods of construction: all details shall support and confirm to the assumptions used in design; masonry should be of quality suited to the stresses adapted, protection against overflowing water shall be ample.

3.6 Theoretical versus practical section of a dam

Considering only the two major forces acting on the dam, i.e. the weight of the dam and the hydrostatic water pressure, the required section of the dam for its stability will be a triangle of base width,

$$B \;\square\; \frac{H}{\;}$$



Where: H = depth of water

s = specific gravity of concrete

For this section, the resultant will pass through the upper middle third point of the base when reservoir is empty and through the lower middle third point when the reservoir is full.

Practical section:

i. The pointed crest of the theoretical dam is unstable to resist shock due to floating objects.
ii. There is need for a free board
iii. There is also need for top width for a roadway

For practical section

i. Crest of the dam shall be a certain thickness depending on the height of the dam. For non-overflow dams, most economical crest width □ 14 % of the height (10 – 15 %) is normal.
ii. Free board is provided and usually 3-4% of the dam height is used as a maximum height of the free board.

3.7 Design procedure of gravity dams

3.7.1 Design methods

The various methods used for the design of concrete gravity dams are as follows:

1. Stability analysis method
    a. Gravity method.
    b. Trial load twist method

i. Joints keyed but not grouted

ii. Joints keyed and grouted

c. Experimental method

i. Direct method

ii. Indirect method

d. Slab analogy method

e. Lattice analogy method

f. Finite element method

2. Zoned (multiple-step) method of determining profile of dam
3. Single step method

Two procedures of design will be discussed in this course: – multiple-step method and single-step method.

3.7.2 Multiple step method of determining profile of gravity dam

This method deals with designing the dam joint by joint (block by block) beginning at the top and making each joint confirm to all gravity dam design requirements. The procedure results in a dam with polygonal face that may be smoothened up for appearance with no appreciable change in stability or economy. The multiple-step method is almost always used for the final design of dams with a height that does not encroach greatly on Zone V.

Zoning of high non-overflow dams

A high gravity dam may be divided into seven zones according to design and stability requirements. The characteristics and limits of these zones are described below.

Zone I: is a rectangular section from the top of the dam to the water surface. The resultant force passes through the mid-point of the base.

Zone II: is also a rectangular section and extends to a depth where the resultant in the reservoir full condition reaches the outer middle third point of the base.

Zone III: upstream face of the dam is vertical but the downstream face is gradually inclined so that the resultant in the reservoir full condition has exactly at the outer middle third point of the base. This zone extends to a depth where the resultant in the reservoir empty condition reaches the inner middle third point of the base.

Zone IV: in this zone both the upstream and downstream faces are inclined so that the resultant both in the reservoir full and empty conditions lie at the middle third point. The zone extends to a point where maximum permissible compressive stress is reached at the *toe* of the dam.

Zone V: the slope of the downstream face is further increased to keep the principal stresses within permissible limits. Resultant in the reservoir full condition is kept well within the middle third section. The resultant in the reservoir empty condition follows the upper middle third section. This zone extends to a depth where the stress at the *heel* of the section reaches the permissible limits in the reservoir empty case.

Zone VI: the slope of the upstream face is rapidly increased so as to keep the principal stress at the heel with in the permissible limits in the reservoir empty condition. The inclination of the downstream face should also be adjusted so that the principal stress at the toe does not exceed the maximum allowable stress. The resultants in both reservoir empty and full conditions lie *within* the middle third section. This zone extends to a point where the slope of the *downstream* face reaches 1:1. This normally happens when the dam is 80 to 90 meters high.

Zone VII: in this zone the inclination of both upstream and downstream faces increase with the height of the dam. Consequently, at some plane the value of $(1 + tan^2 \square)$ may become so great that the principal stress at the downstream face may exceed the allowable limit. If one reaches this zone during design, it is better to avoid it and start again with a fresh design with increased crest width and/or better quality concrete.

Zoning of overflow dams (Spillways)

Zone I: the resultant in the reservoir full condition is outside the middle third point both horizontal and vertical forces are existing. End of zone I is at a depth where resultant intersects downstream middle third point. Upstream face needs reinforcement to take tension.

Zone Ia: this is the zone below zone1. The end of zone Ia is established by the plane where only friction is sufficient to resist sliding.

Zone II: similar to zone II of non overflow dam with the only difference that the downstream face is inclined in overflow dams. The rest of the zones are similar to those of non-overflow dams.

3.7.3 Single Step Method

This method considers the whole dam as a single block. It is used for final design of very high dams that extend well beyond zone V. it can also be used with an accuracy of 2 to 4% on the safe side; for preliminary designs to obtain the area of the maximum section of the dam.

The dam designed by single step method has a straight downstream face. When extended it intersects upstream face at the headwater surface.

Consider the sketch given:

$L = 10\text{-}15\%$ of h_1

$H_{10} = 2L$ (when earthquake is considered)

$= 3L$ (when earthquake is not considered)

$H_6 = 1.33L$

When designing (analyzing) a dam in the single step method, the dam is considered as a single block; and dam dimensions are determined in such a way that rules of Zone IV are satisfied.

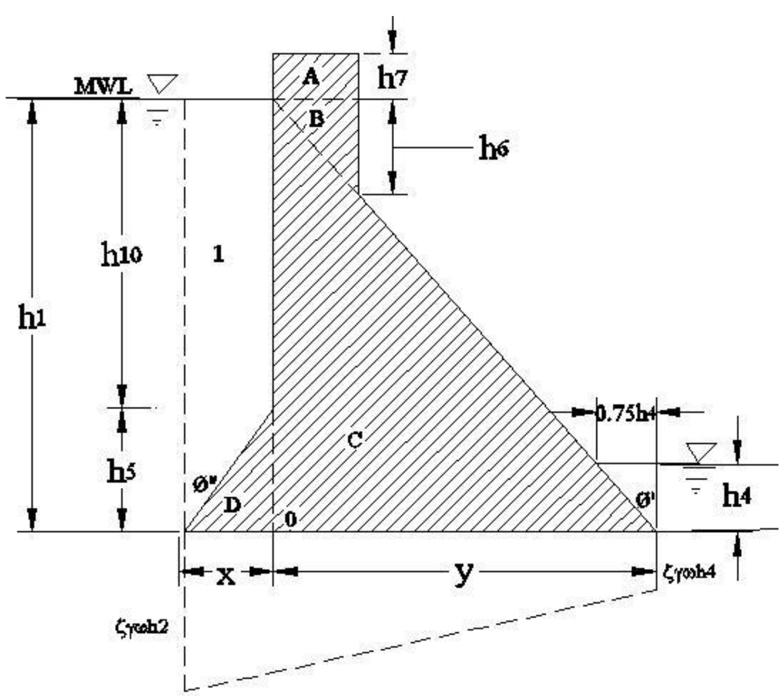


Figure 3-7 Gravity dam section relationship in single step method of design

Comparison of Single step and Multiple step design of gravity dam

- Dams of smaller height can be designed economically by Multiple step method
- High dams beyond zone IV are designed by Single step method so that convex curvature of downstream face and excessive flat slope of upstream face are avoided
- It may be economical to increase the concrete strength through the use of more expensive material, so that even a high dam designed by dividing it into only four zones, thus eliminating zone V and VI.

UNIT III
EARTH DAMS & SPILLWAYS

Instructional objectives

On completion of this lesson, the student shall learn:

1. The functions of a spillways and energy dissipators in projects involving diversion and storage projects
2. Different types of spillways
3. How to determine the shape of an ogee-crested spillway and compute its discharge
4. The spillway profile in the presence of a breast wall
5. Criteria for selecting a particular type of spillway
6. Different types of energy dissipators
7. Design procedure for hydraulic jump and bucket-type energy dissipators
8. Protection measures against science downstream of energy dissipators

Introduction

The previous lessons dealt with storage reservoirs built by impounding a river with a dam and the common types of dams constructed by engineers. However, in rare cases only it is economical or practical for the reservoir to store the entire volume of the design flood within the reservoir without overtopping of dam. Hence, a dam may be constructed to that height which is permissible within the given topography of the location or limited by the expenditure that may be possible for investment. The excess flood water, therefore, has to be removed from the reservoir before it overtops the dam. Passages constructed either within a dam or in the periphery of the reservoir to safely pass this excess of the river during flood flows are called Spillways.

Ordinarily, the excess water is drawn from the top of the reservoir created by the dam and conveyed through an artificially created waterway back to the river. In some cases, the water may be diverted to an adjacent river valley. In addition to providing sufficient capacity, the spillway must be hydraulically adequate and structurally safe and must be located in such a way that the out-falling discharges back into the river do not erode or undermine the downstream toe of the dam. The surface of the spillway should also be such that it is able to withstand erosion or scouring due to the very high velocities generated during the passage of a flood through the spillway.

The flood water discharging through the spillway has to flow down from a higher elevation at the reservoir surface level to a lower elevation at the natural river level on the downstream through a passage, which is also considered a part of the spillway. At the bottom of the channel, where the water rushes out to meet the natural river, is usually provided with an energy dissipation device that kills most of the energy of the flowing water. These devices, commonly called as Energy Dissipators, are required to prevent the river surface from getting dangerously scoured by the impact of the outfalling water. In some cases, the water from the spillway may be allowed to drop over a free overfall, as in Kariba Dam on Zambezi River in Africa, where the free fall is over 100m.

In some projects, like the Indira Sagar Dam on River Narmada, two sets of spillways are provided – Main and Auxillary. The main spillway, also known as the service spillway is the one which is generally put into operation in passing most of the design flood. The crest levels of the auxillary spillways are usually higher and thus the discharge capacities are also small and are put into operation when the discharge in the river is higher than the capacity of the main spillway. Sometimes, an Emergency or Fuse Plug types of spillway is provided in the periphery of the reservoir which operates only when there is very high flood in the river higher than the design discharge or during the malfunctioning of normal spillways due to which there is a danger of the dam getting overtopped.

Usually, spillways are provided with gates, which provides a better control on the discharges passing through. However, in remote areas, where access to the gates by personnel may not be possible during all times as during the rainy season or in the night ungated spillways may have to be provided.

The capacity of a spillway is usually worked out on the basis of a flood routing study, explained in lesson 4.5. As such, the capacity of a spillway is seen to depend upon the following major factors:

- The inflow flood
- The volume of storage provided by the reservoir
- Crest height of the spillway
- Gated or ungated

According to the Bureau of Indian Standards guideline IS: 11223-1985 "Guidelines for fixing spillway capacity", the following values of inflow design floods (IDF) should be taken for the design of spillway:

- For large dams (defined as those with gross storage capacity greater than 60 million m^3 or hydraulic head greater than 60 million m^3 or hydraulic head between (2m and 30m), IDF should be based on the Standard Project Flood(SPF).
- For intermediate dams those with gross storage between 10 and 60 million m^3 or hydraulic head between (2m and 30m), IDF should be based on the Standard Project Flood (SPF).
- For small dams (gross storage between 0.5 to 10 million m^3 or hydraulic head between 7.5m to 12m), IDF may be taken as the 100 year return period flood.

The volume of the reservoir corresponding to various elevation levels as well as the elevation of the crest also affects the spillway capacity, as may be obvious from the flood routing procedure shown in Lesson 4.5.

If the spillway is gated, then the discharging water (Q) is controlled by the gate opening and hence the relation of Q to reservoir water level would be different from that of an ungated spillway. In the example of Lesson 4.5, an ungated spillway considered. Where as, in most practical cases, spillways are provided with gates and the gate operation is guided by a certain predetermined sequence which depends upon the inflow discharge. Hence, for an actual spillway capacity design, one has to consider not only the inflow hydrograph, but also the gate operation sequence.

Apart from spillways, which safely discharge the excess flood flows, outlets are provided in the body of the dam to provide water for various demands, like irrigation, power generation, etc. Hence, ordinarily riverflows are usually stored in the reservoir or released through the outlets, and the spillway is not required to function. Spillway flows will result during floods or periods of sustained high runoff when the capacities of other facilities are exceeded. Where large reservoir storage is provided, or where large outlet or diversion capacity is available, the spillway will be utilized infrequently. This feature may be contrasted with that of a diversion structure-like a barrage-where the storage is almost nil, and hence, the spillway there is in almost continuous operation.

Spillways are ordinarily classified according to their most prominent feature, either as it pertains to the control, to the discharge channel, or to some other component. The common types of spillway in use are the following:

1. Free Overfall (Straight Drop) Spillway
2. Overflow (Ogee) Spillway
3. Chute (Open Channel/Trough) Spillway
4. Side Channel Spillway
5. Shaft (Drop Inlet/Morning Glory) spillway
6. Tunnel (Conduit) spillway
7. Siphon spillway

These spillways are individually treated in the subsequent sections.

The water flowing down from the spillways possess a large amount of kinetic energy that is generated by virtue of its losing the potential head from the reservoir level to the level of the river on the downstream of the spillway. If this energy is not reduced, there are danger of scour to the riverbed which may threaten the stability of the dam or the neighbouring river valley slopes. The various arrangements for suppressing or killing of the high energy water at the downstream toe of the spillways are called Energy Dissipators. These are discussed at the end of this lesson.

Free Overfall Spillway

In this type of spillway, the water freely drops down from the crest, as for an arch dam (Figure 1). It can also be provided for a decked over flow dam with a vertical or adverse inclined downstream face (Figure 2). Flows may be free discharging, as will be the case with a sharp-crested weir or they may be supported along a narrow section of the crest. Occasionally, the crest is extended in the form of an overhanging lip (Figure 3) to direct small discharges away from the face of the overfall section. In free falling water is ventilated sufficiently to prevent a pulsating, fluctuating jet.

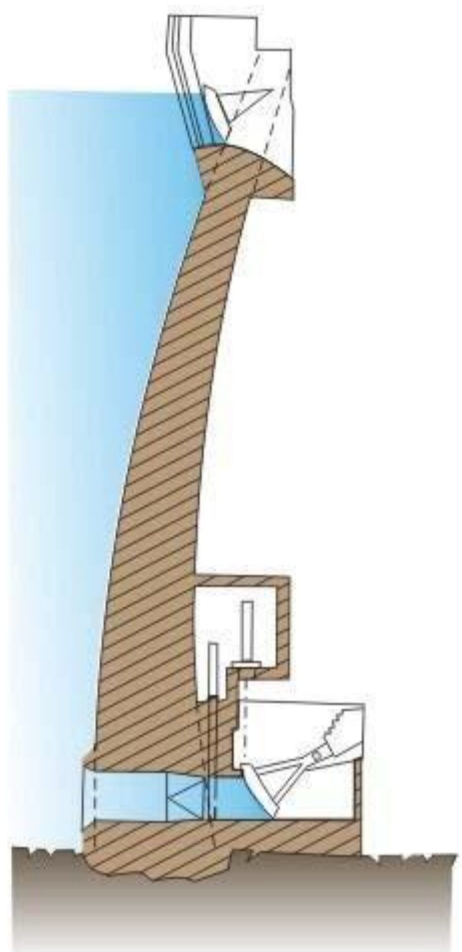
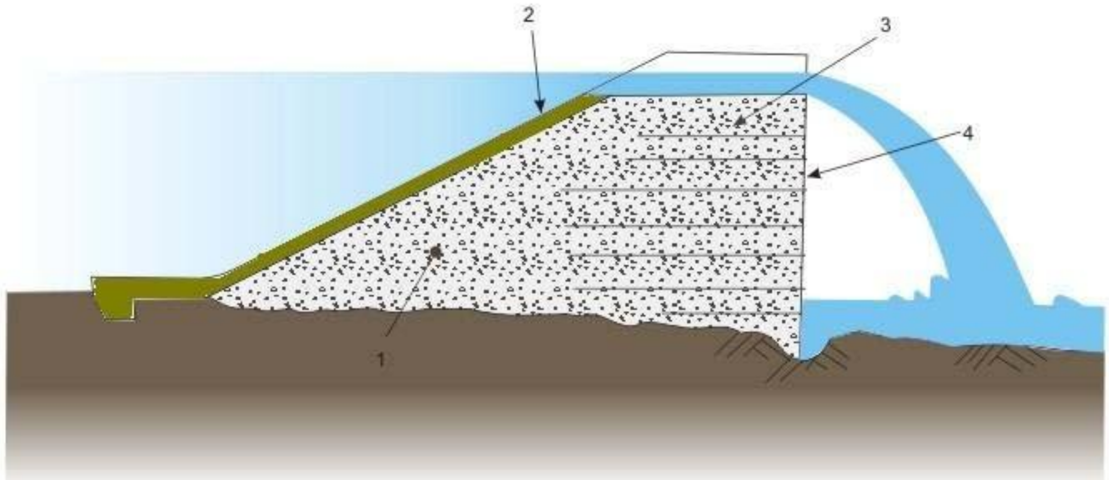


FIGURE 1. Free over fall spillway for an arch dam



LEGEND

1. RANDOM FILL 2. WATERTIGHT MEMBRANE 3. STEEL TENDONS 4. CONCRETE SLABS (1.5 M X 1.5 M).

FIGURE 2. Free over fall spillway for a decked embankment dam

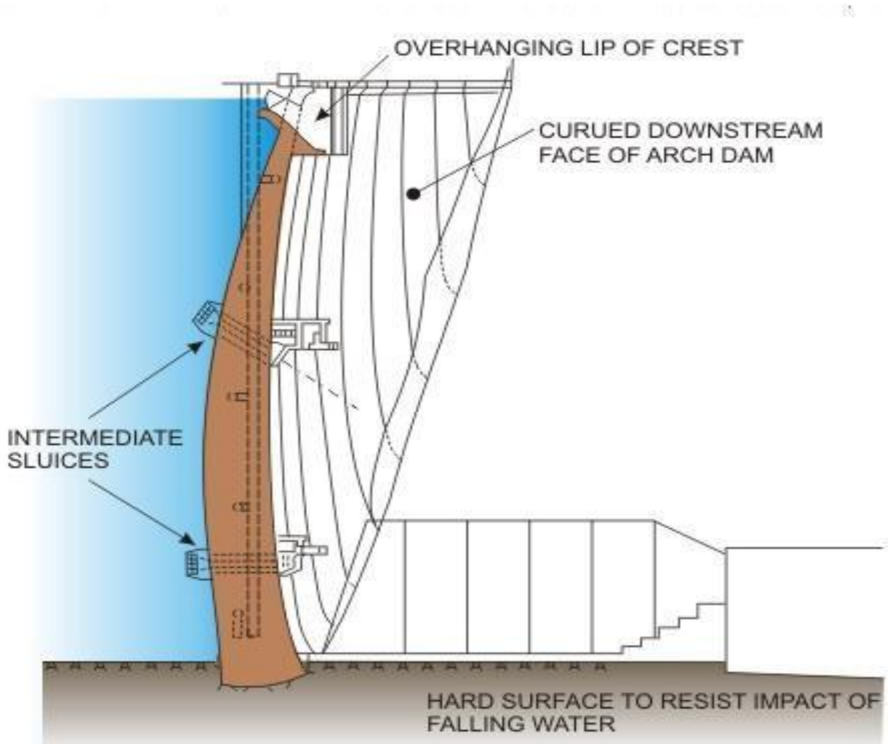


FIGURE 3. Short lip provided for overfall spilling of an arch dam

Where artificial protection is provided at the loose, as in Figure 3, the bottom may not scour but scour may occur for unprotected streambeds which will form deep plunge pool (Figure 4). The volume and the depth of the scour hole are related to the range of discharges, the height of the drop, and the depth of tail water. Where erosion cannot be tolerated an artificial pool can be created by constructing an auxiliary dam downstream of the main structure, or by excavating a basin which is then provided with a concrete apron or bucket.

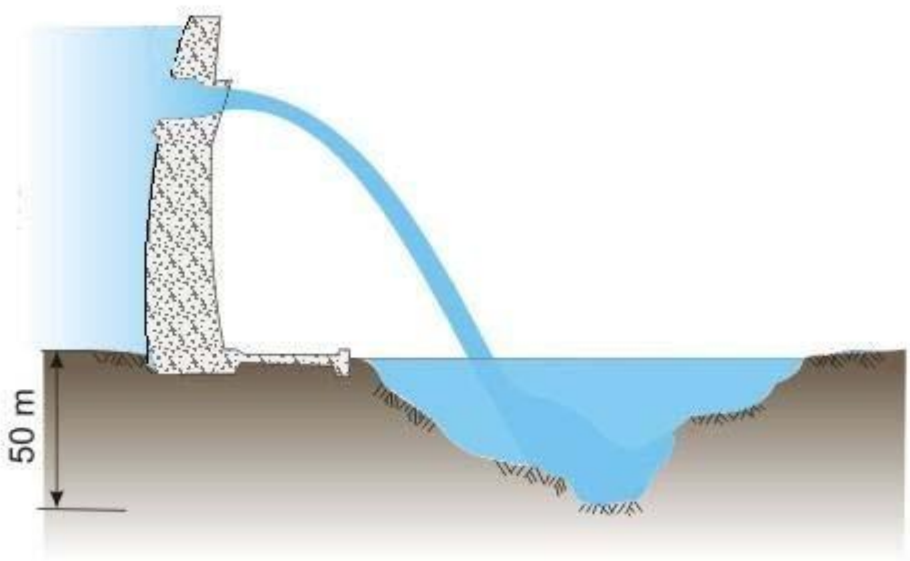


FIGURE 4. Scour below Kariba Dam Spillway , Zimbabwe

Overflow Spillway

The overflow type spillway has a crest shaped in the form of an ogee or S-shape (Figure 5). The upper curve of the ogee is made to conform closely to the profile of the lower nappe of a ventilated sheet of water falling from a sharp crested weir (Figure 6). Flow over the crest of an overflow spillway is made to adhere to the face of the profile by preventing access of air to the underside of the sheet of flowing water. Naturally, the shape of the overflow spillway is designed according to the shape of the lower nappe of a free flowing weir conveying the discharge flood. Hence, any discharge higher than the design flood passing through the overflow spillway would try to shoot forward and get detached from the spillway surface, which reduces the efficiency of the spillway due to the presence of negative pressure between the sheet of water and spillway surface. For discharges at designed head, the spillway attains near-maximum efficiency. The profile of the spillway surface is continued in a tangent along a slope to support the sheet of flow on the face of the overflow. A reverse curve at the bottom of the slope turns the flow in to the apron of a sliding basis or in to the spillway discharge channel.

An ogee crest apron may comprise an entire spillway such as the overflow of a concrete gravity dam (Figure 7), or the ogee crest may only be the control structure for some other type of spillway (Figure 8). Details of computing crest shape and discharges of ogee shaped crest is provided in Section 4.8.9.

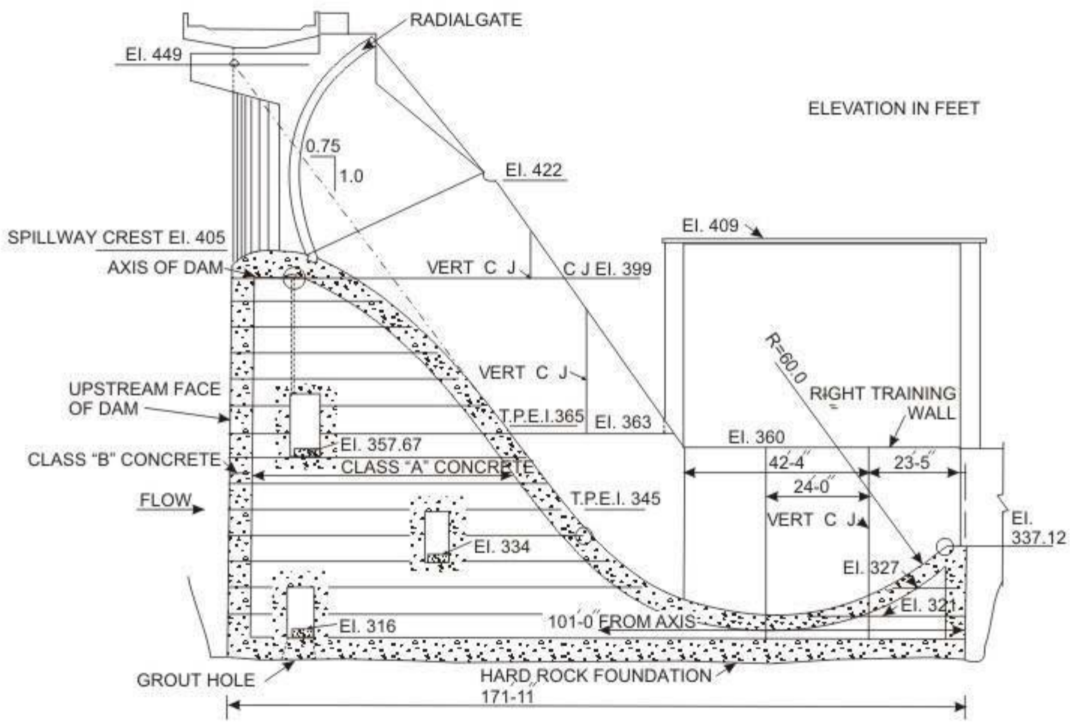


FIGURE 5. Typical overflow (ogee) spillway .Example of Panchet Dam on River Damodar

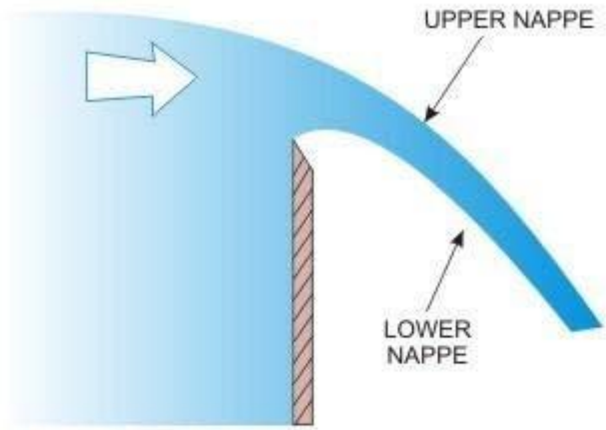


FIGURE 6. Outflow from a free-falling weir , properly ventilated from below

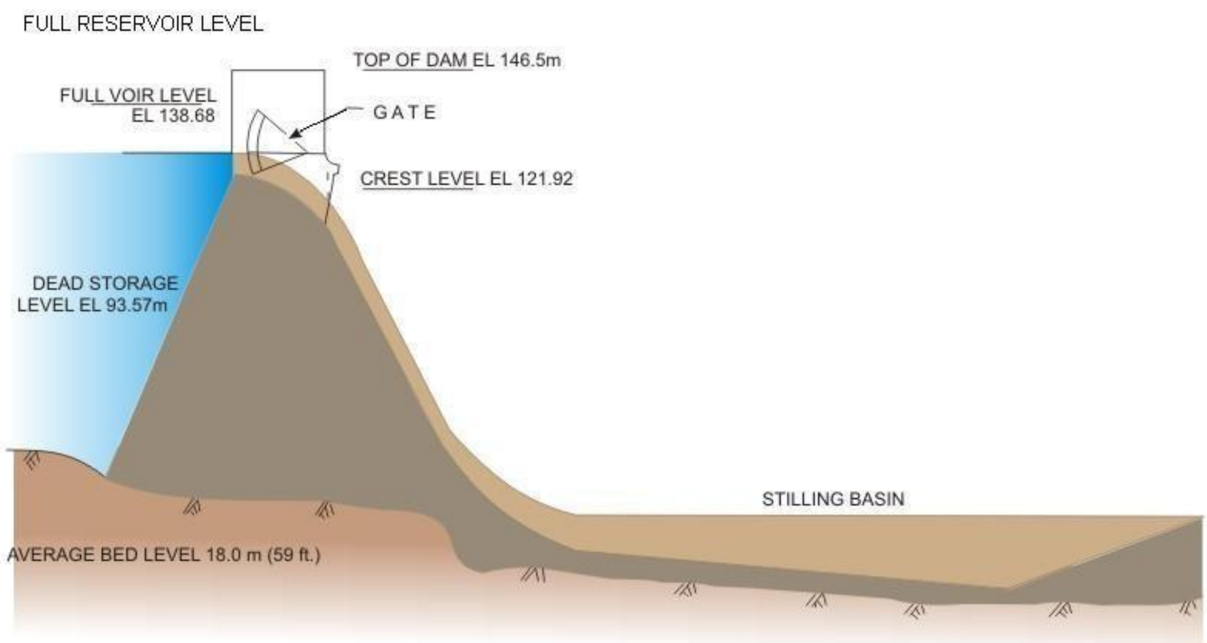


FIGURE 7. Ogee spillway & apron of Sardar Sarovar Dam spillway

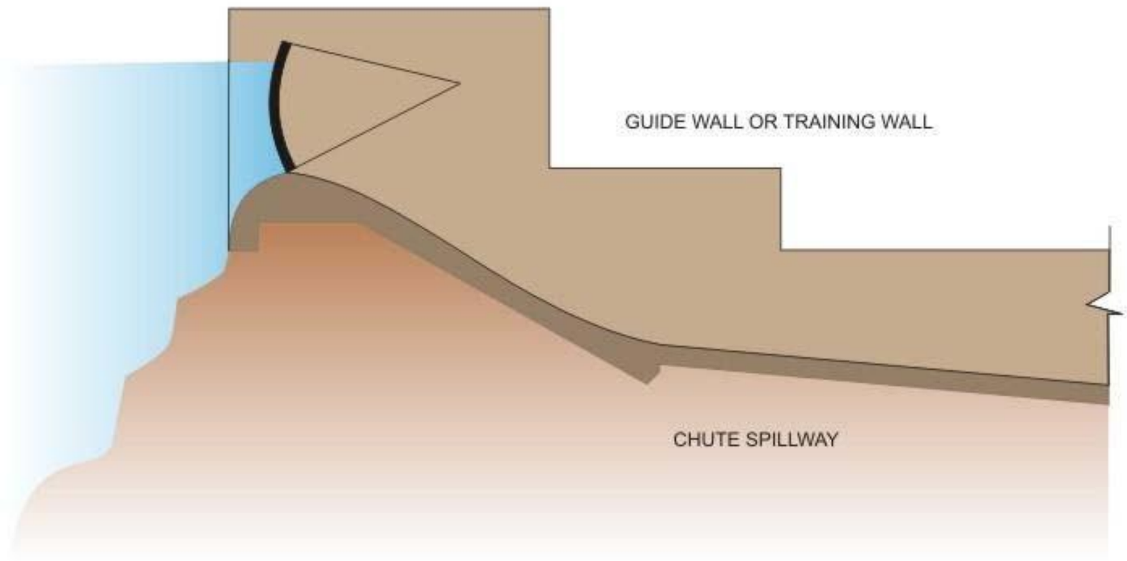


FIGURE 8. Ogee spillway for controlling flow into a chute-type spillway

Chute Spillway

A chute spillway, variously called as open channel or trough spillway, is one whose discharge is conveyed from the reservoir to the downstream river level through an open channel, placed either along a dam abutment or through a saddle (Figure 9). The control structure for the chute spillway need not necessarily be an overflow crest, and may be of the side-channel type (discussed in Section 4.9.4), as has been shown in Figure 10. However, the name is most often applied when the spillway control is placed normal or nearly normal to the axis of the open channel, and where the streamlines of flow both above and below the control crest follow in the direction of the axis.

Generally, the chute spillway has been mostly used in conjunction with embankment dams, like the Tehri dam, for example. Chute spillways are simple to design and construct and have been constructed successfully on all types of foundation materials, ranging from solid rock to soft clay.

Chute spillways ordinarily consist of an entrance channel, a control structure, a discharge channel, a terminal structure, and an outlet channel. Often, the axis of the entrance channel or that of the discharge channel must be curved to fit the topography. For further details, one may refer to the Bureau of Indian Standards Code IS: 5186- 1994 “Design of chute and side channel spillways-criteria”.

