UNIT 3 EARTH AND ROCKFILL DAMS

Structure

3.1 INTRODUCTION

In Unit 1 you learnt about the various types of Reservoir Planning. Then in Unit 2, you were told about the features of design and construction of concrete gravity dams. In this unit you will learn about the features of design and construction of earth and rockfill dams.

Objectives

After studying this unit, you will be able to explain

- the various types of earth dams,
- the causes of failure of earth dams,
- selection of dam depending on the material available,
- the method of stability analysis of an earth dam, and
- the section adopted in case of rockfill dams.

3.2 EARTH DAMS

Earthen dams have been constructed for long times past in India and many other countries. Earlier the design and construction of these dams was based mostly on experience and precedent. However, with the development of the subject of Soil Mechanics these dams are designed and constructed on more rational basis and greater reliance can be placed on them. The increase in knowledge of the behaviour of soil materials has resulted in corresponding increase in confidence in designing earth dams and it is now possible to have embankment dams 200-300 m high.

3.3 TYPES OF EARTH DAMS

There are three main types of earth dams, namely,

1) Homogeneous dam,
2) Zoned dam, and
3) Diaphragm dam.

3.3.1 Homogeneous Dam

The homogeneous dam is a simple embankment which is essentially homogeneous throughout, although a blanket of relatively impervious material may be placed on the upstream face. Levees are often simple embankments, but large dams are rarely constructed in this manner.

3.3.2 Zoned Dam

Zoned dams usually have a central zone of selected soil material, to form a relatively impermeable core, a transition zone along both faces of the core to prevent piping through cracks which may form in the core, and outer zones of more pervious material for stability. This construction is widely used in earth dams and is selected whenever suitable materials are available. Clay, even though highly impermeable, may not make the best core if it shrinks and swells too much. The most satisfactory cores are of clay mixed with sand and fine gravel. Figure 3.1 shows the cross-sections of typical earth dams.

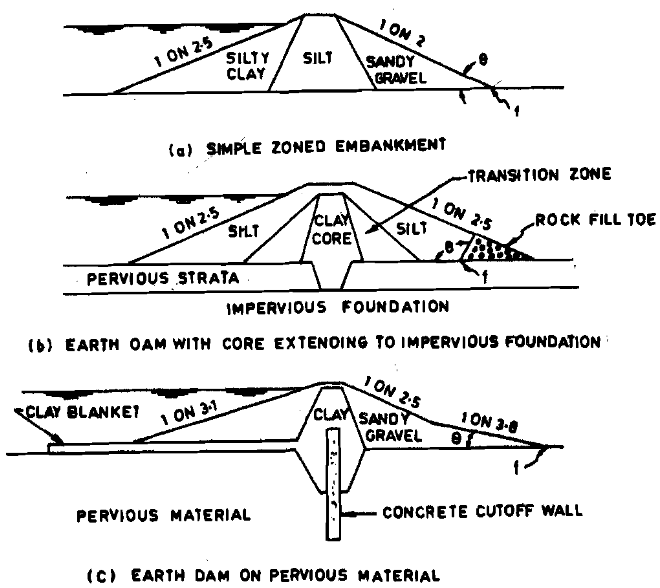


Figure 3.1 : Cross-sections of Typical Earth Dams

3.3.3 Diaphragm Dams

Diaphragm type dams have a thin central section of concrete, steel, or timber which serves as a water barrier, while the surrounding earth or rockfill provides stablity. Thin concrete sections are easily cracked by differential earth loads, and it is difficult to form a perfectly watertight barrier of steel or timber. In addition, the diaphragm must be tied into bedrock or a very impermeable material if excessive underseepage is to be avoided.

SAQ 1

i) What are the various types of earth dams?

ii) What is a homogeneous dam?

iii) What is a zoned dam?

iv) What is a diaphragm dam?

3.4 CAUSES OF FAILURE IN EARTH DAMS

Earth dam failures are caused by improper design quite often based on inadequate investigations, and careless construction and maintenance.

Failures of earth dams may be grouped into the following basic causes:

a) Hydraulic failures,

b) Seepage failures, and

c) Structural failures.

