6. OTHER CANAL STRUCTURES

The two major categories of structures that are built on canals are the regulation works and cross-drainage works. A brief description of some of the important ones is given below.

Regulation Works

Canal Falls

While canals are designed with a slope which is close to the regime slope, the ground slope may differ from it considerably. Many a times, the ground slope is more than the canal slope and this may result in a canal in heavy filling. To overcome this situation, the canal has to be provided with falls (Fig.6.1) which require a masonary or concrete work.

Fig.6.1

The drop in canal bed results in the potential energy of water being converted to kinetic energy and this excess energy has to be dissipated before allowing the flow over the unprotected canal bed. Also, the water surface upstream of the fall also needs to be maintained at its normal level. The fall thus has to be provided with a crest and some means of energy dissipation. The fall can be flumed or unflumed. In a flumed fall, the trapezoidal canal section is contracted to a rectangular section having a width less than the bed width of the canal and expanded back after the works. In unflumed falls, while there is no reduction of the bed width, the section is however converted into a rectangular one.

Only two types of falls are discussed here.

Vertical drop fall

This type of fall depends on a vertical impact for energy dissipation (Fig.6.2). The crest height is determined using the formula

$$Q = C\ L\ H^{3/2}$$

Where C can be taken as 1.71, L is the length of the crest and H the head over the crest.

Fig.6.2

The energy dissipation in this case is by vertical impact of the water on the downstream bed and a pool of water. A cistern of certain depth and length- known as the cistern element is provided. There are many empirical formulae available to compute the length and depth of the cistern such as the one given by UP Irrigation Research Institute (UPIRI) as below is SI units.

$$L_c = 5\sqrt{(E\ H_L)}$$
$$x_c = 0.25(EH_L)^{2/3}$$

in which L_c and x_c are the length and the depth of the cistern respectively as shown in Fig. 6.2. Neglecting velocity head, E can be taken equal to head H over the crest and H_L equal to drop in canal bed levels.

Glacis Fall

Fig.6.3

This type of fall is preferred for larger drops and utilizes the hydraulic jump for energy dissipation (Fig.6.3). The crest is joined to the upstream floor at a slope of 1:1, while the downstream glacis is generally at at slope of 2H:1V. The downstream floor may be carried to a level lower than the canal bed for certain length to provide a cistern, with the length and depth of the cistern being 1.25 E_2 and 0.25 E_2 respectively, where E_2 is the downstream specific energy.

Distributary Head Regulator

This is the work provided at the head of a branch canal or a distributary and serves the purpose of controlling and regulating the flow into the offtake as well as metering of the flow. The arrangement is more or less similar to that of a canal head regulator, with a raised crest, upstream and downstream floors and cutoffs. The width of the regulator and height of crest are fixed such that the offtake may be able to draw its full supply discharge even if the water level in the parent channel is lower than the full supply level. On smaller works, the control is in the form of wooden planks which can be placed in grooves provided in piers for this purpose, while on larger works manually operated gates are provided. Curved vanes or cantilever platform as discussed separately are usually provided to control entry of excess sediment into the offtake

Cross Regulator

Cross regulators are structures constructed across a canal and spanning its entire width. The width is divided into suitable number of spans and provided with gates so as to regulate the flow in the canal downstream of the regulator. Cross regulators serve many purposes such as

(i) If the canal downstream of the cross regulator has to be closed in an emergency, the cross regulator gates can be closed and the discharge diverted to any drain. This requires an escape to be constructed just upstream of the cross regulator.

(ii) The canal water level upstream of the cross regulator can be regulated depending upon the gate openings. This may be required if the canal is carrying less than the full supply discharge and some offtake upstream has to be supplied with its full supply discharge.

Cross Drainage Works

These are works provided at the crossing of a canal and a stream. Depending on whether the canal crosses the stream at top, bottom or at the same level, these are divided into three categories.

Aqueducts and Siphon Aqueducts

Aqueducts are works where the canal crosses over the stream and the high flood level of the stream is lower than the canal bed level so that the flow in the stream remains an open channel flow. The canal section may cross over the stream without any modification i.e. with the banks as they are or with slight modification wherein the outer edges of the banks are replaced by retaining walls. Such works are however suitable only when the stream to be crossed is small. For any major work, the canal is flumed to a rectangular trough – masonary or concrete – and after the crossing restored to its normal section (Fig.6.4).

Fig.6.4

Siphon aqueducts are provided when the high flood level of the stream is higher than the canal bed level. In such a case the flow in the stream becomes a pressure flow through the siphon barrels (Fig.6.5)

Fig.6.5

The design of aqueducts and siphon aqueducts requires consideration of the following factors:

Waterway of stream- The waterway provided in an aqueduct is generally close to the Lacey's regime perimeter. This helps in developing a stable channel upstream of the works without much silting or scouring. The width is divided into suitable spans with the help of piers. In a siphon aqueduct, the velocity in the barrels becomes one of the considerations in deciding the waterway.

Headway i.e. the clearance between the downstream bed of the stream and the bottom of the canal trough should be sufficient so as to prevent the blockage of the barrels. While this may not present much of a problem in aqueducts, lowering of the stream bed upstream of the siphon barrels may have to be resorted to at times (Fig.6.5).

Afflux will be caused by the flow of stream under the canal trough. This results because of the head loss due to constriction, piers or siphon. Afflux can be computed using appropriate formulae for the head loss and is used in determining the hydraulic grade line specially in case of siphon aqueducts.

Fluming of the canal requires contraction as well as expansion transitions. While the splay in contraction can be kept about 1:2, the expansion is generally provided with a splay of 1:3 or more. Suitable design of transitions for contraction and expansion is required and procedures for this design are available.

The considerations of uplift and exit gradient have also to be taken care of. The worst condition for the stream bed being when the stream is dry and the canal is carrying its full supply discharge. In case of siphon aqueducts, the canal trough is also subjected to uplift when the canal is dry and the stream is in high flood.

Superpassages and Siphons

These are works where the stream crosses over the canal. In a superpassage the canal full supply level is lower than the river bed level and the flow in canal is an open channel flow. In a siphon the canal full supply level is higher than the stream bed level and therefore the canal water flows under pressure through barrels under the stream trough.

The design considerations for these works are similar to those for aqueducts and siphon aqueducts. The stream however is not flumed and mostly carried with the original section.

Level Crossing

In this work the canal and stream cross at nearly the same level (Fig.6.6). There is intermixing of the canal and river water and the flow is controlled by regulator gates on the canal as well as the stream. A sill with its top at the canal full supply level is provided on the upstream side of the stream to prevent stagnant water pool in the stream during dry season.

Fig.6.6

Level crossings have a problem with sediment getting deposited in the pool formed at the crossing. This could lead to degradation in the river downstream. Also there is need for constant watch and warning mechanism so that the stream gates could be opened well in time in case a flood has to be passed. The canal may also have to be closed during floods to prevent the river sediment from entering the canal.

Selection of Type

The selection of the type of cross drainage work depends on the relative bed levels of the canal and the stream at the crossing and their discharge. Thus in case the stream is carrying a large discharge, it may not be feasible to siphon it under the canal even though the levels may dictate a siphon aqueduct. The type of crossing can be altered by a suitable realignment of the canal if required, resulting in change in bed levels of both the canal and the stream.

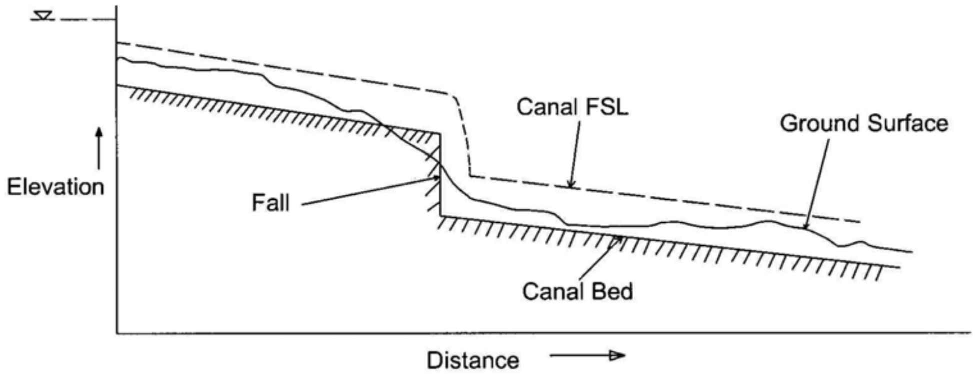


Fig. 6.1 Need for providing falls

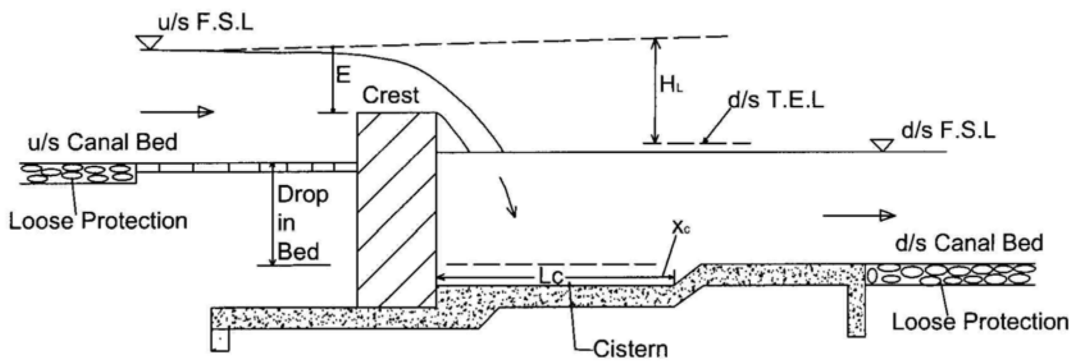


Fig. 6.2 Vertical impact fall (schematic)

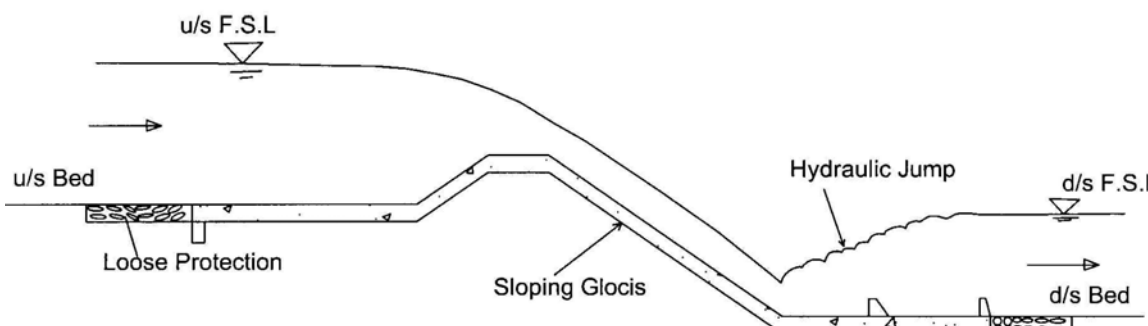


Fig. 6.3 Glacis fall (schematic)

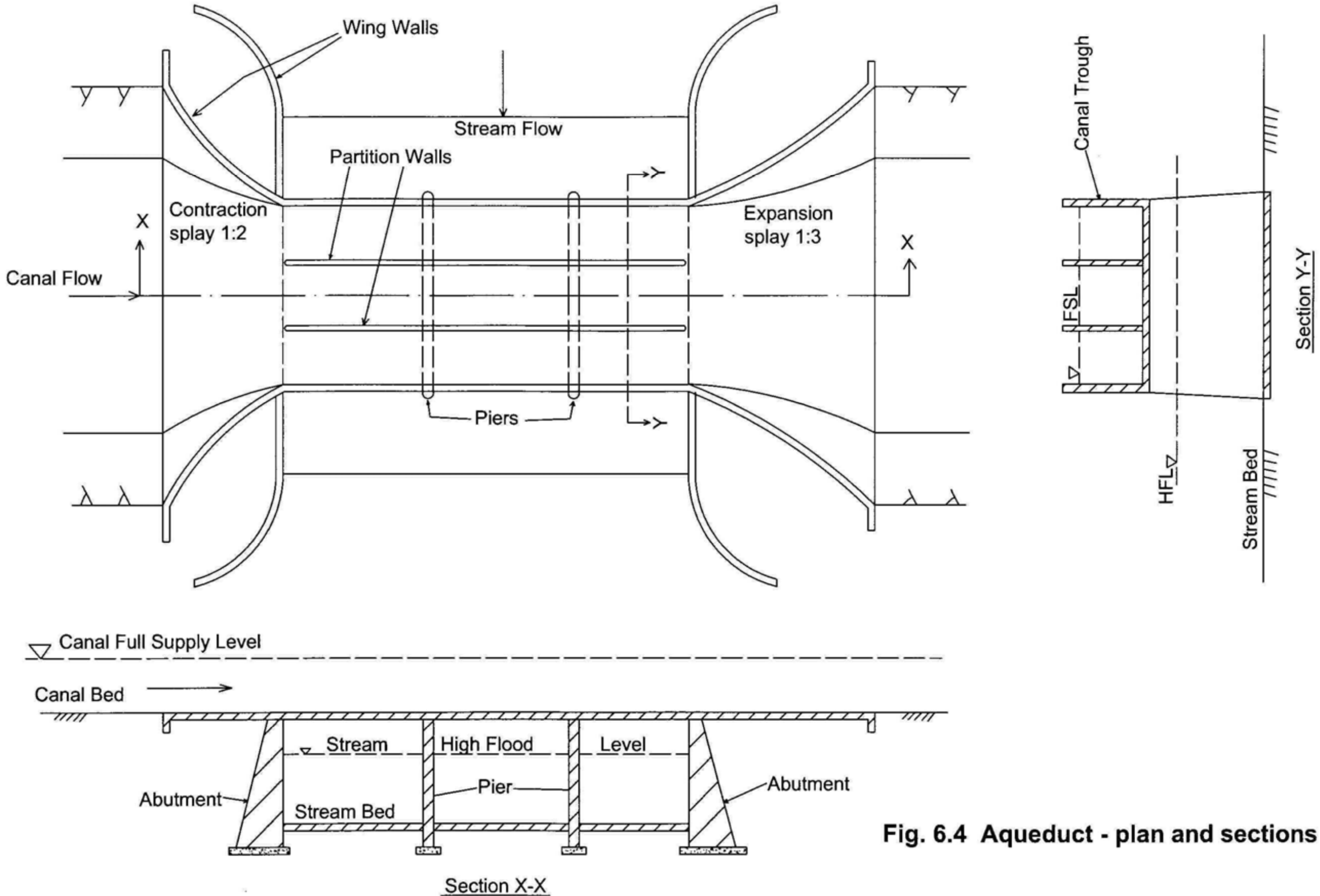


Fig. 6.4 Aqueduct - plan and sections

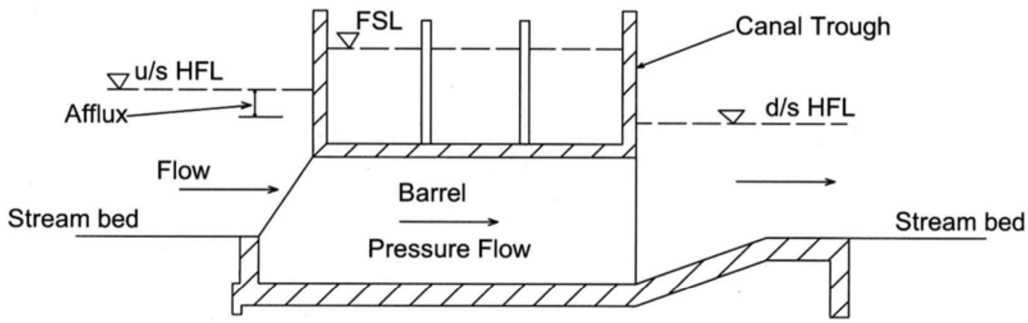


Fig. 6.5 Typical section of siphon aqueduct

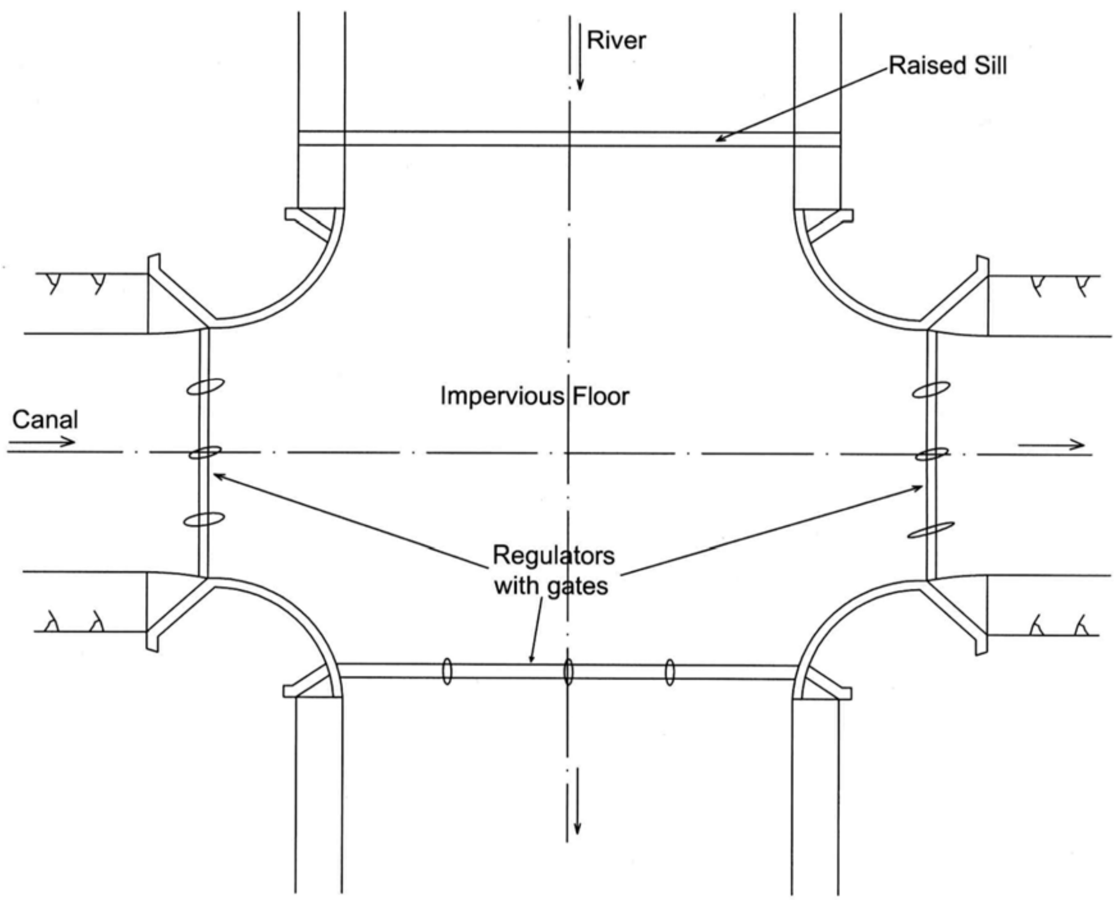


Fig. 6.6 Line diagram of level crossing

7. STORAGE SCHEMES

Storage schemes, as already mentioned, are those wherein the water is stored when available (say monsoon months) and drawn from the storage during lean periods. A reservoir created upstream of a dam constructed on a river provides the storage in such schemes. The purposes for which such schemes are constructed include:

- Irrigation
- Power generation
- Water supply
- Flood control
- Navigation
- Recreation

A scheme which serves more than one of the above purposes is termed as a multipurpose project. The extent to which the requirements of the various purposes can be met depends on the compatibility of the purposes. Thus power generation is the most compatible with other purposes in as much as it does not involve any consumptive use of water. The water released for power generation can always be used for other purposes. Likewise, flood control is the least compatible as this requires availability of empty storage space in the reservoir, while other purposes require availability of water.

PLANNING FOR A STORAGE SCHEME

The main elements of a storage scheme are the dam, spillway, the reservoir. One therefore has to decide the type of dam to be constructed, the type of spillway as well as the reservoir storage capacity. The scheme may be for storing water during the wet season and using it during the lean period in the same year or for using it during the next year also. While the former is called "within year carryover storage", the later is referred to as "over year carryover storage".

Planning for the elements of a storage scheme requires extensive data and investigations to optimize the benefits. These include:

- Estimation of demand – The storage scheme The water these for different purposes the project is supposed to fulfill has to be estimated separately. Since the projects take a long time in completion and are supposed to serve for a fairly long period, the projected population has to be estimated while estimating the demand.
- Hydrologic studies including sedimentation – These include long term data on stream flow, evaporation, water quality, sedimentation, downstream water rights etc. The water availability can only be estimated based on these investigations. Spillway capacity determination and sedimentation of the reservoir is also based on these investigations.

- Geologic studies – The nature of foundations and abutment and the availability of construction material near the site which determine to a large extent the suitability of a given type of dam is a primary objective of these investigations. Further determination of the suitability of the reservoir site, stability of its rim and the possibility of landslides also requires detailed geological investigations.
- Economic aspects – One of the major guiding principles in any project is its cost effectiveness. Estimation of the costs and benefits of the project in economic terms is required for any decision making. Both the capital and recurring costs need to be considered. The benefits cannot always be evaluated in monetary terms as these include the direct benefits as well as called indirect ones. Some of the indirect costs and benefits can be assessed by using a suitable metric wherever possible.
- Social aspects – The construction of a major project at any site is likely to have significant social impact. This may include beneficial effects such as the improved economic conditions, creation of employment opportunities etc. Some adverse effects like submergence of land, increased noise and other activity will also be a part of this. All these impacts need to be evaluated.
- Environmental impact – The environmental impact assessment of any major project has become one of the most important considerations nowadays. The factors to be considered in carrying out an environmental impact analysis are numerous and include things such as submergence of agricultural land, submergence of forest land and its effect on flora and fauna in the region, the loss of biodiversity, the impact of the project on the overall water regime including downstream channel etc. A well carried out study can at times result in some modification of the project plan which minimizes any adverse environmental impact while keeping the beneficial impacts nearly the same.

INVESTIGATIONS

Each of the aforesaid investigations requires a large amount of data and time. All the data required will normally not be available to start with and needs to be collected – an exercise which can be very costly and time consuming. Further, there are likely to be many alternative schemes possible at or close to a given location, each requiring additional data. In order to save time and expenses therefore, investigation for such project are made in three stages.

Preliminary Investigations

These are investigations carried out considering all the abovementioned aspects with the existing data. Some additional data may also be collected without spending much time and with lesser accuracy. The purpose of this is to screen out some alternatives which are considered poor and prepare a short list of promising alternatives and decide upon the additional data to be collected.

Feasibility Investigations

This stage involves collecting data with the desired accuracy and analysis of the alternatives shortlisted as a result of the preliminary investigations. This stage also will cover all the pertinent aspects of the project and forms the basis of a provisional selection of the project plan.

Detailed Investigations

Once the provisionally selected plan is approved, detailed investigations for the same have to be carried out. Fixing up the size of various components of the project, their design incorporating any additional information as well as collecting more data as required are all part of this stage. The planning for construction is also part of the work to be carried out.

DETERMINATION OF RESERVOIR CAPACITY

For Conservation Purposes

The storage capacity required to support a given firm yield can be obtained using the mass curve. Mass curve is a plot of the cumulative inflow into the reservoir – usually in million hectare metres – over a period of time (Fig.7.1). The time is generally taken over a period of years.

Fig.7.1

To determine the storage capacity, historical streamflow records are examined to identify the most adverse sequence and hence to identify the critical period. The critical period begins after a preceding high flow period when the reservoir gets full and ends when the reservoir is refilled after the drought period. The mass curve for this period is plotted. The slope of this curve at any time represents the inflow rate at that time. Likewise, a demand line will have a slope equal to the demand rate. To determine the storage capacity required, a line with a slope equal to the demand rate is drawn starting at the beginning of the critical period till it cuts the mass curve again (AB in Fig.7.1). The maximum ordinate between the mass curve and the demand line (CD) then yields the storage required for this demand rate. In case there are more than one critical periods on record, this exercise may be carried out for all of them and the largest value of storage adopted.

For Flood Control

Flood control requires empty storage space in the reservoir and hence the procedure for determination of such space is somewhat different. The major factor in flood control projects is to limit the peak overflow from the reservoir, which is dictated by considerations of the safe carrying capacity of the downstream channel and prevention of flooding of downstream areas. The procedure used for this is of flood routing through the reservoir. This has been discussed later alongwith the discussion on spillways. The only point to be noted is that if empty space is available in the reservoir then it will be filled up first and the outflow during this period will be zero. Once the water in the reservoir attains the level of the spillway crest, the procedure is similar to that discussed therein. The outflow hydrograph will thus start at a later time and its peak will be lower compared to what it would have been if the reservoir was full to start with. The outflow hydrograph with different values of empty space can be obtained and the one which has a peak equal to that desired on the downstream side then gives the storage to be reserved for flood control.

SEDIMENTATION OF RESERVOIRS

The creation of a reservoir in a river results in reduction of water surface slope and velocities on the upstream side. Consequently the sediment transporting capacity of the stream is reduced and sediment gets deposited in the reservoir. This tends to reduce the storage capacity of the reservoir and is usually accounted for by reserving a portion of the storage for sediment deposition. Sedimentation also determines the useful life of a reservoir.

The sediment deposition depends on the quantity of sediment brought by the river, the size of the reservoir and sluicing arrangements. Since sediment load comes from erosion of the catchment, reduction in rate of sedimentation of a reservoir can be achieved by an appropriate catchment management programme to prevent the erosion. Sluicing through low level outlets in the dam to flush the deposited sediment and construction of detention basins for sediment at the inlet of the reservoir are other measures which can be useful.

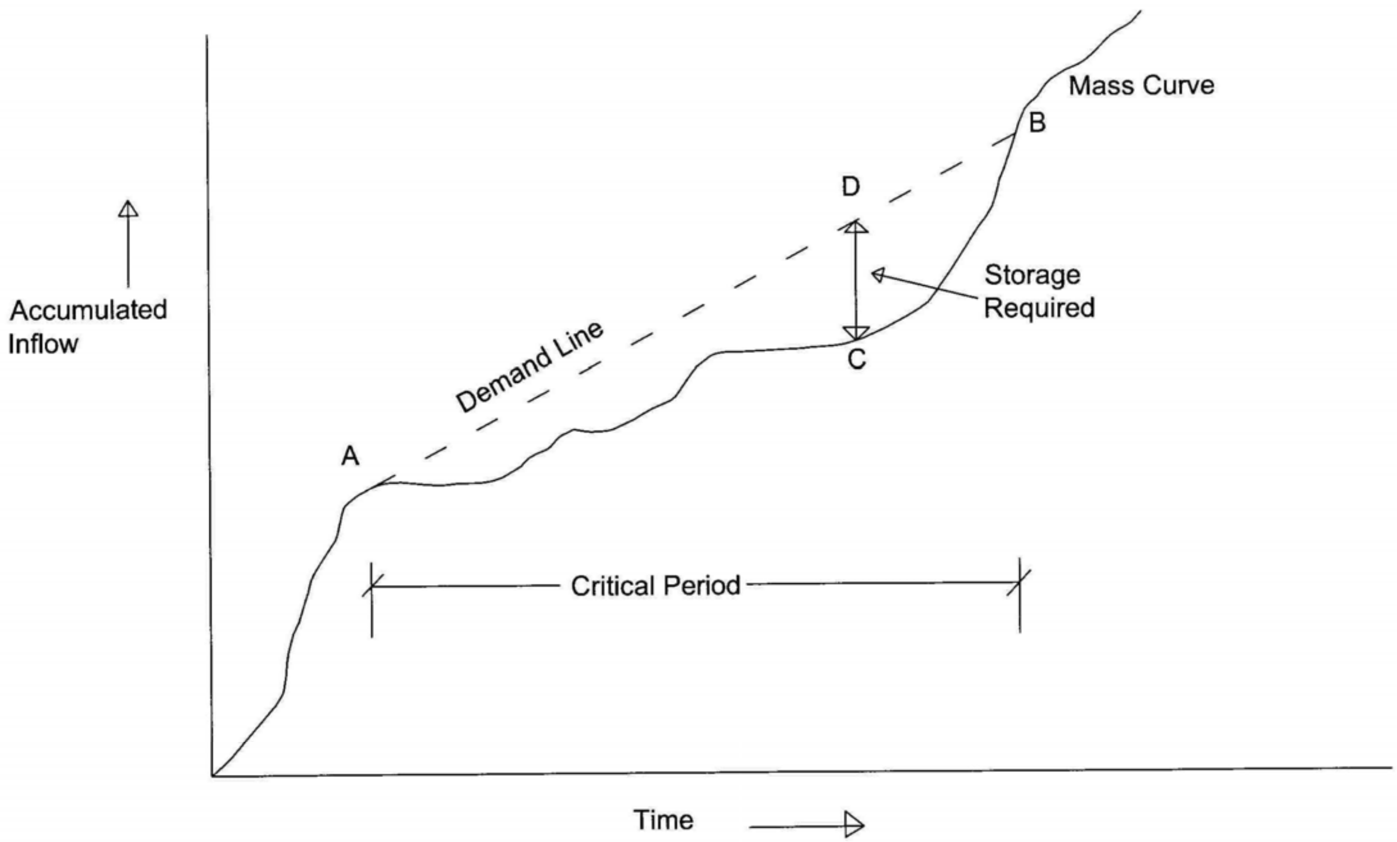


Fig. 7.1 Mass curve & storage capacity determination

8. GRAVITY DAMS

Gravity dams are rigid dams which ensure stability against all loads by virtue of their weight alone. They transfer all the loads to the foundation and hence are built when the foundation is strong rock. A typical section of a gravity dam is shown in Fig.8.1

FORCES ACTING ON A GRAVITY DAM

The forces generally acting on a gravity dam are (Fig.8.1):

Fig.8.1

Dead Load (W_D)

The dead load includes the weight of the dam and is transmitted directly to the foundations.

Water Pressure on the Upstream and Downstream Faces

The water pressure on the upstream face (P_U) depends on the water surface level in the reservoir and acts horizontally. In case the dam has a batter in the upstream side, the load of water over the batter (W_{wu}) is also present and acts vertically. Similarly, the water pressure on the downstream face (P_D) is due to the tail water and acts horizontally while the weight of water on the downstream face (W_{wd}) acts vertically

Uplift

There is always some seepage within the body of the dam as well as through the foundations and this gives rise to an uplift force (U) acting vertically upwards. The uplift pressure is assumed to be equal to the full water pressure at the upstream face and varying linearly to the tail water pressure at the downstream face. Almost all gravity dams are provided with internal drains which modify the uplift pressure distribution. Studies indicate that the pressure at the drains drops to a value equal to the tail water pressure plus one third the difference between the upstream and downstream pressures.

Silt Load

Sediment deposition in the reservoir results in a force (W_S) acting horizontally on the upstream face. This force is assumed to have a hydrostatic distribution. For the dam section having batter on the upstream side, silt load over the batter shall also act vertically down as shown in Fig. 8.1.

Earthquake Force

Earthquakes impart a horizontal as well as a vertical acceleration to the dam and the stored water. This results in additional forces, both in the horizontal and vertical directions. Horizontal and vertical "seismic coefficients" are used to appropriately modify these forces to account for the effect of earthquakes.

Other Forces

These include the force due to the impact of waves on the upstream face of the dam, ice loads in case of dams in extremely cold regions where a sheet of ice may form in the reservoir and thermal loads. Methods for estimation of these are available.

Causes of Failure of Gravity Dams

A gravity dam can fail because of the following three reasons

Overturning

If the moments of the destabilizing forces (such as water pressure on the upstream face and uplift) about the toe of the dam exceed those of the stabilizing forces (mainly the weight of the dam), the dam can overturn. This condition, however will not arise if the condition (iii) discussed below is taken care of.

Sliding

A gravity dam may fail in sliding at any horizontal plane if the sum of the actuating horizontal forces above that plane is more than the resistive forces. The actuating forces are due to water pressure, silt load etc. while the resistive forces are due to friction and the shear strength of the material of the dam. A measure of the stability against sliding is the shear-friction factor of safety (SFF), defined as

$$SFF = (CA + \mu \sum W) / \sum H$$

Where C is the cohesion, A the area of the horizontal plane, μ the coefficient of friction and $\sum W$ and $\sum H$ is the sum of the vertical and horizontal forces respectively.

The value of SFF should obviously be more than 1 and the actual value required depends on the loading conditions considered in the analysis.

Tension at the heel or excessive compression at the toe

If a vertical load W acts on the base, the normal compressive stress at the base will be W/A, A being the area of the base. This however is true only if the vertical load acts through the centre of the base. In case the load W is eccentric, the maximum and minimum stress will be given by (W/A) (1+6e/b) and (W/A) (1-6e/b) respectively, where e is the eccentricity and b the base width. Thus while for e less than or equal to b/6 no tension will develop, for e larger than this, there will be tension at the heel and even the compressive stress at the toe may become quite large. This condition is to be avoided and thus the design is made such that the resultant of all forces crosses the base within the middle one third i.e. an eccentricity of less than or equal to b/6.

Elementary Profile of a Gravity Dam

It can be shown that for a gravity dam subject only to the water pressure on the upstream face, uplift and its own weight, a right angled triangular section with its apex at the water level and adequate base width is a safe profile. Such a profile is called the elementary profile of a gravity dam. The base width of the profile is determined from the criteria of no tension at the heel and no sliding and the greater of the two is adopted.