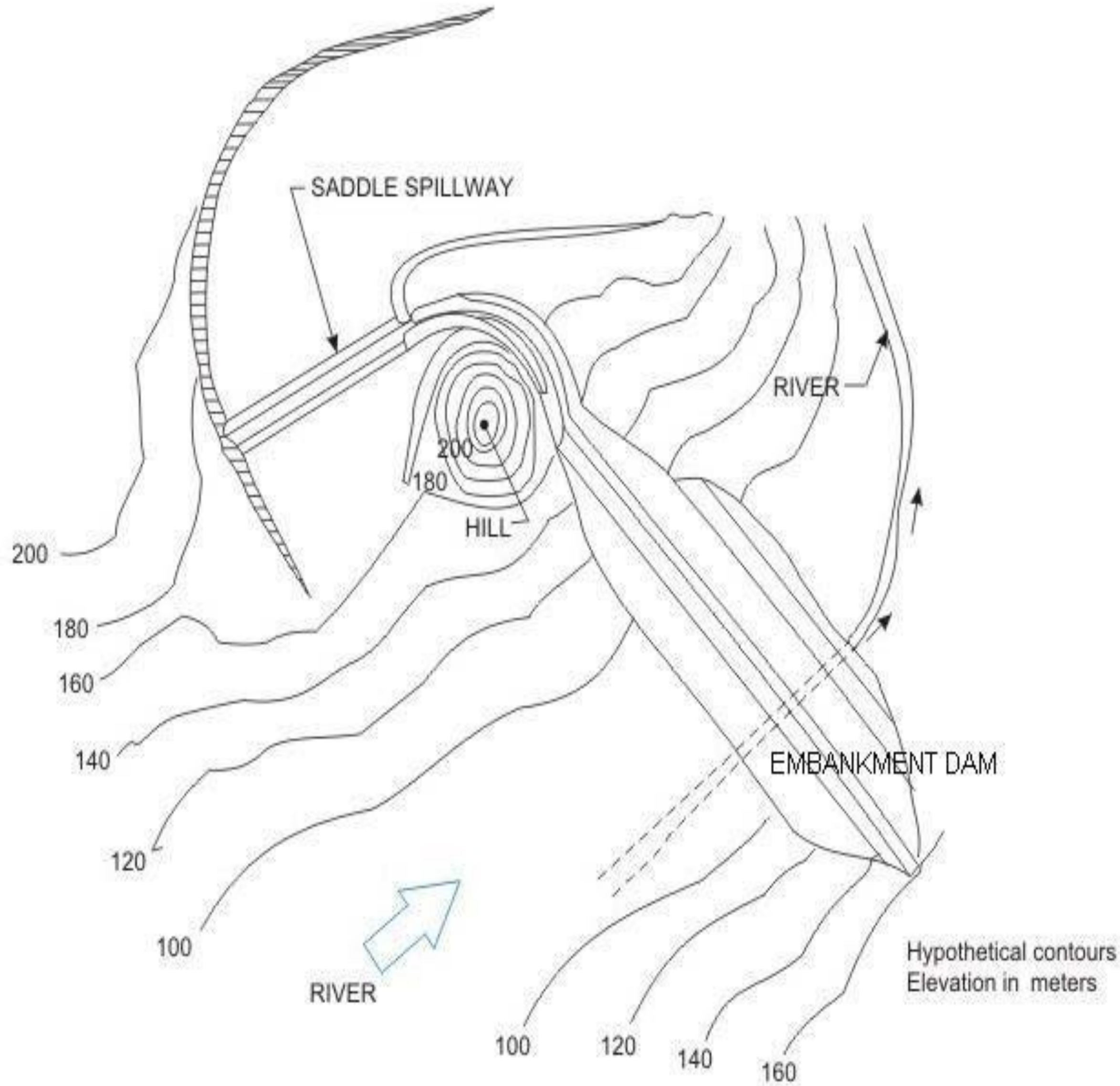


FIGURE 9. Saddle spillway

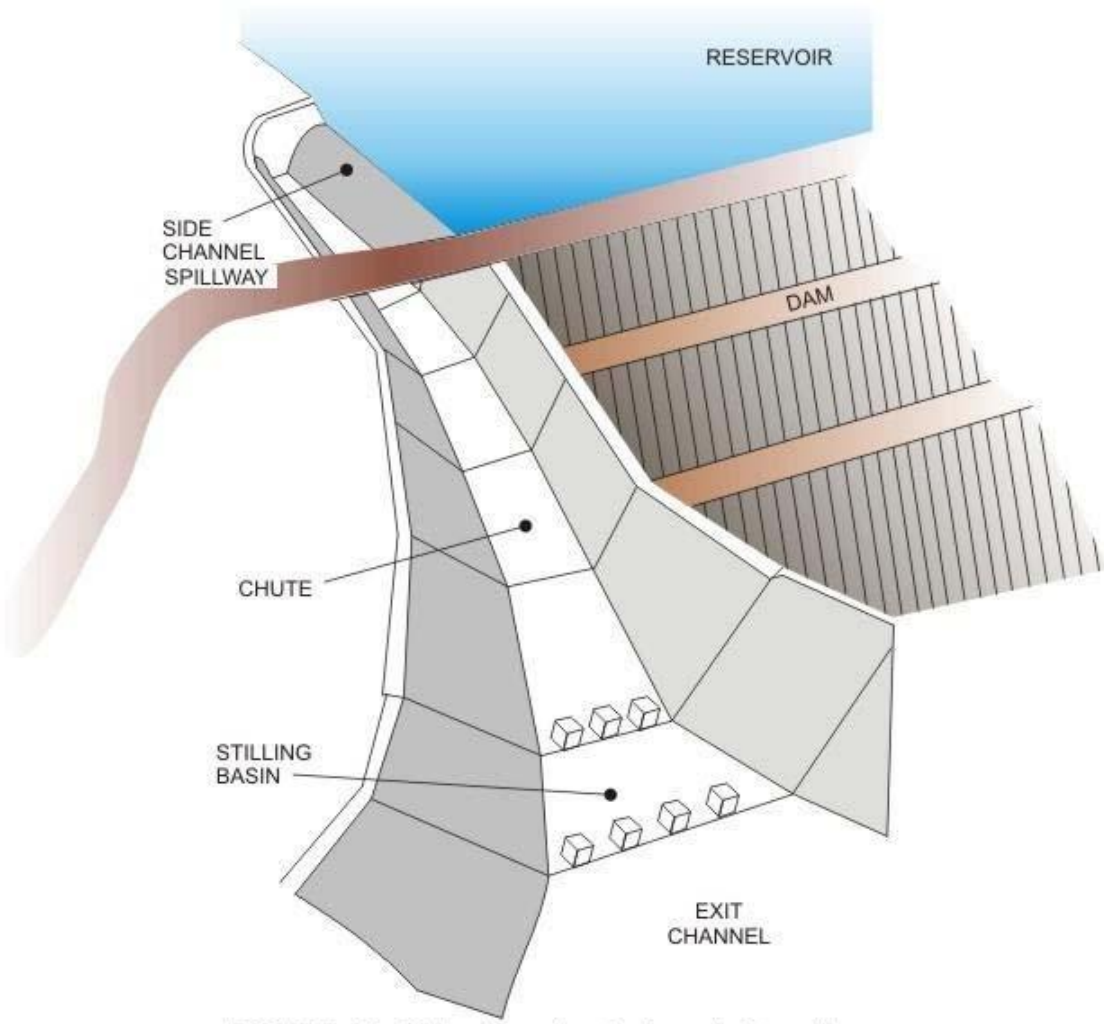


FIGURE 10. Side channel entry to a chute spillway

Side channel Spillway

A side channel spillway is one in which the control weir is placed approximately parallel to the upper portion of the discharge channel, as may be seen from Figure 10. When seen in plan with reference to the dam, the reservoir and the discharge channel, the side channel spillway would look typically as in Figure 11 and its sectional view in Figure 12. The flow over the crest falls into a narrow trough opposite to the weir, turns an approximate right angle, and then continues into the main discharge channel. The side channel design is concerned only with the hydraulic action in the upstream reach of the discharge channel and is more or less independent of the details selected for the other spillway components. Flow from the side channel can be directed into an open discharge channel, as in Figure 10 or 11 showing a chute channel, or in to a closed conduit which may run under pressure or inclined tunnel. Flow into the side channel might enter on only one side of the trough in the case of a steep hill side location or on both sides and over the end of the trough if it is located on a knoll or gently sloping abutment.

Discharge characteristics of a side channel spillway are similar to those of an ordinary overflow spillway and are dependent on the selected profile of the weir crest. Although the side channel is not hydraulically efficient, nor inexpensive, it has advantages which make it adoptable to spillways where a long overflow crest is required in order to limit the afflux (surcharge held to cause flow) and the abutments are steep and precipitous.

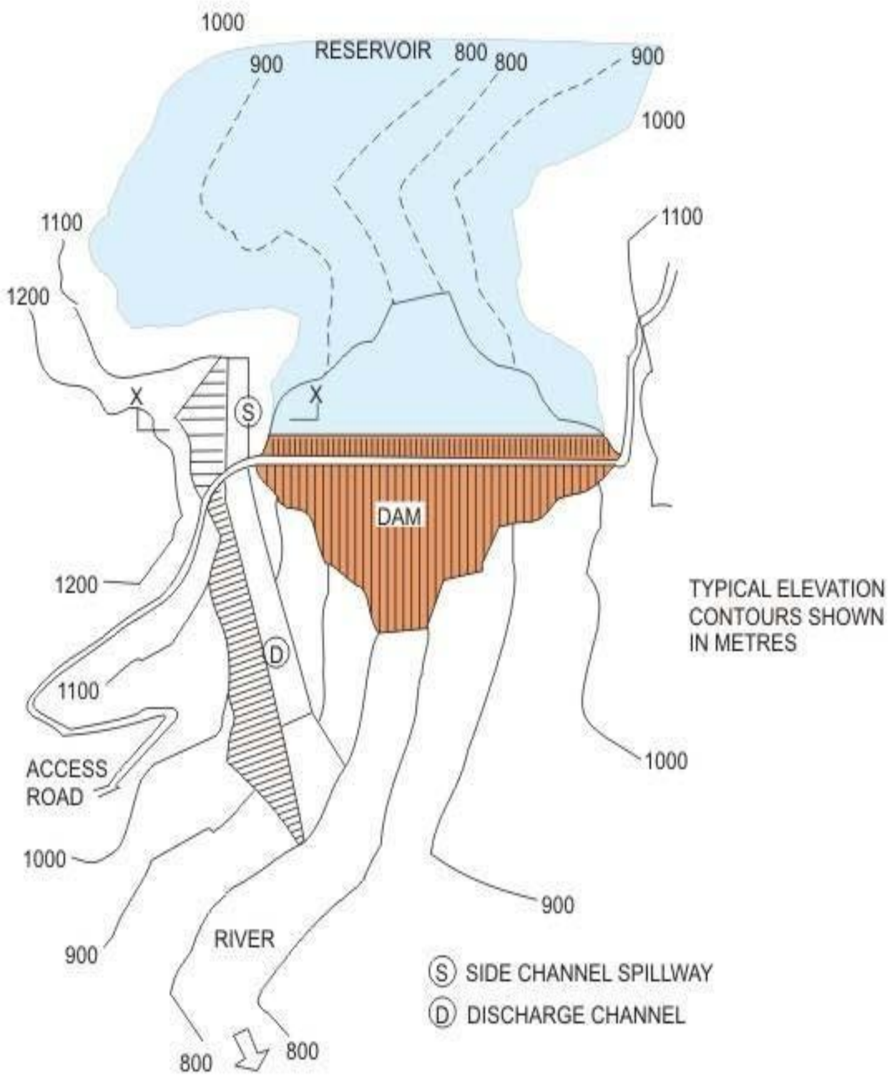


FIGURE 11. Plan of an embankment dam showing side channel spillway and chute channel

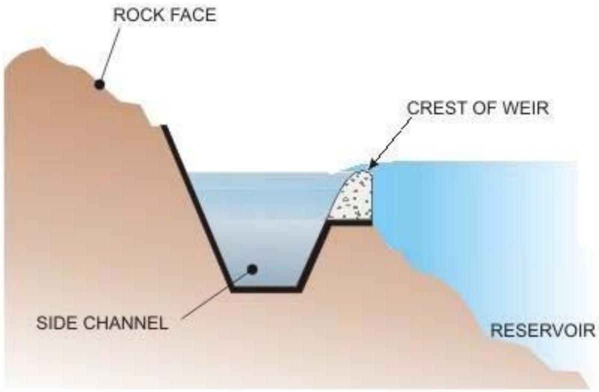


FIGURE 12. Magnified sectional view X-X through the side channel spillway shown in Figure 11

Shaft Spillway

A Shaft Spillway is one where water enters over a horizontally positioned lip, drops through a vertical or sloping shaft, and then flows to the downstream river channel through a horizontal or nearly horizontal conduit or tunnel (Figure 13). The structure may be considered as being made up of three elements, namely, an overflow control weir, a vertical transition, and a closed discharge channel. When the inlet is funnel shaped, the structure is called a Morning Glory Spillway. The name is derived from the flower by the same name, which it closely resembles especially when fitted with anti- vortex piers (Figure 14). These piers or guide vanes are often necessary to minimize vortex action in the reservoir, if air is admitted to the shaft or bend it may cause troubles of explosive violence in the discharge tunnel-unless it is amply designed for free flow.

Discharge characteristics of the drop inlet spillway may vary with the range of head. As the head increases, the flow pattern would change from the initial weir flow over crest to tube flow and then finally to pipe flow in the tunnel. This type of spillway attains maximum discharging capacity at relatively low heads. However, there is little increase in capacity beyond the designed head, should a flood larger than the selected inflow design flood occur.

A drop inlet spillway can be used advantageously at dam sites that are located in narrow gorges where the abutments rise steeply. It may also be installed at projects where a diversion tunnel or conduit is available for use.

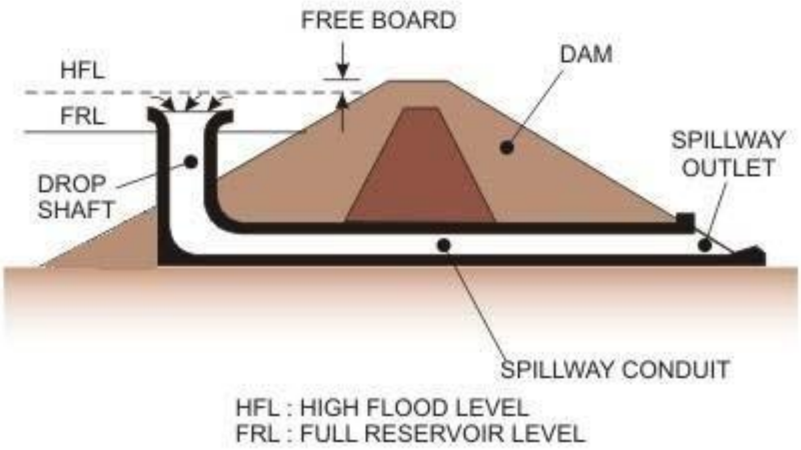


FIGURE 13. Section through a shaft spillway

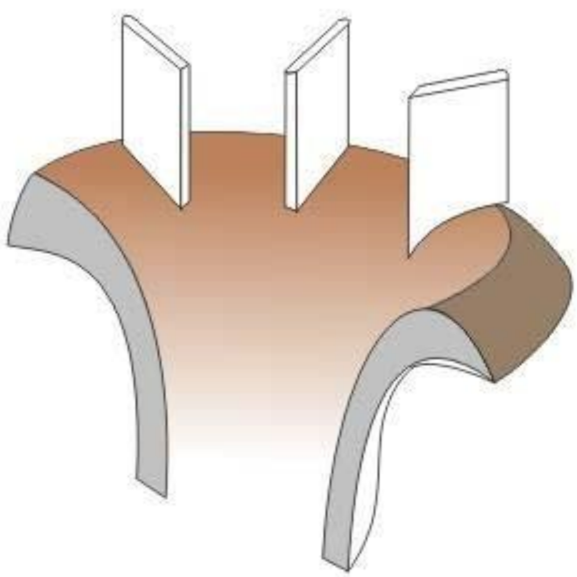


FIGURE 14. Morning glory spillway with anti-vortex piers

Tunnel Spillway

Where a closed channel is used to convey the discharge around a dam through the adjoining hill sides, the spillway is often called a tunnel or conduit spillway. The closed channel may take the form of a vertical or inclined shaft, a horizontal tunnel through earth or rock, or a conduit constructed in open cut and backfilled with earth materials. Most forms of control structures, including overflow crests, vertical or inclined orifice entrances, drop inlet entrances, and side channel crests, can be used with tunnel spillways. Two such examples have been shown in Figs. 15 and 16. When the closed channel is carried under a dam, as in Figure 13, it is known as a conduit spillway.

With the exception of those with orifice or shaft type entrances, tunnel spillways are designed to flow partly full throughout their length. With morning glory or orifice type control, the tunnel size is selected so that it flows full for only a short section at the control and thence partly full for its remaining length. Ample aeration must be provided in a tunnel spillway in order to prevent a fluctuating siphonic action which would result if some part of exhaution of air caused by surging of the water jet, or wave action or backwater.

Tunnel spillways are advantageous for dam sites in narrow gorges with steep abutments or at sites where there is danger to open channels from rock slides from the hills adjoining the reservoir.

Conduit spillways are generally most suited to dams in wide valleys as in such cases the use of this types of spillway would enable the spillway to be located under the dam very close to the stream bed.

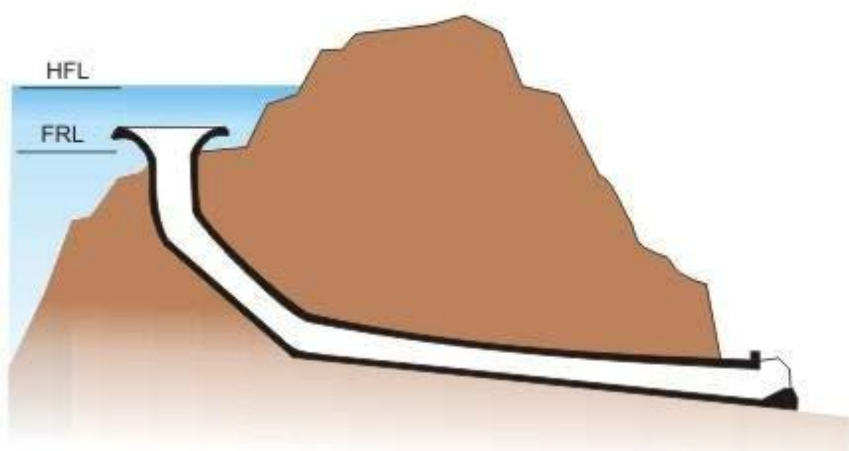


Figure 15. Tunnel spillway with a morning glory entrance.

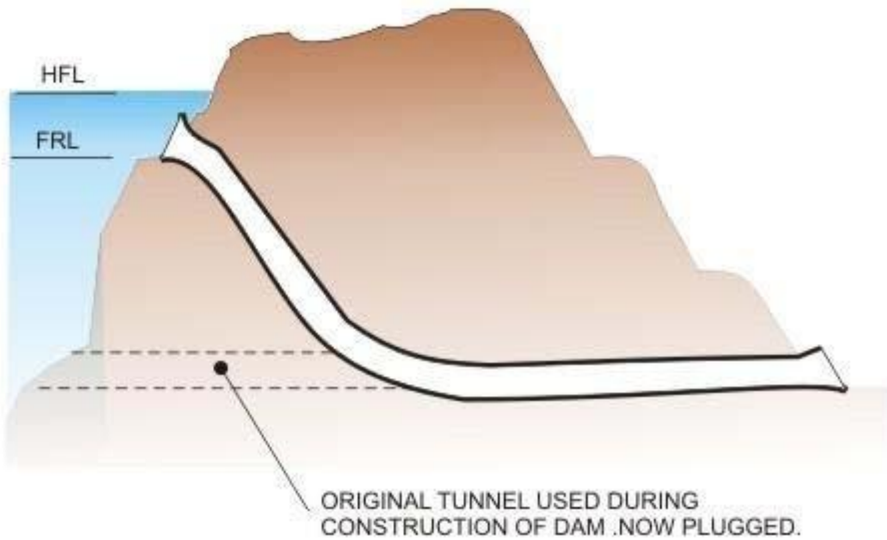


FIGURE 16. Bell mouth entry to a tunnel spillway (Gate control at entrance not shown)

Siphon Spillway

A siphon spillway is a closed conduit system formed in the shape of an inverted U, positioned so that the inside of the bend of the upper passageway is at normal reservoir storage level (Figure 17). This type of siphon is also called a Saddle siphon spillway. The initial discharges of the spillway, as the reservoir level rises above normal, are similar to flow over a weir. Siphonic action takes place after the air in the bend over the crest has been exhausted. Continuous flow is maintained by the suction effect due to the gravity pull of the water in the lower leg of the siphon.

Siphon spillways comprise usually of five components, which include an inlet, an upper leg, a throat or control section, a lower leg and an outlet. A siphon breaker air vent is also provided to control the siphonic action of the spillway so that it will cease operation when the reservoir water surface is drawn down to normal level. Otherwise the siphon would continue to operate until air entered the inlet. The inlet is generally placed well below the Full Reservoir Level to prevent entrance of drifting materials and to avoid the formation of vortices and draw downs which might break siphonic action.

Another type of siphon spillway (Figure 18) designed by Ganesh Iyer has been named after him. It consists of a vertical pipe or shaft which opens out in the form of a funnel at the top and at the bottom it is connected by a right angle bend to a horizontal outlet conduit. The top or lip of the funnel is kept at the Full Reservoir Level. On the surface of the funnel are attached curved vanes or projections called the volutes.

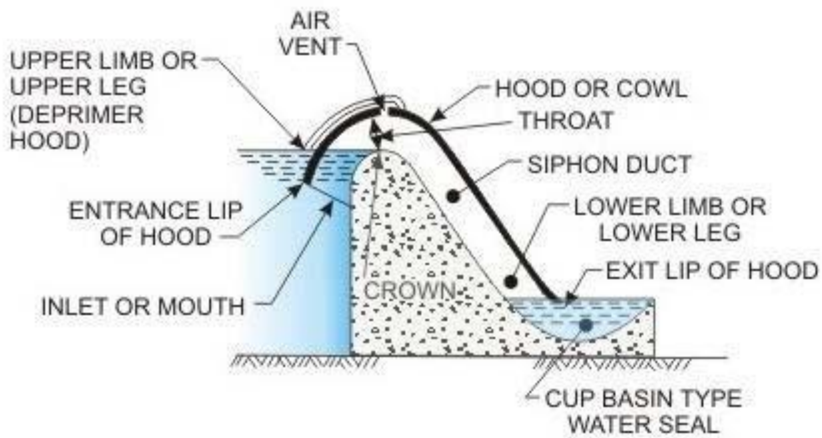


FIGURE 17. Saddle Siphon

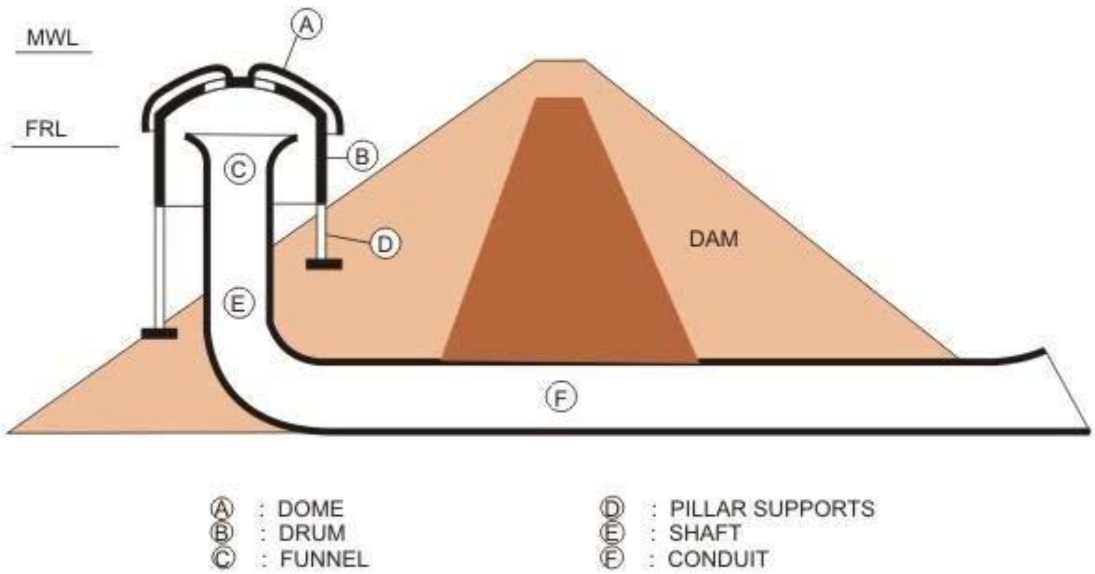


FIGURE 18. Volute of siphon spillway components

Special types of spillways

Apart from the commonly used spillways, a few other types of spillways are used sometimes for a project, which are explained below.

Saddle Spillway

In some basins formed by a dam, there may be one or more natural depressions for providing spillway. They are sometimes preferred for locating main spillway or emergency or auxiliary spillways. A site which has a saddle is very desirable and economical, if the saddle is suitable for locating the spillway. An example for such a spillway may be seen in Figure 9.

Fuse plug

It may be a simple earth bank, flash board or other device designed to fail when overtopped. Such plugs may be used where the sudden release of a considerable volume of water is both safe and not over destructive to the environment. "For example, the saddle spillway of Figure 9 may be constructed as an earthen embankment dam, with its crest at a slightly higher elevation than the High Flood Level (HFL) of the reservoir. In the occurrence of a flood greater than the design flood which may cause rise in the reservoir water would overtop the earthen embankment dam and cause its collapse and allow the flood water to safely pass through the saddle spillway.

Sluice Spillway

The use of large bottom openings as spillways is a relatively modern innovation following the greater reliance on the safety and operation of modern control gates under high pressure. A distinct advantage of this type of spillway is that provision can usually be made for its use for the passage of floods during construction. One disadvantage is that, once built, its capacity is definite whereas the forecasting of floods is still indefinite. A second disadvantage is that a single outlet may be blocked by flood debris, especially where in flow timber does not float. Figure 20 shows an example of a sluice spillway.

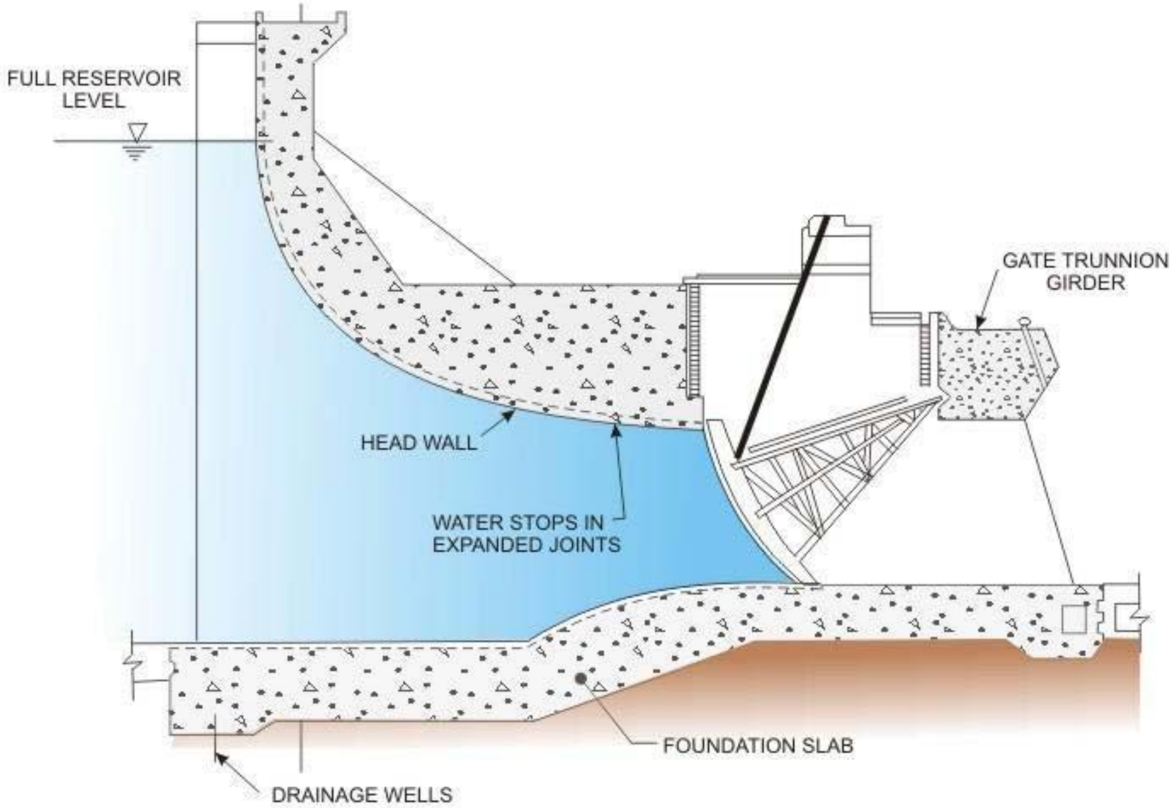


FIGURE 19. Sluice spillway

Duck-bill Spillway

This is a spillway with a rectangular layout projections into the reservoir comprising three straight overflow lengths intersecting at right angles. The layout could be trapezoidal in which case the corner angles will be other than 90 degree. The flow from the three reaches of the spillway interacts in the trough portion and is further conveyed through a discharge carrier on to a terminal structure to provide for energy dissipation. An example of this type of spillway is shown in Figure 21.

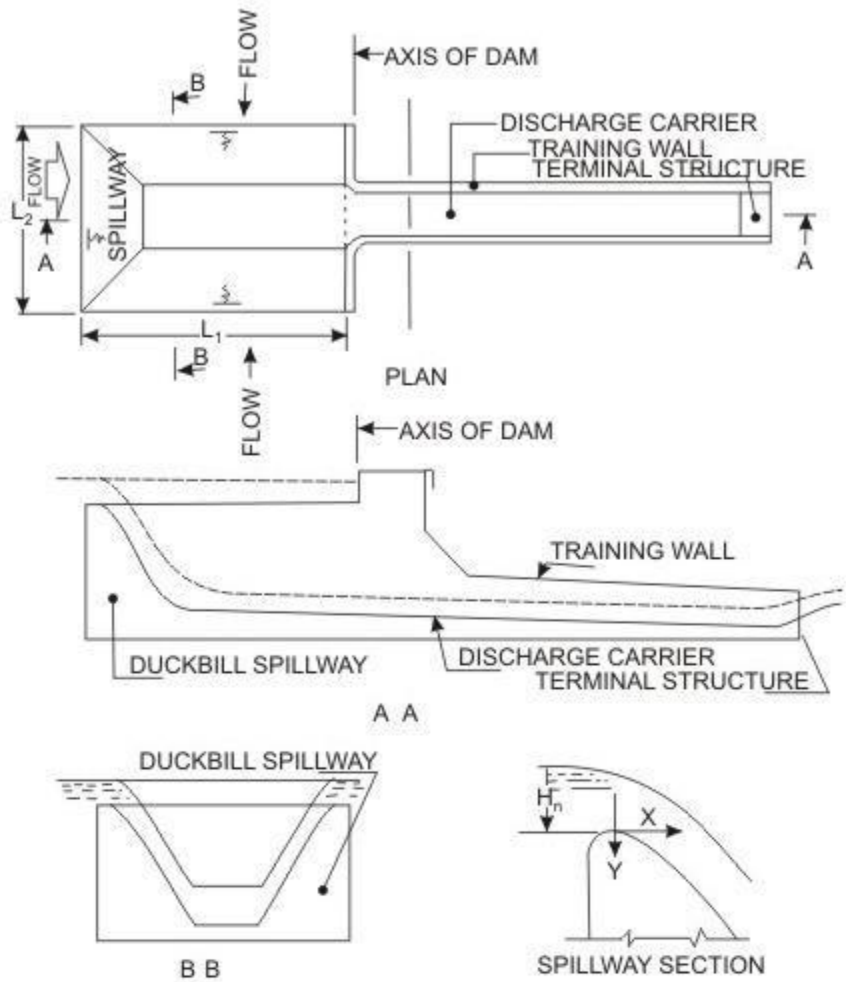


FIGURE 20. Duckbill Spillway

Shape and Hydraulics of Ogee-Crest

Crest shape

The ogee shaped crest is commonly used as a control weir for many types of spillways- Overflow (Figure 5), Chute (Figure 8), Side Channel (Figure 12) etc. The ogee shape which approximates the profile of the lower nappe of a sheet of water flowing over a sharp-crested weir provides the ideal form for obtaining optimum discharges. The shape of such a profile depends upon the head, the inclination of the upstream face of the flow section, and the height of the overflow section above the floor of the entrance channel (which influences the velocity of approach to the crest). The ogee profile to be acceptable should provide maximum possible hydraulic efficiency, structural stability and economy and also avoid the formation of sub atmospheric pressures at the surface (which may induce cavitations).

Ogee crested control structures are also sensitive to the upstream shape and hence, three types of ogee crests are commonly used and shown in Figure 21. These are as follows:

1. Ogee crests having vertical upstream face
2. Ogee crests having inclined upstream face
3. Ogee crests having over hang on up stream face

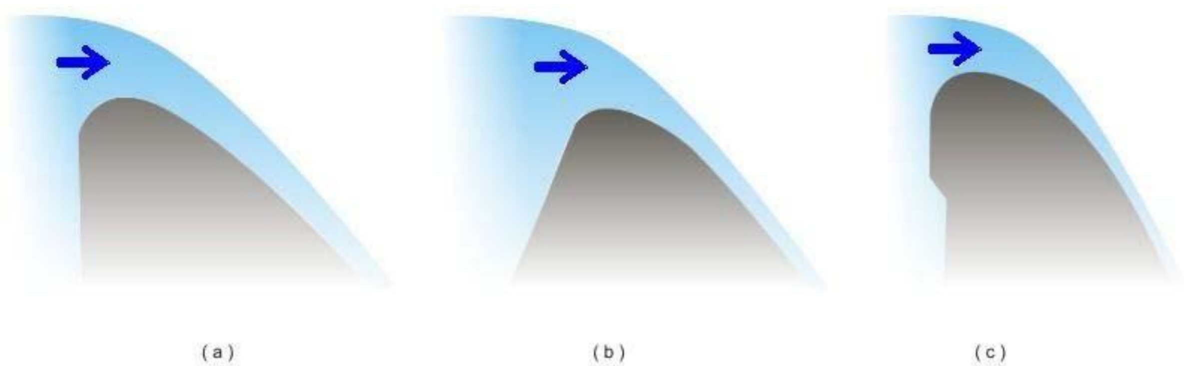


FIGURE 21. Ogee crest control weirs with
(a) Vertical upstream face
(b) Inclined upstream face
(c) Overhangs on the upstream

However, the same general equations for the up stream and down stream quadrants are applicable to all the three cases, as recommended by the Bureau of Indian Standards code IS: 6934-1998 “Hydraulic design of high ogee over flow spillways- recommendations” and are outlined in the following paragraphs.