3.4.1 Hydraulic Failures

Failure of about one third of earth dams is attributed to them. They are produced by surface erosion of the dam by water. Also included are washouts from overtopping (Figure 3.2 (a)), wave erosion of upstream face, scour from the discharge of the spillway, etc. and erosion from rainfall.

3.4.2 Seepage Failures

Seepage of water through the foundation or embankment has been responsible for more than one third of earth dam failures. Seepage is unavoidable in all earth dams and usually it does no harm. However, uncontrolled seepage may cause erosion within the embankment or in the foundation which may lead to piping (Figure 3.2(b)). Piping is the progressive erosion that develops through and under the dam. It begins at a point of concentrated seepage where the gradients are sufficiently high to produce erosive velocities. If forces resisting erosion, that is, cohesion, interlocking effect, weight of soil particles, action of downstream filter, etc. are less than those that tend to cause the erosion, the soil particles are washed away causing piping failure.

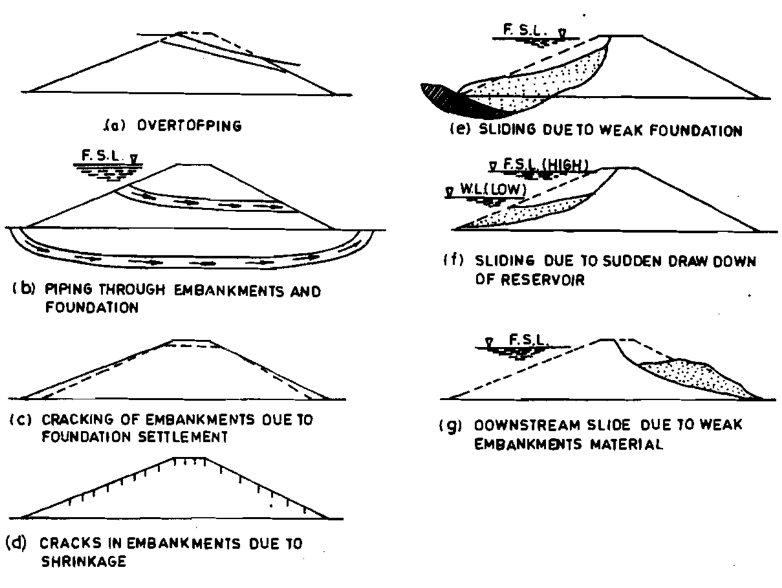


Figure 3.2 : Types of Failures – Earth Dams

Seepage failures are generally caused by (a) pervious foundations, (b) leakage through embankments, (c) conduit leakage, and (d) sloughing.

Pervious Foundations

Presence of strata and lenses of sand or gravel of high permeability or cavities and fissures in the foundation may allow concentrated flow of water from the reservoir causing erosion. Presence of buried channels under the seat of the dam have also been responsible for this type of failure.

Leakage through Embankments

The following are the common causes of embankment leakage which lead to piping:

1) Poor construction control which includes insufficient compaction adjacent to outlet conduits and inadequate bond between embankment and the foundation or between the successive layers of the embankment.

2) Cracking in the embankment or in the conduits caused by foundation settlement (Figure 3.2(c)).

3) Animal burrows.

4) Shrinkage and drying cracks (Figure 3.2(d)).

5) Presence of roots, pockets of gravel or boulders in the embankment.

Sloughing

Failure due to sloughing takes place where downstream portion of the dam becomes saturated either due to choking of filter toe drain, or due to the presence of highly pervious layer in the body of the dam. The process begins when a small amount of material at the downstream toe is eroded and produces a small slide. It leaves a relatively steep face which becomes saturated by seepage from the reservoir and slumps again, forming a higher and more unstable face. This process is continued until the remaining portion of the dam is too thin to withstand the water pressure and complete failure occurs.

3.4.3 Structural Failures

Structural failures of the embankment or its foundation account form about one-fifth of the total number of failures. Structural failures result in slides, such as

a) Foundation slides, and

b) Slides in embankment.