Fig.8.2

No tension criteria

In the case of an elementary profile, if the reservoir is empty the only force acting on the base will be the self weight (W_c) of the dam. This will act through the centre of gravity of the section and thus intersect the base at a distance b/3 from the upstream face. The eccentricity is thus b/2-b/3 i.e. b/6 and therefore satisfies the condition for no tension. Since there is no horizontal force acting under these circumstances, there is no question of overturning or sliding.

For the reservoir full case, the forces acting will be the water pressure (P) and uplift (U) in addition to the self weight of the dam as shown in Fig.8.2 While the former of these forces is horizontal, the later two are vertical and hence the resultant (R) will be inclined to the vertical. In order to satisfy the criteria of no overturning this resultant must intersect the base within itself and not pass out of the base. For no tension the resultant must cut the base within the middle one third as this leads to an eccentricity of less than or equal to b/6. In the limit the resultant must cut the base at the extremity of the middle one third (point E in Fig.8.2). This automatically satisfies the no overturning criteria also. The base width required for this can be worked out as below:

$W_c = \gamma sbh/2$

$P = \gamma h^2/2$

$U = \gamma hb/2$

Since R passes through E, taking moments about E gives

$(\gamma h^2/2)\, h/3 + (\gamma hb/2)\, b/3 = (\gamma sbh/2)\, b/3$

which yields

$b = h/\sqrt{(s-1)}$

No sliding criterion

For no sliding, the friction at the base must exceed the horizontal force acting on the dam i.e. the water pressure. In the limit the two should be equal and thus one can write

$$\mu\,(W_c - U) = P$$

where μ is the coefficient of friction
This, on substitution of the appropriate values yields the required base width as

$$\mu\,(\gamma sbh/2 - \gamma bh/2) = (\gamma h^2/2)$$

or $$b = h/(\mu(s-1))$$

Design and Analysis

The preliminary design of a gravity dam starts with assuming its section. The elementary profile can serve as a guide for the same with provision of freeboard, finite crest width, batter near the bottom of the upstream face etc. The section thus obtained needs to be analysed to determine the stresses in the section and other factors of safety. This can be a simple two dimensional analysis which can be carried out using the gravity method for different loading combinations. An actual dam however, will hardly exhibit two-dimensional behaviour and will also have inspection and drainage galleries and many other features which cannot be accounted for in the gravity analysis. One will then have to resort to a much more complicated three dimensional analysis such as the trial load method or a finite element analysis, before finalizing the design. The details of the design procedure and methods of analysis are beyond the scope of the present discussion.

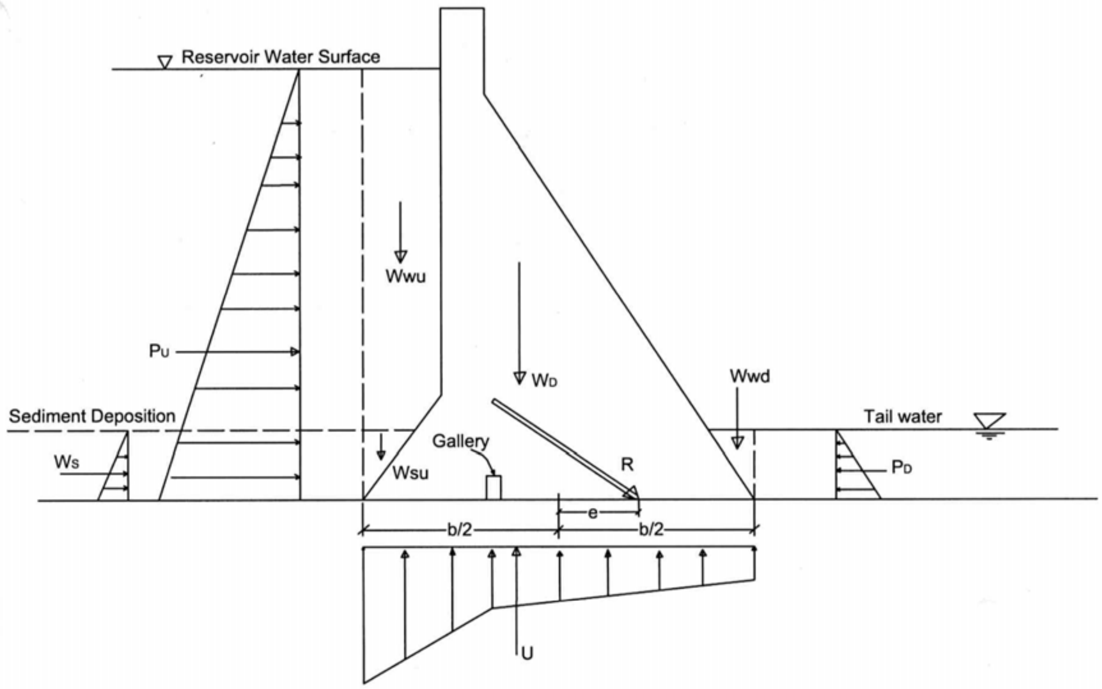


Fig. 8.1 Typical section of a gravity dam & forces acting on it

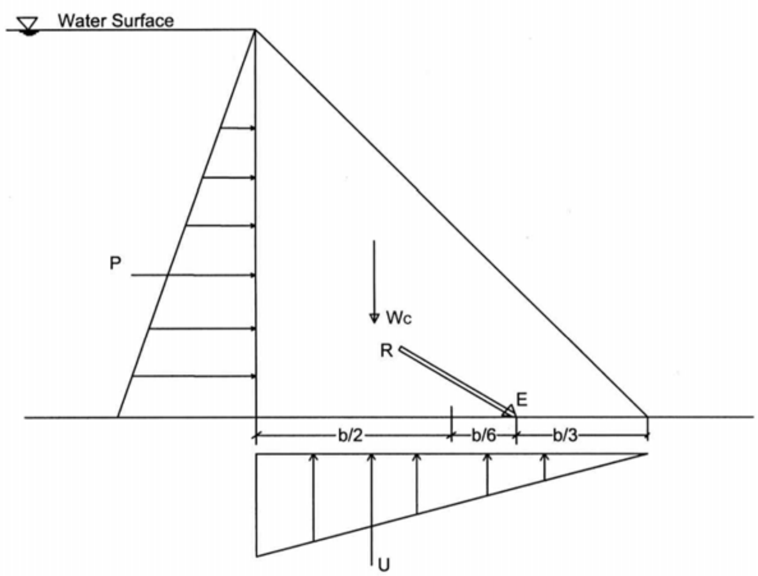


Fig. 8.2 Elementary profile of a gravity dam

9. EMBANKMENT DAMS

Embankment dams are made of earth material and are suitable for situations where the valley is wide and the foundation is weak rock or thick soil deposit and/or the abutments are also weak. This type of dam is more flexible than the rigid dam and can withstand some degree of foundation deformation more easily.

Embankment dams can be of two types:

(i) Earthfill dams
(ii) Rockfill dams

While earthfill dams or earth dams as they are generally called comprise primarily of soil material, rocks form the bulk in a rockfill dam. The design principles of both these being similar, only those for the earth dams are discussed here.

Earth dams can also have two types of sections viz. homogeneous or zoned.

Homogeneous earth dams are constructed of only one type of material and are used only when the height of the dam is small and only one type of material is available economically. In case the height of the dam exceeds about 6m, a modified homogeneous section is used. The modified homogeneous section has some drainage arrangement provided at the downstream side to provide stability as well as help in controlling the effects of seepage. Typical sections of homogeneous dams are shown in Fig.9.1.

Fig.9.1

Zoned earth dams consist of an impervious core flanked by zones of more pervious material called the shelll. The permeability of the material goes on increasing as one moves away from the core. This is the most common type of earth dam and many high earth dams such as Nurek (300m), Oroville (224m) and Ramganga (125m) are of this type. The impervious core may be centrally placed or sloping. Both have their advantages or disadvantages depending on the site conditions and availability of material. While the core makes the section impervious thereby reducing seepage through the dam, the shell supports and protects the core. Typical sections of zoned earth dams are shown in Fig.9.2 The top width of the dam depends on the height and typically varies from about 2.5m to 6m but can be much more for high dams.

Fig.9.2

CRITERIA FOR SAFETY OF EARTH DAMS

The design of earth dams is carried out to conform to the criteria of safety for the same. These are based on observations on existing dams and study of failures. The main criteria can be listed as below:

(i) No overtopping i.e. water must not flow over the dam under any circumstances as this could lead to certain failure of the dam.
(ii) The slopes, both upstream and downstream must be stable under all conditions
(iii) The upstream face must be protected against the action of waves and the downstream one against action of rain.
(iv) The seepage line must be well within the downstream face
(v) There should be no free flow of water through the dam body.
(vi) The seepage through the foundations must be controlled and not allowed to cause piping
(vii) The foundation shear should be within permissible limits

SAFETY AGAINST OVERTOPPING

Majority of the failures of earth dams have been a result of overtopping. In order to avoid this, two steps are necessary. The first involves providing adequate spillway capacity. Liberal provision of spillway capacity in earth dams as compared to rigid dams is mostly resorted to in order to keep the maximum reservoir level within estimated limits. The second step is to provide adequate freeboard – the vertical distance between the dam crest and the still water level in the reservoir. Since the normal reservoir level will be less than the maximum reservoir level i.e. the level when the design flood occurs, the freeboard with respect to the former is termed as the normal freeboard while that with respect to the latter is called the minimum freeboard. The freeboard is also supposed to take into account the safety against overtopping due to the settlement of the dam and foundations. The parameters on which the computation of freeboard is based are the wave height, wave run up and wind set up with an additional margin added for uncertain effects such as settlement, earthquakes etc. The freeboard is given by:

Freeboard = Greater of design wave height or wave run up + wind set up + margin for uncertain effects

The computation for design wave height requires determination of the effective fetch f_e and the wind velocity V for the reservoir. The wave height is then determined by:

$$gH_w/V^2 = 0.0026\,(gf_e/V^2)^{0.47}$$

in which H_w is the wave height in metres, V the wind velocity in m/s and f_e the effective fetch in metres, g being the acceleration due to gravity. The design wave height H_d is taken as 1.67 times the value H_w obtained from the above formula.

The wave run up R depends on the design wave height, the ratio of design wave height to wave length and the embankment slope and roughness and can be determined from curves relating the first three and correction factors available for roughness. The greater of H_d and R is used in computing the freeboard with the proviso that it should not be less than 2.0m for both the normal and minimum freeboard.

STABILITY OF SLOPES

Both the upstream and downstream slopes of an earth dam need to be tested for their stability under different conditions. The conditions for which the testing is required are listed below:

(i) End of construction

Soils derive their strength for withstanding shear from cohesion and friction. The friction is a result of the effective stress between soil particles.

The pore spaces between the soil particles are filled with water and this exerts a pressure called pore pressure. The effective intragranular stress is the total stress minus the pore pressure. The shear strength of the soil can be given by:

$$s = c + (\sigma - u) \tan \varphi$$

In which s is the shear strength, c the cohesion, σ the total stress, u the pore water pressure and φ is the angle of internal friction for the soil.

During construction, soil is compacted after pouring water over it and therefore large pore pressures develop, thereby reducing the effective stress and hence the shear strength. The pore pressures dissipate with time and the shear strength increases. In case of rapid mechanized construction, there is not enough time for the pore pressures to get dissipated and hence it is important to ensure the stability of slopes during construction as well as at the end of construction.

(ii) Reservoir partially filled

The stability of the upstream slope needs to be checked for the condition when the reservoir is partially filled. This is so because under this condition, the upstream slope will be partly submerged and partly dry. The checking is generally done at different levels of water in the reservoir.

(iii) Sudden drawdown

This condition pertains to the rapid removal of water from the reservoir. If a full reservoir is suddenly emptied, the upstream face will not have any weight of water on it thereby reducing the intragranular stress. The pore water pressure will however correspond to the full reservoir level and will take time to dissipate. The effective stress will thus be reduced considerably resulting in decreased shear strength which could lead to the failure of the upstream slope. The upstream slope is therefore checked for this condition.

(iv) Steady seepage

When the reservoir is full and seepage is taking place through the dam, the downstream slope is affected by the seepage. It therefore needs checking for the case of steady seepage.

Besides the above, the stability of slopes also to be ensured for conditions such as earthquake, heavy rainfall with seepage etc. These are however not included in the present discussion.

Method of Stability Analysis

There are many methods of analysis for determining the stability of slopes. However, only the method of slices, also called the Swedish method will be discussed here. In this method, like in many others also, a surface of failure of embankment is assumed and the factor of safety for the same worked out. This process is repeated with a number of trial surfaces and the one with the least factor of safety is called the critical slip surface. If the factor of safety for the critical slip surface is more than one, the slope is taken to be stable. The Swedish method assumes that the slip surface is an arc of a circle. Some guidelines for getting the critical slip circle are also available, which serve to reduce the trials in as much as one could assume slip circles in the vicinity of the one predicted by these guidelines and choose the one with the smallest factor of safety.

Fig.9.3

The following procedure is adopted for analyzing the stability along an assumed slip circle (Fig.9.3).:

(i) The assumed slip circle is divided into a number of slices. The arc at the bottom of a slice should normally be contained wholly in one type of material.

(ii) For each slice, assuming unit length of the dam, the actuating and resisting forces are computed. This is done as follows:

(a) The weight of the slice W is the volume multiplied by the appropriate unit weight.

(b) The normal component of this weight N is given by W $\cos\alpha$, while the tangential component T is W $\sin\alpha$.

(c) The uplift force U acts at the bottom of the slice and can be given by $ub/\cos\alpha$, where u is the average unit pore pressure, b the width of the slice and α is the angle that the normal to the bottom of the slice makes with the vertical.

(d) The net normal force thus is (N-U) and results in a shear strength of (N-U) $\tan\varphi$.

(e) The shear strength due to cohesion C is $cb/\cos\alpha$ and therefore the total shear strength is C + (N-U) $\tan\varphi$, where φ is the angle of internal friction.

(f) As is clear from the free body diagram of a slice as shown, the actuating force i.e. one which tends to cause the sliding is T, while the resisting force is C+(N-U) $\tan\varphi$.

(g) There are also forces F_L and F_R acting on the left and right side of the slice. It is difficult to estimate these and they are generally assumed to cancel out.

(iii) The computations are carried out in a tabular form and the sum of the actuating and resisting forces S and T is obtained.

(iv) The factor of safety is computed as

$$F.S. = \sum S / \sum T$$

In case the pore water pressures are not directly taken into account in the analysis i.e. if u is ignored, the same can be indirectly taken care of to some extent by using the submerged unit weight of the soil for computation of the resisting forces and the saturated weight for computing the actuating forces for the soil mass below the phreatic line. The moist weight is used in computing these for the soil mass above the phreatic line.

The factor of safety for both the upstream and downstream slopes for all conditions must be greater than one, though the actual acceptable value will depend on a number of factors such as the condition under which the slope is being tested, the shape and size of the dam, the site conditions etc.

In addition to being stable, the upstream slope of the dam has to be protected against wave action. This is generally done by providing stone pitching on the upstream slope or providing dumped rip rap over a suitable filter. Likewise, the downstream slope needs protection against rain, which can cause deep gullies on this slope. The best protection against rain is turfing – growing grass over this face. In situations where this is not feasible for one reason or the other, the downstream face is also protected with pitching or dumped rip rap.

SEEPAGE CONSIDERATIONS

An earth dam being composed of earth material, water is bound to seep through it as well as through the foundations. Though an impervious core is provided in the earth dams, this is never truly impervious, but only has a low permeability compared to the shell. Certain quantity of water thus will seep through the core also. Controlling the quantity of seepage and the effects of seepage through the dam and its foundations is thus very important both from the point of view of water conservation as well as the safety of the dam.

The two dimensional seepage through homogeneous, isotropic and incompressible porous media is governed by the Laplace equation

$$(\partial^2 h/\partial x^2) + (\partial^2 h/\partial y^2) = 0$$

Where h is the seepage head.

The solution of the aforesaid equation with appropriate boundary conditions will therefore yield all the needed information about the seepage through an embankment dam or its foundations. There are various means of solving the equation including the graphical, analytical and numerical methods. The present discussion will be confined mainly to the graphical method and results obtained therefrom.

The graphical method of solving the Laplace equation involves drawing the flownet for given boundary conditions. In the case of an embankment dam, the topmost streamline, also called the phreatic line is not known beforehand and hence the difficulty in drawing the flownet. This is tackled by making use of the solution given by Kozeny, according to which, the phreatic line for an embankment dam with a parabolic upstream face and a horizontal drain as shown in Fig.9.4. will be a parabola with its focus F at the start of the horizontal drain and vertex at V where

$$\text{FV} = \frac{1}{2}\left(\sqrt{d^2 + h^2} - d\right)$$

And $\text{FG} = 2\ \text{FV}$

The discharge per unit length of the dam will then be given by

$$q = K\ (\sqrt{(d^2 + h^2)} - d)$$

where K is the coefficient of permeability of the embankment material.

Fig.9.4

However, since the actual embankment dam sections do not have a parabolic upstream face or may not have a filter which is horizontal, the phreatic line in such cases will not be given by the Kozeny's parabola (also referred to as the base parabola).