1. Ogee crests with vertical upstream face

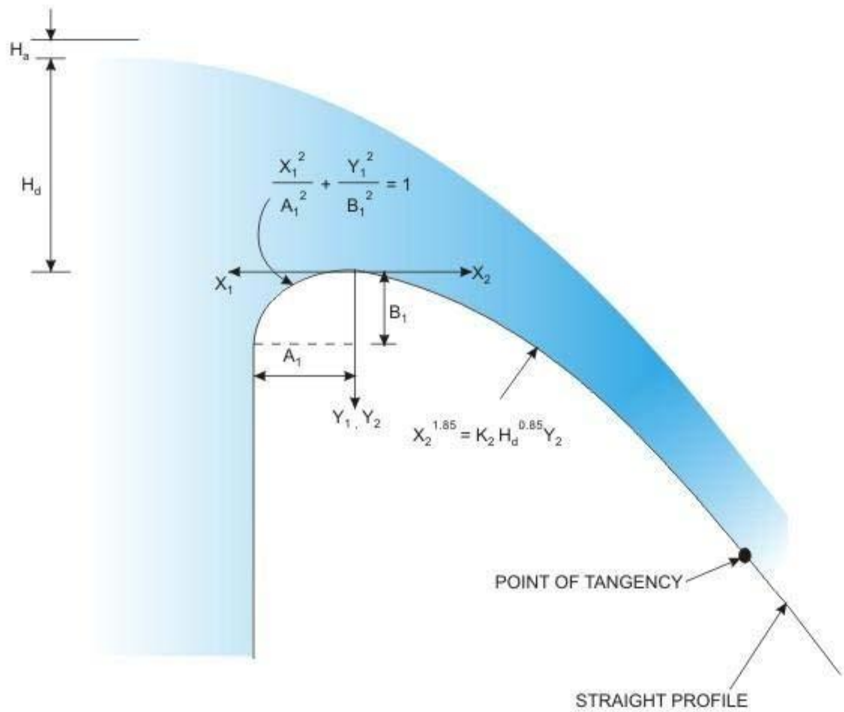


FIGURE 22. Ogee crest shape with vertical upstream wall . Coefficients to be determined from Figure 21

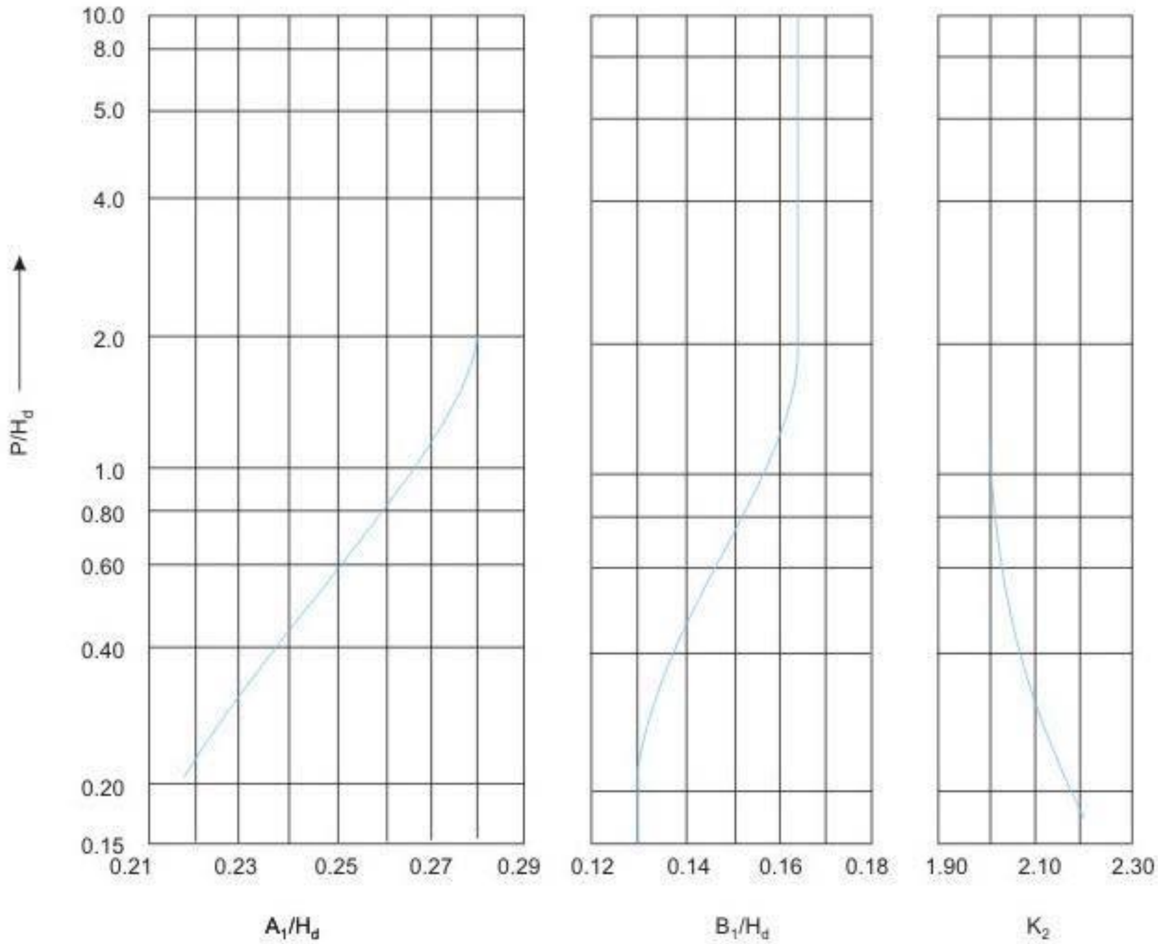


FIGURE 23. Coefficients for Figure 22

2. *Ogee crests with sloping up stream face*

In this case, the desired inclination of the upstream face is made tangential to the same elliptical profile as provided for a crest with a vertical face. The down stream face equation remains unchanged.

3. *Ogee crests with overhang*

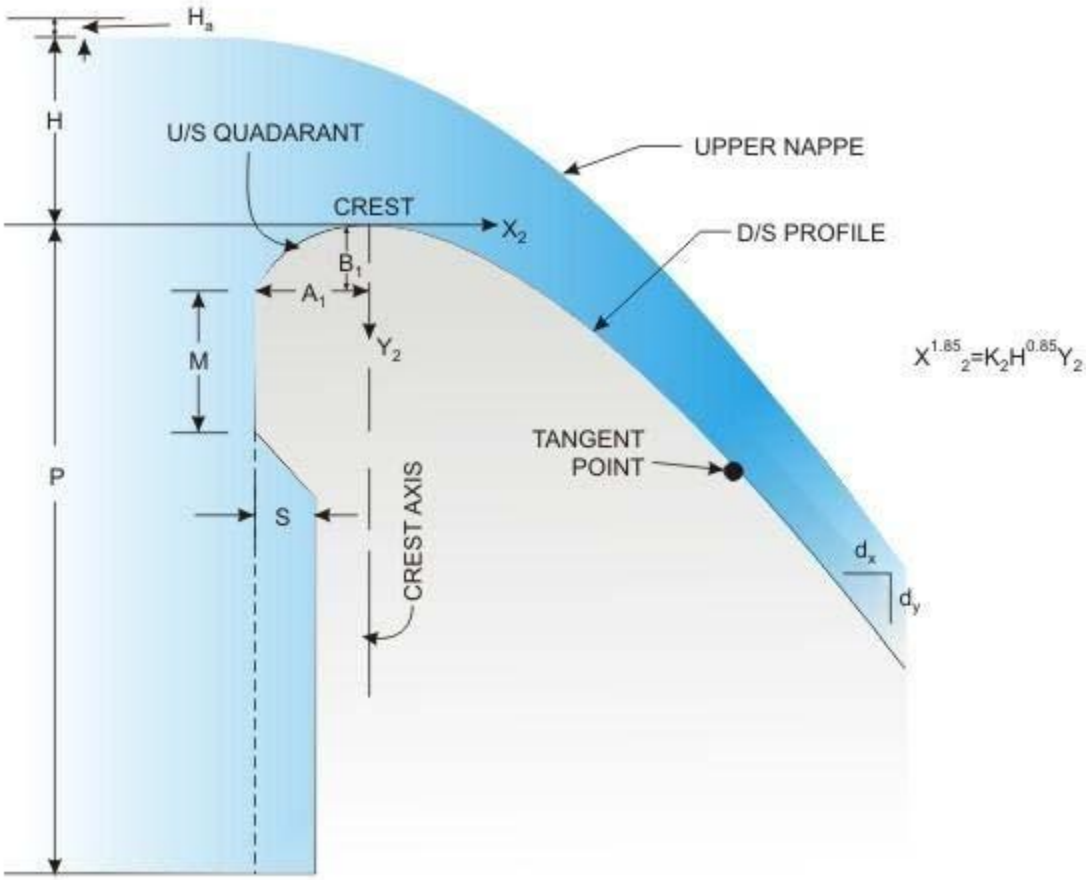


FIGURE 24. Overhang details of ogee crest

Whenever structural requirements permit, the upstream vertical face of an ogee crested spillway (Figure 22) may be offset inside, (Figure 24). It is recommended that the ratio of the rises M to the design head H_d, should be at least 0.6 or greater for flow conditions to be stable. The crest shapes on the up stream and downstream may be provided the same as for an ogee crest with vertical up stream wall if the condition M/ H_d >0.6 is satisfied.

Discharge characteristics of ogee crests-uncontrolled flow

For an ogee crested control weir for a spillway without any control with a gate, the free flow discharge equation is given as

Where Q is the discharge (in m^3/s), C_d is the coefficient of discharge, L_e is the effective length of crest (in m), including velocity of approach head. The discharge coefficient, C_d, is influenced by a number of factors, such as:

1. Depth of approach
2. Relation of the actual crest shape to the ideal nappe shape
3. Upstream face slope
4. Downstream apron interference, and
5. Downstream submergence

The effect of the above mentioned factors on the variation of discharge and calculation for effective length are mentioned in the following paragraphs.

1. Effect of depth of approach

For a high sharp-crested ogee shaped weir, as that of a Overfall spillway of a large dam, the velocity of approach is small and the lower nappe flowing over the weir attains maximum vertical contraction. As the approach depth is decreased, the velocity of approach increases and the vertical contraction diminishes. For sharp-crested weirs whose heights are not less than about one-fifth of the head producing the flow, the coefficient of discharge remains fairly constant with a value of about 1.82 although the contraction diminishes. For weir heights less than about one-fifth the head, the contraction of the flow becomes increasingly suppressed and the crest coefficient decreases. This is the case of an ogee crested chute spillway control section. When the weir height becomes zero, the contraction is entirely suppressed and the weir turns into a broad crested one, for which the theoretical coefficient of discharge is 1.70. The relationship of the ogee crest coefficient of discharge C_d for various values of P/H_d where P is the height of the weir above base and H_d is the design head, is given in Figure 25. The coefficients are valid only when ogee is formed to the ideal nappe shape.

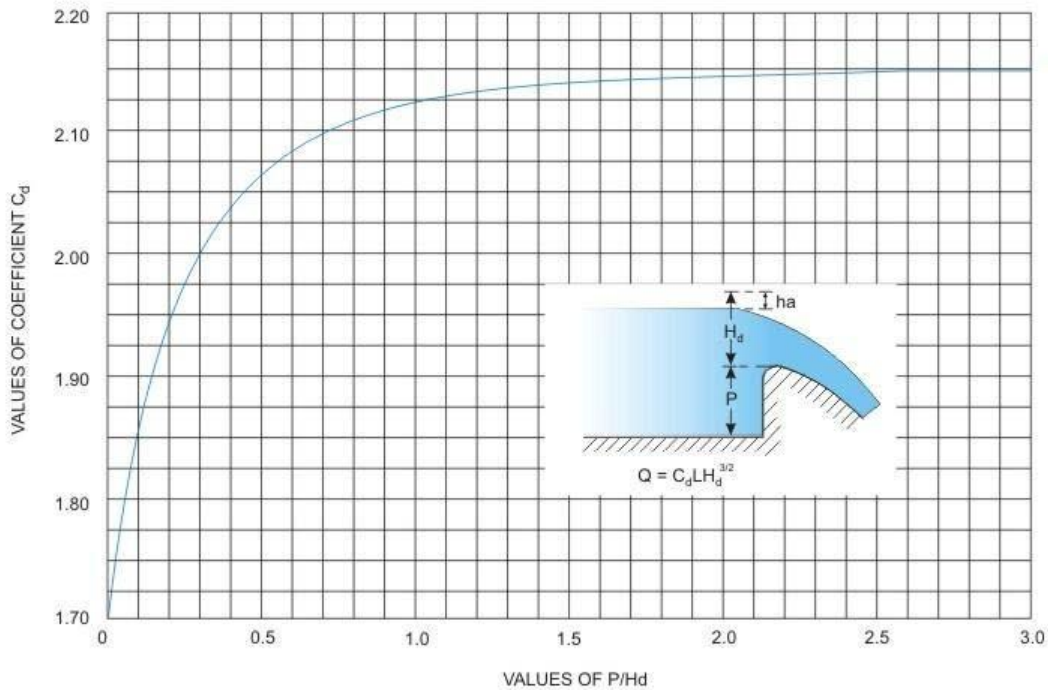


FIGURE 25. Coefficient of discharge (C_d) variation due to height of a vertical faced ogee crest

2. Effect of the crest shape differing from the ideal nappe shape

When the ogee crest is formed to a shape differing from the ideal nappe shape or when the crest has been shaped for a head larger or smaller than the one under consideration, the coefficient of discharge will differ from that given in the previous section. A wider crest shape will reduce the coefficient of discharge while a narrower Crest Shape will reduce the coefficient. The application of this concept is required to deduce the discharge flowing over a spillway when the flow is less or more than the design discharge. The variation of the coefficient of discharge in relation to H/Hd, where H is the actual head and Hd is the design head, is shown in Figure 26.

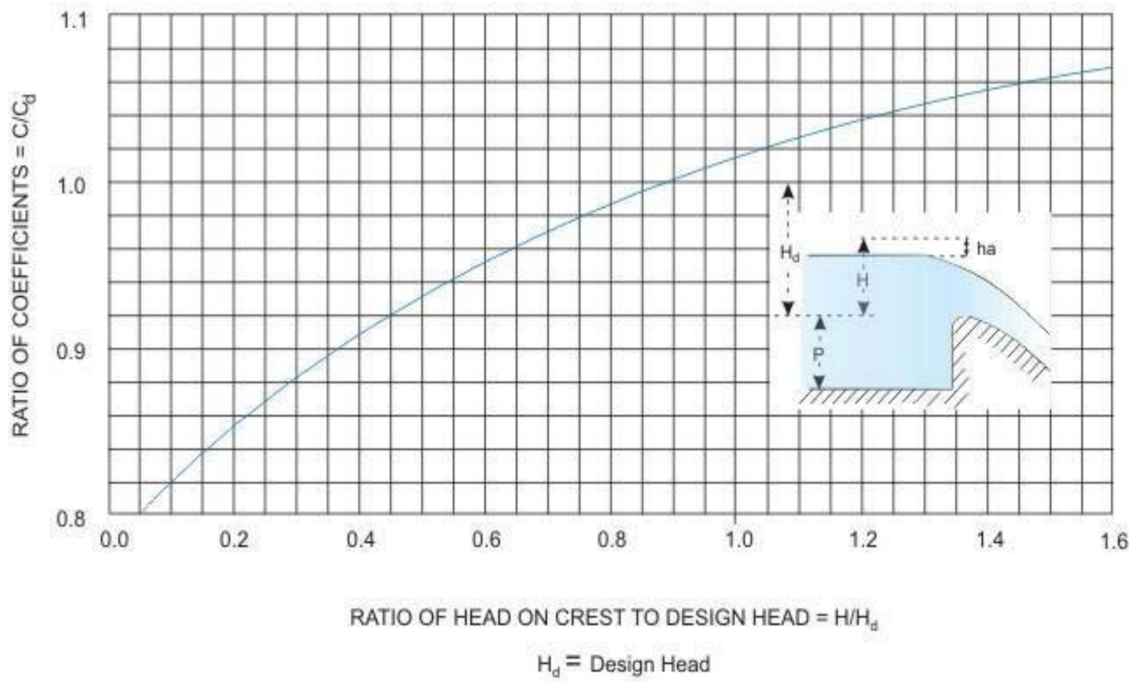


FIGURE 26. Coefficient of discharge other than the design head

3. Effect of upstream face slope

For small ratio of P/H_d where P is the height of the weir and H_d the design head, as for the approach to a chute spillway, increase of the slope of upstream face tends to increase the coefficient of discharge, as shown in Figure 27. This figure shows the ratio of the coefficient for ogee crest with a sloping face to that with vertical face. For large ratios of P/H_d, the effect is a decrease of the coefficient. The coefficient of discharge is reduced for large ratios P/H_d only for relatively flat upstream slopes.

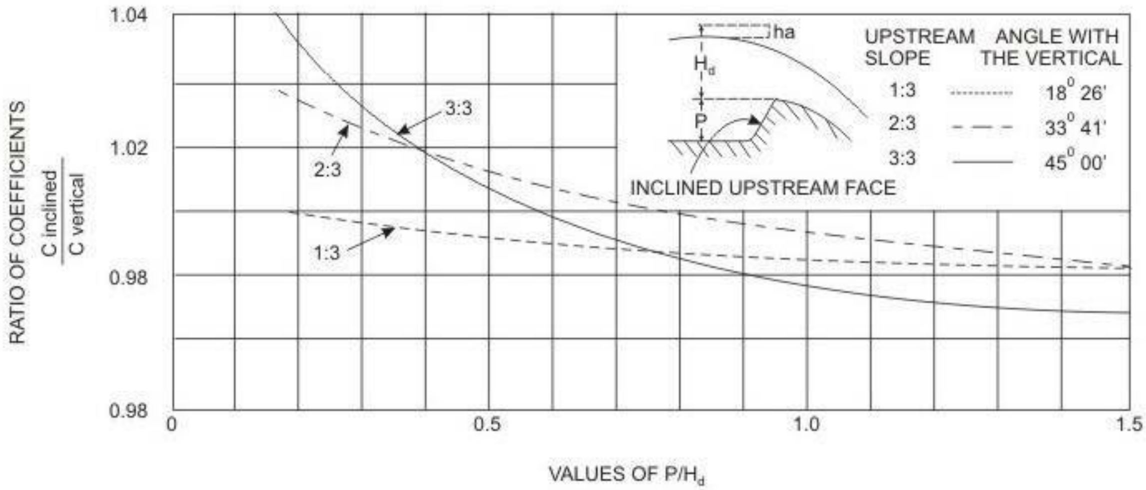


Figure 27. Coefficient of discharge variation with upstream face inclination.

4. Effect of downstream apron interference and downstream submergence

This condition is possible for dams of relatively small heights compared to the natural depth of the river, when the water level downstream of the weir crest is high enough to affect the discharge, the condition being termed as submerged. The conditions that after the coefficient of discharge in this case are the vertical distance from the crest of the over flow to the downstream apron and the depth of flow in the downstream channel, measured above the apron.

Five distinct characteristic flow conditions can occur below an overflow crest, depending on the relative positions of the apron and the downstream water surface:

A. The flow will continue at supercritical stage
B. A partial or incomplete hydraulic jump will occur immediately downstream from the crest
C. A true hydraulic jump will occur
D. A drowned jump will occur in which the high-velocity jet will follow the face of the overflow and then continue in an erratic and fluctuating path for a considerable distance under and through the slower water, and
E. No jump will occur - the jet will break away from the face of the overflow and ride along the surface for a short distance and then erratically intermingle with the slow moving water underneath.

According to USBR (1987), the relationship of the floor positions and downstream submergences which produce these distinctive flows can be shown in a graph as in Figure 28.

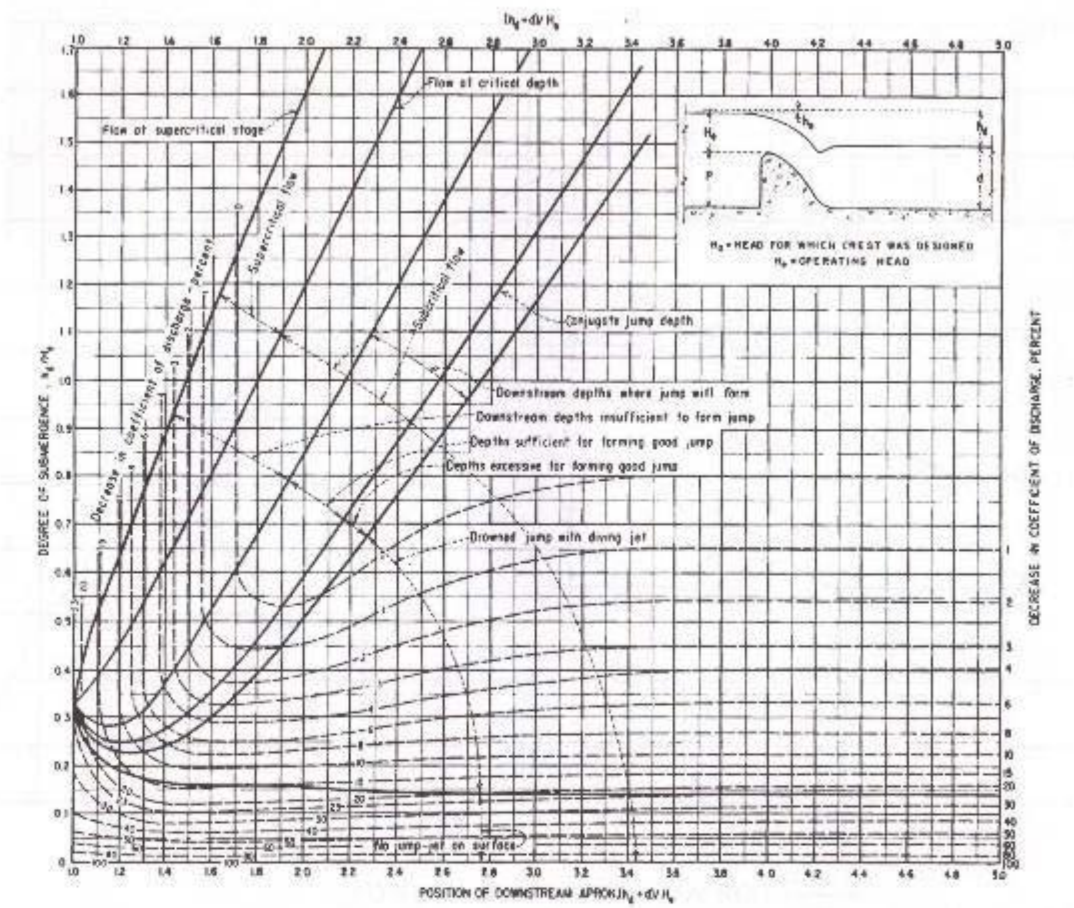


FIGURE 28. Effects of downstream influences on flow over weir crests

(Courtesy: United States Bureau of Reclamation, Design of Small Dams)

Usually for large dams the cases A,B or C with dominate and the decrease in the coefficient of discharge is due principally to the back pressure effect of the downstream apron and is independent of any submergence effect due to tail water. Cases D and E can be expected t o be found in low-height dams like small height diversion or navigation dam. Figure 29, adapted from USBR (1987), shows the effect of downstream apron conditions on the coefficient of discharge. It may be noted that this curve plots the same data represented by the vertical dashed lines of Figure 28 in a slightly different form. As the downstream apron level nears the crest of the overflow $\left(\frac{h_d + d}{H_e}\right.$ approaches 1.0), where h_d is the difference of total energy on upstream and the water level downstream, d is the downstream water depth and H_e is the total energy upstream measured above the crest of the weir, the coefficient of discharge is about 77 percent of that for un-retarded flow. From Figure 29 it can be seen that when the ratio of $\frac{h_d + d}{H_e}$ values exceed about 1.7, the downstream floor position has little effect on the coefficient, but there is a decrease in the coefficient caused by tail water submergence. Figure 30 shows the ratios of the coefficient of discharge where affected by tailwater conditions, to that coefficient for free flow conditions. This curve plots the data represented by the horizontal dashed lines on Figure 28 in a slightly different form. Where the dashed lines of Figure 28 are curved, the decrease in the coefficient is the result of a combination of tail-water effects and downstream apron position.

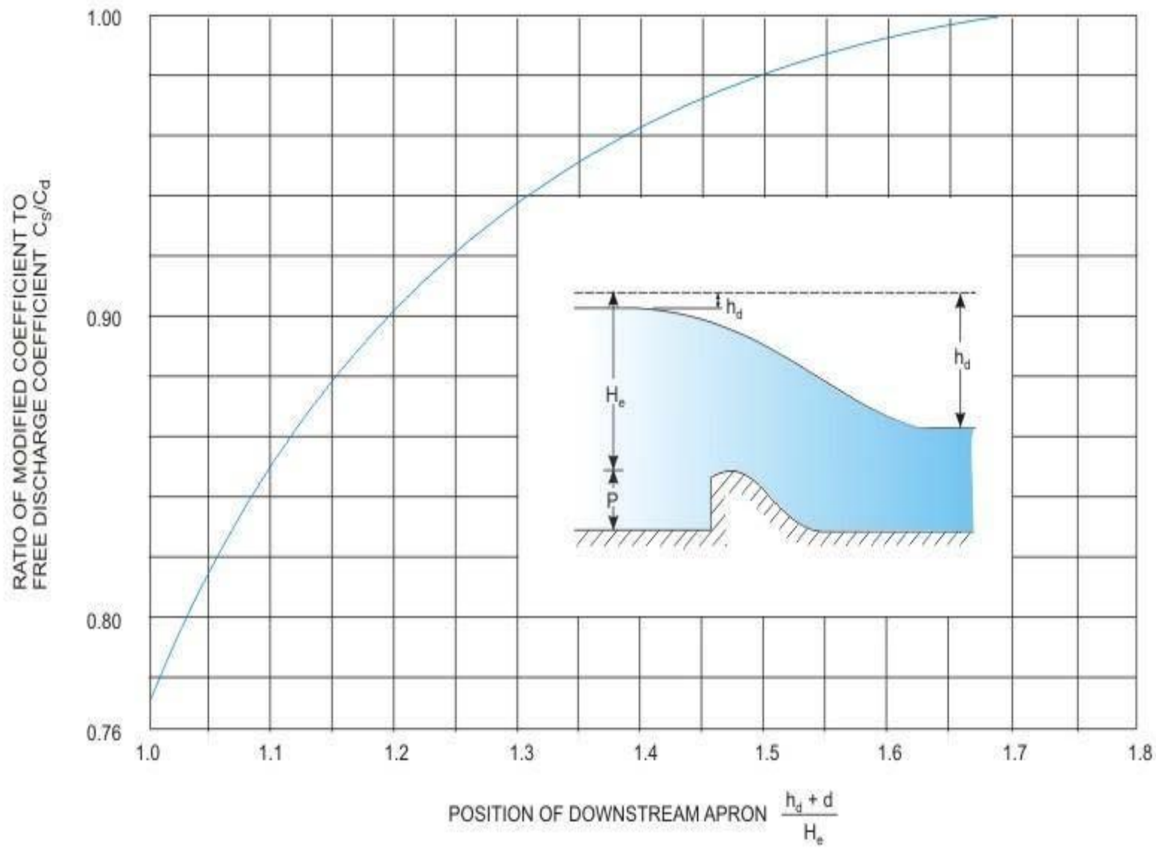


FIGURE 29. Ratio of discharge coefficients resulting from the effect of the apron on flow

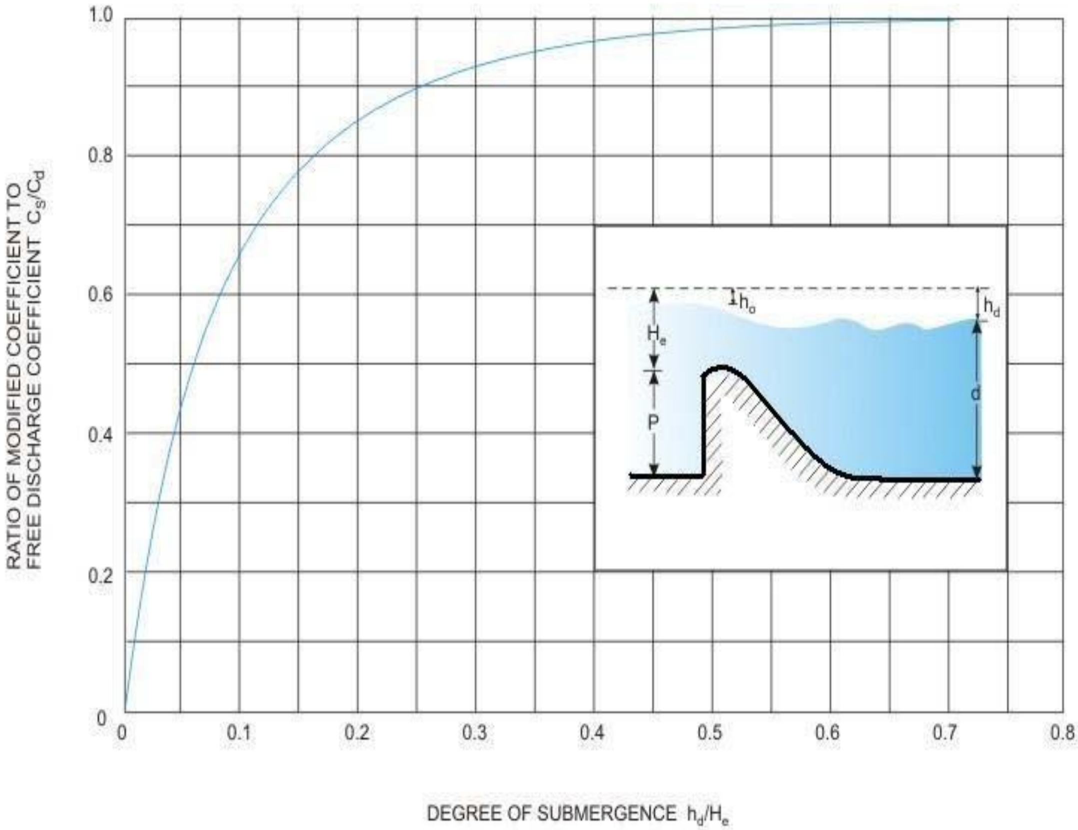


FIGURE 30. Ratio of discharge coefficients caused by tailwater effects

If the ordinate of Figure 30 is changed from $\frac{h_d}{H_e}$ to $1 - \frac{h_d}{H_e}$, that is, equal to $\frac{H_e - h_d}{H_e} = \frac{h}{H_e}$ where h is the downstream water depth measured above crest, then the curve of Figure 30 may be transposed as in Figure 31.

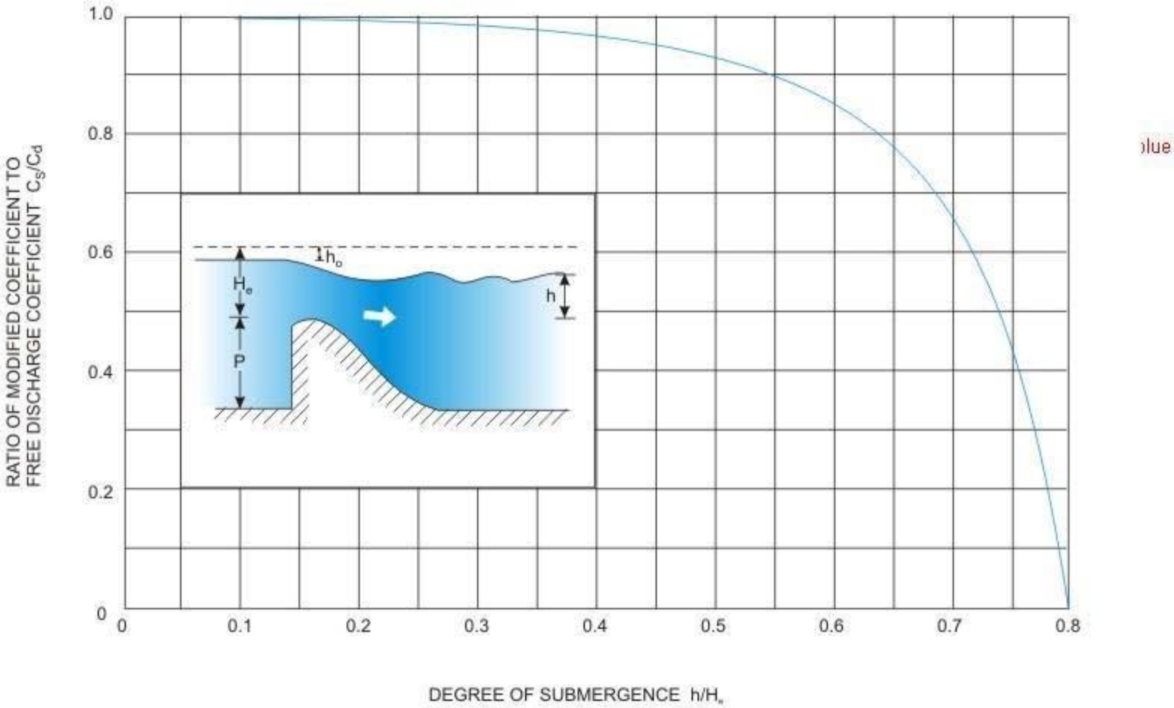


FIGURE 31. Ratio of submerged discharge coefficient to that without submergence effect
This figure is similar to that given in FIGURE 30, but with different ordinate value

The total head on the crest He, does not include allownces for approach channel friction losses due to curvature into the inlet section, and inlet or transition losses. Where the design of the approach channel results in appreciable losses, they must be added to H_e to determine reservoir elevations corresponding to the discharges given by the discharge equation.

Where the crest piers and abutments are shaped to cause side contractions of the overflow, the effective length, L_e, will be the net length of the crest, L. The effect of the end contractions may be taken into account by reducing the net length of crest as given below:

$$L_e = L - 2(N K_p + K_a) H_e$$

Where L, L_e and H have been explained before, N is the number of piers and K_p and K_a are the pier and abutment contraction coefficients. The reason for the reduction of the net length may be appreciated from Figure 32.