Foundation Slides

Seams of weathered rocks, shales, soft clay strata are responsible for the foundation failure in which the top of the embankment cracks and subsides, and the lower slope moves outward and large mud waves are formed beyond the toe (Figure 3.2(e)). Another form of foundation failure occurs because of excessive water pressure in confined seams of silt or sand. Pore water pressure in the confined cohesionless seams, artesian pressure in the abutments or consolidation of clays interbedded with the sands or silt, reduces the strength of the soil until it is not capable of resisting the shear stresses induced by the embankments or its water load. The movement develops very rapidly without warning.

Slides in Embankment

Embankment slides can occur when the slope is too steep for the strength of soil (Figure 3.2(f) and (g)). Usually the movement develops slowly and is preceded by cracks on the top or the slope near the top. Failure of this type is usually due to faulty design and construction. The shearing strength of the soil is reduced by development of pore pressures during construction. In high dams embankment slides may occur during dissipation of pore pressures.

SAQ 2

What are the various types of failures in earth dams?

3.5 DAM SELECTION TO SUIT AVAILABLE MATERIALS

For dams advantage should be taken of every local resource to reduce the cost of the project without sacrificing the efficiency and quality of the final structure. If suitable soils can be found in nearby borrow pits, an embankment dam may prove to be the most economical.

The type of earth dam would be dictated essentially by the materials available at or near the site as also the foundations. For economy, the design of earth dam should be adopted to utilise the available materials with little processing. Thus, if nothing but sand is available the design should utilize the sand in the natural state for the bulk of the dam, limiting the imported material like clay or silt for providing impervious member to the minimum. The available materials and their excavation costs dictate the type of earthfill structures to be considered.

The amount of each type of material available from borrow areas influences the type of earth dam chosen. If materials are to be used with stockpiling, they must be used early in the construction period. Materials from a borrow area may also be available only at certain times of the year and only to certain depths due to groundwater or climatic conditions. Available excavation machinery, in some cases, force a change in the type of dam. Haul distances from the borrow area influence cost and therefore the type of dam. Environmental considerations suggest that a borrow area should ideally be in the area to be submerged by the reservoir.

Thus the type of dams that may be selected depending upon the materials available are:

1) Homogeneous dam,
2) Zoned dam,
3) Diaphragm dam, and
4) Rockfill dam.

3.5.1 Homogeneous Dam

This dam may be selected when there is an abundant supply of a single material. Also when the more readily available materials are neither clearly pervious or impervious. The material for drains usually involves limited volumes and hence can be processed materials.

3.5.2 Zoned Dam

A zoned dam may be considered where sufficient quantities of both pervious and impervious materials are available. The sandy materials should have good shear strength and be free-draining, relative to the core or impervious section. If the material contains sufficient fines to be relatively impermeable when compacted in the embankment will serve as a good core (Figure 3.3).

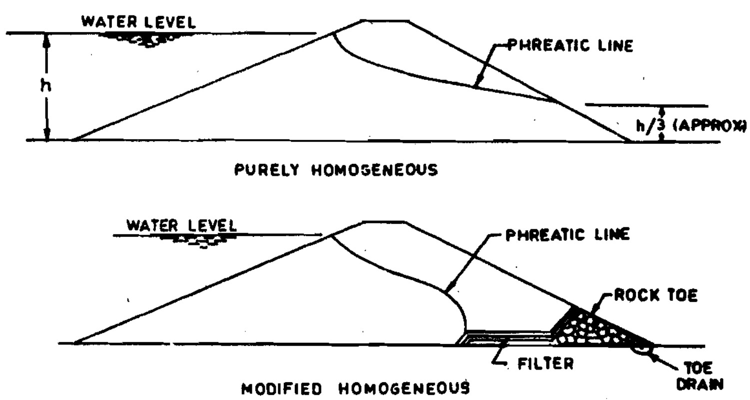
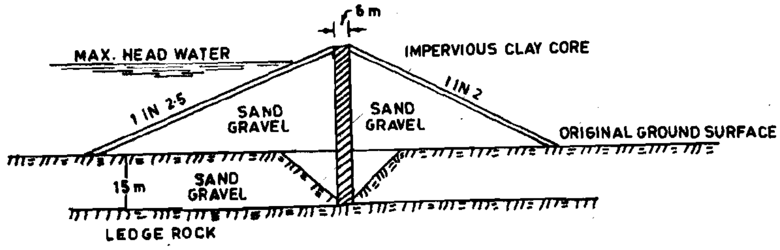