Fig.9.5

Casagrande obtained the flownets for a variety of sections of embankment dams and found that the phreatic line by and large followed the base parabola with departures at the entrance and exit points. Thus, while for a homogeneous section with no drains, the base parabola will have its focus at the point D (Fig.9.5), it will start at a point H where EH is equal to 0.3 EK. The actual phreatic line will however, start at the point E, normal to the face AB and take a reverse curve to join the base parabola tangentially. At the exit end, the actual phreatic line will join the face CD tangentially at I, while the base parabola cuts this face at J. The value of $\Delta a/(a+\Delta a)$ for different values of the angle α has been given by Casagrande.

α^{o}	30	60	90	135
$\frac{\Delta a}{a + \Delta a}$	0.36	0.32	0.26	0.13

Fig.9.6

In case the section has a drain other than horizontal such as a rock toe (Fig.9.6), the focus of the base parabola will be at F and the starting point will be at H as for the case discussed above. The corrections at the entry and exit will also be determined in a manner similar to what has been discussed above.

Having drawn the phreatic line, the complete flownet can be drawn for the embankment and the requisite information regarding seepage through the same obtained.

The discharge through the dam can still be determined by the equation

$$q = K\left(\sqrt{\left(d^2 + h^2\right)} - d\right)$$

with the value of d being taken as the horizontal distance between the point H and the focus of the base parabola.

For a zoned embankment section, generally the shell is many times more permeable than the core and hence the aforesaid analysis needs to be done for the core only, which can be taken as homogeneous. Analysis on lines similar to the above can also be carried out if the above condition is not satisfied or in cases where there is a variation of permeability in the horizontal and vertical directions. These cases have, however not been discussed here.

As already mentioned, all earth dams are provided with some drainage at the downstream end. This ensures that the phreatic line does not cut the downstream face thereby preventing the chances of sloughing of the downstream slope. In addition, such a drain also controls the outgoing seepage water such that it does not remove soil particles i.e. prevents piping.

The seepage through the foundations can become an important parameter, specially if the dam is founded on pervious material and can be analysed using a flownet. Reduction of the quantity of water seeping through the foundations is important in such cases. This can be done by use of cutoffs- partial or complete. These are shown in Fig.9.7. The cutoff can be a rolled earth one, which is economical if the depth of the pervious foundation is relatively small, with a partial cutoff being effective only to a limited extent. Other means such as a slurry trench filled with clay and bentonite mixture, sheet piles or concrete cut off wall can also be used for somewhat larger depths of pervious material.

Fig.9.7

In case the depth of the pervious strata is large, a horizontal impervious blanket can be used to reduce the quantity of seepage through the foundations. The horizontal blanket is provided on the upstream side (Fig.9.8) and consists of relatively impervious material with thickness of the order of 0.75m to even 3m or more.

Fig.9.8

The reduction in seepage with a blanket can be computed as below:

In the absence of the blanket, the seepage discharge through the foundation can be given by

$$Q = K_f\,(H/x_c)D_f$$

where K_f and D_f are the permeability and depth of the foundation respectively.

If a blanket of length x_b is provided as shown, the discharge through the foundation is given by

$$Q_b = K_f\,((H/(x_c+x_b))D_f$$

The reduction in discharge p is thus

$$p = Q_b/Q = x_c/(x_c+x_b)$$

It may however be mentioned that the above is a simplified analysis and the effectiveness of the blanket will reduce with increasing length.

The seepage water should not get a free flow path through the body of the dam, because in such a case the flowing water can dislodge soil particles and create a cavity within the dam body by piping. This could ultimately lead to the failure of the dam. Such a free flow path can generally be available along the outside of outlet pipes etc. embedded within the dam and as such these are best avoided in an earth dam. In case it becomes necessary to embed the same, extreme care has to be taken.

FOUNDATION SHEAR

The foundation of an embankment dam must be safe in shear. Thus the shear strength of the foundation material should be more than the shear to which it is subjected. The distribution of shear on the foundation is not uniform. The factor of safety being the ratio between the shear strength and the shear intensity, it will vary with the distribution and should be more than one at the location of maximum shear. Simplified procedures are available, which can give a good idea of the shear as well as its distribution and should be used to check the safety of the foundation.

Design Procedure

The design of an embankment dam starts with an assumed section based on the availability of material, foundation conditions etc. and its modification based on the criterion of safety as discussed above. Economy also plays a very important role in selecting from a number of alternatives available before the section is finalised.

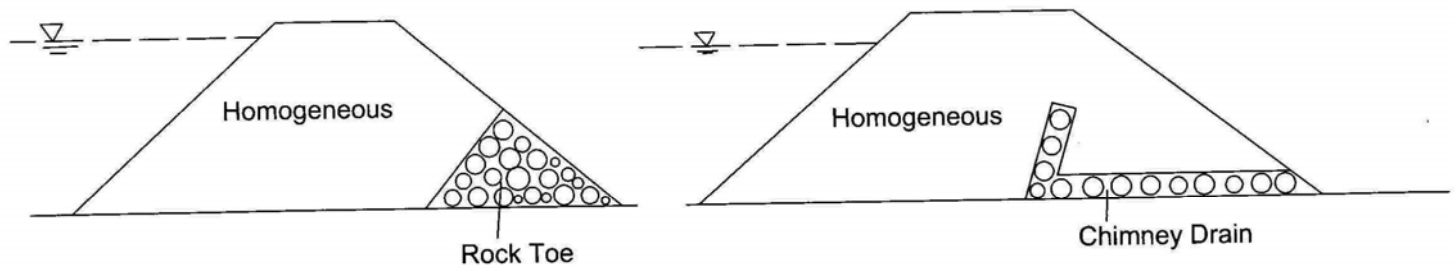


Fig. 9.1 Typical section - homogeneous embankment dams

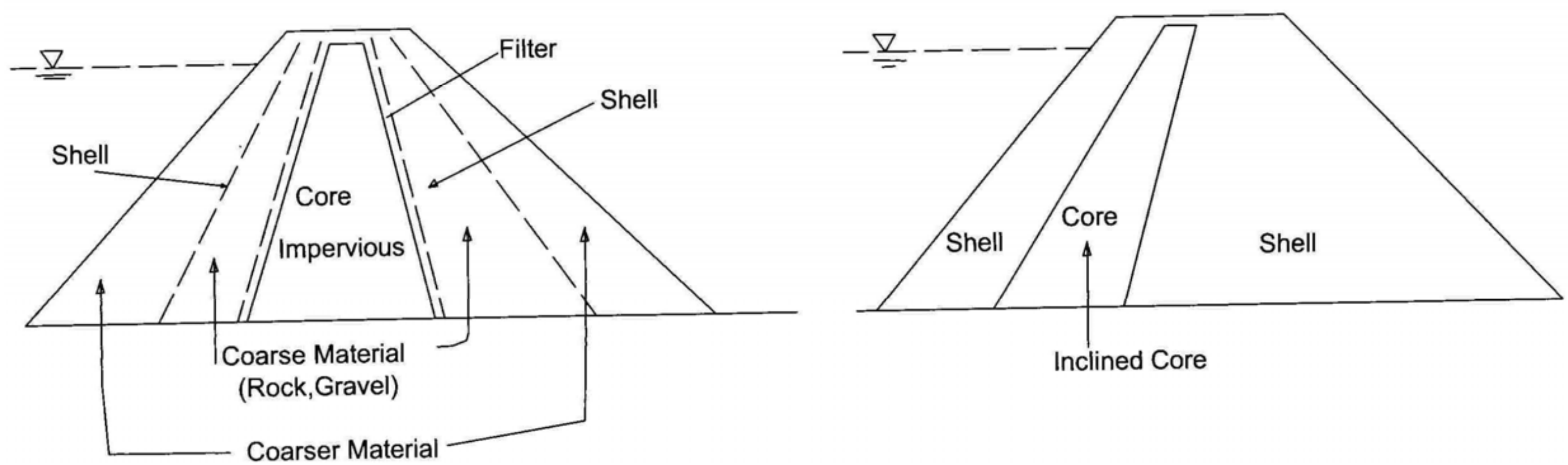


Fig. 9.2 Typical section - zoned embankment dams

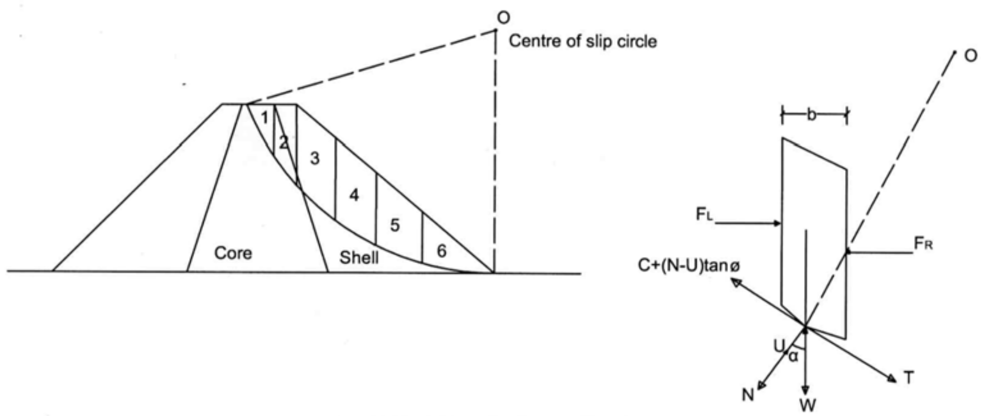


Fig. 9.3 - Slip circle method

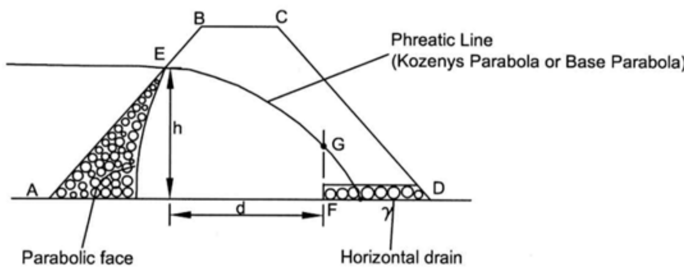


Fig. 9.4 - Kozeny's base parabola

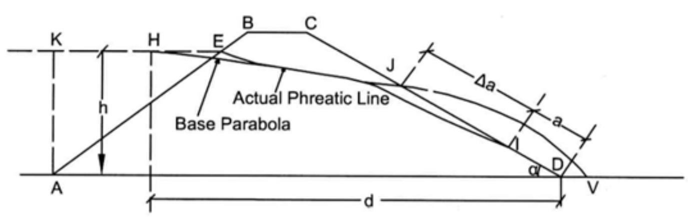


Fig. 9.5 - Modifications of kozeny's parabola

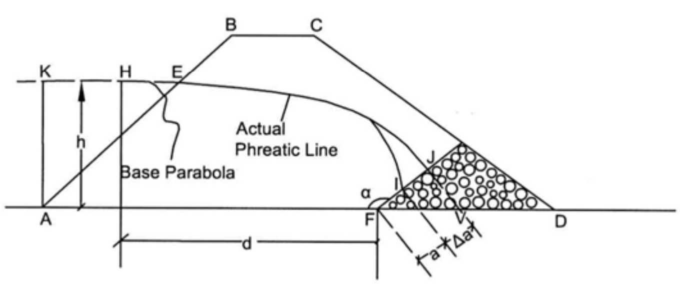
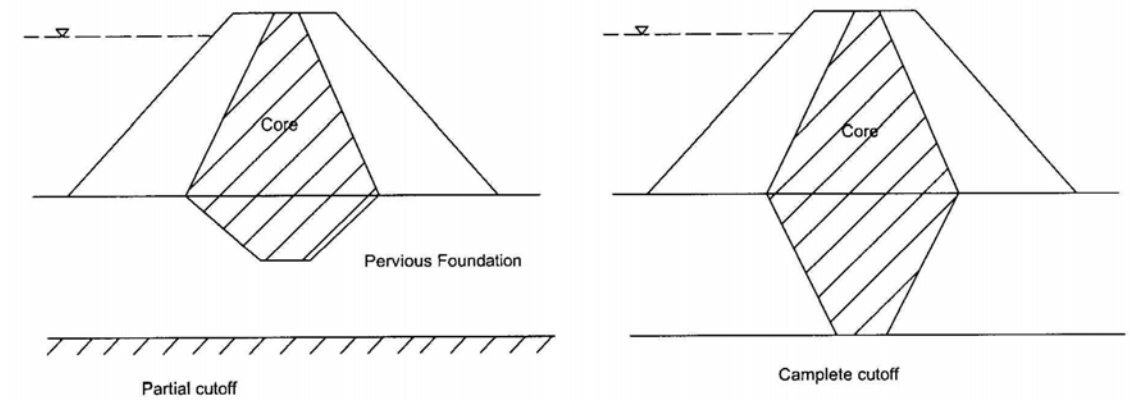


Fig. 9.6 - Phreatic line determination - rock toe



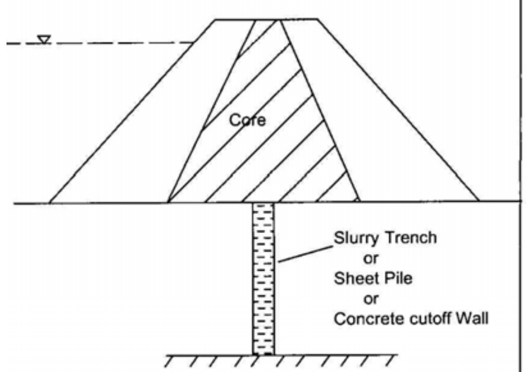


Fig. 9.7 Cutoffs

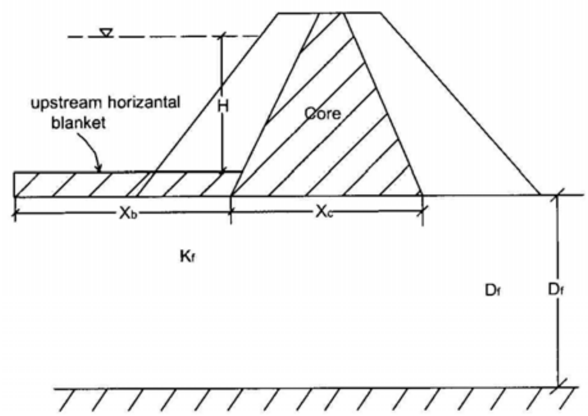


Fig. 9.8 Horizontal blanket

10. SPILLWAYS

Spillways are structures constructed to provide safe release of flood waters from a dam to a downstream area - normally the river on which the dam has been constructed.

Every reservoir has a certain capacity to store water. If the reservoir is full and flood waters enter the same, the reservoir level will go up and may eventually result in overtopping of the dam. To avoid this situation, the flood has to be passed to the downstream and this is done by providing a spillway which draws water from the top of the reservoir. A spillway can be a part of the dam or separate from it.

Spillways can be controlled or uncontrolled. A controlled spillway is provided with gates which can be raised or lowered. Controlled spillways have certain advantages as will be clear from the discussion that follows.

When a reservoir is full, its water level will be the same as the crest level of the spillway. This is the normal reservoir level. If a flood enters the reservoir at this time, the water level will start going up and simultaneously water will start flowing out through the spillway. The rise in water level in the reservoir will continue for some time and so will the discharge over the spillway. After reaching a maximum, the reservoir level will come down and eventually come back to the normal reservoir level. The top of the dam will have to be higher than the maximum reservoir level corresponding to the design flood for the spillway, while the effective storage available is only upto the normal reservoir level. The storage available between the maximum reservoir level and the normal reservoir level is called the surcharge storage and is only a temporary storage in uncontrolled spillways. Thus for a given height of the dam, part of the storage – the surcharge storage – is not being utilised. In a controlled spillway, water can be stored even above the spillway crest level by keeping the gates closed. The gates can be opened when a flood has to be passed. Thus controlled spillways allow more storage for the same height of the dam.

Many parameters need consideration in designing a spillway. These include the inflow design flood hydrograph, the type of spillway to be provided and its capacity, the hydraulic and structural design of various components and the energy dissipation downstream of the spillway. The topography, hydrology, hydraulics, geology and economic considerations all have a bearing on these decisions.

For a given inflow flood hydrograph, the maximum rise in the reservoir level depends on the discharge characteristics of the spillway crest and its size and can be obtained by flood routing. Trial with different sizes can then help in getting the optimum combination.

FLOOD ROUTING

Flood routing through a reservoir involves determination of the outflow over the spillway and change in reservoir elevation corresponding to a given inflow hydrograph. Any inflow into the reservoir causes a change in the reservoir elevation as well as in outflow and the continuity equation dictates that the inflow must equal the outflow plus the change in storage. This forms the basis for flood routing. The known quantities being the inflow hydrograph, the reservoir elevation versus storage curve and the reservoir elevation versus the spillway discharge curve (Fig.10.1).