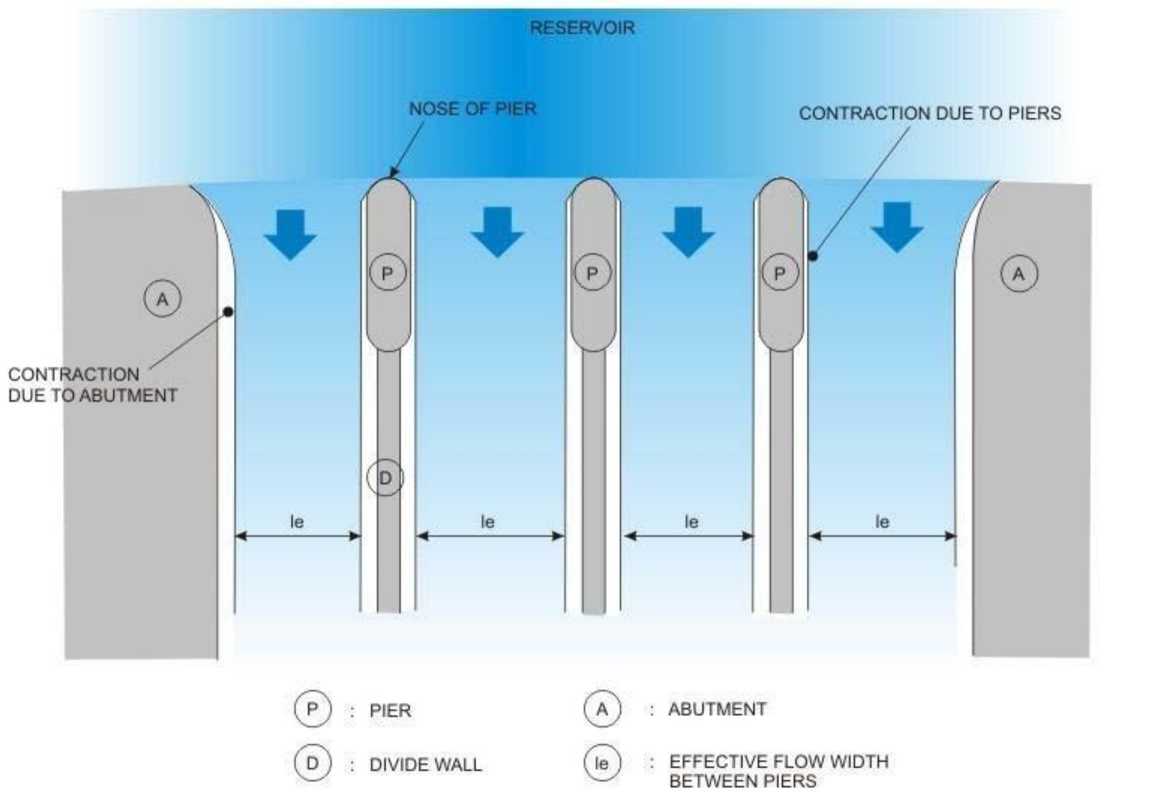


FIGURE 32. Abutment and pier contractions shown on a spillway plan

The pier contraction coefficient K_p depends upon the following factors:

1. Shape and location of the pier nose
2. Thickness of the pier
3. Head in relation to the design head
4. Approach velocity

For the condition of flow at the design head, the average values of pier contraction coefficients may be assumed as shown in Figure 33.

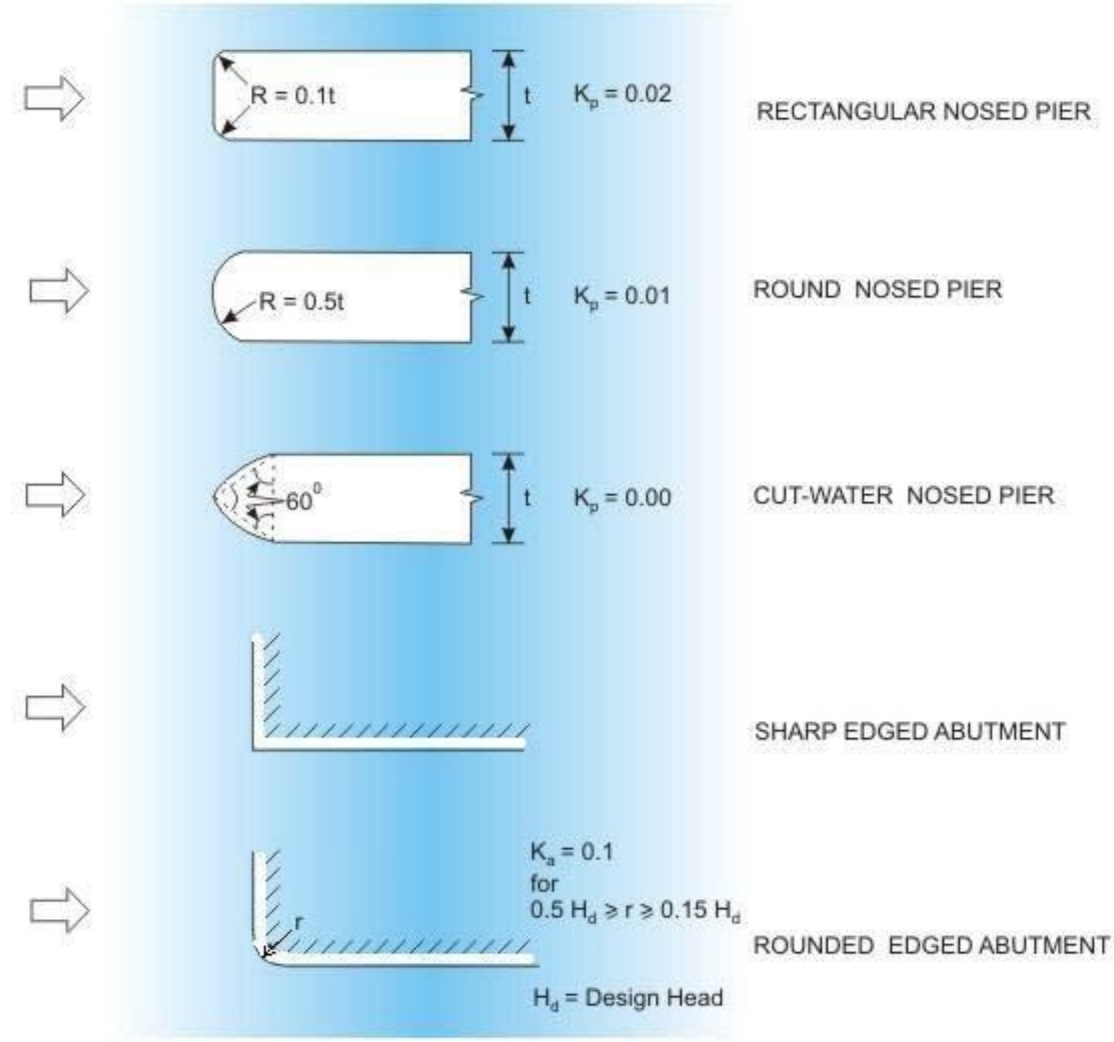


FIGURE 33. Recommended values of K_p and K_a

The abutment contraction coefficient is seen to depend upon the following factors:

1. Shape of abutment
2. Angle between upstream approach wall and the axis of flow
3. Head, in relation to the design head
4. Approach velocity

For the condition of flow at the design head, the average value of abutment contraction coefficients may be assumed as shown in Figure 33.

For flow at head other than design head, the values of K_p and K_a may be obtained from graphical plots given in IS: 6934-1973 "Recommendations for hydraulic design of high ogee overflow spillways".

Discharge characteristics of ogee crests-controlled spillway

The discharge for gated crests at partial gate opening is similar to flow through a low- head orifice and may be computed by the following equation recommended by the Bureau of Indian Standards code IS:6934-1998 "Hydraulic design of high ogee overflow spillways-recommendations".

$$Q = C_g \cdot G_0 \cdot L_e \sqrt{2gH_e} \tag{4}$$

Where Q is the discharge (in m^3/s), C_g is the gated coefficient of discharge, G_0 is the gate opening (in m), L_e is the effective length of crest, g is the acceleration due to gravity, and H_e is the hydraulic head measured from the centre of the orifice (in m).

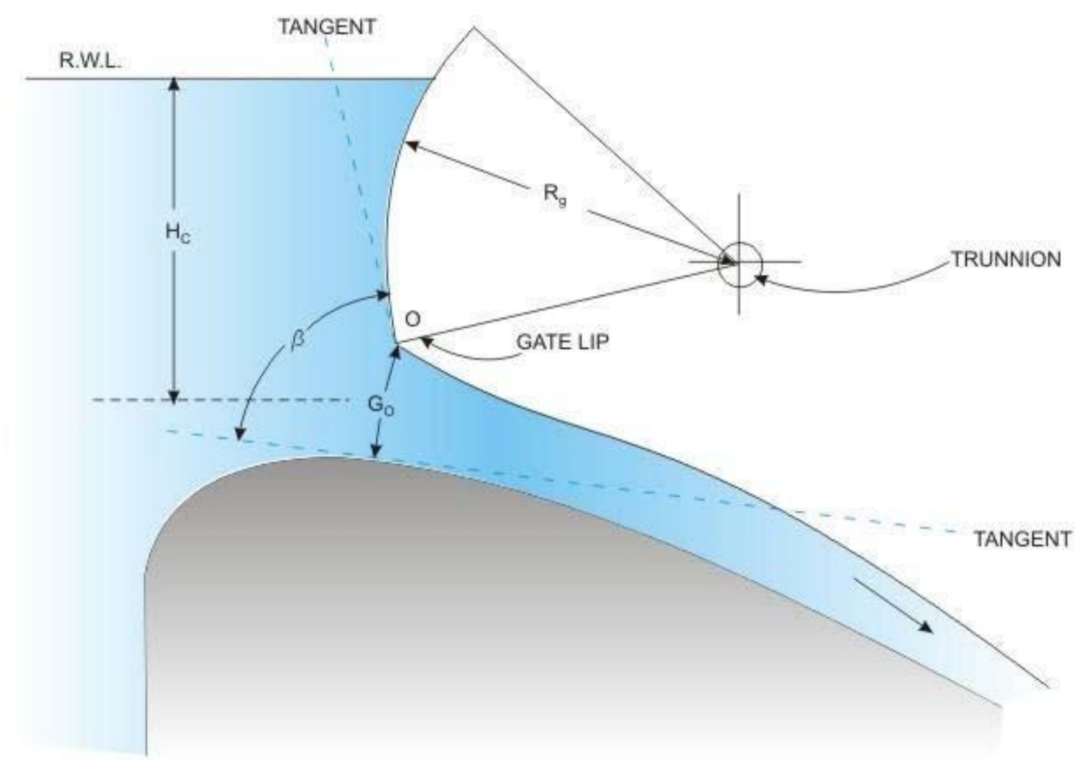


FIGURE 34. Partially opened radial gate discharging flow

Usually for high head spillways, radial gates are common and Figure 34 shows the position of a partially opened radial gate over an ogee-crested spillway. The gate opening G_0 may be seen to be measured as the shortest distance from the gate lip to the ogee crest profile meeting at G. The angle β is seen to be measured between the tangent at G and the tangent of the radial gate at gate lip. Figure 35 presents a curve relating the coefficient of gated discharge C_g with the angle β.

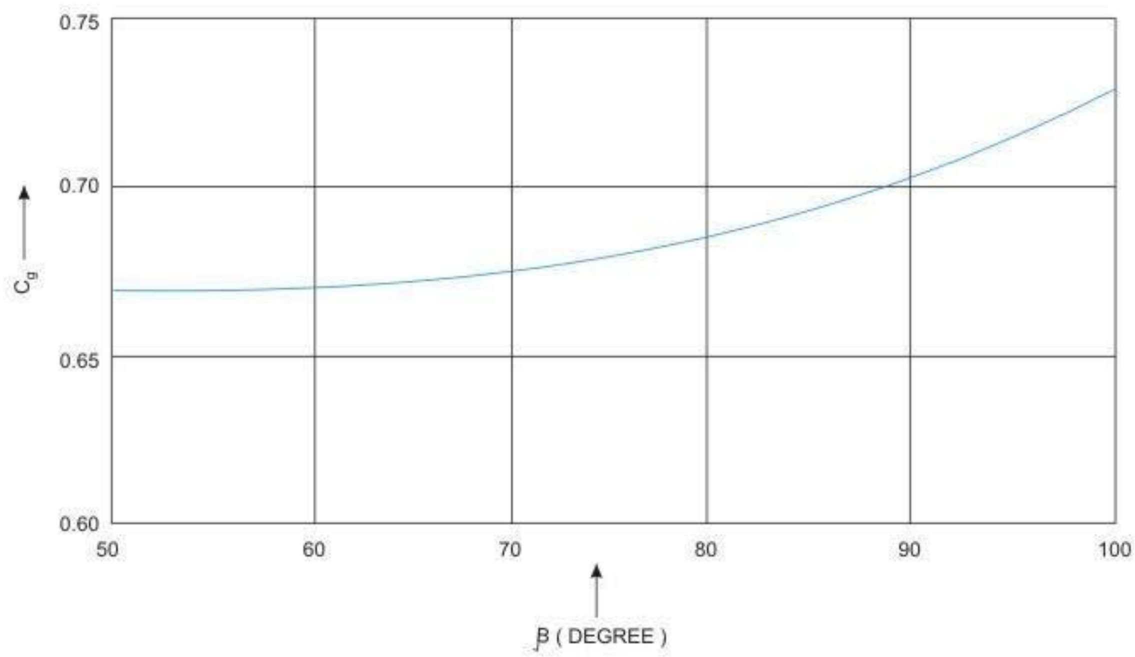


FIGURE 35. Coefficient of gated discharge Cg variation with lip angle β

The curve presents an average value of C_g determined for various approach and downstream conditions and may be used for preliminary design purposes. In fact, it may be noticed that the discharge equation mentioned above for calculating flow through a gated spillway as recommended in IS: 6934-1998 may not be strictly correct as the gate opening becomes larger, comparable to the hydraulic head H_C.

Spillway profile with breast wall

Spillways, generally the ogee-crested type, are sometimes provided with a breast wall from various considerations such as increasing the regulating storage of flood discharge, reducing the height of the gate, minimizing the cost of gate operating mechanism, etc.

For the spillways with breast wall, the following parameters are required to be determined:

a) Profile of the spillway crest including the upstream and downstream quadrants,
b) Profile of the bottom surface of the breast wall, and
c) Estimation of discharge efficiency of the spillway.

The flow through a spillway with breast wall has been idealised as two-dimensional flow through a sharp edged orifice in a large tank. The following guidelines for determining the parameters mentioned above may be used for preparing preliminary designs and studies on hydraulic model may be conducted for confirming or improving on the preliminary design. Figure 36 shows pertinent details of various profiles of the spillway with a breast wall.

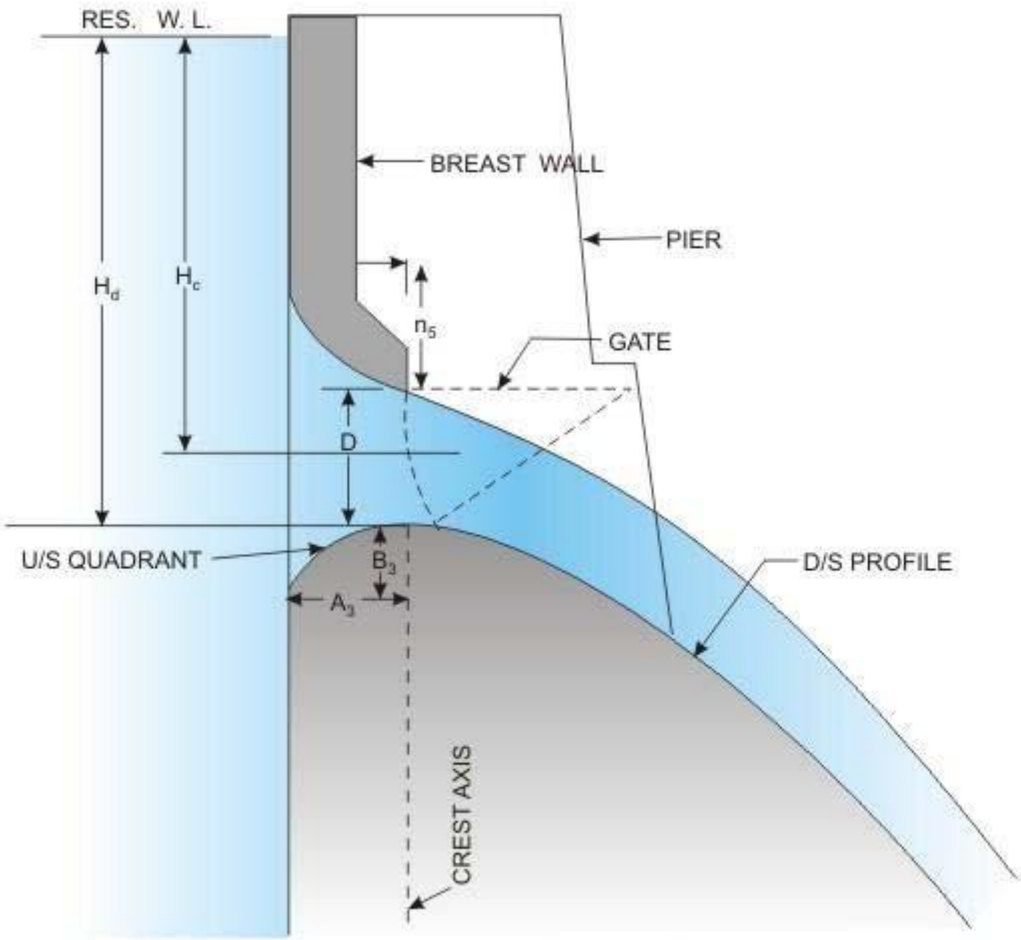


FIGURE 36. Spillway with breast wall

Ogee Profile - Upstream Quadrant'

The upstream quadrant may conform to an ellipse with the equation:

$$\frac{X^2}{A_3^2} + \frac{Y^2}{B_3^2} = 1 \qquad (5)$$

where

$$A_3 = 0.541\ D\ (H_d/D)^{0.32}_{0.04} \text{ and}$$

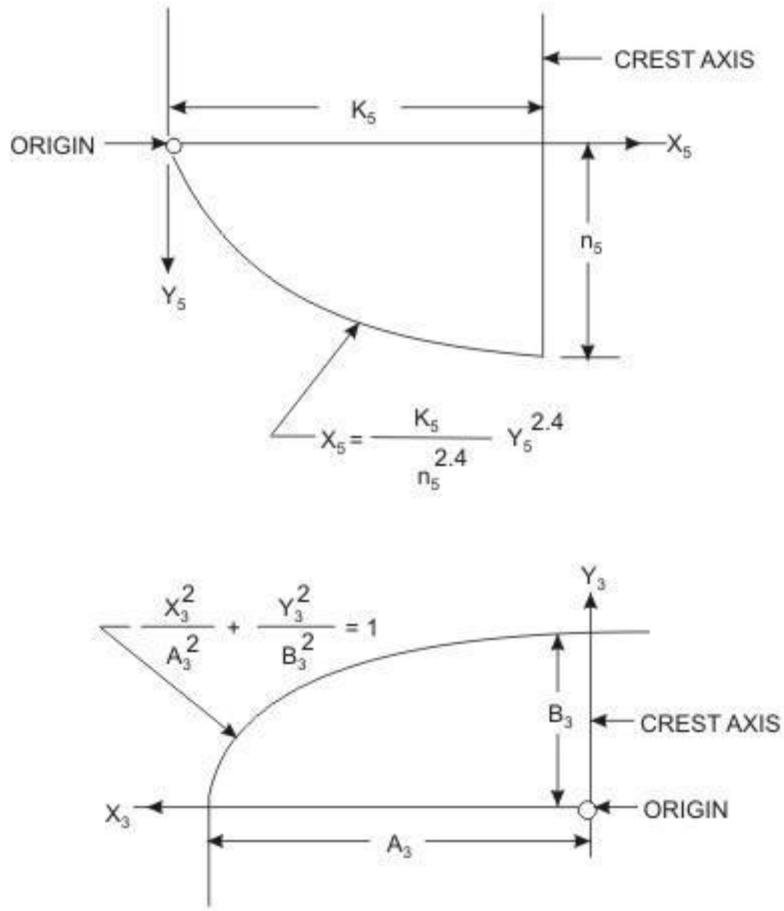


FIGURE 37. Upstream profile of ogee crest and bottom profile of breast wall details

Discharge Computation

The discharge through the breast wall spillway may be estimated by the equation:

$$Q \; C_b \; L \; D \left\{ 2g \left(\; c \; \frac{v_a^2}{2g} \right) \right\}^{0.5} \qquad (12)$$

The following equation relates C_h with the parameter (H/H_d) in the range of H/H_d = 0.8 to 1.33.

$$C_b = 0.148631 + 0.945305(H/H_d) - 0.326238(H/H_d)^2 \quad (13)$$

Typical values of C_b are:

0.80	0.696
1.00	0.769
1.15	0.797

Selection of spillways

The Bureau of Indian Standards code IS: 10137-1982 "Guidelines for selection of spillways and energy dissipators" provide guidelines in choosing the appropriate type of spillway for the specific purpose of the project. The general considerations that provide the basic guidelines are as follows:

Safety Considerations Consistent with Economy

Spillway structures add substantially to the cost of a dam. In selecting a type of spillway for a dam, economy in cost should not be the only criterion. The cost of spillway must be weighed in the light of safety required below the dam.

Hydrological and Site Conditions

The type of spillway to be chosen shall depend on:

a) Inflow flood;
b) Availability of tail channel, its capacity and flow hydraulics;
c) Power house, tail race and other structures downstream; and
d) Topography

Type of Dam

This is one of the main factors in deciding the type of spillway. For earth and rockfill dams, chute and ogee spillways are commonly provided, whereas for an arch dam a free fall or morning glory or chute or tunnel spillway is more appropriate. Gravity dams are mostly provided with ogee spillways.

Purpose of Dam and Operating Conditions

The purpose of the dam mainly determines whether the dam is to be provided with a gated spillway or a non-gated one. A diversion dam can have a fixed level crest, that is, non-gated crest.

Conditions Downstream of a Dam

The rise in the downstream level in heavy floods and its consequences need careful consideration. Certain spillways alter greatly the shape of the hydrograph downstream of a dam. The discharges from a siphon spillway may have surges and break-ups as priming and depriming occurs. This gives rise to the wave travelling downstream in the river, which may be detrimental to navigation and fishing and may also cause damage to population and developed areas downstream.

Nature and Amount of Solid Materials Brought by the River

Trees, floating debris, sediment in suspension, etc, affect the type of spillway to be provided. A siphon spillway cannot be successful if the inflow brings too much of floating materials. Where big trees come as floating materials, the chute or ogee spillway remains the common choice.

Apart from the above, each spillway can be shown as having certain specific advantages under particular site conditions. These are listed below which might be helpful to decide which spillway to choose for a particular project.

Ogee Spillway

It is most commonly used with gravity dams. However, it is also used with earth and rockfill dams with a separate gravity structure; the ogee crest can be used as control in almost all types of spillways; and it has got the advantage over other spillways for its high discharging efficiency.

Chute Spillway

a) It can be provided on any type of foundation,
b) It is commonly used with the earth and rockfill dams, and
c) It becomes economical if earth received from spillway excavation is used in dam construction.

The following factors limit its adaption:
a) It should normally be avoided on embankments;
b) Availability of space is essential for keeping the spillway basins away from the dam paving; and
c) If it is necessary to provide too many bends in the chute because of the topography, its hydraulic performance can be adversely affected.

Side Channel Spillways

This type of spillway is preferred where a long overflow crest is desired in order to limit the intensity of discharge, It is useful where the abutments are steep, and it is useful where the control is desired by the narrow side channel.

The factor limiting its adoption is that this type of spillway is hydraulically less efficient.

Shaft Spillways (Morning Glory Spillway)

a) This can be adopted very advantageously in dam sites in narrow canyons, and
b) Minimum discharging capacity is attained at relatively low heads. This characteristic makes the spillway ideal where the maximum spillway outflow is to be limited. This characteristic

becomes undesirable where a discharge more than the design capacity is to be passed. So, it can be used as a service spillway in conjunction with an emergency spillway.

The factor limiting its adoption is the difficulty of air-entrainment in a shaft, which may escape in bursts causing an undesirable surging.

Siphon Spillway

Siphon spillways can be used to discharge full capacity discharges, at relatively low heads, and great advantage of this type of spillway is its positive and automatic operation without mechanical devices and moving parts.

The following factors limit the adoption of a siphon spillway:

It is difficult to handle flows materially greater than designed capacity, even if the reservoir head exceeds the design level; Siphon spillways cannot pass debris, ice, etc; There is possibility of clogging of the siphon passage way and breaking of siphon vents with logs and debris; In cold climates, there can be freezing inside the inlet and air vents of the siphon; When sudden surges occur and outflow stops; The structure is subject to heavy vibrations during its operation needing strong foundations; and Siphons cannot be normally used for vacuum heads higher than 8 m and there is danger of cavitation damage.

Overfall or Free Fall Spillway

This is suitable for arch dams or dams with downstream vertical faces; and this is suitable for small drops and for passing any occasional flood.

Tunnel or Conduit Spillway

This type is generally suitable for dams in narrow valleys, where overflow spillways cannot be located without risk and good sites are not available for a saddle spillway. In such cases, diversion tunnels used for construction can be modified to work as tunnel spillways. In case of embankment dams, diversion tunnels used during construction may usefully be adopted. Where there is danger to open channels from snow or rock slides, tunnel spillways are useful.

Energy dissipators

Different types of energy dissipators may be used along with a spillway, alone or in combination of more than one, depending upon the energy to be dissipated and erosion control required downstream of a dam. Broadly, the energy dissipators are classified under two categories – Stilling basins or Bucket Type. Each of these are further sub- categorized as given below.

Stilling basin type energy dissipators

They may fundamentally be divided into two types.

a) Hydraulic jump type stilling basins

1. Horizontal apron type (Figure 38)

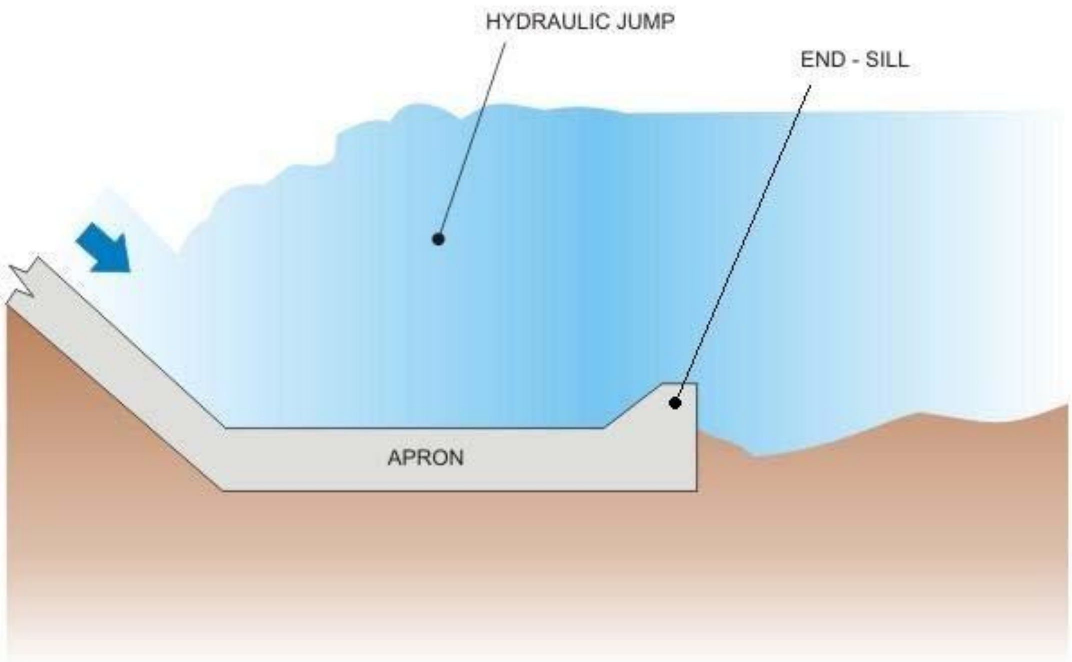


FIGURE 38. Horizontal apron stilling basin with end-sill

2. Sloping apron type (Figure 39)

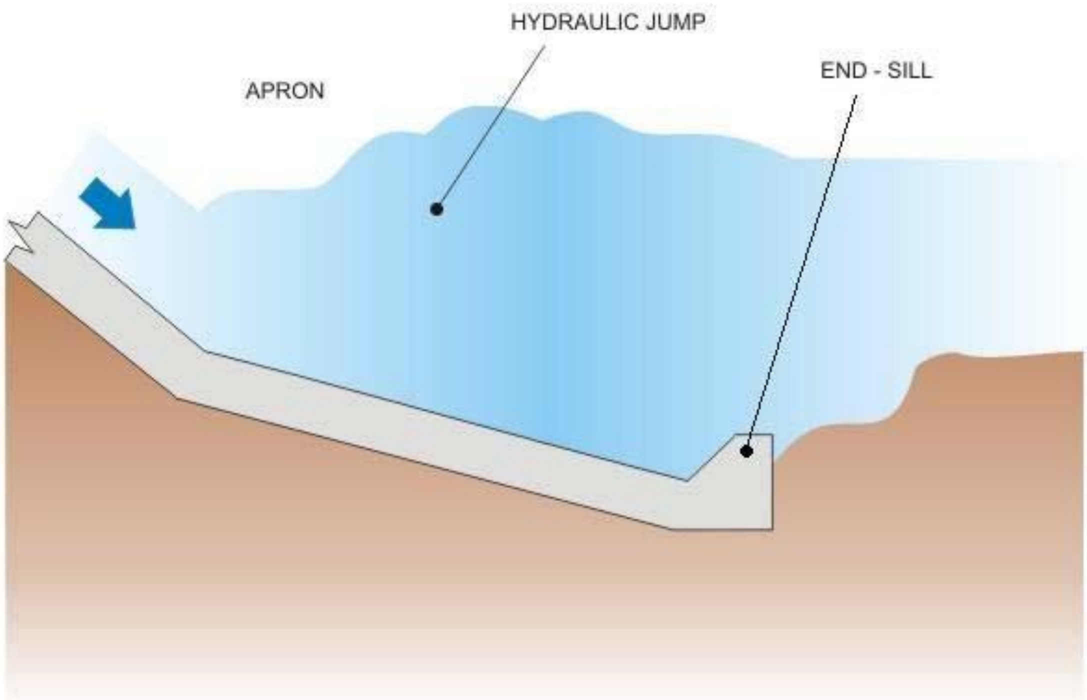


FIGURE 39. Sloping apron stilling basin with end-sill

b) Jet diffusion type stilling basins

1. Jet diffusion stilling basins (Figure 40)

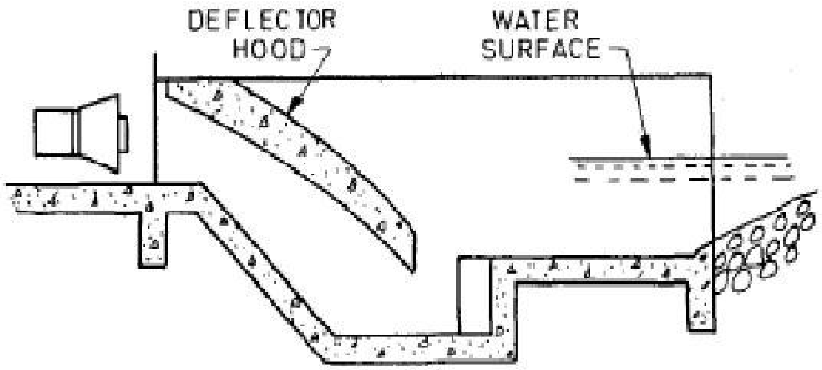
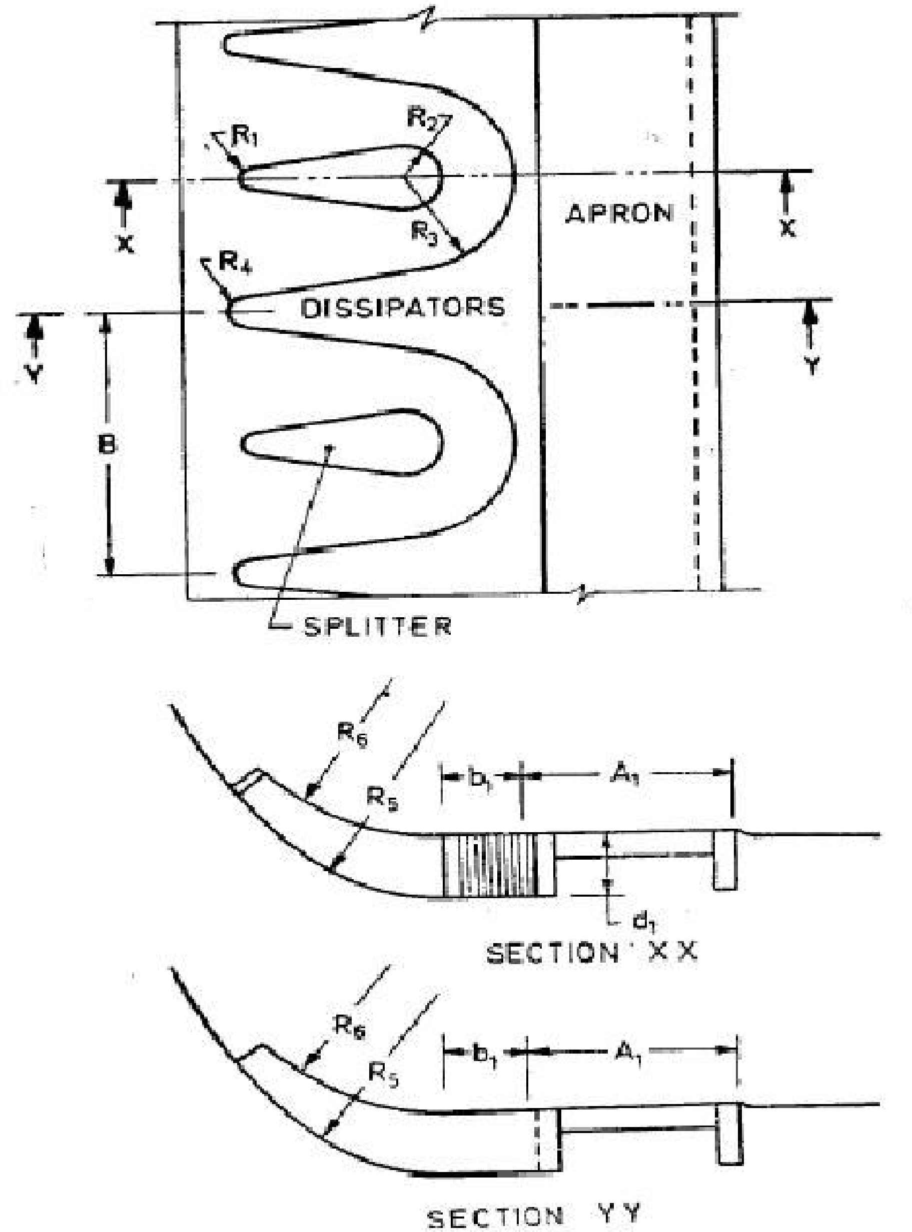