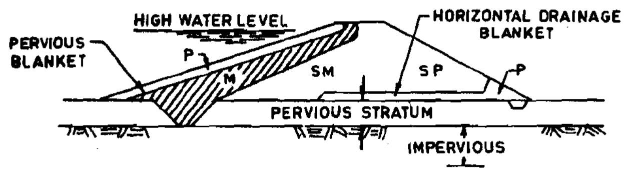
Figure 3.3 : Homogeneous Type Earth Dams

3.5.3 Diaphragm Dam

This type of section is adopted when large quantities of pervious material, say sand, gravel or rock are available and due to inadequate quantities of impervious material being available a thin diaphragm of impervious material of clay, cement concrete or other material is provided in the central or upstream face of the dam (Figure 3.4).



(a) VERTICAL DIAPHRAGM



P = PERVIOUS, M = IMPERVIOUS, SM = SEMI-IMPERVIOUS, SP = SEMI - PERVIOUS

(b) INCLINED DIAPHRAGM

Figure 3.4 : Diaphragm Type Earth Dams

3.5.4 Rockfill Dam

Where appurtenant features such as spillway, tunnel and outlet works provide the greater portion of rockfill materials such a dam may be selected. Figure 3.5 shows sections of typical rockfill dams. If the rock is strong, hard, durable, relatively dense, and flat or platy fragments are absent then it is acceptable. If the materials are capable of resisting weathering, then they may be used in the outer portions of the upstream and downstream slopes.

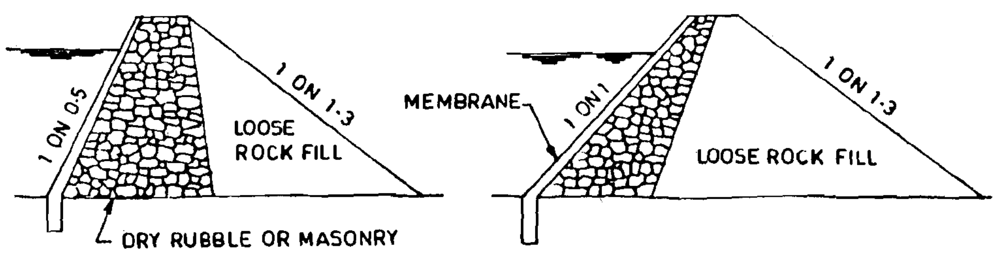


Figure 3.5 : Sections of Typical Rockfill Dams

SAQ 3

i) How would you select a homogeneous dam depending upon the materials available?

ii) How would you select a zoned dam depending upon the materials available?

iii) How would you select a diaphragm dam depending upon the materials available?

iv) How would you select a rockfill dam depending upon the materials available?

3.6 TOP WIDTH

The top or crest width of an earth dam depends on several considerations such as

a) nature of embankment materials and minimum allowable percolation distance through the embankment at normal reservoir water level,

b) height and importance of structure,

c) required width to provide embankment mass for resistance to earthquake shock, and

d) roadway requirements.

A common practice is fairly well represented by the formula

$$B_t = (5/3)\sqrt{H} \quad ...(3.1)$$

where,

B_t = crest width (m), and

H = height of dam (m).

However, the top width should be not less than 4 m for maintenance purposes.

SAQ 4

On what considerations will you select the top w dth of an earth dam?

3.7 FREEBOARD

The required allowance for waves is based on the effect of wind of maximum velocity blowing down the reservoir and setting up a wave splash on the dam face. Various empirical formulae depending upon wind velocity and reservoir fetch have been suggested for computing wave heights. The Stevenson-Moliter formula which is normally used is given by

$$h_w = 0.032\sqrt{(FV)} + 00.763 - 0.271\,F^{1/4} \quad \text{where} \quad F < 32 \text{ km} \quad ...(3.2)$$

and
$$h_w = 0.032\sqrt{(FV)} \quad \text{where} \quad F > 32 \text{ km} \quad ...(3.3)$$

where,

h_w = wave height (m),

F = fetch (km), and

V = wind velocity (km/hr).