Fig.10.1

The basic book keeping equation can be written as

$$(I_1 + I_2)\, \Delta t/2 - (O_1 + O_2)\, \Delta t/2 = (S_2 - S_1)$$

where Δt is a interval of time, I, O and S are the inflow, outflow and storages respectively with the subscript 1 corresponding to the beginning and 2 to the end of the time period respectively.

To carry out the process, a suitable time interval is chosen, beginning at the start of the inflow hydrograph. The inflows at the beginning and end of this interval are obtained from the inflow hydrograph. The initial reservoir level – usually the same as the spillway crest level – being known, the outflow and storage corresponding to this is read from the relevant curves. A value for the reservoir elevation at the end of the period is then assumed and the outflow and storage corresponding to this are also read. If these values satisfy the continuity equation as given above then this becomes the reservoir elevation at the end of the period, otherwise the trial elevation is revised till the above equation is satisfied. This value then gives the elevation and outflow at the end of the time period. With these values as the initial values, the process is repeated for the next time interval and so on till the whole of the outflow hydrograph has been obtained. This computation also yields the maximum reservoir elevation.

TYPES OF SPILLWAYS

There are different types of spillways that can be provided depending on the suitability of site and other parameters. Generally a spillway consists of a control structure, a conveyance channel and a terminal structure, but the former two may be combined in the same for certain types. The more common types are briefly described below.

Ogee Spillway

The Ogee spillway is generally provided in rigid dams and forms a part of the main dam itself if sufficient length is available. The crest of the spillway is shaped to conform to the lower nappe of a water sheet flowing over an aerated sharp crested weir. The profile has been studied extensively by the United States Bureau of Reclamation (USBR) (Fig.10.2).

Fig.10.2

The profile to the right of the crest is given by

$$y/H_d = -k\,(x/H_D)^n$$

where the value of k and n depends on the slope of the upstream face of the spillway and is available in the form of curves, being 0.5 and 1.87 respectively for a vertical upstream face. H_D is the design head which is taken as 0.75 H_{max}, being maximum expected head over the spillway.

The profile to the left of the crest is given by a double circle as shown in the figure. The values of the parameters defining these circles can also be read from curves given by USBR.

The profile given by the above equation to the right is continued till a point at which the tangent to the curve has a slope equal to the slope of the downstream face of the dam. Thereafter it continues at the same slope and given a reverse curve near the bottom.

The discharge over an ogee crest is given by

$$Q = C\,L\,H^{3/2}$$

Where L is the effective length of the crest, H the head over the crest and C is a coefficient which depends- besides other factors – on the ratio of H to the design head H_D.

If the spillway is operated at heads less than the design head, the sheet of water will have a tendency to press against the spillway surface resulting in positive pressures over the surface and in reducing the value of C. At the design head, the pressures over the surface will be atmospheric and at larger heads, these will be below atmospheric i.e. negative. The negative pressures will result in increased value of C and thus are advantageous from the discharging capacity point of view. Large negative pressures could however cause stability problems. The operating head therefore is not allowed to exceed the design head by more than a certain amount. This can be ensured by designing the crest for a head which is about 75-80 % of the head expected for the design flood.

Chute (Trough) Spillway

In this type of spillway, the water, after flowing over a short crest or other kind of control structure, is carried by an open channel (called the "chute" or "trough") to the downstream side of the river (Fig.10.3). The control structure is generally normal to the conveyance channel. The channel is constructed in excavation with stable side slopes and invariably lined. The flow through the channel is supercritical. The spillway can be provided close to the dam or at a suitable saddle away from the dam where site conditions permit.

Fig.10.3

This type of spillway is ideally suited for embankment dams and for rigid dams in narrow valleys where the river bed immediately downstream of the dam is of erodible material.

Side Channel Spillway

Side channel spillways are located just upstream and to the side of the dam (Fig.10.4). The water after flowing over a crest enters a side channel which is nearly parallel to the crest. This is then carried by a chute to the downstream side. Sometimes a tunnel may be used instead of a chute.

Fig.10.4

The crest is usually an ogee profile and generally straight in plan though shapes like “L” or “U” have also been sometimes used.

This type of spillway is specially suited in dams on narrow valleys where sufficient length for the spillway crest may not be available otherwise or when a large crest length is required to keep the rise in reservoir level low.

Shaft (Morning Glory or Gloryhole) Spillway

This type of spillway utilizes a crest circular in plan, the flow over which is carried by a vertical or sloping tunnel on to a horizontal tunnel nearly at the streambed level and eventually to the downstream side (Fig.10.5). The diversion tunnels constructed during the dam construction can be used as the horizontal conduit in many cases.

Fig.10.5

The crest can be a standard crest or a flat crest. While the former has a larger discharge coefficient, the later requires smaller funnel diameter and hence economical if excavation has to be carried out. The standard crest conforms to the lower nappe of flow over a circular sharp crested weir. The ideal condition favouring this type of spillway is when there is a rock outcrop in the reservoir somewhat upstream of the dam.

Problems frequently encountered in this type of spillway are vortex action, instability of flow and cavitation. Radial piers are generally used at the crest to suppress vortex formation.

Siphon Spillway

As the name indicates, this spillway works on the principle of a siphon. A hood provided over a conventional spillway forms a conduit (Fig.10.6). With the rise in reservoir level water starts flowing over the crest as in an ogee spillway. The flowing water however, entrains air and once all the air in the crest area is removed, siphon action starts. Under this condition, the discharge takes place at a much larger head. The spillway thus has a larger discharging capacity. The inlet end of the hood is generally kept below the reservoir level to prevent floating debris from entering the conduit. This may cause the reservoir to be drawn down below the normal level before the siphon action breaks and therefore arrangement for depriming the siphon at the normal reservoir level is provided.

Fig.10.6

One of the important aspects of the siphon spillway is its priming and therefore priming devices such as a joggle or baby siphon are used to ensure early priming. Cracking of the hood can lead to depriming by allowing entry of air. The spillway is therefore provided in batteries so that the whole spillway does not get deprimed by the cracking of one portion of the hood.

While this kind of spillway has a larger discharge for the same rise in reservoir level, it has problems of vibration and noise. Cavitation can also be a problem in some cases.

TERMINAL STRUCTURES – ENERGY DISSIPATION

The water flowing over a spillway loses a large amount of its potential energy. A good percentage of this is converted into kinetic energy and subsequently the flow at the toe of the spillway is a high velocity flow. If allowed as such to flow in the river, it is likely to cause considerable bed erosion and as such some sort of energy dissipation is required before allowing this flow into the river. Generally two major types of energy dissipating devices are used for spillways. These are the hydraulic jump type stilling basin and the bucket type energy dissipators.

Hydraulic Jump type Stilling Basins

As the name indicates, these basins employ the hydraulic jump as the energy dissipation mechanism. The characteristics of the jump- such as the length, efficiency in energy dissipation etc.- depend on the initial Froude number and the tailwater conditions. Appurtenances such as chute blocks, baffle blocks and end sill are also used to increase the efficiency as well as to decrease the length of the basin (Fig.10.7).

Fig.10.7

A comprehensive study of this type of basins was carried out by the United States Bureau of Reclamation and certain types of basins have been recommended by them for various initial Froude numbers and inflow velocities. These basins- referred to as USBR Type II, Type III etc.- are suitable for different ranges of initial Froude numbers. The appurtenances to be used in each type are also specified. The dimensions of the basin as well as of the appurtenances to be employed in each case are given in the form of curves, which can be used to design the basin for a given set of conditions.

Bucket Type Energy Dissipators

These are used when the tailwater depth is either too low or too high for the formation of a hydraulic jump, rendering a jump type basin uneconomical. The bucket can be a ski-jump bucket or a roller bucket.

1. **Ski Jump Bucket**

Fig.10.8

This type of bucket is used when the tailwater depth is quite low for the formation of a jump. The water leaves the bucket as an upturned jet (Fig.10.8) and strikes the river bed somewhat downstream of the spillway. During its trajectory, the jet splits into smaller jets and part of the energy is dissipated due to air friction. The bulk of the energy dissipation however takes place due to the impact of the jet on the water and river bed downstream. This also requires that the river bed be comprised of hard rock to withstand the impact of the jet.

2. **Roller Bucket**

Fig.10.9

This type of bucket is used when the tailwater depth is too large for the formation of a jump. The water entering the bucket forms a roller- called the bucket roller- within the bucket and another one – called the ground roller- just downstream of the bucket (Fig.10.9 a). While the former is anticlockwise, the latter moves in a clockwise direction. Energy dissipation takes place because of the interaction between the two rollers and the intermingling of the inflow with the same. The ground roller has a tendency to pile up loose material against the bucket lip and if some of this enters the bucket, it will keep moving with the bucket roller and can cause objectionable abrasion in the concrete surface. To avoid this, a slotted bucket is sometimes used instead of a solid bucket. The slotted bucket has teeth and gaps (Fig.10.9 b) and leads to better flow conditions downstream, besides allowing any material that may enter the bucket to leave through the gaps.

SPILLWAY GATES

There are three major types of gates provided in spillways. These are (Fig.10.10):

Fig.10.10

1. **Vertical Lift Gates**

   These gates are made of steel plate and move in gate grooves provided in the supporting piers. They move vertically in their own plane and are operated from a hoist chamber, which has to be at a higher elevation than the raised position of the gates.

2. **Tainter Gates**

   Also called radial gates, these are segments of a cylinder made of steel plate and connected to a trunnion at the centre of the arc. The hoist chamber is suitably located and normally does not have to be as high as in case of vertical lift gates.

3. **Drum Gates**

   These are in the form of a floating drum which is hinged at the top and sits in a float chamber within the spillway crest. Raising of the gate is accomplished by allowing water under pressure into the float chamber, while for lowering the same another valve is used to empty the float chamber.