FIGURE 40. Jet diffusion stilling basin

(Image courtesy: IS 10137)

2. Interacting jet dissipators (Figure 41)



$R_1 = 0{\cdot}03B$
$R_2 = 0{\cdot}13B$
$R_3 = 0{\cdot}37B$
$R_4 = 0{\cdot}03B$
$R_5 = 0{\cdot}66B$

$R_6 = 0{\cdot}66B$
$d_1 = 0{\cdot}18B$
$A_1 = 0{\cdot}71B$
$b_1 = 0{\cdot}25B$

where B is the bay width

FIGURE 41. Interacting jet dissipators

(Image courtesy: IS 10137)

3. Free jet stilling basins (Figure 42)

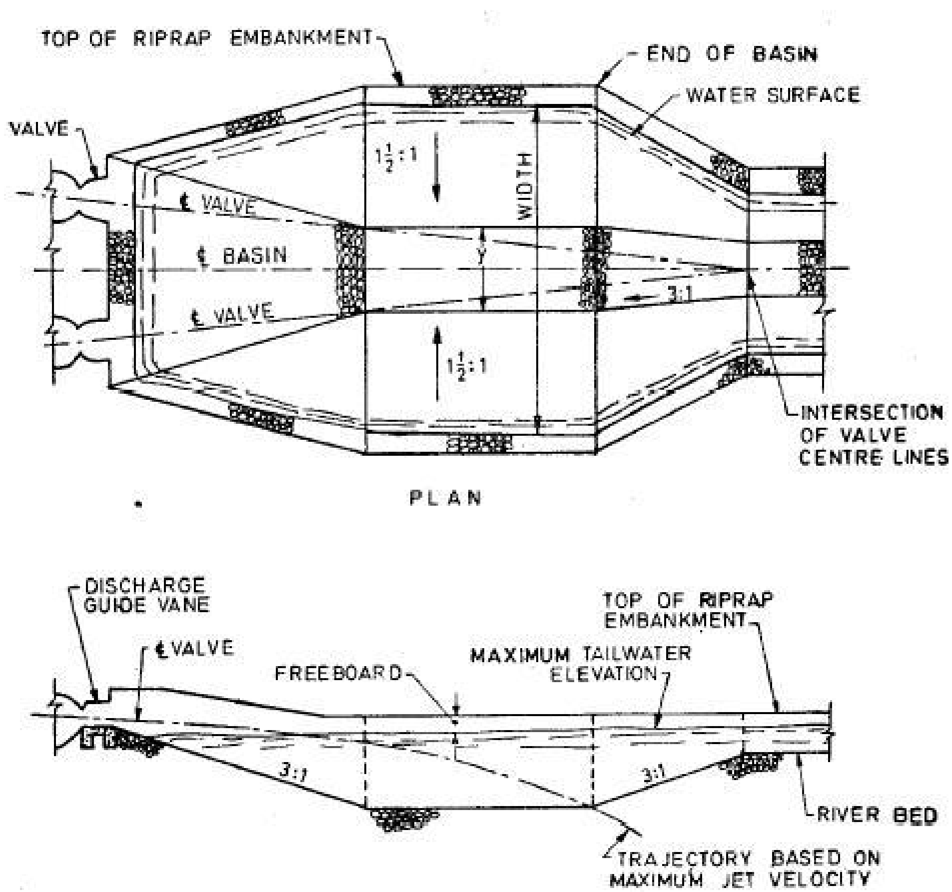


FIGURE 42. Free jet stilling basin

(Image courtesy: IS 10137)

4. Hump stilling basins (Figure 43)

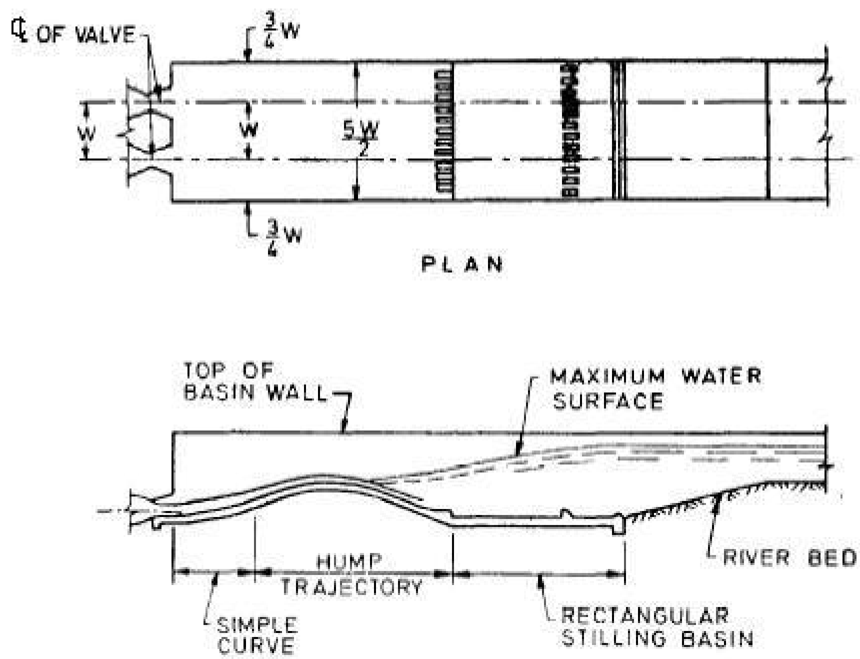


FIGURE 43. Hump stilling basin

(Image courtesy: IS 10137)

5. Impact stilling basins (Figure 44)

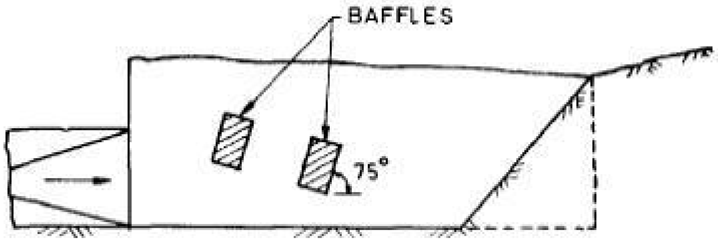


FIG. 15 IMPACT STILLING BASIN WITH INCLINED BAFFLES

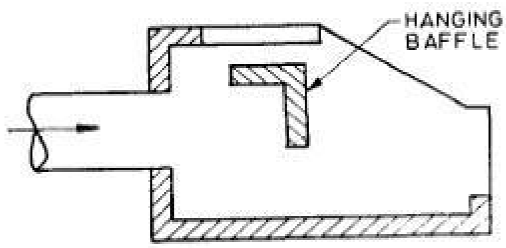


FIGURE 44. Impact stilling basin

(Image courtesy: IS 10137)

Bucket type energy dissipators

This type of energy dissipators includes the following:

1. Solid roller bucket
2. Slotted roller bucket
3. Ski jump (Flip/Trajectory) bucket

The shapes of the different types of bucket-type stilling basins have been given in section 4.8.14. Usually the hydraulic jump type stilling basins and the three types of bucket-type energy dissipators are commonly used in conjunction with spillways of major projects. The detailed designs of these are dealt in subsequent sections.

Since energy dissipators are an integral part of a dam's spillway section, they have to be viewed in conjuction with the latter. Two typical examples have been shown in Figures 45 and 46, though it must be remembered that any type of energy dissipator may go with any type of spillway, depending on the specific site conditions.

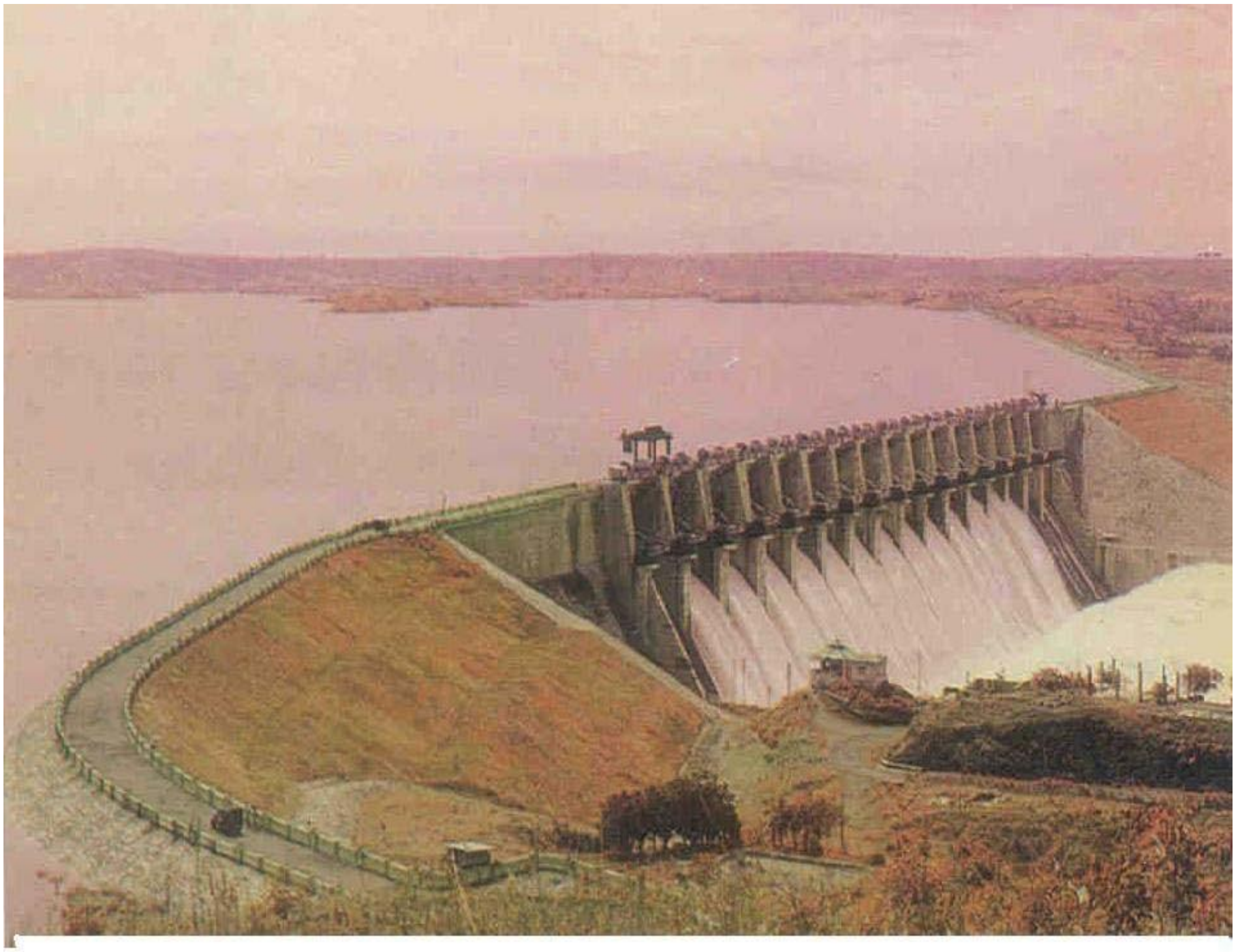


FIGURE 45. Mahi Bajaj Sagar Dam across river Mahi in Rajasthan showing ski-jump bucket energy dissipators in action

(Image courtesy: Web-site of Ministry of Water Resources, Government of India)

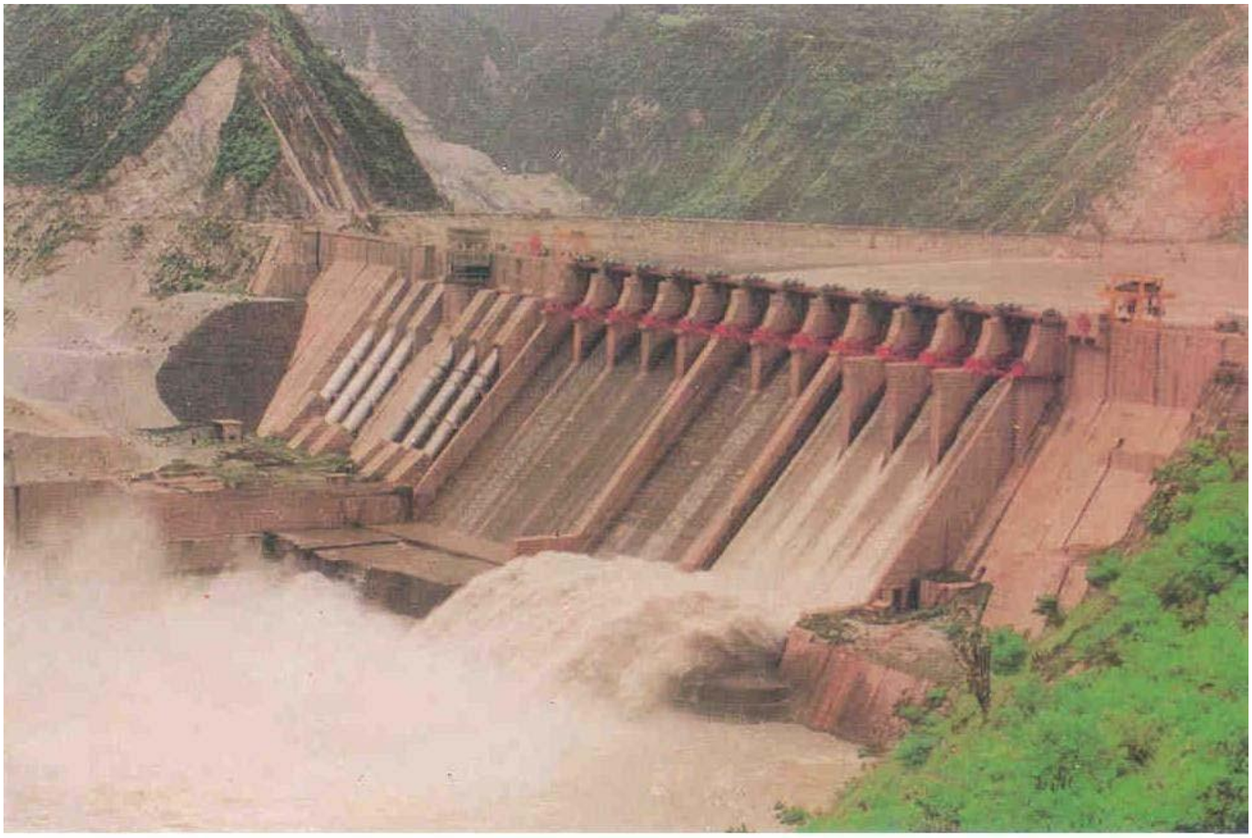
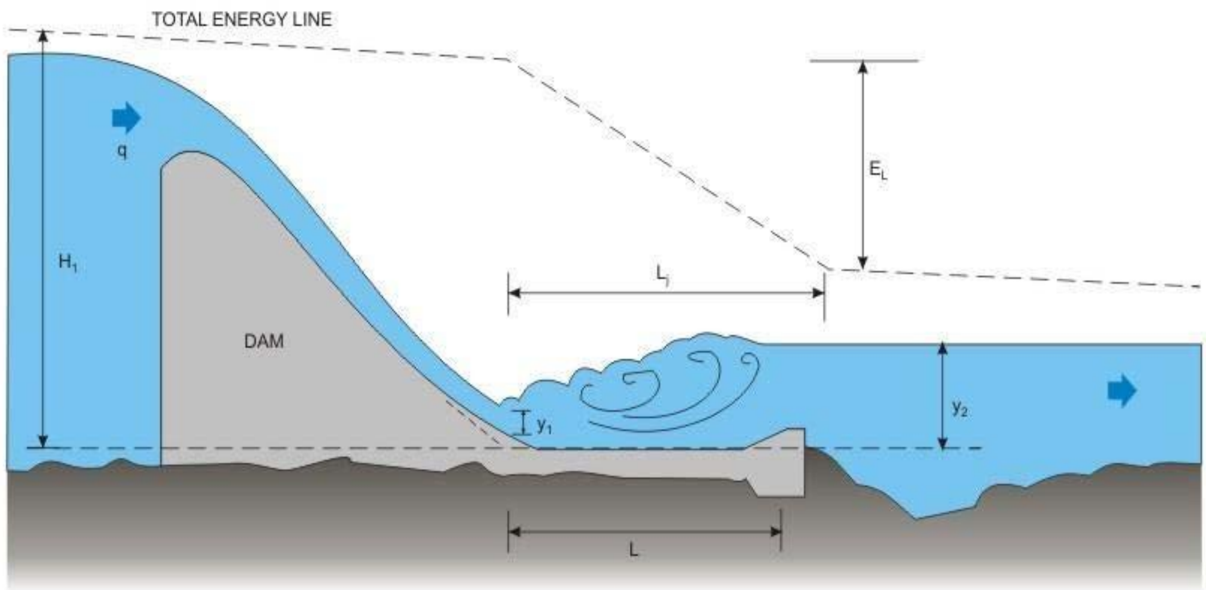


FIGURE 46. Salal project on river Chenab showing energy being dissipated by ski-jump bucket type energy dissipators

(Image courtesy: Web-site of Ministry of Water Resources, Government of India)

Design of Hydraulic Jump Stilling Basin type energy dissipators

A hydraulic jump is the sudden turbulent transition of supercritical flow to subcritical. This phenomena, which involves a loss of energy, is utilized at the bottom of a spillway as an energy dissipator by providing a floor for the hydraulic jump to take place (Figure 47). The amount of energy dissipated in a jump increases with the rise in Froude number of the supercritical flow.



L : LENGTH OF STILLING BASIN APRON

L_j : LENGTH OF HYDRAULIC JUMP

q : DISCHARGE PER UNIT WIDTH

H_1 : TOTAL ENERGY UPSTREAM OF JUMP

y_1 : PRE-JUMP (SUPER CRITICAL FLOW) DEPTH

y_2 : POST-JUMP (SUB CRITICAL FLOW) DEPTH

E_L : ENERGY LOST IN JUMP

FIGURE 47. Definition sketch of hydraulic jump & associated parameters

The two depths, one before (**y1**) and one after (**y2**) the jump are related by the following expression:

$$\frac{y_1}{y_2} = \frac{1}{2}\left(-1+\sqrt{1+8F_1^2}\right) \qquad (14)$$

Where F_1 is the incoming Froude number = $\frac{V_1}{\sqrt{gy_1}}$

Alternatively, the expression may be written in terms of the outgoing Froude number F_2 $\left(=\frac{V_2}{\sqrt{gy_2}}\right)$ as

$$\frac{y_2}{} = \frac{1}{2}\left(-1+\sqrt{1+8F_2^2}\right)$$

$$y_1 \quad 2 \tag{15}$$

where **V1** and **V2** are the incoming and outgoing velocities and ***g*** is the acceleration due to gravity.

The energy lost in the hydraulic jump (**E_L**) is given as:

$$E_L = \frac{\left(y_2 - y_1\right)^3}{4\, y_1 y_2} \qquad (16)$$

In most cases, it is possible to find out the pre-jump depth (**y_1**) and velocity (**V_1**) from the given value of discharge per unit width (**q**) through the spillway. This is done by assuming the total energy is nearly constant right from the spillway entrance up to the beginning of the jump formation, as shown in Figure 47. **V_1** may be assumed to be equal to $\sqrt{2gH_1}$ **,** where **H_1** is the total energy upstream of the spillway, and neglecting friction losses in the spillway. The appropriate expressions may be solved to find out the post- jump depth (**y_2**) and velocity (**V_2**).

The length of the jump (**L_j**) is an important parameter affecting the size of a stilling basin in which the jump is used. There have been many definitions of the length of the jump, but it is usual to take the length to be the horizontal distance between the toe of the jump upto a section where the water surface becomes quite level after reaching a maximum level. Because the water surface profile is very flat towards the end of the jump, large personal errors are introduced in the determination of the jump length.

Bradley and Peterka (1975) have experimentally found the length of hydraulic jumps and plotted them in terms of the incoming Froude number (**F_1**), and post-jump depth (**y_2**) as shown in Figure 48. It is evident that while **L_j/y_2** varies most for small values of **F_1**, at higher values, say above 5 or so, **L_j/y_2** is practically constant at a value of about 6.1.

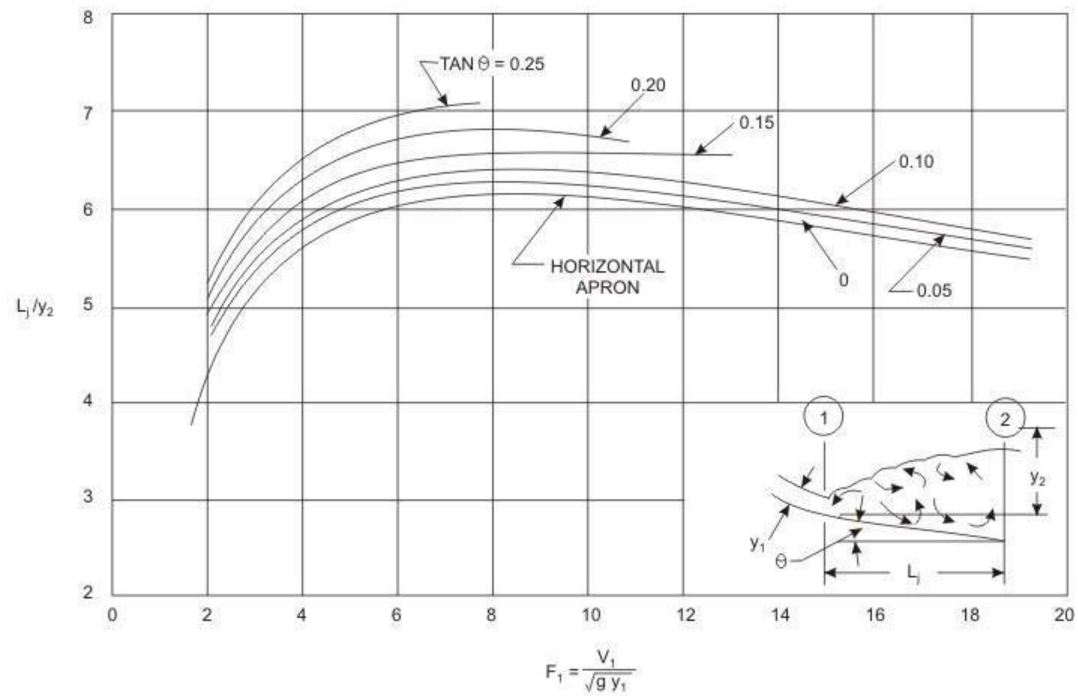


FIGURE 48. Length of hydraulic jump on a horizontal or inclined floor

The depth of water in the actual river downstream of the stilling basin (**y2***) is determined from the river flow observations that have been plotted as a stage-discharge curve (Figure 49).

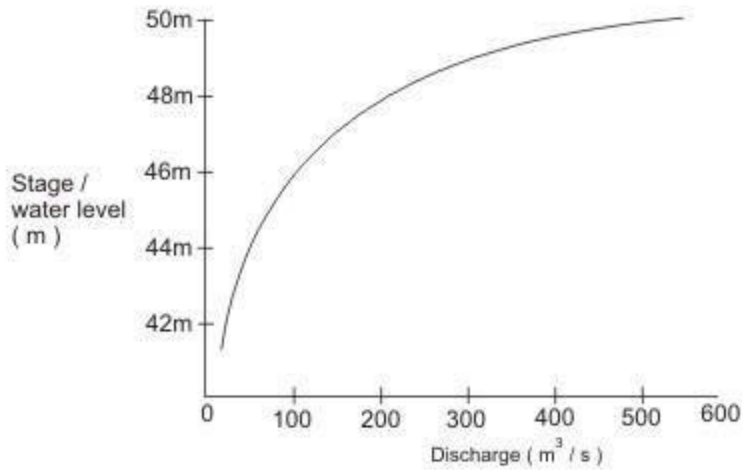


FIGURE 49. A typical stage-discharge curve for a river

Subtracting the stilling basing apron level from the stage or water level corresponding to the total discharge passing through the spillway gives the tail-water depth (**$y2^*$**). Since the stage-discharge curve gives indications about the tail-water of the spillway, it is called the Tail-Water Rating Curve (TRC), usually expressed as the water depth (**$y2^*$**) versus unit discharge (**q**), as shown in Figure 50(a).

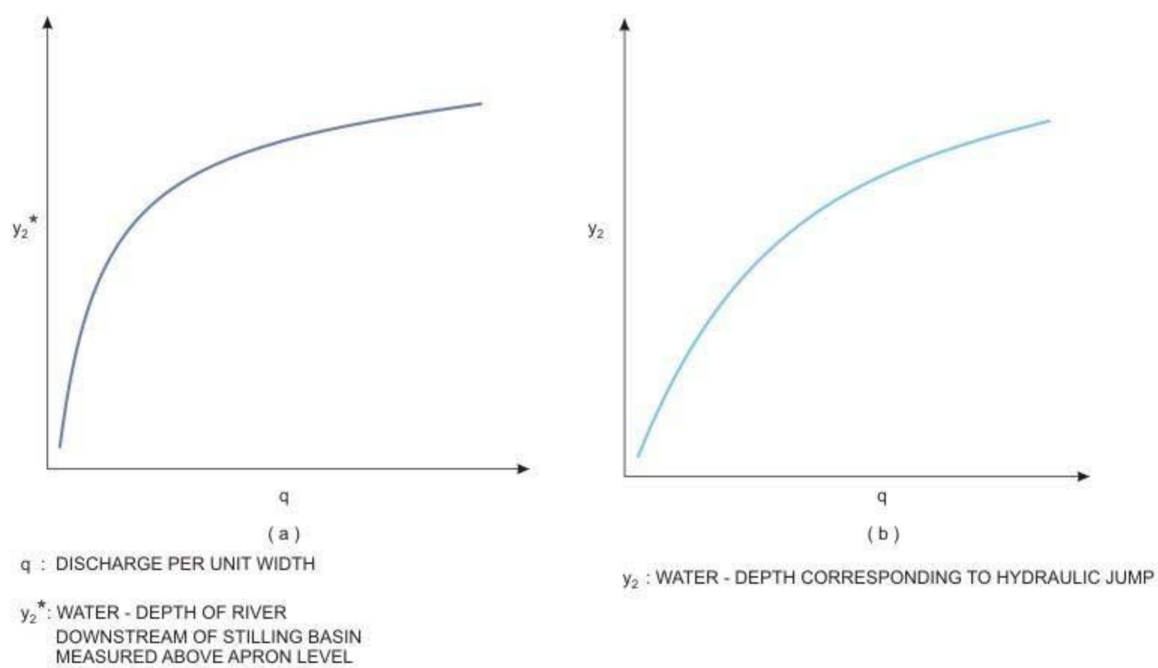


FIGURE 50. Water Level curves
(a) : Tail - water Rating Curve (TRC)
(b) : Jump Rating Curve (JRC)

At the same time, using the formula relating unit discharge (**q**) with the post-jump depth (**y2**), a similar graph may be obtained, as shown in Figure 50(b). Since this graph gives indication about the variation of the post-jump depth, it is called the Jump Rating Curve (JRC).

In general, the JRC and TRC would rarely coincide, if plotted on the same graph, as shown in Figure 51.

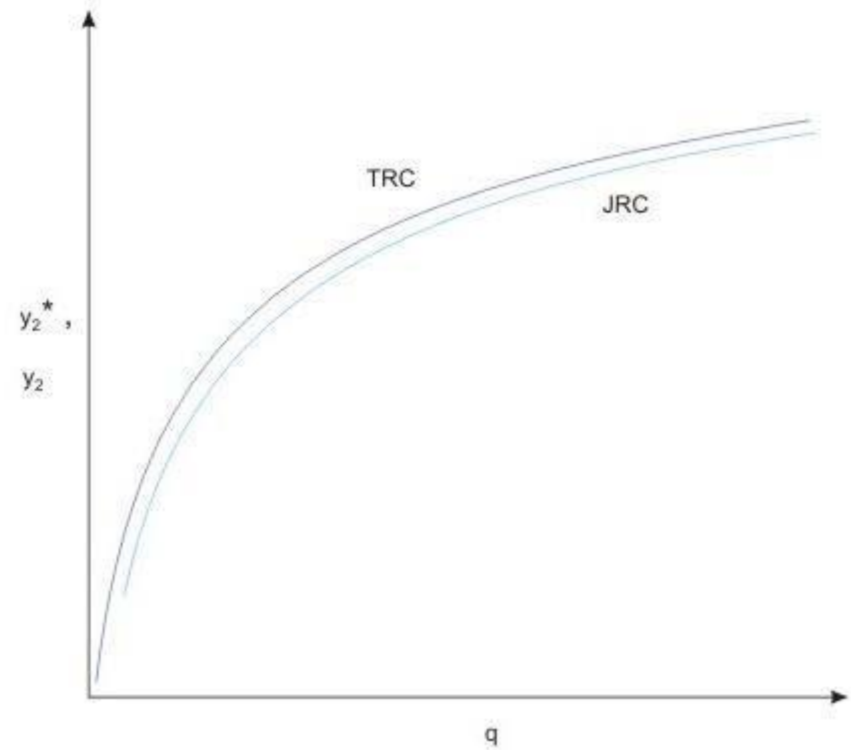


FIGURE 51. TRC & JRC coinciding

At times, the TRC may lie completely below the JRC (Figure 52), for all discharges, in which case the jump will be located away from the toe of the spillway resulting in possible erosion of the riverbed.