On a sloping surface the wave rides along the slope upto a vertical height of 1.5 times the wave height above the reservoir level, hence free board is 1.5 h_w.

A distinction is made between the normal and minimum free boards. Normal freeboard is defined as the difference in elevation between the crest of the dam and normal reservoir water surface. Table 3.1 gives the freeboard to be provided according to the fetch.

Table 3.1 : Recommended Values of Freeboards

Fetch (km)	Minimum Freeboard (m)	Normal Freeboard (m)
< 1.5	1.00	1.25
1.5	1.25	1.50
4.0	1.50	1.80
8.0	1.80	2.50
15.0	2.20	3.00

It is further recommended that the freeboard given in the table be increased by 50 per cent if a smooth pavement is provided as protection on the upstream slope.

SAQ 5

How do you define freeboard for an earth dam?

3.8 STABILITY ANALYSIS

Every soil mass which has a slope at its end is subject to shear stresses on internal surfaces in the soil mass, near the slope. This is due to the force of gravity which tries to pull down the portions of the soil mass, adjoining the slope. If, however, the shearing resistance of the soil is greater than the shearing stress induced along the most severely stressed or critical internal surface, the slope will remain stable; and if on the other hand the shearing resistance of the soil, at any time after the construction of the slope becomes less than the induced shearing stress, the portion of the soil mass between the slope and the critical internal surface will slide down along this surface, until, the new slope formed by the sliding mass makes the shearing stress less than the shearing strength of the soil. The stability of slope of earthwork thus depends on the shear resistance or strength of the soil.

The well known method of investigating stability of slope is the Swedish method, devised by Swedish engineers in 1922, which is simple, and therefore, is more commonly used.

In this method, the curved slip surface is taken to be an arc of a circle with a certain centre. There will be a number of such likely slip circles with their respective centres. It is necessary to pick up the most dangerous of critical slip circle which is the circle along which the soil has the least shear resistance. The centre of this circle is located by trial and error. Below are shown methods of locating the critical slip circle in the case of various types of soils, assuming that the soil slope is homogeneous structure i.e., it consists of one type of soil only.

a) **Cohesive Soils**

The slip circle for cohesive soils is shown in Figure 3.6. Let PHV be any slip circle with centre O_1, X_1 is the distance of the centroid G of the area DPHV from the centre, O_1. The position of G can be obtained in the same manner as the centroid of an irregular plane area.

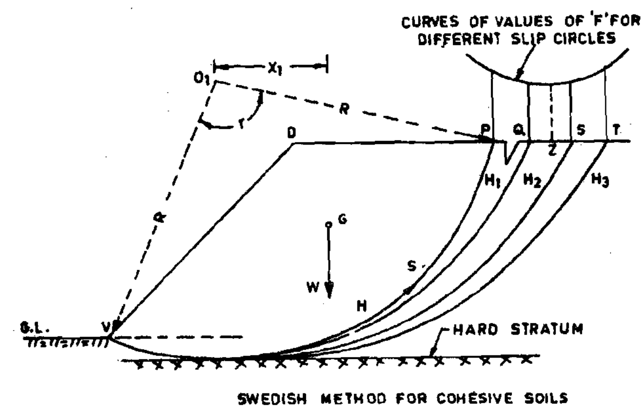


Figure 3.6 : Slip Circle Method

Actuating moment which may cause a slip along surface PHV = $W \times x_1$ (kgm) ...(3.4)

where, W = weight of soil mass of area $DPHV$ and length of 1 m.

For cohesive soils $\phi = 0$ and, its shearing resistance depends on cohesion only and is the same along the entire surface PHV.

The maximum resisting moment mobilized by the soil to prevent the likely occurrence of a slip = cohesive shear resistance developed along PHV (for 1 m length) $\times O_1V$ (kgm)

$$= LCR \text{ (kgm)} \qquad \text{...(3.5)}$$

where,

L = length of arc PHV (m) = $R\,\alpha$,

C = unit cohesion (kg/m^2),

R = radius of likely slip circle (m), and

α = angle subtended by arc PHV at O_1 (radians).