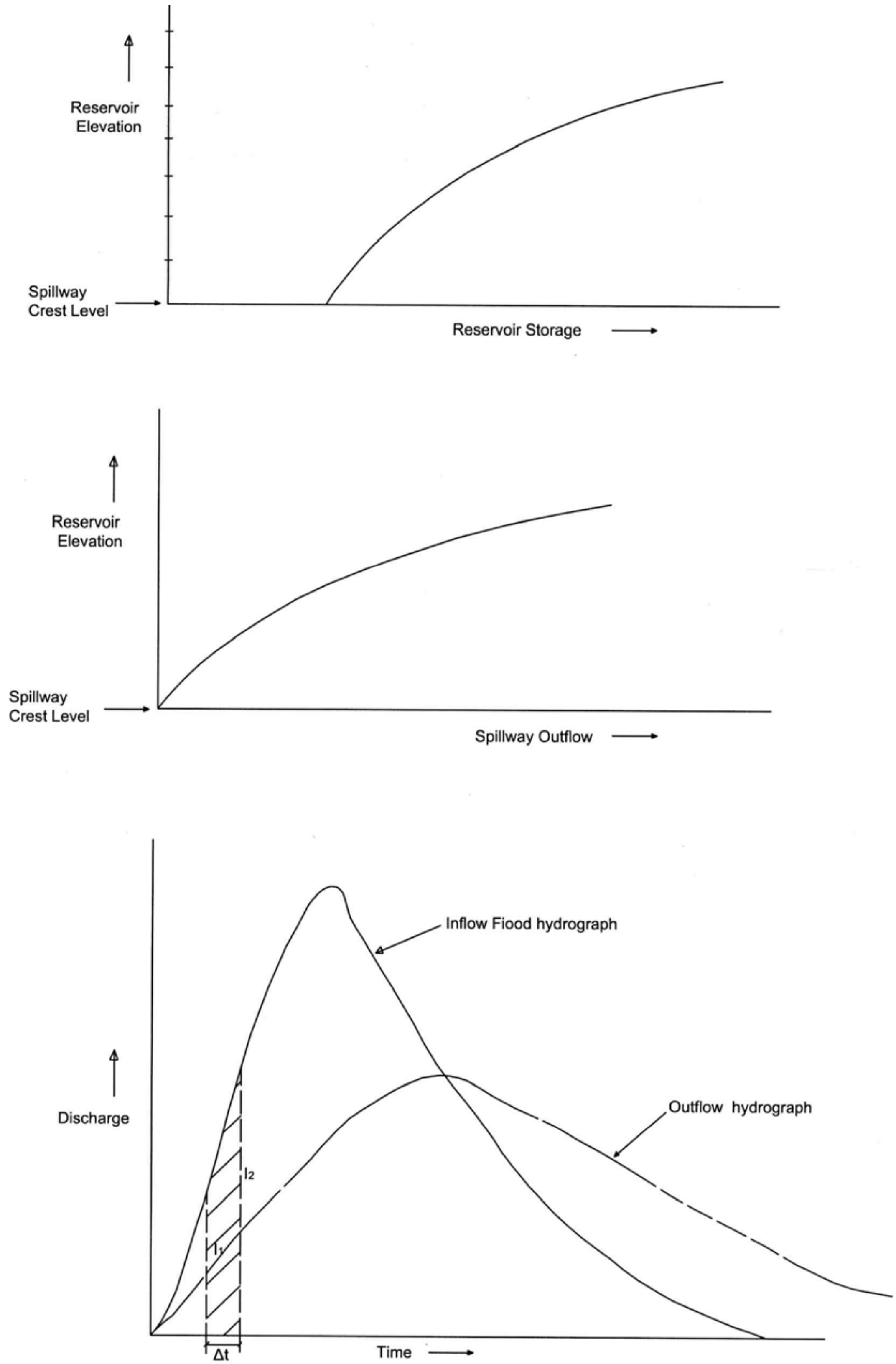


Fig. 10.1 Flood routing through a reservoir

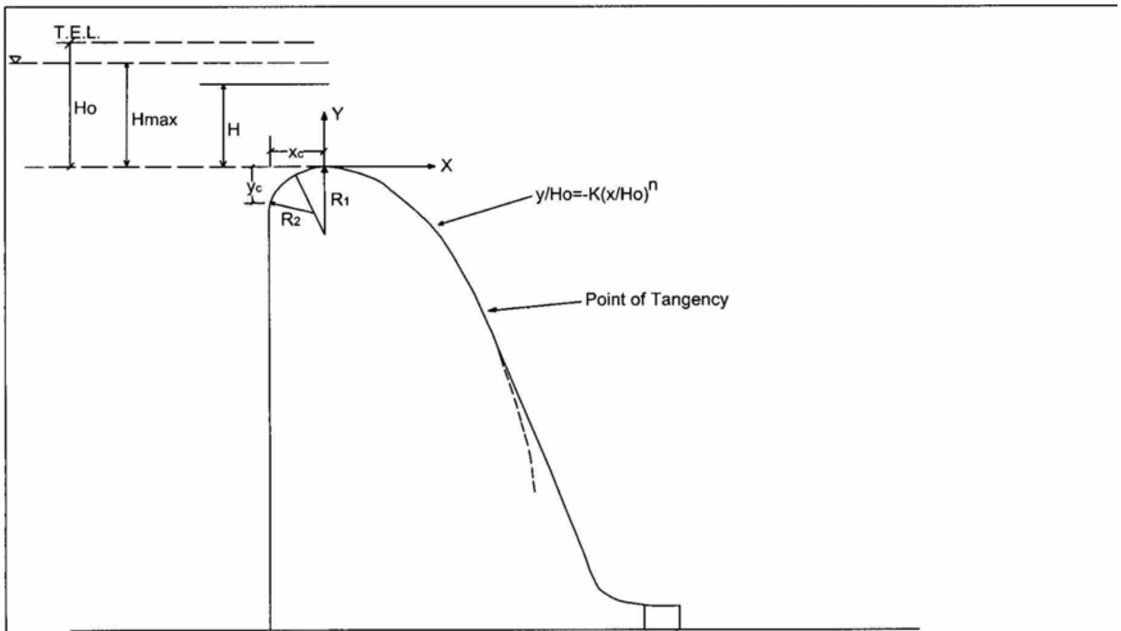


Fig. 10.2 Ogee spillway

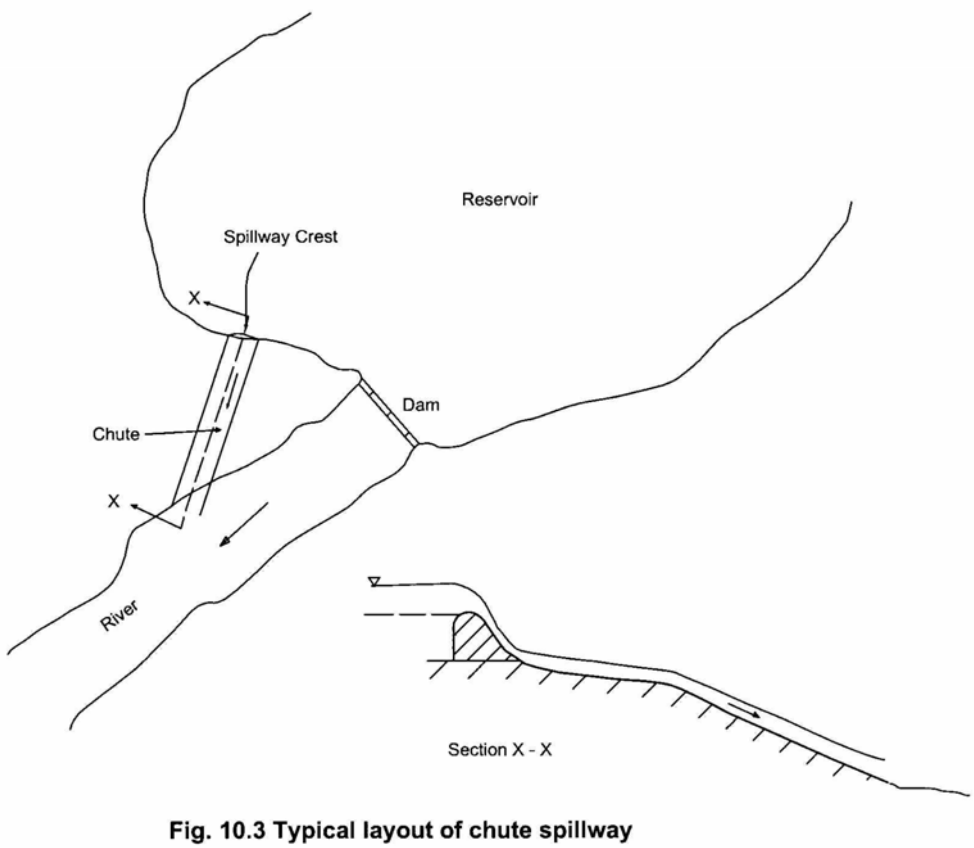


Fig. 10.3 Typical layout of chute spillway

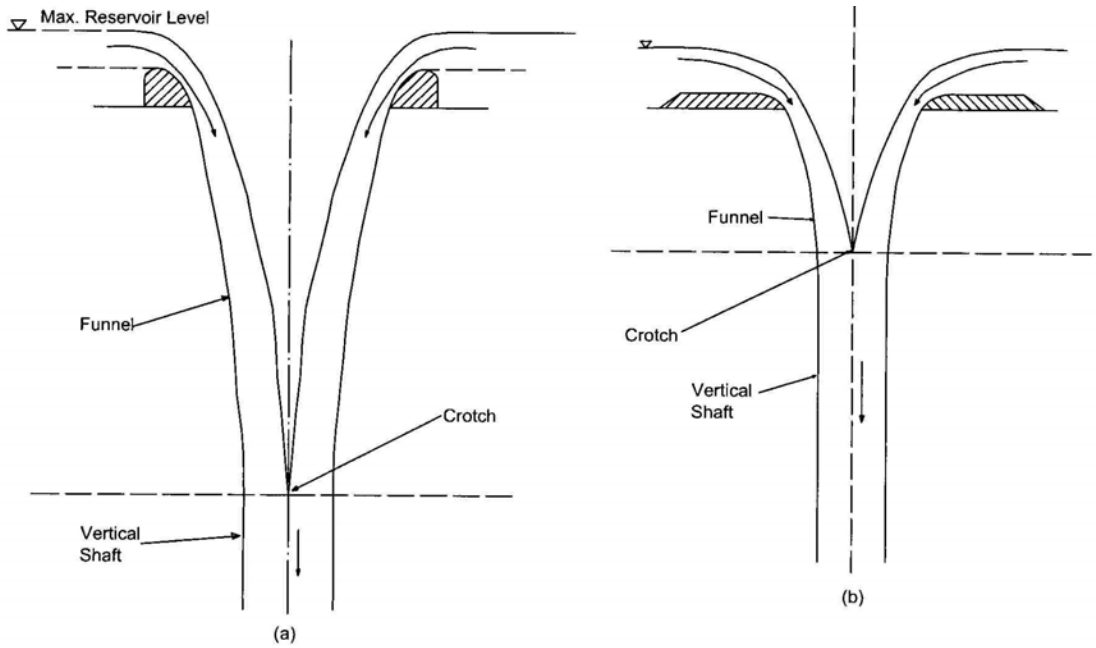


Fig. 10.5 Shaft Spillway (a) Standard Crested (b) Flat Crested

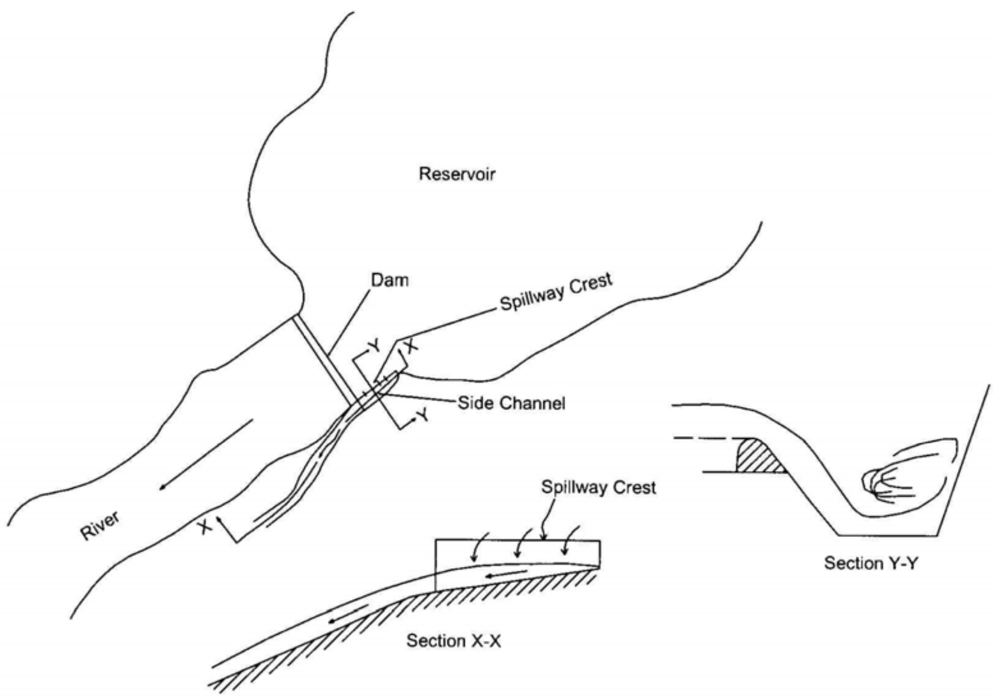


Fig. 10.4 Side Channel Spillway

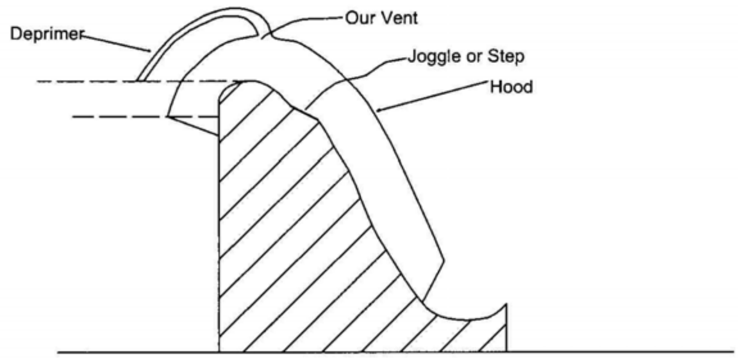


Fig. 10.6 Line diagram of siphon spillway with joggle

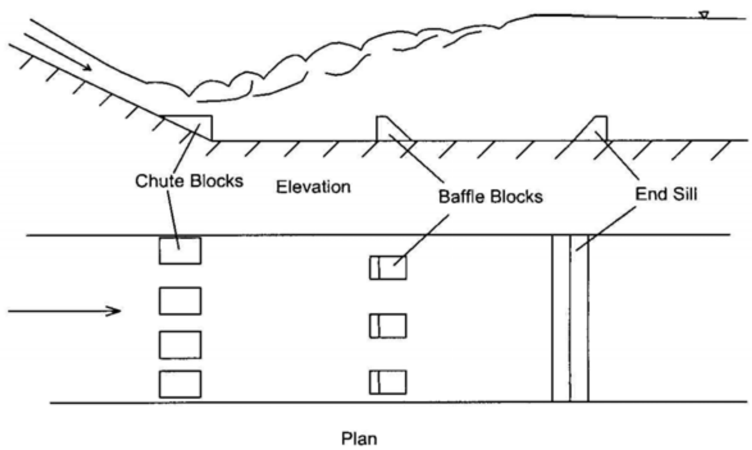


Fig. 10.7 Typical basin with appurtenances

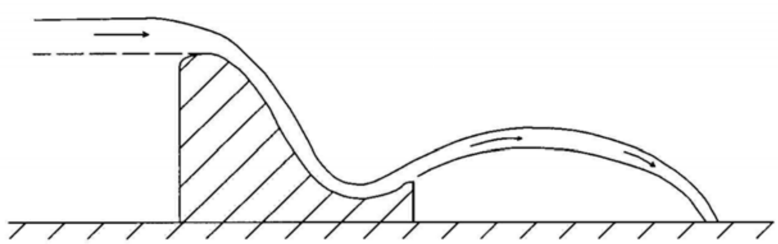


Fig. 10.8 Ski jump bucket

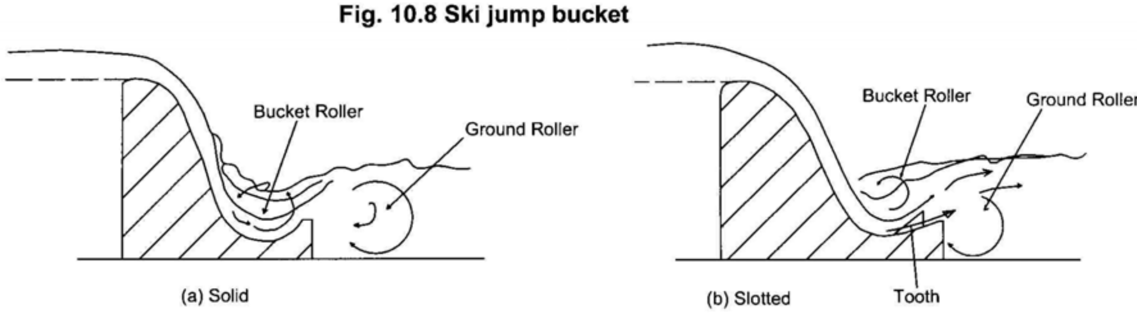


Fig. 10.9 Roller bucket

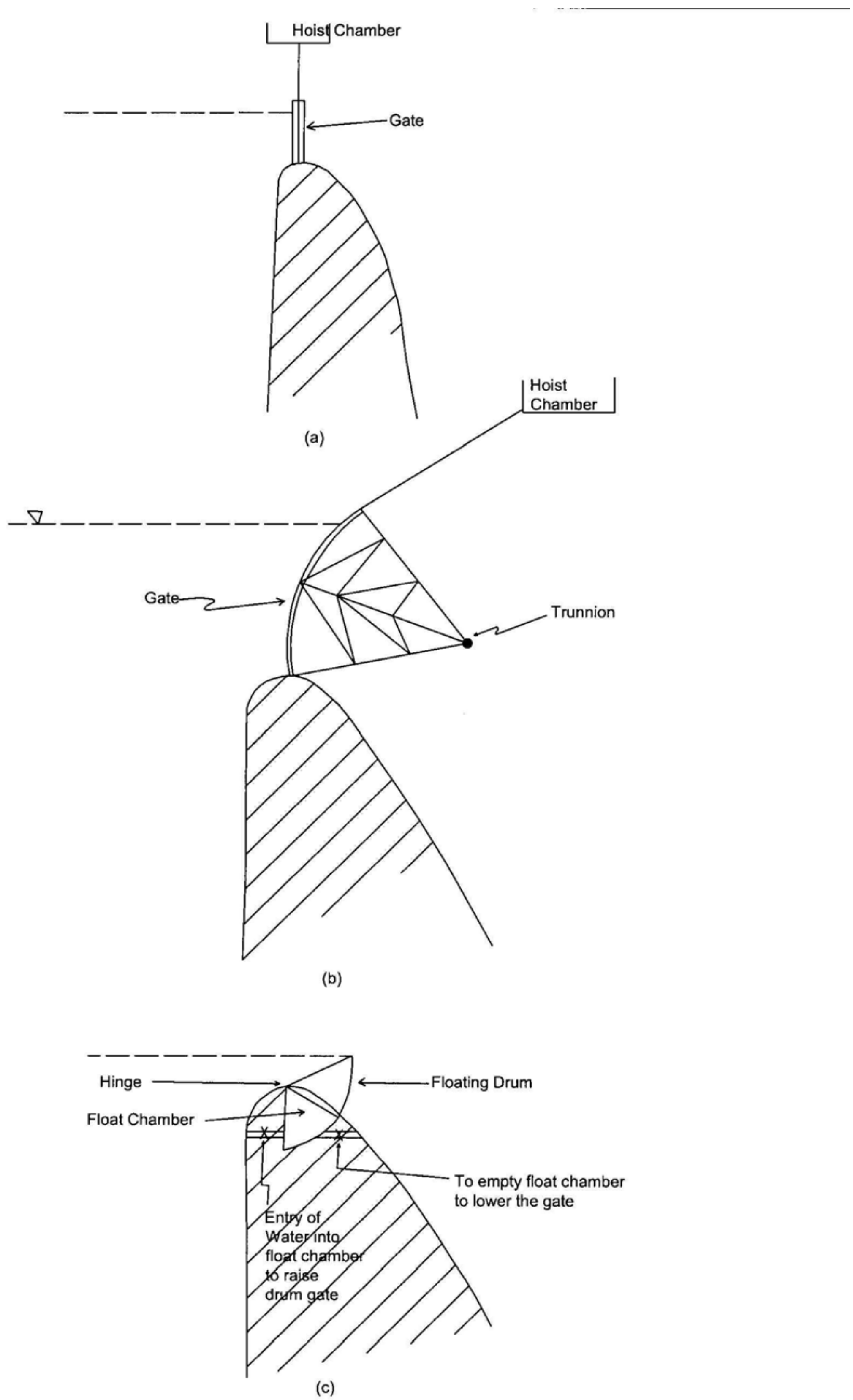


Fig. 10.10 (a) Vertical lift gate (b) tainter gate (c) drum gate

11. SOME SPECIAL PROBLEMS OF HILLY STREAMS

Hilly streams have many characteristics which are quite different from those in the alluvial plains. Thus most of the hilly streams would have steep longitudinal slopes, high velocity flow which generally does not follow the logarithmic distribution, large bed roughness and narrow cross section with steep banks. Many of the techniques and formulae used for alluvial rivers will not be applicable to such streams. This section is intended to discuss very briefly some of these differences.

FLOW CHARACTERISTICS

The flow in hilly streams depends on the slope and bed material size. This would vary from the origin of the stream to the foothills where the stream enters the plains, with the slope and bed material size decreasing continuously. Thus while in the upper reaches the bed will have predominance of boulders with very little finer material, further down gravel will predominate till in the plain region it will mostly be sand.

Streams have been classified differently by different investigators. The important flow types at low flows are however nearly same for these classifications. Thus one could have a cascade flow where the flow is highly turbulent around large size roughness elements (boulders), step pool flow with sequential highly turbulent flow over steps with relatively tranquil flow through intervening pools or plane bed flow with boulders protruding through otherwise uniform flow (Fig.11.1)

Fig.11.1

ESTIMATION OF FLOOD FLOWS

Due to the low time of concentration, hilly streams generally have flash floods i.e. the flood hydrograph has a small time base with rapid rise and fall. The estimation of flood flows in the absence of data would require estimation of the bed resistance in addition to the cross section, depth of flow and longitudinal slope. The estimation of bed resistance using the procedures available for alluvial rivers is likely to result in overestimation of the resistance and hence the flow.

RIVER TRAINING

There is hardly any possibility of a hilly stream changing its course. This is so because of the type of cross section which is characterized by high banks. River training works of the type already discussed therefore are not necessary for these streams. The possibility of landslides is however always there, specially due to sloughing or by undercutting in case one of the banks is being attacked by the stream. Bank protection measures may be required in such situations. This is usually done by providing toe walls at sites vulnerable to landslides. The toe walls have to be designed to withstand the impact of boulders which will roll along during floods

DIVERSION STRUCTURES

The type of diversion structures discussed earlier i.e. weir with a raised crest or barrage are not suitable for the boulder reaches. The boulders rolling along with the flow during floods are likely to damage such structures or the gates. Alternative designs such as the trench weir – discussed earlier – which are not likely to suffer major damage by rolling boulders, are more suitable under such conditions. The canals also are contour channels. Sediment exclusion is generally not a problem as most of the sediment is coarse and moves along the bed. The traditional sediment exclusion devices are not suitable as they can be blocked by large size particles or suffer damage by boulders. The use of vortex chambers to exclude coarser material in case of power channels is possible in gravel bed streams.