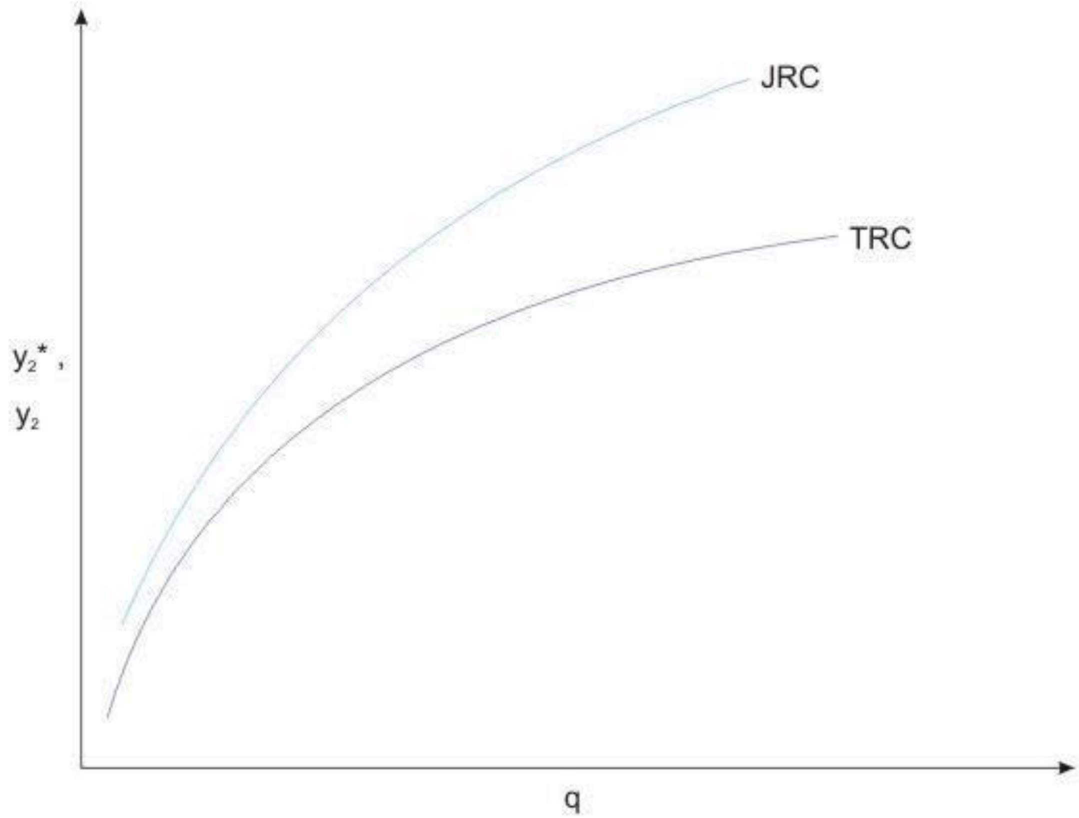


FIGURE 52. TRC below JRC for all discharges

If the TRC is completely above the jump would be located so close to the spillway to make it submerged which may not dissipate the energy completely. (Figure 53)

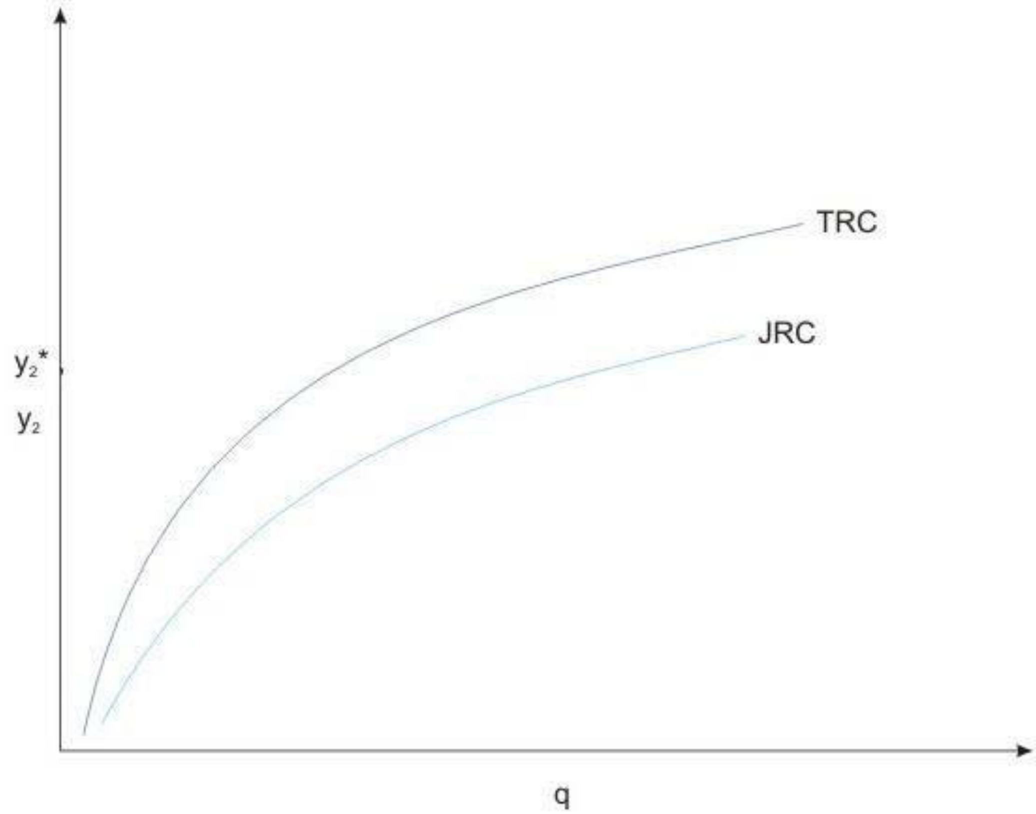


FIGURE 53. TRC above JRC for all discharges

It may also be possible in actual situations that the TRC may be below the JRC for some discharges above for the rest, as shown in Figs. 54 and 55.

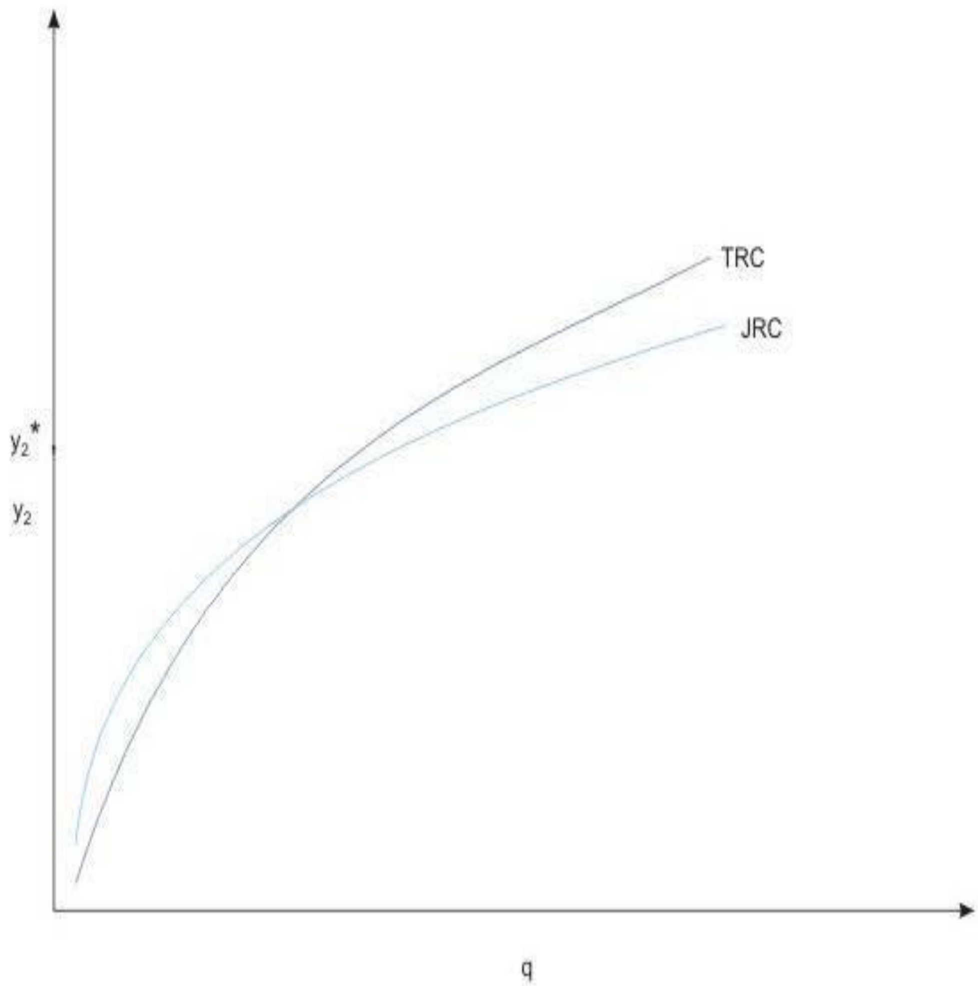


FIGURE 54. TRC below JRC for low discharges and above for high discharges

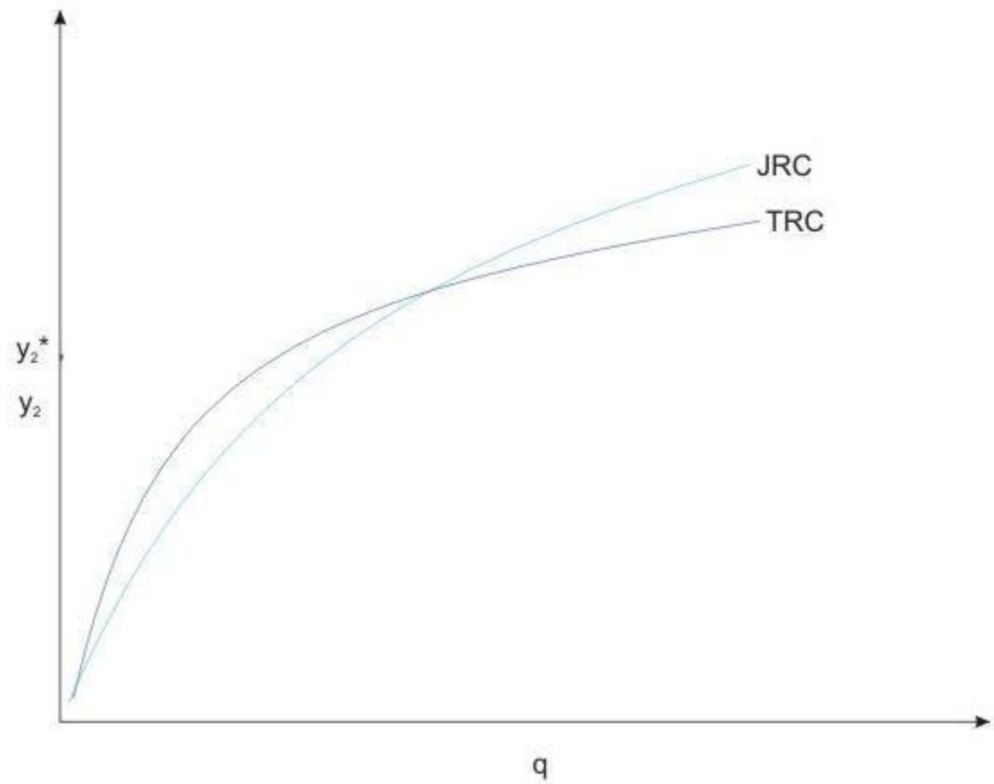


FIGURE 55. TRC above JRC for low discharges and below for high discharges

In these cases two, favourable location of jump may not be possible. In view of the above situations, the following recommendations have been made for satisfactory performance of the hydraulic jumps.

Case1 (Figure 51)
This is the ideal case in which the horizontal apron provided on the riverbed downstream from the toe of the spillway would suffice. The length of the apron should be equal to the length of the jump corresponding to the maximum discharge over the spillway.

Case2 (Figure 52)
It is apparent that the tail water depth as provided by the natural river is not sufficiently for the jump to form. This may be over come by providing a stilling basin apron that is depressed below the average riverbed level (Figure56) or by providing a sill or baffle of sufficient height at the end of the spillway (Figure 57)

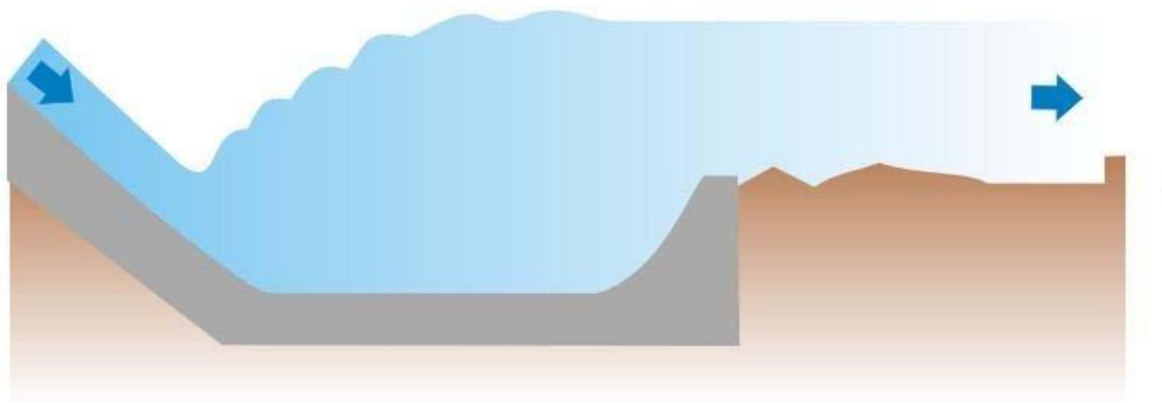


FIGURE 56. Depressed floor of stilling basin apron

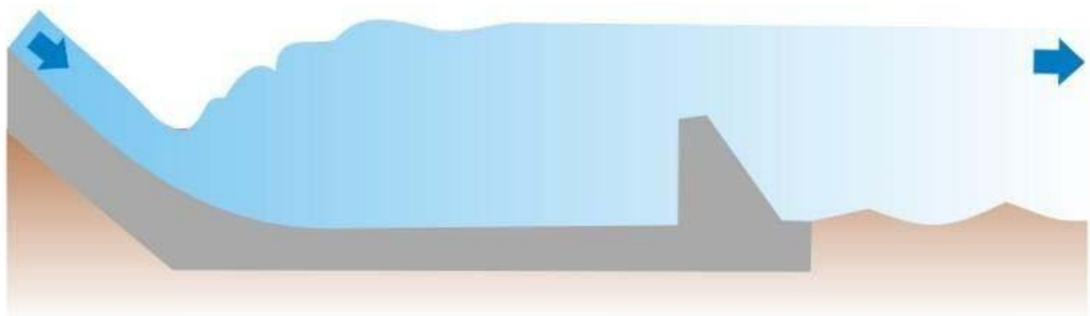


FIGURE 57. High end - sill or baffle at toe of stilling basin

Case 3 (Figure 53)
Since this situation results in submergence results in submergence of the jump, it is necessary to raise the floor in order to form a clear jump. In practice, it is done by providing an inclined apron of the stilling basin (Figure 58).

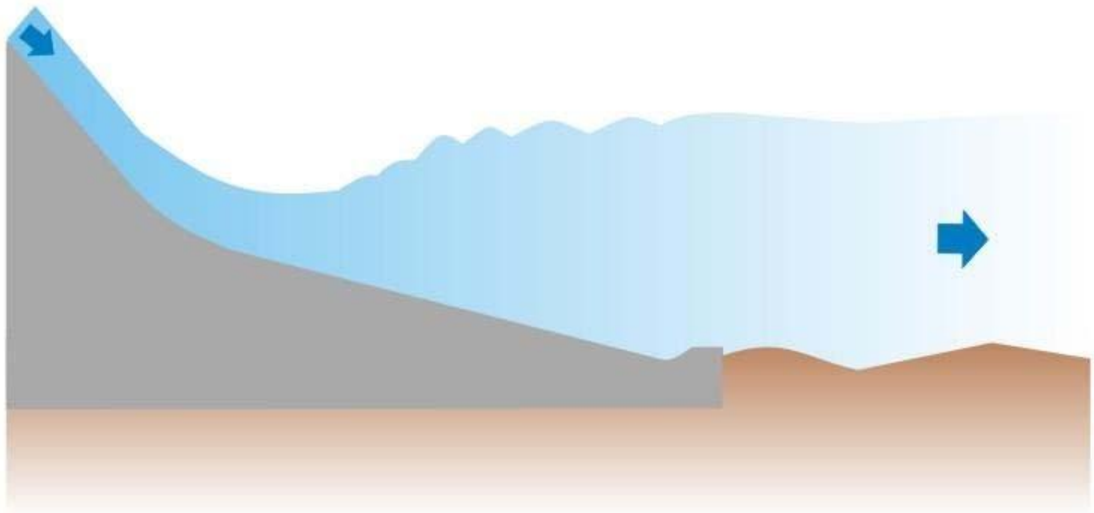


FIGURE 58. Inclined stilling basin

Case 4 (Figure 54)
This situation may be taken care of by providing an inclined floor in the upper portion of the stilling basin and providing either a depressed floor in the lower portion of the basin or provide a baffle at the end of the basin.

Case 5 (Figure 55)
In this case a sloping apron may be provided which lies partly above and partly below the riverbed. So that the jump will form on the higher slope at low discharges and on the lower slope at high discharges.

The type of Stilling Basins that may be provided under different situations is recommended by the Bureau of Indian Standards code IS: 4997-1968 “Criteria for design of hydraulic jump type stilling basins with horizontal and sloping aprons”. In all, these are four types of basin shapes recommended. Types I and II are meant for basins with horizontal floors and types III and IV for basins with inclined floors.

Design of bucket-type energy dissipators

Hydraulic behaviour of bucket type energy dissipator depends on dissipation of energy through:

a. Interaction of two rollers formed, one in the bucket, rolling anti-clockwise (if the flow is from the left to the right) and the other downstream of the bucket, rolling clockwise; or
b. Interaction of the jet of water, shooting out from the bucket lip, with the surrounding air and its impact on the channel bed downstream.

Bucket type energy dissipators can be either:

a) Roller bucket type energy dissipator; or
b) Trajectory bucket type energy dissipator.

The following two types of roller buckets are adopted on the basis of tailwater conditions and importance of the structure:

a) Solid roller bucket, and
b) Slotted roller bucket.

These are shown in Figure 59.

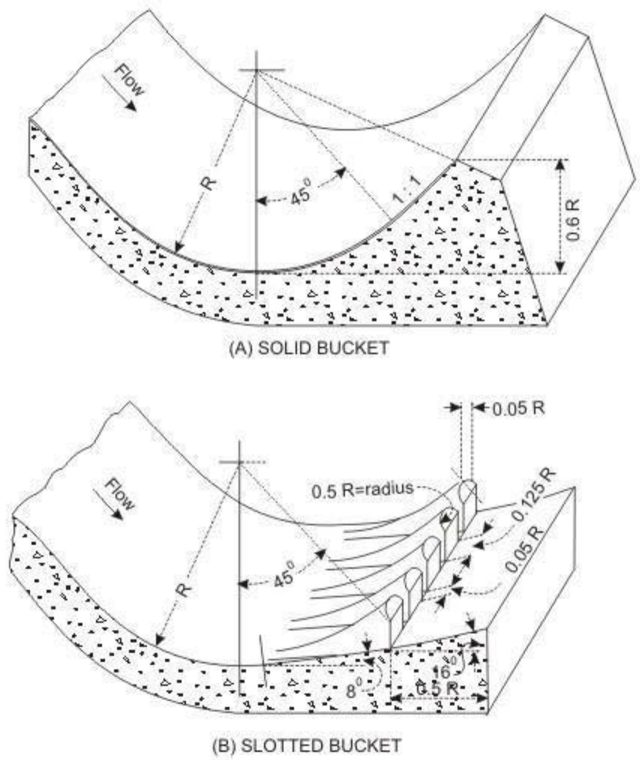


FIGURE 59. Roller buckets ; (A) Solid ; (b) Slotted

Roller bucket type energy dissipator is preferred when:

a) Tailwater depth is high (greater than 1.1 times sequent depth preferably 1.2 - times sequent depth), and
b) River bed rock is sound.

Trajectory bucket type energy dissipator is generally used when:

a) Tailwater depth is much lower than the sequent depth of hydraulic jump, thus preventing formation of the jump;
b) By locating at higher level it may be used in case of higher tailwater depths also, if economy permits; and

c) Bed of the river channel downstream is composed of sound rock.

This is shown in Figure 60.

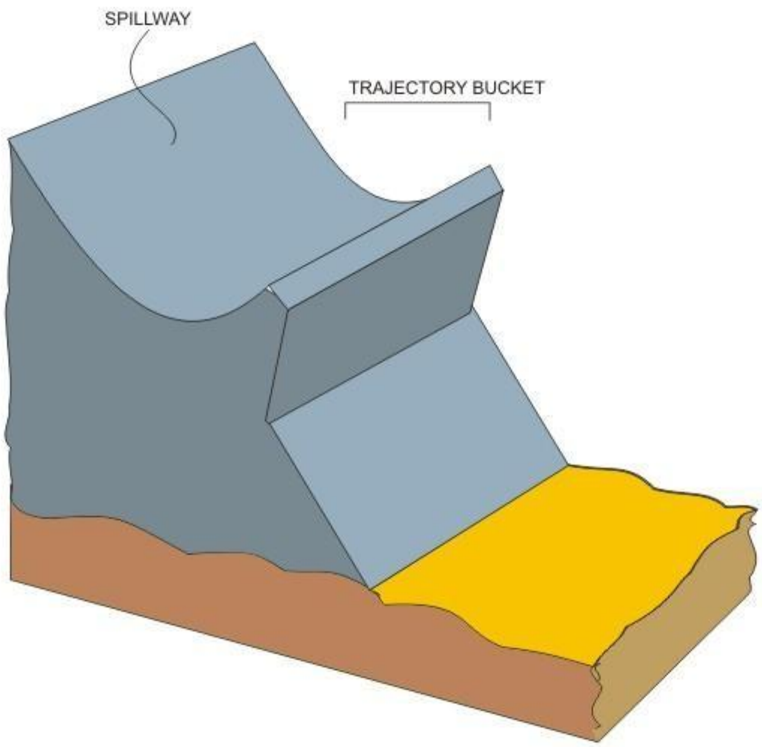


FIGURE 60. Trajectory bucket type energy dissipator

Action of the various types of bucket-type energy dissipators is given below:

Hydraulic Design of Solid Roller Bucket

An upturn solid bucket is used when the tailwater depth is much in excess of sequent depth and in which dissipation of considerable portion of energy occurs as a result of formation of two complementary elliptical rollers, one in bucket proper, called the surface roller, which is anticlockwise (if the flow is to the right) and the other downstream of the bucket, called the ground roller, which is clockwise.

In the case of solid roller bucket the ground roller is more pronounced and picks up material from downstream bend and carried it towards the bucket where it is partly deposited and partly carried away downstream by the residual jet from the lip. The deposition in roller bucket is more likely when the spillway spans are not operated equally, setting up horizontal eddies downstream of the bucket. The picked up material which is drawn into the bucket can cause abrasive damage to the bucket by churning action.

For effective energy dissipation in a solid roller bucket, both the surface or dissipating roller and the ground or stabilizing roller, should be well formed. Otherwise, hydraulic phenomenon of sweep out or heavy submergence occurs depending upon which of the rollers is inhibited.

Design Criteria - The principal features of hydraulic design of solid roller bucket consists of determining:

a) The bucket invert elevation,
b) The radius of the bucket, and
c) The slope of the bucket lip or the bucket lip angle.

The various parameters are shown in Figure 61 (a).

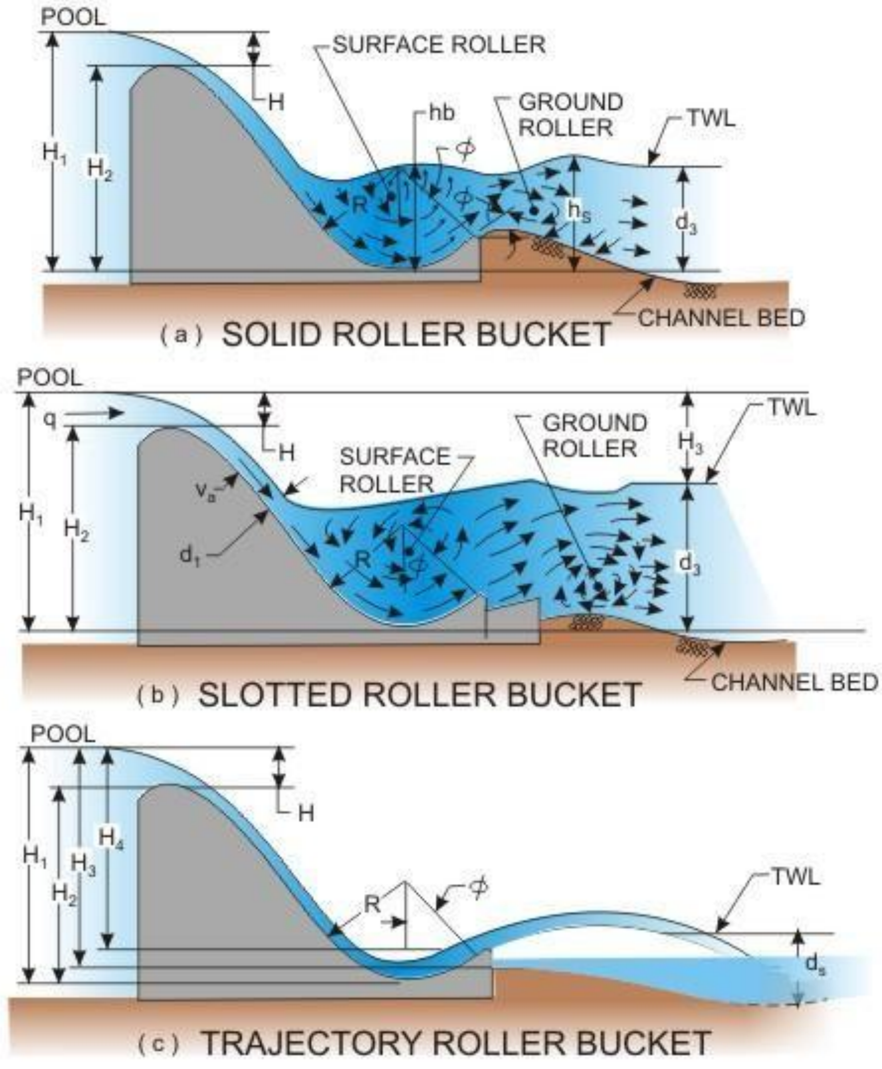


FIGURE 61. Sketches for bucket type energy dissipators

An example of the use of a solid roller bucket is the energy dissipator of the Maithan Dam Spillway (Figure 62)

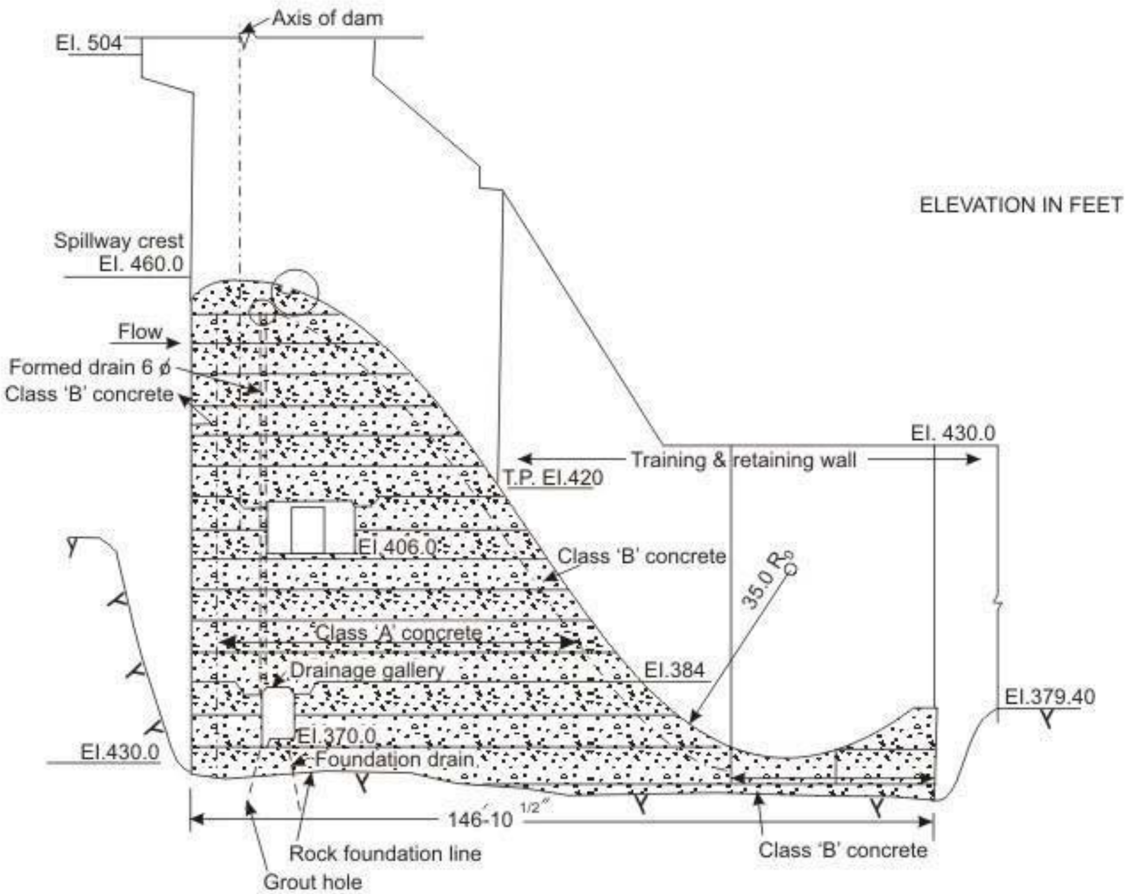


FIGURE 62. Maithon Dam spillway

Drawal of Bed Materials - A major problem with the solid roller bucket would be the damage due to churning action, caused to the bucket because of the downstream bed material brought into the bucket by the pronounced ground roller. Even in a slotted roller bucket downstream material might get drawn due to unequal operation of gates. The channel bed immediately downstream of the bucket shall be set at 1 to 1.5 m below the lip level to minimize the possibility of this condition. Where the invert of the bucket is required to be set below the channel general bed level the channel should be dressed down in one level to about 1 to 1.5 m below the lip level in about 15 m length downstream and then a recovery slope of about 3 (horizontal) to 1 (vertical) should be given to meet the general bed level as shown in Figure 62. Careful model studies should be done to check this tendency. If possible, even provision of solid apron or cement concrete blocks may be considered to avoid trapping of river bed material in the bucket as it may cause heavy erosion on the spillway face, bucket and side training wall.

In the case of slotted roller bucket a part of the flow passes through the slots, spreads laterally and is lifted away from the channel bottom by a short apron at the downstream end of the bucket. Thus the flow is dispersed and distributed over a greater area resulting in a less violent ground roller. The height of boil is also reduced in case of slotted roller bucket. The slotted bucket - provides a self-cleaning action to reduce abrasion in the bucket.

In general the slotted roller bucket is a improvement over the solid roller bucket for the range of tailwater depths under which it can operate without sweepout or diving. However, it is necessary that specific model experiments should be conducted to verify pressure on the teeth so as to avoid cavitation conditions. In case of hydraulic structures in boulder stages slotted roller buckets need not be provided. Heavy boulders rolling down the spillway face can cause heavy damage to the dents thereby making them ineffective and on the contrary, increasing the chances of damage by impact, cavitation and erosion.

Hydraulic Design of Slotted Roller Bucket

An upturned bucket with teeth in it used when the tailwater depth is much in excess of sequent depth and in which the dissipation of energy occurs by lateral spreading of jet passing through bucket slots in addition to the formation of two complementary rollers as in the solid bucket.

In the slotted roller bucket, a part of the flow passes through the slots, spreads laterally and is lifted away from the channel bottom by a short apron at the downstream end of the bucket. Thus the flow is dispersed and distributed over a greater area providing less violent flow concentrations compared to those in a solid roller bucket. The velocity distribution just downstream of the bucket is more akin to that in a natural stream, that is, higher velocities at the surface and lower velocities at the bottom. While designing a slotted roller bucket, for high head spillway exceeding the total head of 50 m or so, specific care should be taken especially for design of the teeth, to ensure that the teeth will perform cavitation free. Specific model tests should therefore be conducted to verify pressures on the teeth and the bucket invert should accordingly be fixed at such an elevation as to restrict the subatmospheric pressures to the permissible magnitude.

Design Criteria - The principal features of hydraulic design of the slotted roller bucket consists of determining in sequence:

a) bucket radius;
b) bucket invert elevation;
c) bucket lip angle; and
d) bucket and tooth dimensions, teeth spacing and dimensions and profile of short apron.

The various parameters are shown in Figure 61(b)

An example of the use of a slotted roller bucket is the energy dissipator provided in the Indira Sagar Dam Spillway (Figure 63).

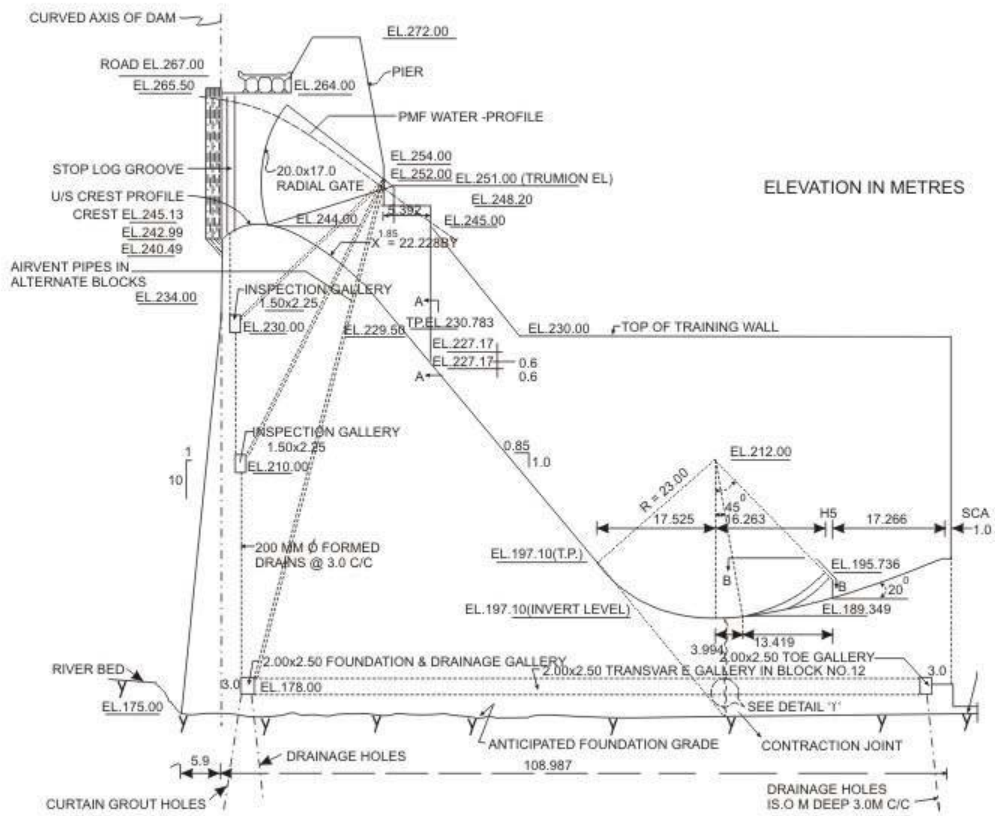


FIGURE 63. Indira Sagar Dam Spillway

Hydraulic Design of Trajectory Bucket Type Energy Dissipator

An upturn solid bucket used when the tailwater depth is insufficient for the formation of the hydraulic jump, the bed of the river channel downstream comprises sound rock and is capable of withstanding, without excessive scour, the impact of the high velocity jet. The flow coming down the spillway is thrown away from toe of the dam to a considerable distance downstream as a free discharging upturned jet which falls into the channel directly, thereby avoiding excessive scour immediately downstream of the spillway. There is hardly any energy dissipation within the bucket itself. The device is used mainly to increase the distance from the structure to the place where high velocity etc. hits the channel bed, thus avoiding the danger of excessive scour immediately downstream of the spillway. Due to the throw of the jet in the shape of a trajectory, energy dissipation takes place by

a) internal friction within the jet,
b) the interaction between the jet and surrounding air,
c) the diffusion of the jet in the tailwater, and
d) the impact on the channel bed.