Hence, maximum resisting moment = $CR.R\,\alpha = CR^2\,\alpha$ (kgm).

Factor of safety against slipping along surface PHV = $\frac{\text{Maximum resisting moment}}{\text{Actuating moment}}$

$$= \frac{CR^2\,\alpha}{W \times x_1} \qquad ...(3.6)$$

Various other slip circles like the above one, e.g. QH_1V, SH_2V, TH_3V, etc. are drawn to scale and the factor of safety is found, as shown above, for each such slip circle. The slip circle giving the minimum factor of safety is the critical slip circle, because, the failure, if it occurs, will occur along this slip circle. This critical slip circle is located as shown below.

The values of the factor of safety worked out for various slip circles say four, are plotted as shown in Figure 3.6 and a curve is drawn passing through the tops of the ordinates representing the four values of the factors of safety corresponding to four slip circles. The lowest point on this curve is noted and a vertical line is drawn through it, meeting the top of soil mass at a point say Z; V and Z then mark the two points through which the required critical slip circle will pass.

b) **C- ϕ Soil**

The shear strength of such a soil due to cohesion and internal friction varies from point to point along the slip circle. The equation for shear strength of this soil is

$$S = C + P_n \tan\phi \text{ (kg/m}^2\text{)} \qquad ...(3.7)$$

where C is constant along the slip circle but P_n which is the pressure normal to the slip circle due to the weight of soil varies from point to point along the slip circle. The method of locating the critical slip circle is as follows:

Figure 3.7 shows the Swedish method for C- ϕ soil. The whole soil mass adjoining the slope, is divided into vertical strips or slices as shown in Figure 3.7(a). The fifth strip from the left in the figure is taken for the study and is shown on a bigger scale in Figure 3.7(b).

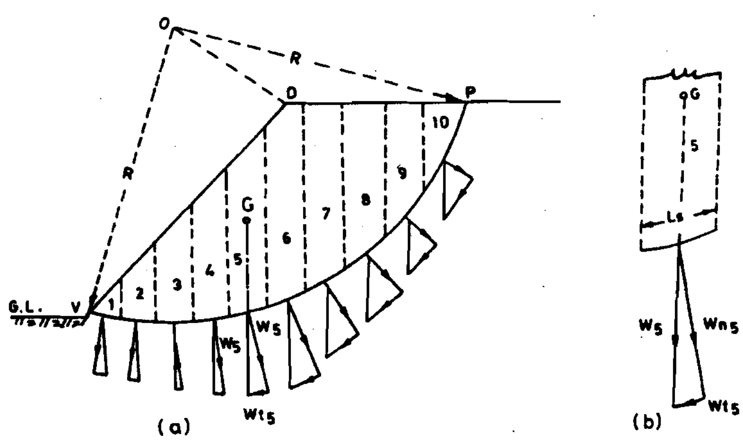


Figure 3.7 : Swedish Method for C – ϕ Soil

Taking 1 m of the strip, its equilibrium is practically due to two forces only, namely, its weight W_5 and its shearing strength S_5 along the curved surface of length l_5 m at the bottom of the strip. The weight W_5 is equivalent to two forces, namely, W_{n5} which is normal to the curved surface and, the tangential force W_{t5} which is tangential to the curved surface as shown. Thus the equation will be

$$S_5 = C.\,l_5 + W_{n5}.\,\tan\phi \text{ (kg)}$$

where,

$C.l_5$ = cohesive strength per m length of the strip (kg), and

$W_{n5}.\tan\phi$ = frictional strength per m length of the strip.

The moment of this shear strength is = $R.S_5$ (kg-m)

$$\text{PHV} = \sum R\,[C.\,l_5 + W_{n5.}\ \tan\ \phi] \quad \text{(kg)}$$

and the moment of shear strength along the entire slip surface,

$$\text{PHV} = \sum R\,[C.\,l_5 + W_{n5.}\ \tan\ \phi]$$

$$= R \sum\ [C.\,l_5 + W_{n5.}\ \tan\ \phi]$$

$$= R \sum\ [CL + \tan\ \phi.\ W_n \quad \text{(kg-m)}$$

where,

L = length of the whole slip surface PHV.