The waterway and scour in alluvial rivers is generally determined using the Lacey's theory. This theory is however not applicable to boulder or gravel beds. While the determination of the waterway is not an issue in hilly streams because they do not have wide flood plains, the determination of the likely scour may be needed. This could be done by estimating the largest size of boulder that may get displaced during floods. The size of such a boulder multiplied by an appropriate factor is likely to give an estimate of the scour.

The loss of water by seepage underneath the structures is also likely to be more in these streams because of the larger sediment size and hence increased permeability.

It may however be mentioned that the problems will vary depending upon the topography, the bed material, the high flood discharge and many other parameters. No single prescription can work for the boulder reach as well as the gravel reach equally and a detailed discussion of all these is beyond the scope of the present work.

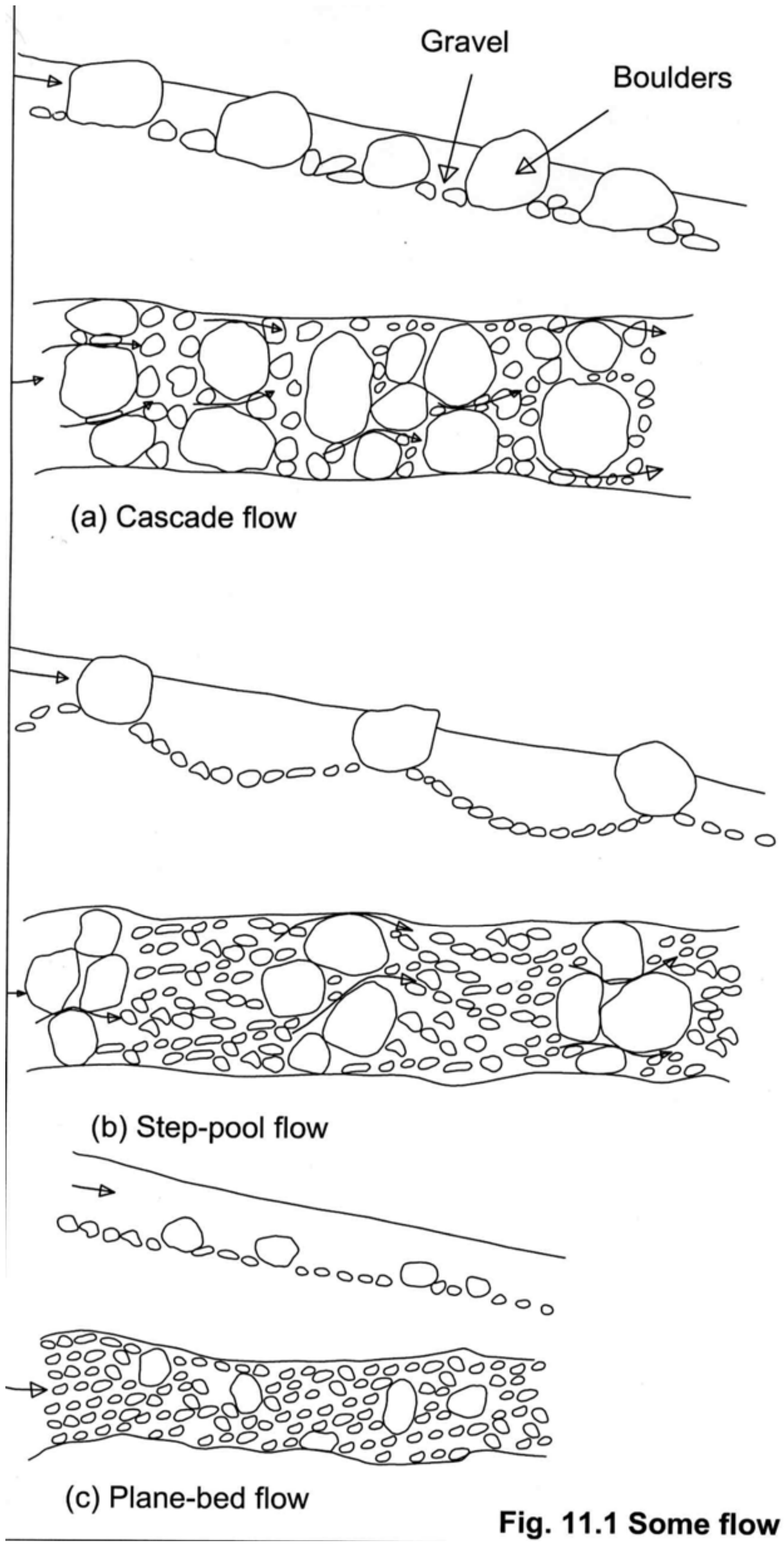


Fig. 11.1 Some flow types in hilly streams

12. GROUND WATER

Ground water is the water occurring under the earth's surface and fills the pores and fractures of the subsurface medium. It can be divided into two zones viz. the saturated or phreatic and the unsaturated or vadose zones. The saturated zone has all its pores filled with water while in the unsaturated zone, pores contain gases alongwith water (Fig.12.1).

Fig.12.1

Water Table: The top of the saturated zone, where the pressure is equal to the atmospheric pressure, is known as the water table. It is however worth mentioning that there is a capillary rise depending on the pore size and the medium can be saturated even in the capillary zone but the pressure there is below atmospheric.

Aquifer, Aquitard and Aquiclude: A geologic formation from which significant quantities of ground water can be obtained is called an aquifer. The aquifers may sometimes be separated vertically by formations which allow very little or no water to flow through them. These formations are termed as aquitards if they allow very little flow through them and called aquicludes if the flow through them is nearly zero.

Aquifers can be further divided into unconfined or confined (Fig.12.2). In an unconfined aquifer, the water table can rise or fall with the change in the quantity of stored water. A confined aquifer, on the other hand has an aquitard or aquiclude at top and thus the water in the same is under a pressure greater than atmospheric.

Fig.12.2

Ground water movement takes place both in unconfined and confined aquifers depending on the hydraulic gradient. Thus an aquifer acts both as storage as well as a conduit for ground water.

Some definitions, characterizing aquifers, useful in the study of ground water are given below:

Porosity- is the ratio of the volume of pores to the total volume of the porous medium.

Specific Yield – A saturated soil formation will yield certain amount of water under gravity. The ratio of this volume to the total volume of the formation is called the specific yield of the formation.

Specific Retention – This is the ratio of the volume of water retained by the formation after it has been drained under gravity to the total volume of the formation. The sum of the specific yield and the specific retention thus are equal to the porosity.

Storage Coefficient – is defined for confined aquifers and is the volume of water a confined aquifer will release per unit surface area of the aquifer for a unit change in the head.

Hydraulic Conductivity (K) – Also called the coefficient of permeability, is a measure of the ease with which a fluid will pass through it. It is defined as the quantity of water that flows through a unit cross sectional area of the medium in unit time under a unit hydraulic gradient.

Transmissivity (T) – is generally used for confined aquifers and is the product of the hydraulic conductivity and the thickness of the saturated portion of the aquifer.

DARCY'S LAW

The flow of liquids through porous media is governed by the Darcy's law, which states that the rate of flow through a saturated medium is proportional to the hydraulic gradient and the cross-sectional area of flow.

Mathematically, Darcy's law can be expressed as:

$$Q = - K A (dh/ds)$$

In which, Q is the rate of flow, A the cross-sectional area and dh/ds represents the hydraulic gradient which is negative as h decreases in the direction of flow, K being a constant of proportionality which is equal to the hydraulic conductivity of the medium.

The aforesaid relationship applies so long as the flow is laminar, which is the case most of the times as far as ground water is concerned. This law is considered to be valid in a Reynold's number range from 1 to 10, with the Reynold's number Re being defined as

$$Re = V d \rho/\mu$$

In which V is the velocity, d the average grain diameter, ρ and μ are the density and viscosity of fluid respectively.

The above equation can also be written as:

$$V = - K (dh/ds)$$

where V is the apparent velocity of flow

The velocity components u, v and w in the x, y and z directions respectively can be written as:

$$u = -K_x\ (\partial h/\partial x)$$

$$v = -K_y\ (\partial h/\partial y)$$

$$w = -Kz\ (\partial h/\partial z)$$

in which $K_{x,}$ $K_{y\ \text{and}}$ K_z are the coefficients of permeability in the x, y and z directions respectively.

For a homogeneous, isotropic and incompressible medium, substitution of the above in the equation of continuity for fluid motion yields the following equation, known as Boussinesq's equation, for a confined aquifer

$$(\partial^2 h/\partial x^2) + (\partial^2 h/\partial y^2) + (\partial^2 h/\partial z^2) = (S/T)\ ((\partial h/\partial t)$$

For an unconfined aquifer, one obtains the Dupuit's equation, which is based on the assumption that the free surface curvature is small enough so as to make the w component of the velocity negligible. This equation has the following form

$$(\partial^2 H^2/\partial x^2) + (\partial^2 H^2/\partial y^2) = (2n/K)\ ((\partial H/\partial t)$$

H being the hydraulic head and n the porosity of the medium

For steady flow the aforesaid equations can be written as

$$(\partial^2 h/\partial x^2) + (\partial^2 h/\partial y^2) + (\partial^2 h/\partial z^2) = 0$$

$$(\partial^2 H^2/\partial x^2) + (\partial^2 H^2/\partial y^2) = 0$$

Well Hydraulics

Wells are structures used to pump water from an aquifer. They consist of an intake which is a slotted pipe with screen to prevent fine sediment from entering the well and a vertical casing pipe to convey the water to the surface.

When water is pumped through a well, the water table/piezometric-head in the vicinity of the well is lowered (Fig.12.3). The surface of the lowered water table/piezometric-head is called the cone of depression and the distance from the centre of the well to the point to which the cone of depression extends is known as the radius of influence of the well. The vertical distance between the original water table/piezometric-head at any point and the same after pumping starts is called the drawdown at that point. For given aquifer and well characteristics, these quantities are a function of the pumping rate. The quantity of water discharged per unit drawdown at the well is known as the specific yield of the well.

Fig.12.3

As the pumping from a well starts, the radius of influence and drawdown for an extensive aquifer start increasing with time and attain a constant value once the rate of pumping becomes equal to the rate of recharge. The well is then said to have attained equilibrium conditions.

An expression for the discharge through a well can be obtained by writing the Boussinesq/Dupuit equations for confined/unconfined aquifers in the radial coordinate system and integrating them with the appropriate boundary and initial conditions. For equilibrium conditions, this results in the following equations:

For confined aquifers

$$Q = -2\pi T\,(h_0 - h_w) / (\ln(r_0/r_w))$$

While for unconfined aquifers one gets

$$Q = -\pi K\,(H_0^2 - H_w^2) / (\ln(r_0/r_w))$$

For non-equilibrium conditions, the solution of the relevant equation for a confined aquifer was obtained by Theis and takes the following form:

$$d = Q\,W(u)/(4\pi T)$$

where d is the drawdown at time t since the beginning of pumping at a distance r from the well and W(u) is known as the well function and is tabulated as a function of u, which is given by

$$u = r^2\,S/(4Tt)$$

For a fully penetrating well in an unconfined aquifer, the solution was given by Boulton and can be expressed as

$$d = Q\,(1+C_k)\,V(t', r')$$

C_k being a correction factor which depends on t' and V(t', r') is the Boulton's well function the values of the same being available as a function of t' and r', which are defined as

$$t' = Kt / (SH_0)$$

and

$$r' = r / H_0$$

Pumping Tests

One of the important parameters in ground water development is the determination of aquifer characteristics such as hydraulic conductivity and transmissivity. Pumping tests are carried out for their determination. The arrangement consists of a pumping well and some observation wells at different distances from the pumping well. The observation wells are used to measure the water table/piezometric-head levels. These levels are recorded before the start of pumping and subsequently at different times after starting pumping at a constant rate. Under equilibrium conditions, knowing the drawdown at different observation wells and the discharge Q, the value of K and T can be obtained from the relevant discharge equations. Thus for two observation wells at distance r_1 and r_2 from the pumping well one can get the following expressions:

For an unconfined aquifer

$$K = Q \ln (r_2 / r_1) / (H_2^2 - H_1^2)$$

And for a confined aquifer

$$T = Q \ln (r_2 / r_1) / (2(h_2 - h_1))$$

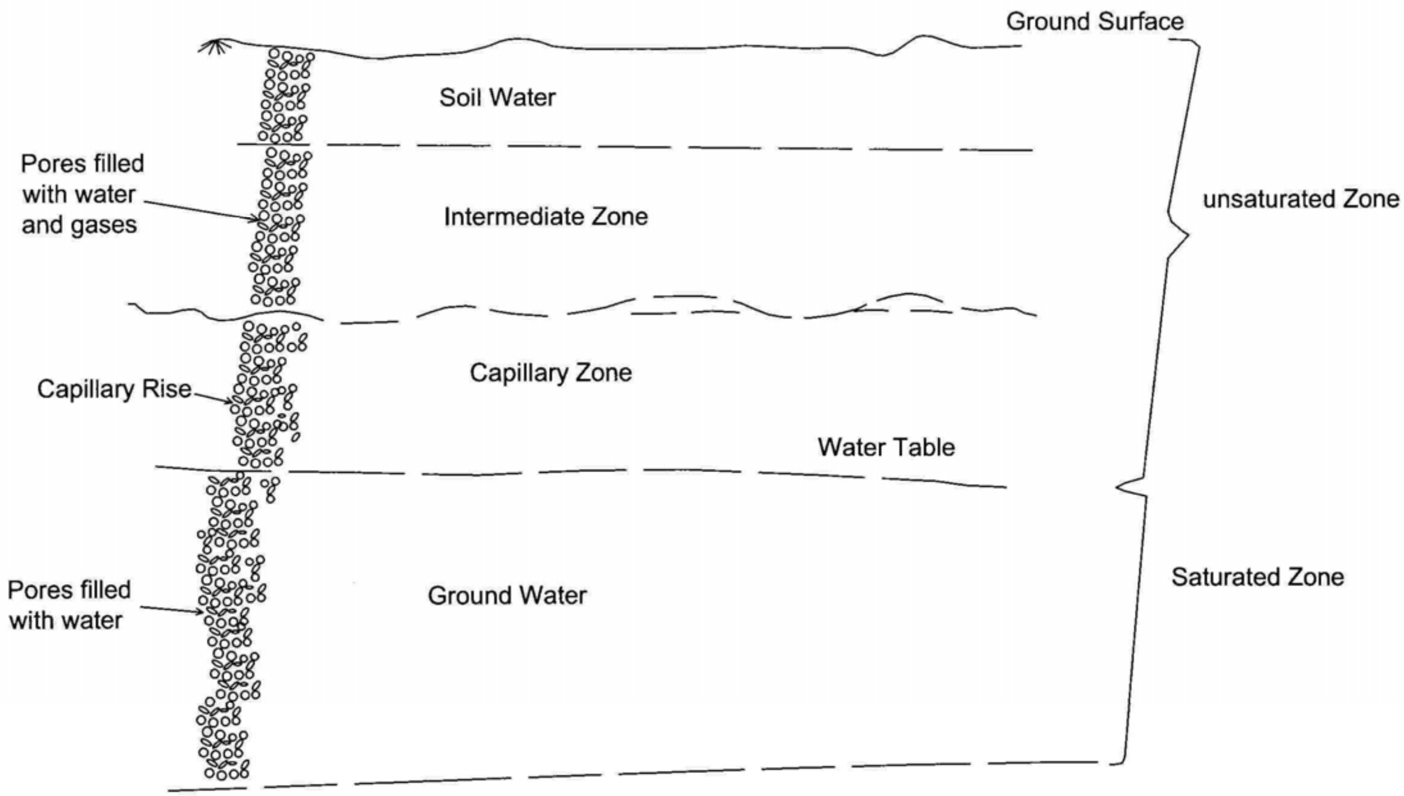


Fig. 12.1 Occurance of ground water

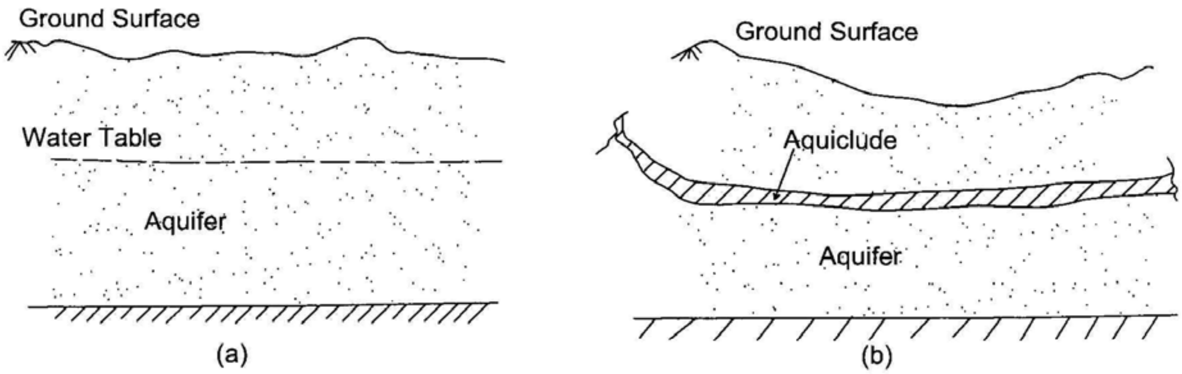


Fig. 12.2 Aquifers (a) unconfined (b) confined

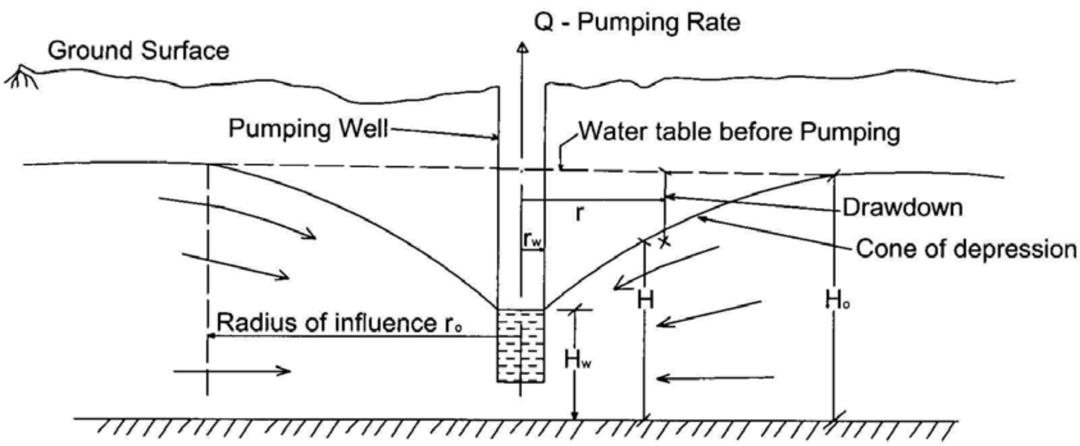


Fig. 12.3 Pumping from unconfined aquifer

13. DECISION SUPPORT SYSTEMS IN WATER RESOURCES

It is being increasingly realised that water resources related decisions cannot be taken in isolation. While surface water and ground water have to be dealt with together because of the strong interaction between these components, there are a number of other interacting parameters that need to be considered in planning and managing water resources. These range from the water quality, environmental, economical and socio-economical, land use and stakeholder perceptions to sustainability and the like. It is also being realised that water related decisions have to be taken with the basin as a unit rather than an isolated catchment, as any intervention somewhere is likely to have an impact elsewhere.