When the tailwater depth is insufficient for the formation of the hydraulic jump and the bed of the channel downstream comprises sound rock which is capable of withstanding the impact of the high velocity jet, the provision of a trajectory bucket is considered more suitable as provision of conventional hydraulic jump type apron or a roller bucket involves considerable excavation in hard strata forming the bed. It is also necessary to have sufficient straight reach in the downstream of a skijump bucket. The flow coming down the spillway is thrown away in air from the toe of the structure to a considerable distance as a free discharging upturned jet which falls on the channel bed *d/s.* The hard bed can tolerate the spray from the jet and erosion by the plunging jet would not be a significant problem for the safety of the structure. Thus, although there is very little energy dissipation within the bucket itself, possible channel bed erosion close to the downstream toe of the dam is minimized. In the trajectory bucket, only part of the energy is dissipated through interaction of the jet with the surrounding air. The remaining energy is imparted to the channel bed below. The channel bed should consist of sound, hard strata and should be free from laminations, joints and weak pockets to withstand the impact of jet. The design of the trajectory bucket presupposes the formation of large craters or scour holes at the zone of impact of the jet during the initial years of operation and, therefore, the design shall be restricted to sites where generally sound rock is available in the river bed. Special care shall be taken to concrete weak pockets in the bed located in a length of

Design Criteria - The principal features of hydraulic design of tra jectory bucket consist of determining:

a) Bucket shape,
b) Bucket invert elevation, radius or principal geometrical parameters of the bucket, lip elevation and exit angle, trajectory length, and
c) Estimation of scour downstream of the spillway.

The various parameters are shown in Figure 61(c)

An example of the use of a trajectory bucket is the one provided in the Srisailam Dam Spillway (Figure 64).

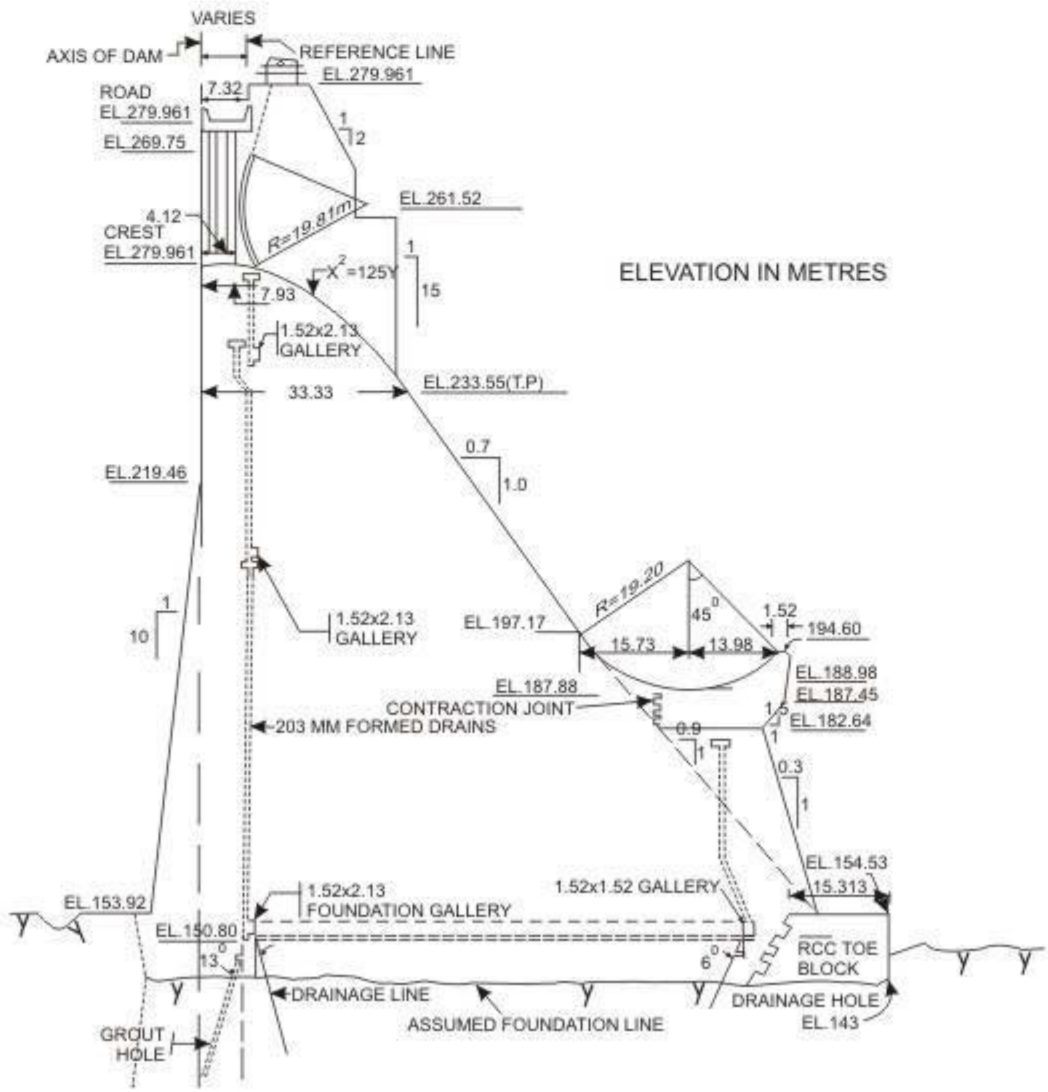


FIGURE 64. Srisailam Dam Spillway

Further details about the design of bucket type energy dissipators may be had from the Bureau of Indian Standards Code IS: 7365-1985 "Criteria for hydraulic design of bucket type energy dissipators"

Protection of downstream of spillways from scour

It may be noted that inspite of the provision of the best suited energy dissipator for a specific spillway under the prevailing site conditions, there may be still some energy is expected to be maximum for the trajectory type spillway, followed by the solid and slotted roller buckets and finally the hydraulic jump type stilling basins. In order to protect the downstream riverbed from these undesirable scour, the following types of protection works have been recommended by the Bureau of Indian Standards code IS:

13195-1991 “Preliminary design, operation and maintenance of protection works downstream of spillways-guidelines”.

1. *Training Walls at the Flanks of the Spillways-* Training walls extended beyond the end-sill of the stilling basins or buckets generally serve to guide the flow into the river channel, protect the wrap-rounds of the adjacent earth dams,
river banks or power house bays and tail race channels. To this extent, the training walls are - considered to be downstream protection works.

2 *Protective Aprons Downstream of Bucket Lips or End-sills of Stilling Basins- Protective* aprons of concrete laid on fresh rock or acceptable strata immediately downstream of bucket lips or end-sill of stilling basin, protect the energy dissipator against undermining due to excessive scour during or after construction of the spillways. A suitable concrete key is normally provided, at the downstream end of the apron. Where the normal river bed level is higher than the end-sill and a recovery slope is , provided, it sometimes becomes necessary to lay a concrete apron on such a recovery slope also for protection.

3. *Concrete Blocks or Concrete Filling on River Bed Downstream of Energy Dissipator -* Concrete blocks or concrete fillings are sometimes provided on the river bed downstream of energy dissipators to safeguard against excessive scour and prevent further scour.

4. *Protective Pitchings on Natural or Artificial Banks Downstream of Spillways-* Protective pitchings of stone rip rap, masonry or concrete blocks are provided on natural river banks or artificially constructed embankments of diversion channels, power house tail race channels or guide banks, for protecting them against high velocity flows or waves.

Figure 65 shows the various types of protection works that may typically be used downstream of a spillway.

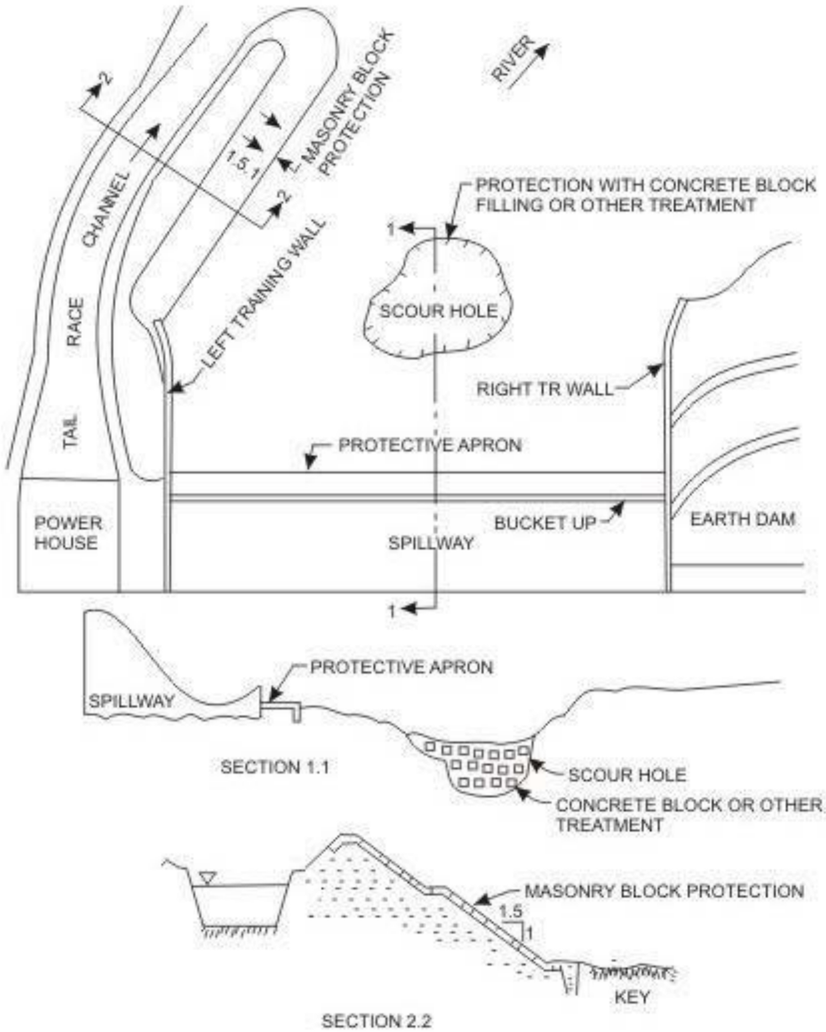


FIGURE 65. Different types of protection works downstream of a typical spillway project (Courtesy: IS 13195)

The importance of providing protection below a spillway, especially of the trajectory type may be noted from the incidence of deep scour on the downstream of the Srisailam dam spillway.

Case Study

Srisailam dam spillway (Figure 64) across river Krishna was constructed during 1977-83. It is a 137 m high concrete dam, with 12 spans of 18'3 m **x** 16'8 m. The river bed is composed of quartzites and shales. In the immediate downstream vicinity of the spillway, there were horizontal shear zones 0'2 m to 0'9 m thick, where the quartzites are crushed and sheared. During the monsoons of 1977 to 1980, the construction stage flood passed over the partially constructed spillway bays, spilling over 7 bays which were at different levels having a maximum difference of level of 23 m. The difference in level between the lip of the ski-jump bucket and downstream rock was about 44 m.

Shorter throw of the water spilling over the bucket lip, as a cascading flow caused deep scour in the immediate vicinity of the bucket lip. During subsequent floods, the scour holes were concreted and leveled as protective aprons in some part of the spillway. Such aprons were however, subjected to repeated damage and undermining. By April 1985, depth of scour below blocks 11 to 13 reached from 9 m to 22 m below the protective apron. Cavities of undermining below the apron were also present at a depth of 6 to 9 m.

The protection work consisted of providing an underwater massive concrete block touching the apron and filling the eroded cavities below the apron. The water level at downstream toe varied from the top of existing apron to about 1.5 m below it.

The scheme involved forming 4 cells with steel cylinder walls and filling concrete in each cell followed by concrete capping. Heavy concrete blocks (approximate 1 metre cube) were placed downstream of the cylinder watls to further protect the rock from the water jump damage.

Since the construction of the above protection works, the spillway was completed to final levels and crest gates have also been installed. Hydraulic model studies were conducted to evolve an operation of the spillway in such a way that the throw of the trajectory fall further away from the toe of the dam. This together with the protective measures already implemented is expected to prevent further erosion at the toe of the dam.

Outlet works enable stored water to be released from the reservoir to users, and also to empty the dam within a reasonable period for safety reasons. Multilevel draw-offs are frequently provided to draw well-oxygenated water from just below the surface of the reservoir.

1. Introduction

The main purpose of a spillway is to safely pass moderate floods and to prevent a dam from failing during very major floods. It is therefore essential that the spillway designer cope with frequent flood releases without significant CHAPTERS erosion or other related damage.
provides a structure that complies with all safety requirements at the least combined lifetime cost of the spillway and dam *(see Large Dams)*.

The spillway system has to safely pass the Recommended Design Flood (RDF) with adequate freeboard. However, when the system passes the Safety Evaluation Flood (SEF), it might not cause the dam to fail catastrophically, but could cause substantial damage to the structure and its surroundings.

2. Spillway Types

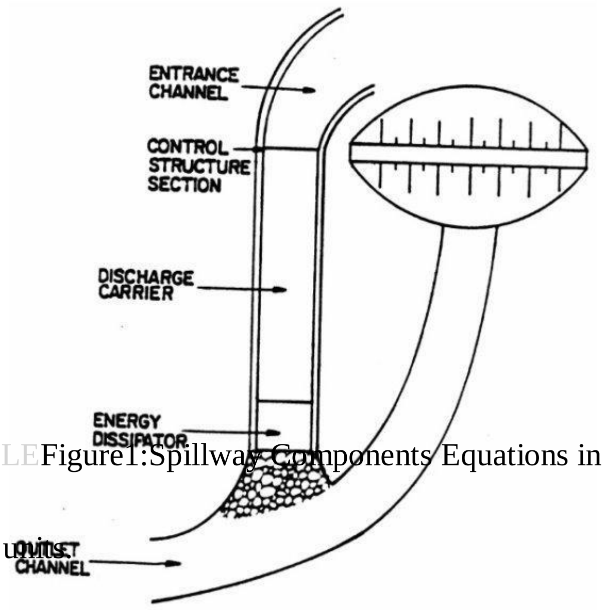
2.1 Service spillways can be either uncontrolled or gated spillways - both are able to flood peaks, as well as being cheaper to build and maintain. Free overspill crest spillways, shaft spillways and syphon spillways also fall into this category.

2.3 Gated spillways should be backed-up by auxiliary spillways as the gates may be subject to automatic operation malfunction, human error and debris blockage. These spillways enable storage to be maximised by controlling water levels. They also allow pre-releases, but if incorrectly operated, can aggravate downstream flooding. Gated spillways are generally more complex and more costly to build and maintain than uncontrolled spillways.

2.4 Auxiliary spillways are uncontrolled spillways used in combination with service spillways and sometimes also with flood outlets, specifically at dams without a service spillway. The auxiliary spillway might take the form of either a fuse plug or fuse gate, which must be designed to function automatically when required without aggravating downstream floods.

The choice of a spillway type depends on a number of factors including the dam type, topography, geology, flood discharge and the frequency and duration of overflows. The main components of spillways influenced by these factors and which have to be combined in the most economically effective manner are the following as shown in Figure 1:

- the entrance tunnel
- the control structure
- the discharge carrier
- the energy dissipater
- the outlet channel



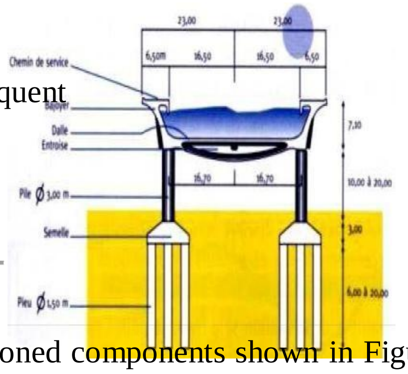


Figure1:Spillway Components Equations in subsequent sections are in metric units.

3. Service Spillway Design

The design of spillways is conveniently also divided into the above mentioned components shown in Figure 1 as follows:

UNIT IV
DIVERSION HEAD WORKS

Diversion Head Works

The works, which are constructed at the head of the canal, in order to divert the river water towards the canal, so as to ensure a regulated continuous supply of silt-free water with a certain minimum head into the canal, are known as ***diversion heads works***.

Objective of Diversion Head Works

- To rise the water level at the head of the canal.
- To form a storage by constructing dykes (embankments) on both the banks of the river so that water is available throughout the year
- To control the entry of silt into the canal and to control the deposition of silt at the head of the canal
- To control the fluctuation of water level in the river during different seasons

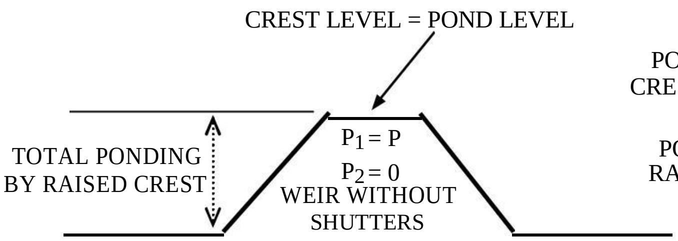
Selection of Site for Diversion Head Works

- At the site, the river should be straight and narrow
- The river banks should be well defined.
- The valuable land should not be submerged when the weir or barrage is constructed.
- The elevation of the site should be much higher than the area to be irrigated.
- The site should be easily accessible by roads or railways.
- The materials of construction should be available in vicinity of the site.
- The site should not be far away from the command area of the project, to avoid transmission loss.

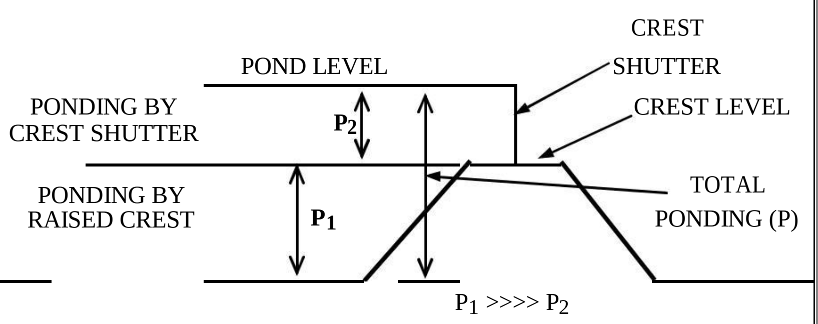
Weir and Barrage

It is a barrier constructed across the river to raise the water level on the upstream side of the obstruction in order to feed the main canal.

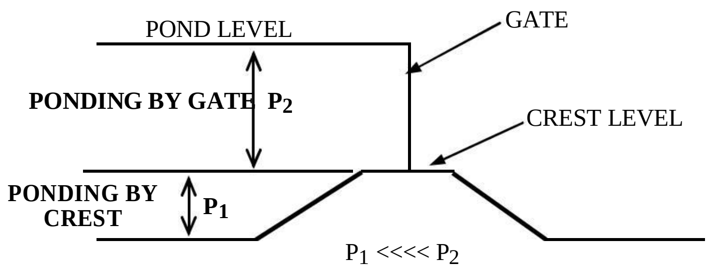
The ponding of water can be achieved either only by a raised crest across the river or by a raised crest supplemented by gates or shutters, working over the crest.



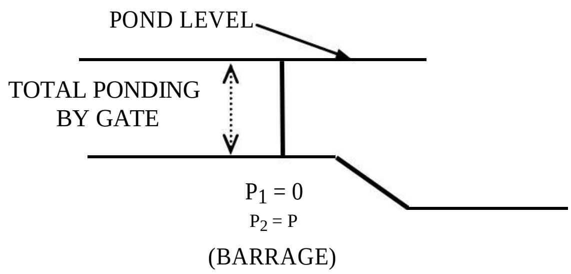
(a) Fig: Weir without shutters



(b) Fig: Weir with shutters



(c) Fig: Barrage with a small raised crest



(d) Fig: Barrage without any raised crest

5. **Weir**

If the major part or the entire ponding of water is achieved by a raised crest and a smaller part or nil part of it is achieved by the shutters, then this barrier is known as a ***weir***.

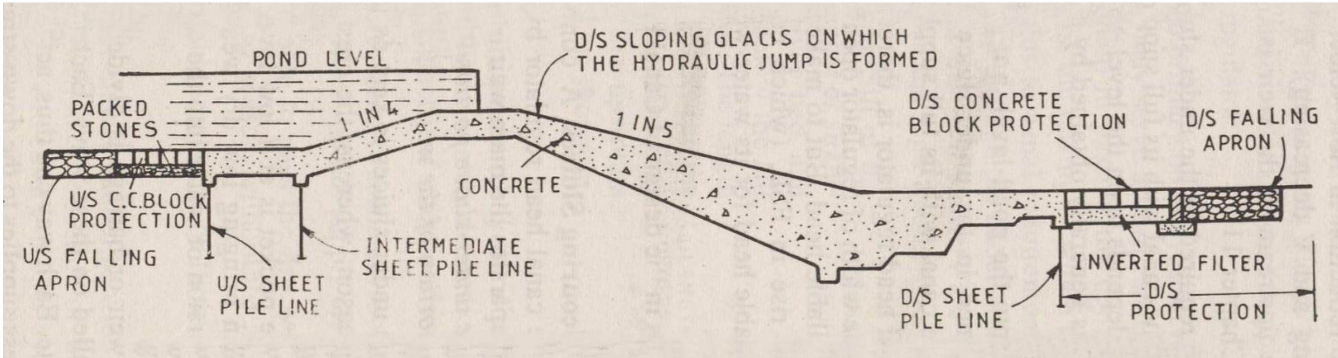


Fig: A typical cross-section of a modern concrete weir

<u>Gravity and Non-gravity weirs</u>:

When the weight of the weir (i.e. its body and floor) balances the uplift pressure caused by the head of the water seeping below the weir, it is called a gravity weir.

On the other hand, if the weir floor is designed continuous with the divide piers as reinforced structure, such that the weight of concrete slab together with the weight of divide piers keep the structure safe against the uplift then the structure may be called as a non-gravity weir.

- In the latter case, RCC is to be used in place of brick piers
  - Considerable savings may be obtained, as the weight of the floor can be much less than what is required in gravity weir.

<u>Types of weirs</u>

- Masonry weirs with vertical drop
- Rock-fill weirs with sloping aprons
- Concrete weirs with sloping glacis

<u>Masonry weirs with vertical drop</u>

Masonry weir wall is constructed over the impervious floor. Cut-off walls are provided at both ends of the floor. Sheet piles are provided below the cut off walls. The crest shutters are provided to raise the water level, if required. The shutters are dropped down during flood. The masonry weir wall may be vertical on both face or sloping on both face or vertical on downstream face and sloping in upstream face.

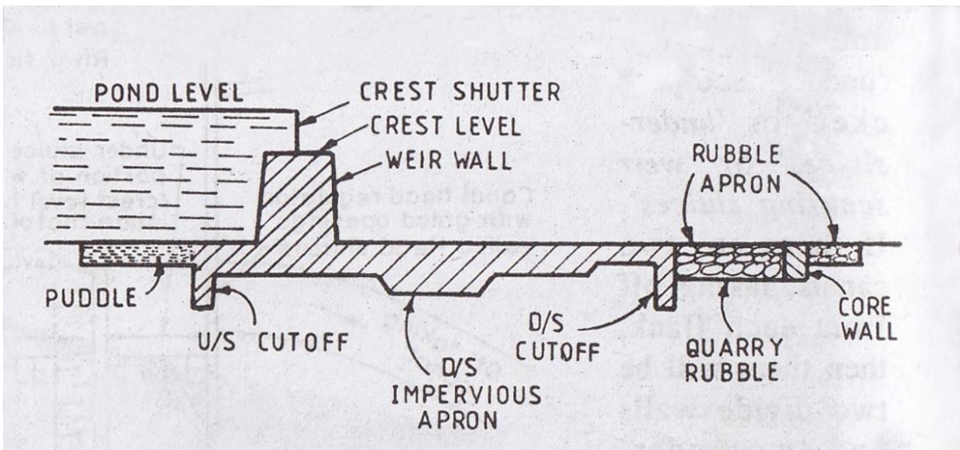


Fig. Masonry weir

Rock-fill weirs with sloping aprons

It consists of masonry breast wall which is provided with adjustable crest shutter. The upstream rock-fill portion is constructed with boulders forming a slope of 1 in 4. The boulders are grouted with cement mortar. The downstream sloping apron consists of core walls. The intermediate spaces between the core walls are filled up with boulders maintaining a slope of 1 in 20. The boulders are grouted properly with cement mortar.

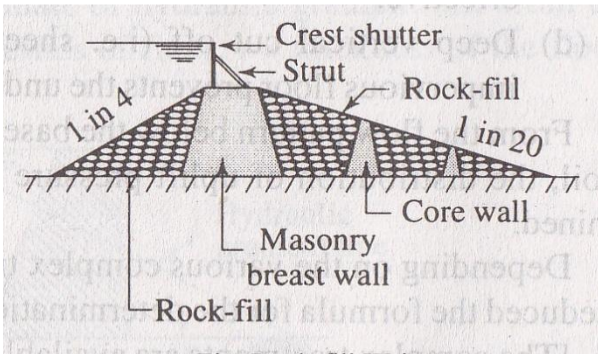


Fig: Rock-fill Weir

Concrete weir

Now-a-days, the weir is constructed with reinforced cement concrete. The impervious floor and the weir are made monolithic. The cut off walls are provided at the upstream and downstream end of the floor and at the toe of the weir. Sheet piles are provided below the cut-off walls. The crest shutters are also provided which hare dropped down during the flood.

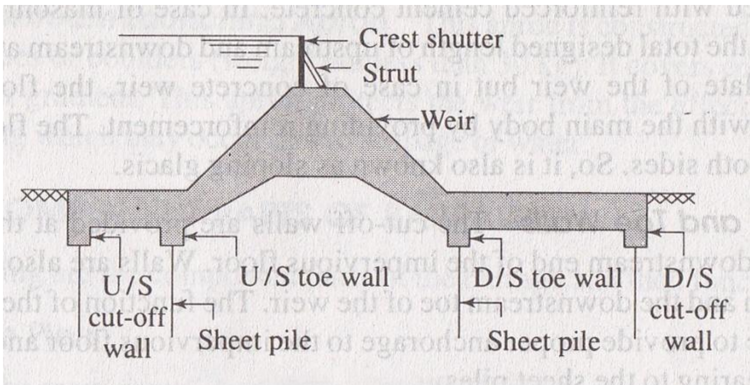


Fig: Concrete Weir

Barrage

- If most of the ponding is done by gates and a smaller or nil part of it is done by the raised crest, then the barrier is known as a ***barrage*** *or a river regulator*.

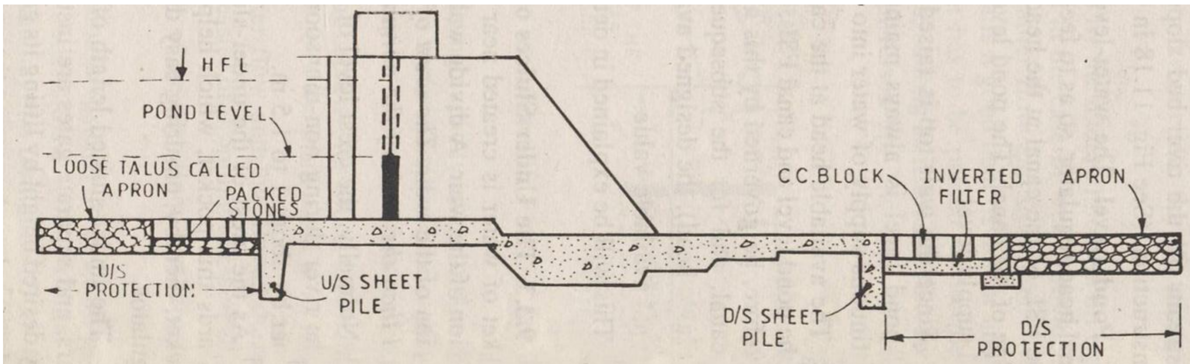


Fig: A typical cross-section of a barrage

Afflux:

1. The rise in the highest flood level (HFL) upstream of the weir due to construction of the weir across the river is called.
1. In case of weir, the afflux caused during high floods is quite high. But in case of a barrage, the gates can be opened during high floods and the afflux will be nil or minimum.

Choice between a weir and a barrage

The choice between a weir and a barrage is largely governed by cost and convenience in working.

800 A shuttered weir will be relatively cheaper but will lack the effective control possible in the case of a barrage.

700 A barrage type construction can be easily supplemented with a roadway across the river at a small additional cost. Barrages are almost invariably constructed now-a-days on all important rivers.

Difference between Barrage and Weir

SL	Barrage	Weir
(a)	Low set crest	High set crest
(b)	Ponding is done by means of gates	Ponding is done against the raised crest or partly against crest and partly by shutters
(c)	Gated over entire length	Shutters in part length
(d)	Gates are of greater height	Shutters are of smaller height, 2 m
(e)	Gates are raised clear off the high floods to pass floods	Shutters are dropped to pass floods
(f)	Perfect control on river flow	No control of river in low floods
(g)	Gates convenient to operate	Operation of shutters is slow, involve labour and time
(h)	High floods can be passed with minimum afflux	Excessive afflux in high floods
(i)	Less silting upstream due to low set crest	Raised crest causes silting upstream
(j)	Longer construction period	Shorter construction period
(k)	Silt removal is done through under sluices	No means for silt disposal
(l)	Road and/or rail bridge can be constructed at low cost	Not possible to provide road-rail bridge
(m)	Costly structure	Relatively cheaper structure

Layout of a Diversion Head Works and its components

A typical layout of a canal head-works is shown in figure below. *Such a head-works consists of*:

- Weir proper
- Under-sluices
- Divide wall
- River Training works
- Fish Ladder
- Canal Head Regulator
- River Training Works e.g. Guide bank, Marginal bunds, spur and groyne etc.
- Shutters and Gates
- Silt Regulation Works

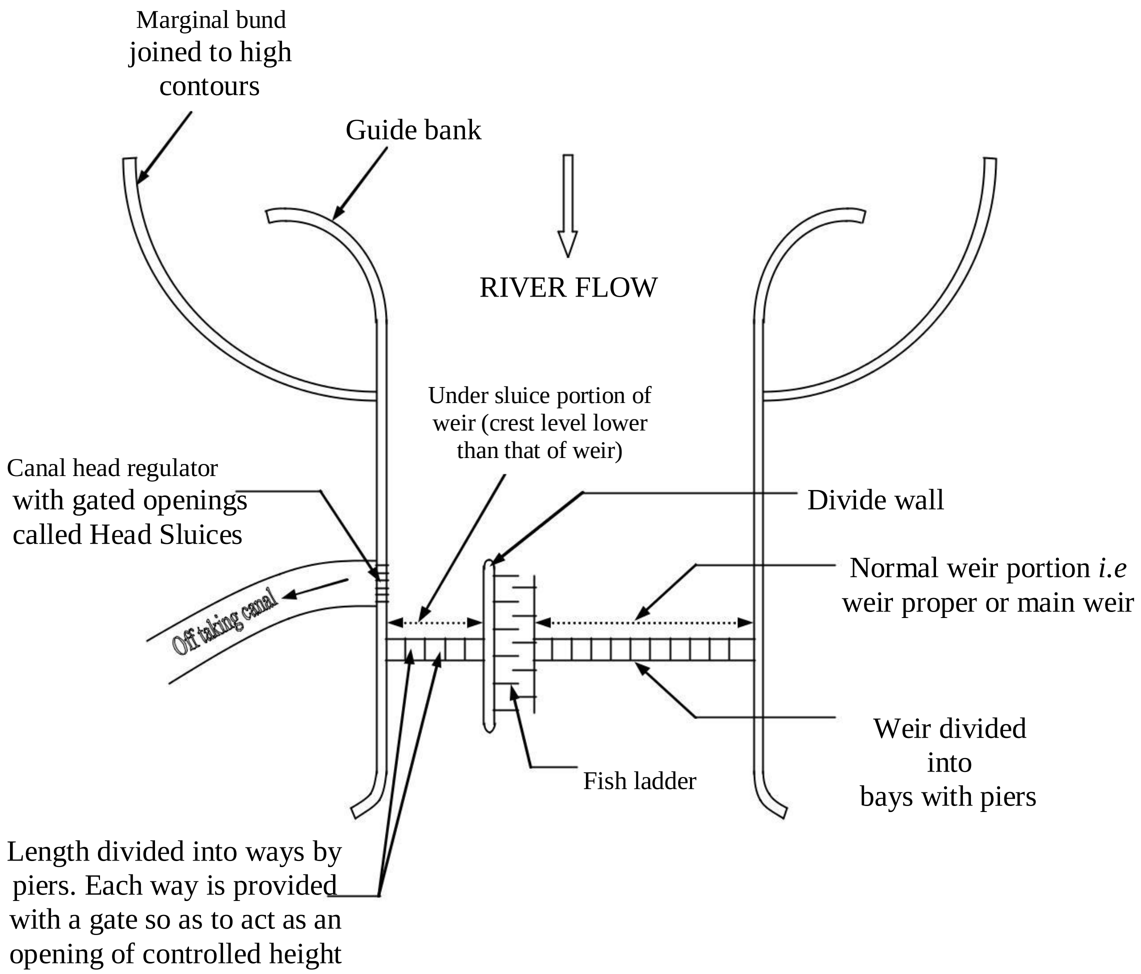


Fig: Typical layout of diversion head-works

- **Weir Proper:**

It is a barrier constructed across the river. It aims to raise the water level in order to feed the canal.