The actuating moment along slip surface at bottom of the strip = $R.\ W_{t5}$ (kg-m).

Hence, the actuating moment along the entire slip surface PHV

$$= R.\ \sum W_{t5} \quad \text{(kg-m)}$$

The factor of safety, against shear failure, along the slip surface PHV $= \dfrac{\text{Resisting moment}}{\text{Actuating moment}}$

or, in general, the factor of safety $= \dfrac{CL + \tan\phi.\ \sum W_n}{\sum W_t}$

The weight of each strip 1 m long, is proportional to its area and this latter can be found accurately by a planimeter. W_n and W_t of each strip can be found graphically by drawing the triangle of forces for each strip (Figure 3.7 (a)).

The factor of safety is found for each of other slip circles and the critical circles is located as usual. It should be carefully noted, that the tangential components of the weights of the first few strips near the toe of the slope, will resist the slipping tendency and these components must therefore, be taken with their proper sign.

Location of Critical Slip Surface

The location of the critical slip surface by trial and error entails much loss of time. To save time in locating the centre of critical slip circle for homogeneous sections, Fellenius construction for locating the line on which the centre of the critical circle is most likely to lie, is generally adopted. Various circles with centres on this line are tried until the one with minimum factor of safety is found.

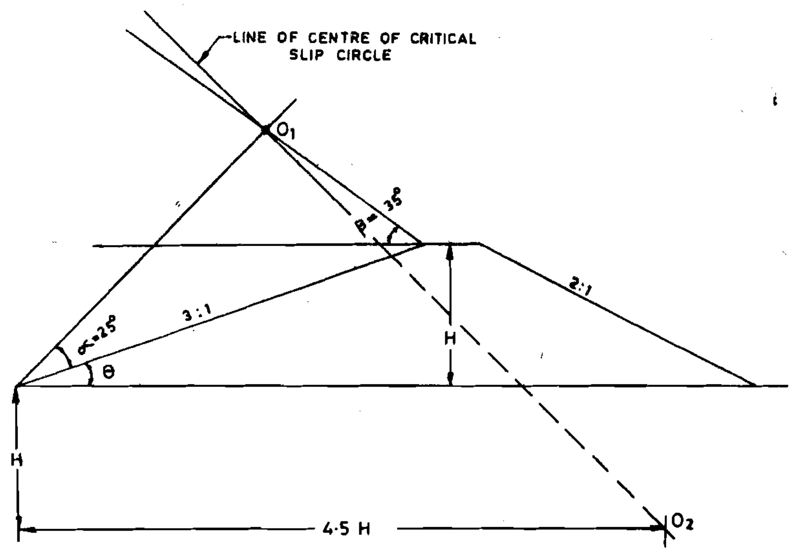


Figure 3.8 : Fellenius Method of Locating Line of Centre for Critical Slip Circle

According to the Fellenius construction, the position of the line on which the centre of the critical circle lies depends only on the height and slopes of the embankment. Referring to Figure 3.8, the values of α and β are taken from Table 3.2 for the embankment slope. From point A and B these angles are drawn to meet at point O_1. Point O_2 is then located at a horizontal distance of 4.5 *H*, starting directly below point A, and at a depth equal to *H*, the height of the embankment. The line joining O_1 and O_2 is the line on which the centre of the critical circle lies. The maximum depth to which the rupture can occur is limited by the presence of hard stratum underneath.

Table 3.2 : Recommended Values of α and β for Fellenius Construction

Slope (*H*:*V*)	α	β
1 : 1	27.5°	37°
2 : 1	25°	32°
3 : 1	25°	35°
4 : 1	25°	36°
5 : 1	25°	37°

SAQ 6

i) How will you test the stability of an earth dam constructed of cohesive soils?

ii) How will you test the stability of an earth dam constructed of C-ϕ soils?

3.9 ROCKFILL DAM SECTION

Rockfill dams (Figure 3.2) can be divided into two main types:

a) Impervious membrane type which depends on an impervious membrane of cement concrete, asphaltic concrete, or rarely of steel or timber for water tightness.

b) Earth core type with an earth core sloping at 1.3:1 or 1.4:1 (*H*:*V*) placed against a dumped rockfill. If the rockfill is placed in layers, the core may be placed at any slope or even centrally.