A holistic view as above requires modelling of an entire system and different processes, as against the purely hydrologic and hydraulic modelling of a catchment. The models have not only increased in complexity but have to cover diverse fields as mentioned above. The advances in the field of Artificial Intelligence have given rise to added modelling capabilities, while those in the field of data collection and management have meant availability of large data sets.

The rapid advances in the fields of computing and communications – the Information Technology revolution as it is often referred to – have made it possible to handle the aforesaid problems and integrate knowledge and understanding of water resources with developments in information technology to improve decision making in the water sector.

Decision Support Systems are computer based interactive systems to help in decision making. These may be data oriented or model oriented. While data oriented systems perform data retrieval and analysis, model oriented systems make use of various kinds of models to aid the decision making process.

Decision Support Systems traditionally have three major components viz. a user interface, a database and a model base, besides the management tools necessary for the intended purpose.

The user interface component is to present and receive information from a user and various types of interfaces have been developed for the same. The database component manages data while the model base keeps track of the various models and methods that may be needed in any given Decision Support System.

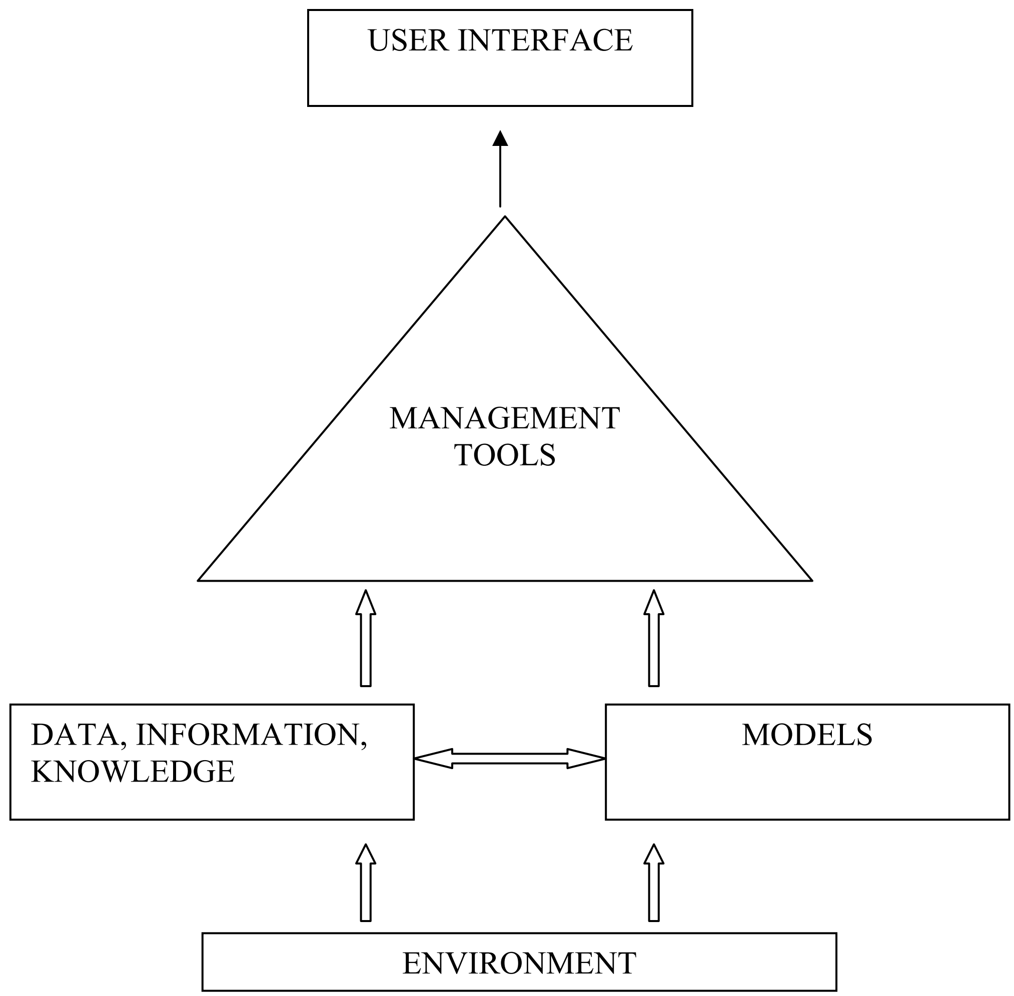


Fig. 13.1: Typical decision support system

Some of the issues connected with the various components are briefly discussed below:

DATA MANAGEMENT

In addition to collection, analysis and presentation is necessary to transform data to information, which can be used by the decision-maker.

Data Types

Data can be temporal, spatial or attribute data. Attribute data generally record information about the other two data types. While temporal data may reside in different formats in a database, spatial data is mostly in a GIS platform.

Data Storage

Data Warehouse and Data Mart are the basic architectural options to interface with the Decision Support Systems. The data in a Data Warehouse contains multiple subject areas while Data Marts are a scaled down version of Data Warehouse and usually subject specific.

Data Retrieval

Data can be retrieved either by on line processing or by Data Mining facilities. Data Mining allows knowledge discovery in databases through further exploration of data by discovering implicit patterns such as correlations and trends.

Data mining algorithms would normally be comprised of a model representation to discover patterns, model evaluation for predictive validity and a search for optimisation. To this end, data mining tools usually incorporate some form of Artificial Intelligence in relational databases and employ analytic techniques, which may include pattern recognition, heuristics, inductive reasoning, neural networks and fuzzy logic.

Data Interchange

Any DSS may use a number of models, which require data from different types of sources such as time series database systems, relational database management systems, GIS or data synthesis systems. Sometimes these sources may be with different organisations and all the data in one particular type of source may also not always be in the same format. The data along with information about the data- metadata as it is referred to- has to be exchanged between what are called the Information Providers and Information Consumers and therefore a standard data exchange system is a necessity.

MODELLING ISSUES

Modelling has come a long way from the early computational models and simulation. Five generations of models have been identified. The first three were concerned with performing numerical tasks, custom models for a particular region or system and generalisation of these custom models respectively. The fourth generation models built on the work of the first three generations and combined codes and products from several third generation products. Another important development was to establish the distinction between tool-maker and tool-user.

The fifth generation models combined the tools of computational hydraulics with the tools of artificial intelligence and incorporated sophisticated visualisation tools to allow the non-experts to better comprehend the results.

Soft Computing Techniques

It may not be out of place to mention here some of the Artificial Intelligence (Soft Computing) techniques, which have emerged in recent years as a complement tool to mathematical approaches and are being increasingly used in water resources applications.

Expert Systems- These are knowledge-based or rule-based systems, which use the knowledge base and inference engines to solve problems. These however do not have the ability to learn or adapt to new situations. A schematic of a knowledge-based expert system is shown in Fig.13.2.

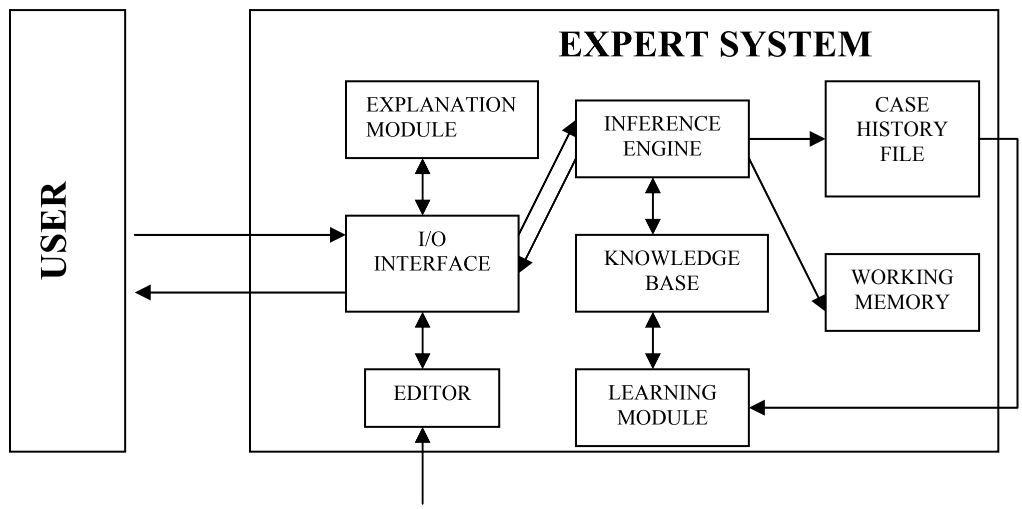


Fig.13.2: Schematic of a typical knowledge-based expert system

Artificial Neural Networks (ANN)- They have been developed as a generalisation of mathematical models of neural biology. They are characterised by their architecture (single layer, bilayer or multilayer), topology (connectivity pattern, direction of information flow etc.) and learning regime (supervised or unsupervised). A typical multilayer ANN is shown in Fig.13.3.

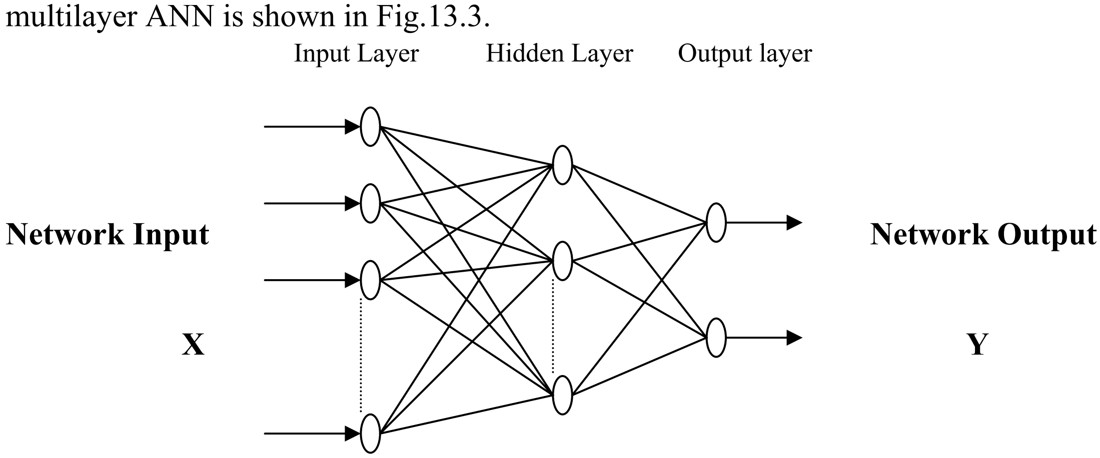


Fig 13.3: A Typical multilayer ANN

Agents- an agent is something that perceives and acts in an environment. It is a self contained software program specialised in obtaining a set of goals by autonomously performing tasks. Agents could be simple reflex; model-based reflex, goal-based or utility based. All the types can improve their performance through learning. Fig. 13.4 shows a schematic of learning agents.

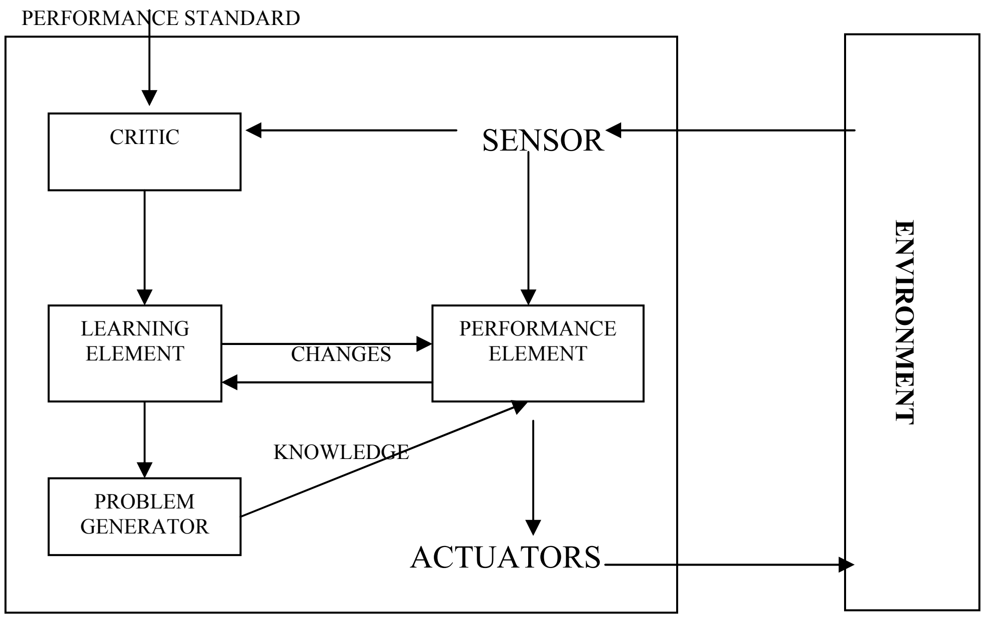


Fig 13.4: Schematic of learning agents (after Russell and Norvig)

Evolutionary Computing- This includes genetic algorithm, simulated annealing, evolutionary programming, genetic programming etc. Genetic algorithm is probably the most used. It starts with a random population of states, each representing a solution to the problem at hand and generating new states through reproduction, mutation and crossover. At every generation, a fitness function judges the quality of the population and as such only the good string gets access to the next generation. Genetic programming is closely related to genetic algorithms, the only difference being that the representations that are mutated and combined are programs rather then strings.

Model Integration

One of the major problems faced by water resources engineers is to integrate various models covering different disciplines and having varied origins. The approaches for same have been

1. Integrating the model implementations for a given set of packages. This however does not seem to be very useful as such an exercise will have to be carried out separately for all combinations of packages.

2. Providing shells for pre- and post-processing to generate configurations for a range of models. This is also considered as a partial solution only.

3. Developing integration frameworks and integrating models with the framework. Most of the current effort is directed primarily towards this approach.

USER INTERFACE

The user interface is the component of the system, which facilitates the interaction between the user and the system, the major types being:

1. Command-driven interfaces- these usually require an explicit command conforming to the defined syntax.
2. Menu-driven interfaces- these provide the user with a list of options and a simple way of selection such as in bar or pull-down menus.
3. Direct-manipulation interfaces (DMI)- in this type of interface the user can manipulate information by direct action. Graphical User Interface (GUI) is the most popular implementation of DMI. This makes use of visual objects to implement its model and the objects can be manipulated via pointing devices such as a mouse.
4. Special-purpose interfaces- these are generally used to control an imbedded computer system.

The growing complexity of interfaces and the need for flexibility has given rise to the development of "intelligent interfaces" which can anticipate and adapt to the needs of different users, can learn new techniques and concepts and provide explanation of their actions.