- **Under-sluices:**

The under sluices are the openings provided at the base of the weir or barrage. These openings are provided with adjustable gates. Normally, the gates are kept closed. The crest of the under-under sluice portion of the weir is kept at a lower level (1~1.5 m) than the crest of the normal portion of the weir. The suspended silt goes on depositing in front of the canal head regulator. When the silt deposition becomes appreciable the gates are opened and the deposited silt is loosened with an agitator mounting on a boat. The muddy water flows towards the downstream through the scouring sluices. The gates are then closed. But, at the period of flood, the gates are kept opened.

***The main functions of under-sluices are*:**

i. To maintain a well defined deep channel approaching the canal head regulator.

i. To ensure easy diversion of water into the canal through the canal head regulator even during low flow.

1. To control the entry of silt into the canal

i. To help scouring and of the silt deposited over the under-sluice floor and removing towards the downstream side.

- To help passing the low floods without dropping the shutters of the weir.

- **The divide wall:**

o The divide wall is a masonry or concrete wall constructed at right angle to the axis of the weir.

p The divide wall extends on the upstream side beyond the beginning of the canal head regulator; and on the downstream side, it extends upto the end of the loose protection of the under-sluices.

q The divide wall is a long wall constructed at right angles in the weir or barrage, it may be constructed with stone masonry or cement concrete. On the upstream side, the wall is extended just to cover the canal head regulator and on the downstream side, it is extended up to the launching apron

The main functions of the divide walls:

o It separates the 'under-sluices' with lower crest level from the 'weir proper' with higher crest level.

o It helps in providing a comparatively less turbulent pocket near the canal head regulator, resulting in deposition of silt in this pocket and, thus, to help in the entry of silt-free water into the canal.

a It helps to keep cross-current, if any, away from the weir.

Fish Ladder

iii. It is device by which the flow energy can be dissipated in such a manner as to provide smooth flow at sufficiently low velocity, not exceeding 3 to 3.5 m/s.

iv. A narrow opening including suitable baffles or staggering devices in it is provided adjacent to the divide wall.

v. The fish ladder is provided just by the side of the divide wall for the free movement of fishes. Rivers are important source of fishes.

vi. There are various types of fish in the river. The nature of the fish varies from type to type. But in general, the tendency of fish is to move from upstream to downstream in winters and from downstream to upstream in monsoons. This movement is essential for their survival. Due to construction of weir or barrage, this movement gets obstructed, and is detrimental to the fishes.

vii. In the fish ladder, the fable walls are constructed in a zigzag manner so that the velocity of flow within the ladder does not exceed 3 m/sec

viii. The width, length and height of the fish ladder depend on the nature of the river and the type of the weir or barrage.

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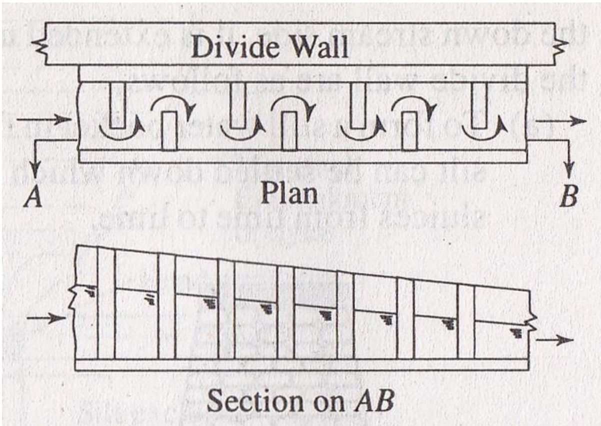


Fig: Fish Ladder

Canal Head Regulator or Head sluices

A structure which is constructed at the head of the canal to regulate flow of water is known as canal head regulator. It consists of a number of piers which divide the total width of the canal into a number of spans which are known as bays. The piers consist of number tiers on which the adjustable gates are placed. The gates are operated form the top by suitable mechanical device. A platform is provided on the top of the piers for the facility of operating the gates. Again some piers are constructed on the down stream side of the canal head to support the roadway.

***Functions of Canal Head Regulator*:**

vi. It regulates the supply of water entering the canal o It controls the entry of silt in the canal

o It prevents the river-floods from entering the canal

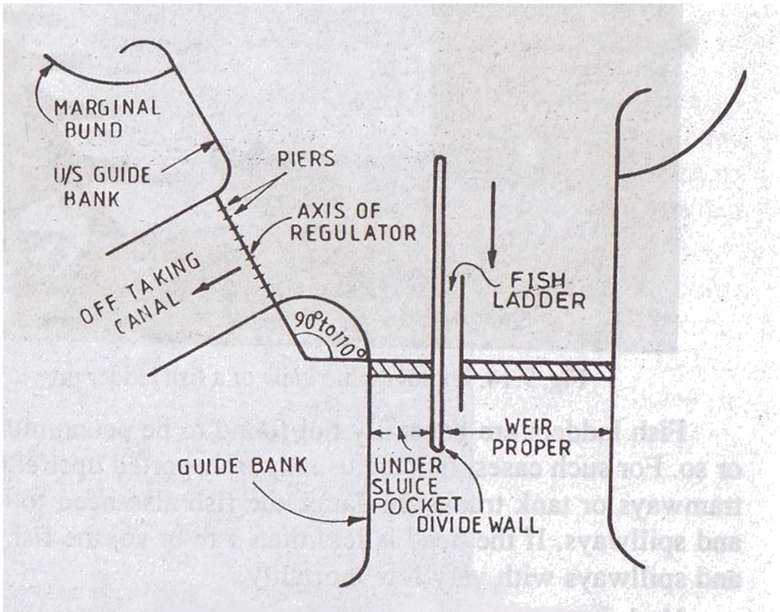


Fig: Alignment of a canal head regulator

The water from the under-sluice pocket is made to enter the regulator bays, so as to pass the full supply discharge into the canal. The maximum height of these gated openings, called head sluices will be equal to the difference of Pond Level and Crest Level of the regulator.

- The entry of silt into the canal is controlled by keeping the crest of the head regulator by about 1.2 to 1.5 meters higher than the crest of the under-sluices.
- If a silt-excluder is provided, the regulator crest is further raised by about 0.6 to 0.7 meter.
- Silt gets deposited in the pocket, and only the clear water enters the regulator bays.
- The deposited silt can be easily scoured out periodically, and removed through the under-sluice openings.

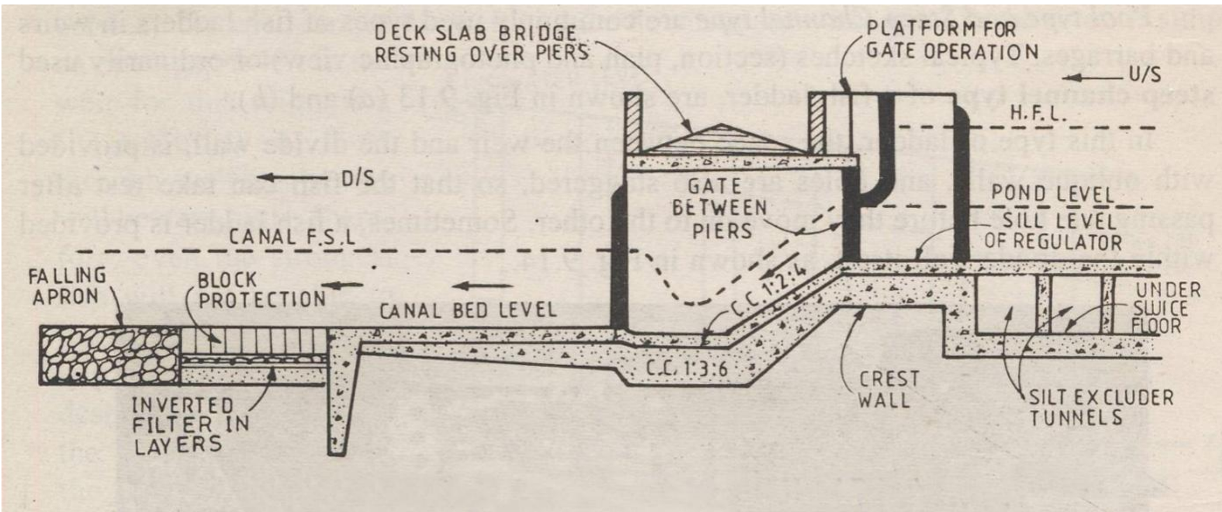


Fig: A typical section through a Canal Head Regulator (CHR)

River Training Works

River training works are required near the weir site in order to ensure a smooth and an axial flow of water, and thus, to prevent the river from outflanking the works due to a change in its course.

The river training works required on a canal headwork are:

H Guide banks
I Marginal bunds
J Spurs or groynes

(a) Guide Bank

When a barrage is constructed across a river which flows through the alluvial soil, the guide banks must be constructed on both the approaches to protect the structure from erosion.

Guide bank serves the following purposes:

- It protects the barrage from the effect of scouring and erosion.
- It provides a straight approach towards the barrage.
- It controls the tendency of changing the course of the river.
- It controls the velocity of flow near the structure.

- **Marginal Bunds**

The marginal bunds are earthen embankments which are constructed parallel to the river bank on one or both the banks according to the condition. The top width is generally 3 m to 4 m. The side slope on the river side is generally 1.5: 1 and that on the country side is 2:1.

The marginal bunds serve the following purposes:

- It prevents the flood water or storage water from entering the surrounding area which may be submerged or may be water logged.
- It retains the flood water or storage water within a specified section.
- It protects the towns and villages from devastation during the heavy flood.
- It protects valuable agricultural lands.

ii. Spurs or groynes

a.<u>Spurs</u>

These are temporary structures permeable in nature provided on the curve of a river to protect the river bank from erosion. These are projected from the river bank towards the bed making angles 60^{o} to 75^{o} with the bank of the river. The length of the spurs depends on the width of the river and the sharpness of the curve. <u>The function of the spurs is to break the velocity of flow and to form a water pocket on the upstream side where the sediments get deposited</u>. Thus the reclamation of land on the river bank can be achieved. *The spurs may be of the following types*:

- Bamboo Spur
- Timber Spur
- Boulder Spur

i. **Bamboo Spur:** In this type of spur, a box like compartment is prepared by driving bamboo piles at 15 cm centre to centre. The piles are secured by bamboo bracings. The hollow space is filled up with sand bags. It is permeable in nature and water can seep through its body. This type of spur is suitable for small rivers. This is purely temporary and requires repair work every year. The length of bamboo piles depends on bed condition.

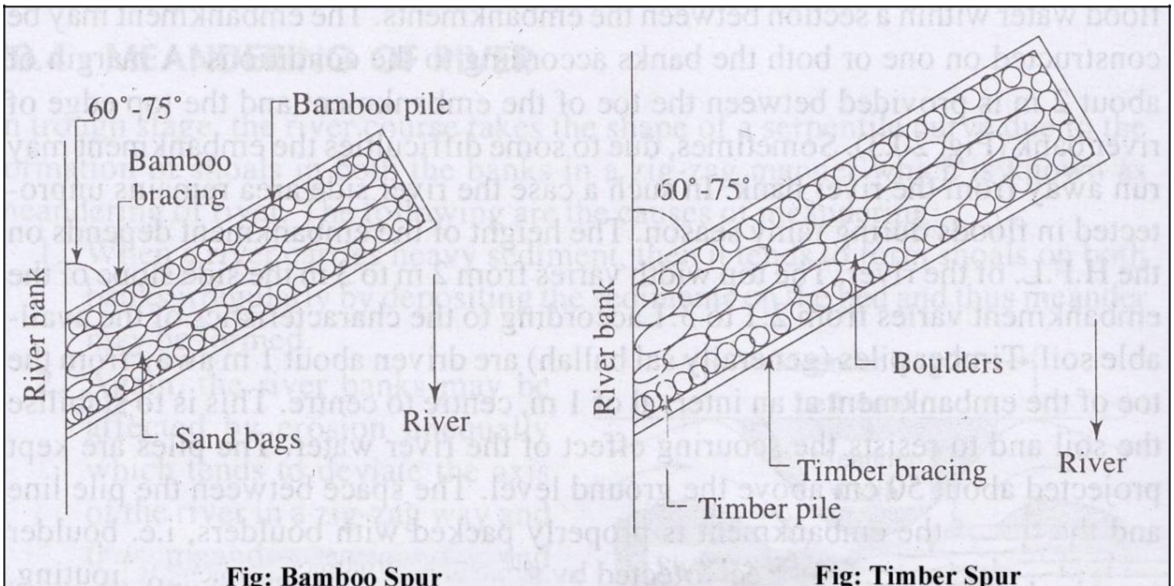


Fig: Bamboo Spur **Fig: Timber Spur**

i. **Timber Spur:** In this type, a box like compartment is prepared by driving timber piles at 15 cm to 30 cm centre to centre. The piles are secured by wooden bracings. The hollow space is filled up by boulders. This spur is permeable but stable. This is recommended for bi rivers with high velocity of flow. The length of the timber piles depend on bed condition.

ii. **Boulder Spur:** In this type of spur, boulders are enclosed in G.I wire net in circular shape. The boulder should be heavy, varying from 30 kg to 50 kg and the wire net should be made of 4 mm diameter G.I wires. It is laid from the river bank towards the bed making an angle of 60^o-75^o with the bank. This type of spur is recommended for the rivers where velocity of flow is very high.

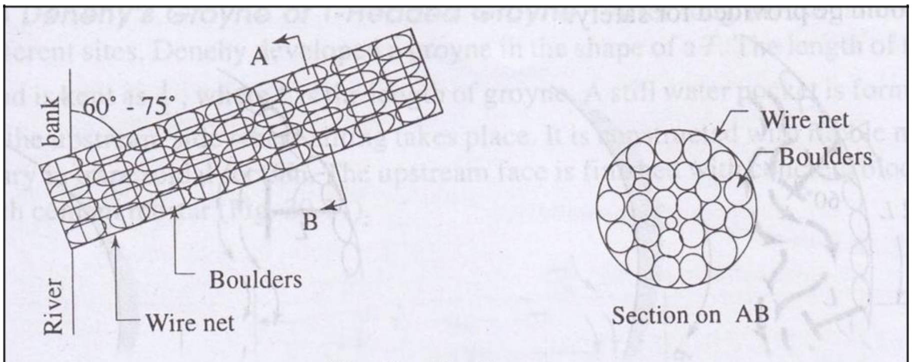


Fig: Boulder Spur

a. <u>Groynes</u>

The <u>function of groynes is similar to that of spur</u>. But <u>these are impervious permanent structures</u> constructed on the curve of a river to protect the river bank from erosion. They extend from the bank towards the bed by making an angle of 60^o to 75^o with the bank. The angle may be towards the upstream or downstream. Sometimes, it is made perpendicular to the river bank. These are constructed with rubble masonry in trapezoidal section and the surface is finished with stone pitching or concrete blocks.

G The stone pitching or the concrete blocks are set with rich cement mortar. o
The length of the groyne depends on the width and nature of the river.

- o The top width varies from 3 m to 4 m. The side slope may be 1½: 1 or 2:1.
- o The groynes are provided in series throughout the affected length of the river bank.
- o The spacing between the adjacent groynes is generally kept as 2*L*, where *L* is the length of the groyne.
- o These are recommended for the river where the permanent solution of erosion control is extremely necessary.

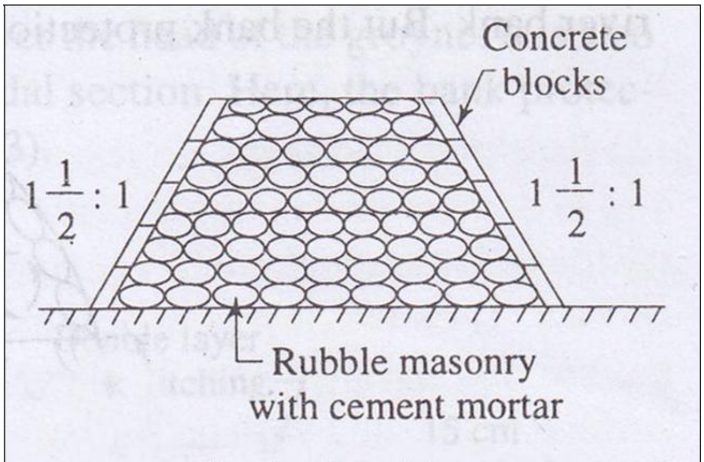


Fig: Section of Groyne

***The groynes may be designated as follows*:**

I **Attracting Groyne:** The groyne which is constructed obliquely to the bank by making an angle of 60 to 75o towards the downstream is known as attracting groyne, here the flow of water is attracted towards the bank, and the velocity of flow is reduced to such a extent that it can not cause any erosion to the bank. However, a bank protected of stone pitching is provided for safety.

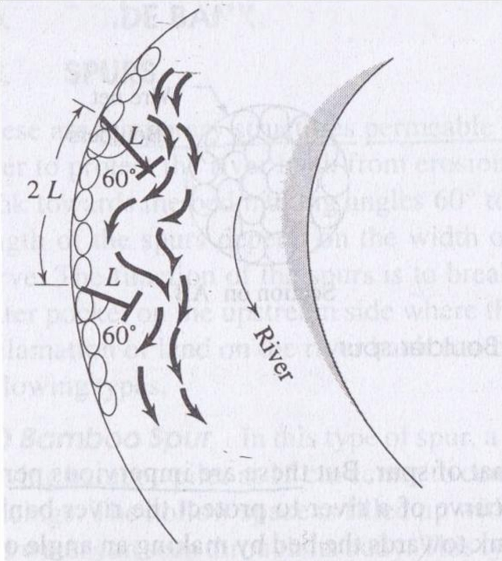


Fig: Attracting groyne

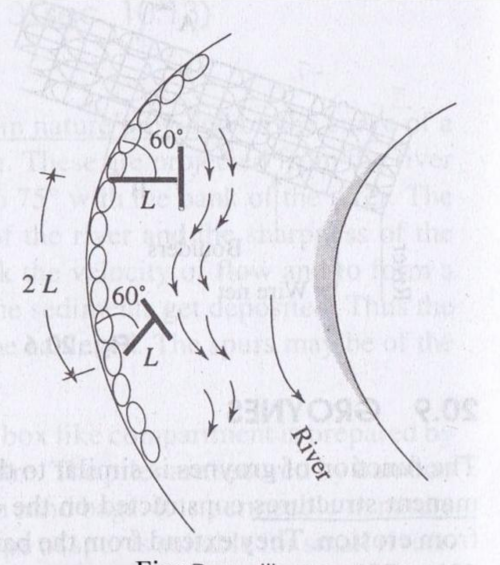


Fig: Repelling groyne

= **Repelling Groyne:** A groyne which is aligned towards upstream at an angle of 60^{o} to 75^{o} with the river bank is known as repelling groyne. A still water pocket is formed on the upstream where silting takes place. Here, the bank protection is not necessary, because the flow of water does not touch the bank and there is no effect of erosion on the bank. But still boulder pitching should be provided for safety.

= **Deflecting Groyne:** The groyne which is constructed perpendicular to the river bank is known as deflecting groyne. Here the flow of water is deflected from bank by the perpendicular obstruction i.e. groyne. The flow of water follows an undulating path just outside the head of the groyne. An eddy current is formed on the upstream side of the groyne. This eddy current will not affect the river bank. But the bank protection is provided for safety.

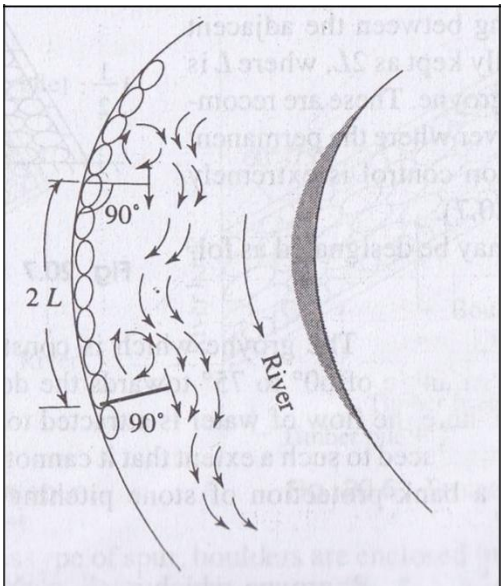


Fig: Deflecting Groyne

Modification of Groyne:

- **Denehy's Groyne or T-Headed Groyne:** After long investigation in different sites, Denehy developed a groyne in the shape of a T. The length of the head is kept as ½, where L is the length of groyne. A still water pocket is formed on the upstream side where silting takes place. It is constructed with rubble masonry in trapezoidal section. The upstream face is finished with concrete blocks with cement mortar.

- **Hockey Head Groyne:** Another development is hockey head groyne. Here, the head of the groyne is curved towards the downstream in the shape of a hockey stick. It behaves like an attracting groyne. But it allows the water to flow smoothly over the head of the groyne. It is also constructed with rubble masonry in trapezoidal section. Here, the bank protection by stone pitching is necessary.

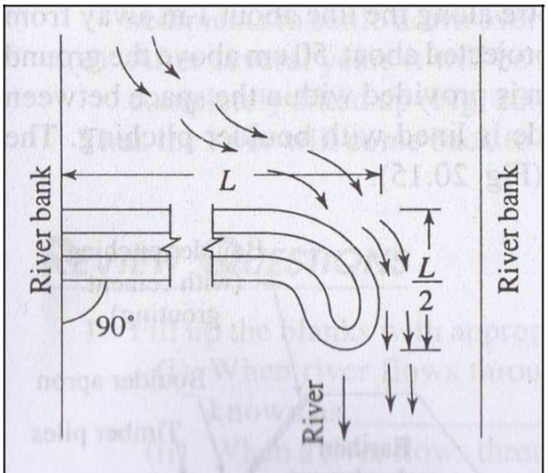


Fig: Hockey Head Groyne

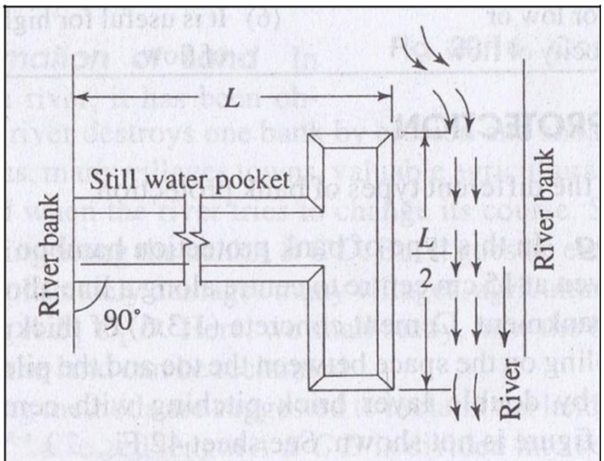


Fig: Denehy's Groyne

Comparison between spur and groyne

	Spur	Groyne
(1)	It is a temporary structure.	It is a permanent structure.
(2)	It is permeable.	It is impermeable.
(3)	It is constructed with bamboo pile, timber pile, sand bag, boulders etc.	It is constructed with rubble masonry with cement mortar.
(4)	It requires repair works.	It does not require any repair work.
(5)	It is recommended for small rivers.	It is recommended for large rivers.
(6)	It is useful for low or medium velocity of flow	It is suitable for high velocity of flow.

Shutters and Gates:

Functions of shutters and gates are:

- They maintain pond level.
- They raise water level during low flow.

Pond Level

The water level required in the under-sluice pocket upstream of the Canal Head Regulator, so as to feed the canal with its full supply, is known as Pond Level.

The FSL of the canal at the head depends upon the level of the irrigated areas and the slope of the canal. Pond Level = Canal FSL + 1.0 to 1.2 m

Silt Regulation works

The entry of silt into a canal, which takes off from a head works, can be reduced by constructed certain special works, called silt control works.

These works may be classified into the following two types:

- Silt Excluders
- Silt Ejectors

(a) Silt Excluders

Silt excluders are those works which are constructed on the bed of the river, upstream of the head regulator. The clearer water enters the head regulator and silted water enters the silt excluder. In this type of works, the silt is, therefore,, removed from the water before in enters the canal.

(b) Silt Ejectors

Silt ejectors, also called silt extractors, are those devices which extract the silt from the canal water after the silted water has traveled a certain distance in the off-take canal. These works are, therefore, constructed on the bed of the canal, and little distance downstream from the head regulator.

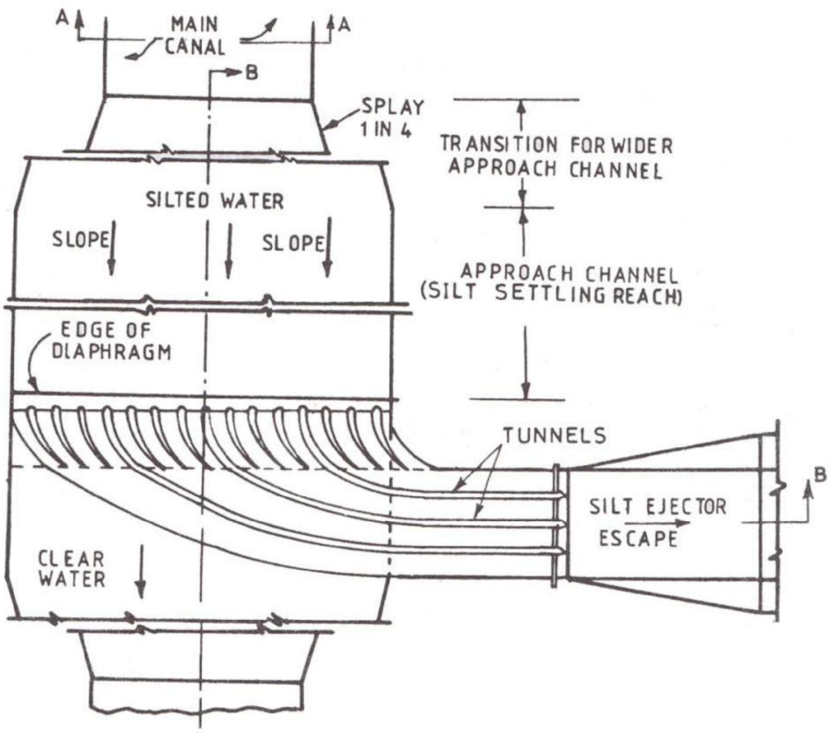


Fig: Plan of Silt Ejector

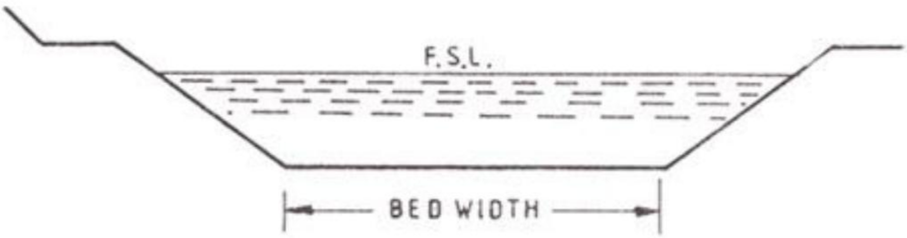


Fig: Section A-A
(Main Canal)

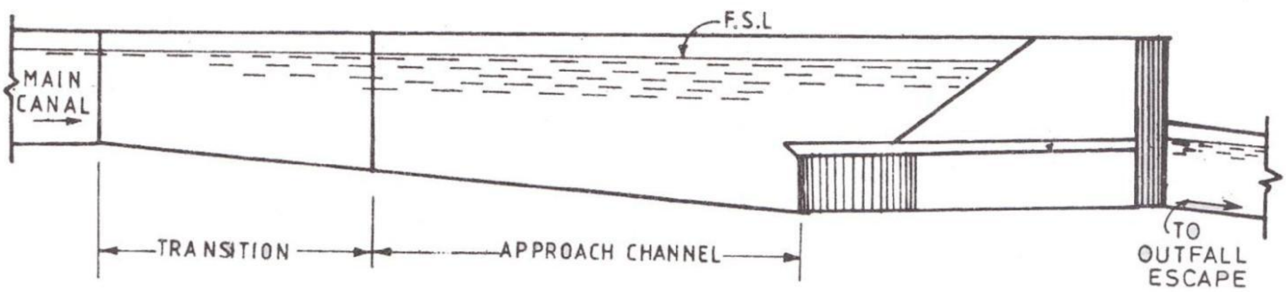


Fig: L-section along tunnel
(Section B-B)

UNIT V
CANAL FALLS & REGULATIONS – DRAINAGE WORKS

Definition:

A cross drainage work is a structure carrying the discharge from a natural stream across a canal intercepting the stream.

Canal comes across obstructions like rivers, natural drains and other canals.

The various types of structures that are built to carry the canal water across theabove mentioned obstructions or vice versa are called cross drainage works.

It is generally a very costly item and should be avoided by

Types of cross drainage works

Depending upon levels and discharge, it may be of the following

types:

Cross drainage works carrying canal across the drainage: the

structures that fall under this type are:

1. An Aqueduct
2. Siphon Aqueduct

Aqueduct

When the HFL of the drain is sufficiently below the bottom of the canal such that the drainage waterflows freely under gravity, the structure is known as Aqueduct.

Crossing works: (aqueducts)

Siphon Aqueduct:

In case of the siphon Aqueduct, the HFL of the drain is much higher above the canal bed, and water runs under siphonic action through the Aqueduct barrels.

The drain bed is generally depressed and provided with pucci floors, on the upstream side, the drainage bed may be joined to the pucca floor either by a vertical drop or by glacis of 3:1. The downstrean rising slope should not be steeper than 5:1. When the canal is passed over the drain, the canal remains open for inspection throughout and the damage caused by flood is rare. However during heavy floods, the foundations are succeptible to scour or the waterway of drain may get choked due to debris, tress etc.

Cross Drainage Works

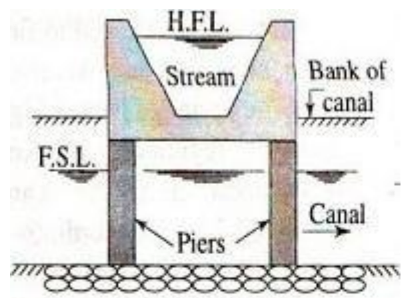
Cross drainage works carrying drainage over canal.

The structures that fall under this type are:

- Super passage
- Canal siphon or called syphon only

Super passage:

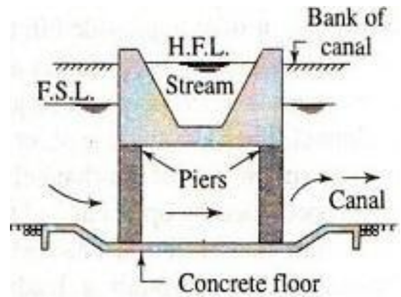
- The hydraulic structure in which the drainage is passing over the irrigation canal is known assuper passage. This structure is suitable when the bed level of drainage is above the flood surface level of the canal. The water of the canal passes clearly below the drainage
- A super passage is similar to an aqueduct, except in this case the drain is over the canal.
- The FSL of the canal is lower than the underside of the trough carrying drainage water. Thus, the canal water runs under the gravity.
- Reverse of an aqueduct



Super passage

Canal Syphon:

- If two canals cross each other and one of the canals is siphoned under the other, then the hydraulic structure at crossing is called "canal siphon". For example, lower Jhelum canal is siphoned under the Rasul-Qadirabad (Punjab, Pakistan) link canal and the crossing structure is called "L.J.C siphon"
- In case of siphon the FSL of the canal is much above the bed level of the drainage trough, so that the canal runs under the siphonic action.
- The canal bed is lowered and a ramp is provided at the exit so that the trouble of silting is minimized.
- Reverse of an aqueduct siphon
- In the above two types, the inspection road cannot be provided along the canal and a separate bridge is required for roadway.



Siphon super passage

For economy, the canal may be flumed but the drainage trough is never flumed.

Selection of suitable site for cross drainage works

- The factors which affect the selection of suitable type of cross drainage works are:
- Relative bed levels and water levels of canal and drainage
- Size of the canal and drainage.
- The following considerations are important
- When the bed level of the canal is much above the HFL of the drainage, an aqueduct is the obvious choice.
- When the bed level of the drain is well above FSL of canal, super passage is provided.
- The necessary headway between the canal bed level and the drainage HFL can be increased by shifting the crossing to the downstream of drainage. If, however, it is not possible to change the canal alignment, a siphon aqueduct may be provided.
- When canal bed level is much lower, but the FSL of canal is higher than the bed level of drainage, a canal siphon is preferred.
- When the drainage and canal cross each other practically at same level, a level crossing may be preferred. This type of work is avoided as far as possible.

Factors which influence the choice / Selection of Cross Drainage Works

1. The considerations which govern the choice between aqueduct and siphon aqueduct are:
2. Suitable canal alignment
3. Suitable soil available for bank connections
4. Nature of available foundations
5. Permissible head loss in canal
6. Availibility of funds

Compared to an aqueduct a super passage is inferior and should be avoided whenever possible. Siphon aqueduct is preferred over siphon unless large drop in drainage bed is required.

Classification of aqueduct and siphon aqueduct

Depending upon the nature of the sides of the aqueduct or siphon aqueduct it may be classified under three headings:

Type I:

Sides of the aqueduct in earthen banks with complete earthen slopes. The length of culvert should be sufficient to accommodate both, water section of canal, as well as earthen banks of canal with aqueduct slope.

Sides of the aqueduct in earthen banks, with other slopes supported by masonry wall. In this case, canal continues in its earthen section over the drainage but the outer slopes of the canal banks are replaced by retaining wall, reducing the length of drainage culvert.

Type II:

Sides of the aqueduct made of concrete or masonry. Its earthen section of the canal is discontinued and canal water is carried in masonry or concrete trough, canal is generally flumed in this section.