The main structural element of the rockfill dams is quarry run rock, dumped or placed in layers. This material being highly pervious requires a membrane or earth core for water tightness. On hard rock formations, and if suitable core material is not available, the membrane type would normally be used. Where suitable core materials are available, the core type would generally be found more economical. Also for large heights likelihood of cracking and leakage through membrane makes core type preferable.

Material for Rockfill Dams

The rock used in rockfill construction should be sound and should neither disintegrate during long exposure to water and weather nor split and crush under the heavy stresses to which it will be subjected. Massive igneous and metamorphic rocks are satisfactory.

Sedimentary rocks like sandstones and limestones have also been used satisfactorily.

In earlier practice of dumped construction large sized rocks, weighing 3 - 5 t and larger were used. In more recent practice, however, smaller rock is preferred.

Foundation Requirements

Large rockfill dams are usually founded upon bed rock. It is, however, not necessary to look for rock foundations of as high quality as for concrete or masonry dams. For dams of moderate or low heights, river bed sand gravel and boulder, when dense and compact, with shear strength of the same order as that of the rockfill, may be retained and need not be excavated.

Dam Cross-section of Membrane Type

Dumped rockfill adopts a natural slope of about 1.3:1 to 1.4:1 (*H* : *V*), and the downstream slope is usually kept at this natural rock slope (Figure 3.9). If no rehandling of dumped rock is desired, the upstream slope will also be the same. However, in a number of dams the upstream slope has been made steeper, about 0.75:1 (H:V), to reduce the quantity of rock as well as the area of the membrane. This requires rehandling of the rock and a thick zone of "rubble cushion" to resist rock pressure.

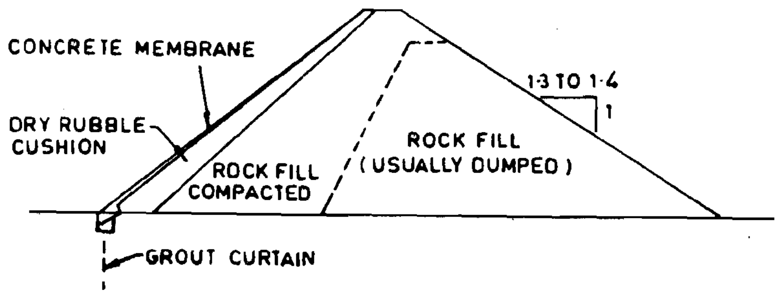


Figure 3.9 : Membrane Type Rockfill Dam

The transition between the main rockfill and the face membrane was built in the past of hand or derrick laid rock. The more common practice now is to provide well graded rockfill of small boulder-gravel material compacted in layers of 30 - 45 cm thickness. It acts as a cushion distributing the water load from the membrane to the main rockfill evenly. On self supporting rock slope it may be 1.5 - 3 m thick at the top increasing at 5 per cent of height towards the bottom.

A rockfill dam of the proportions given above is automatically safe against sliding. It has practically no pore pressures, good seismic resistance, and is inherently a stable structure.

Membranes

The most common type is reinforced cement concrete membrane. The thickness is usually about 30 cms at the top increasing at about 0.5 - 1 per cent of the height. The reinforcement is usually 0.5 per cent of the sectional area of the slab in each direction. The slab is generally poured directly on the rubble cushion and is provided with joints 10 - 12 m apart. The horizontal joints are tight while the vertically inclined expansion joints permit movement. Reinforcement does not pass the joints. Water seals have to be provided at the joints.

Asphaltic concrete membranes cost less than reinforced concrete and are more flexible and hence less susceptible to cracking. They can also be constructed more rapidly. The asphaltic concrete is made of aggregates from 25 mm size to fine rock dust with 8 - 10 per cent by weight of asphaltic binder.

Rockfill Dams with Earth Cores

Rockfill dams with earth cores are just like earth dams except that the shell material is free draining rockfill.

Typical sections of Oroville dam and Nurek dam are shown in Figure 3.10 and Figure 3.11, respectively. It is necessary to provide adequate filter between the core and the rockfill on

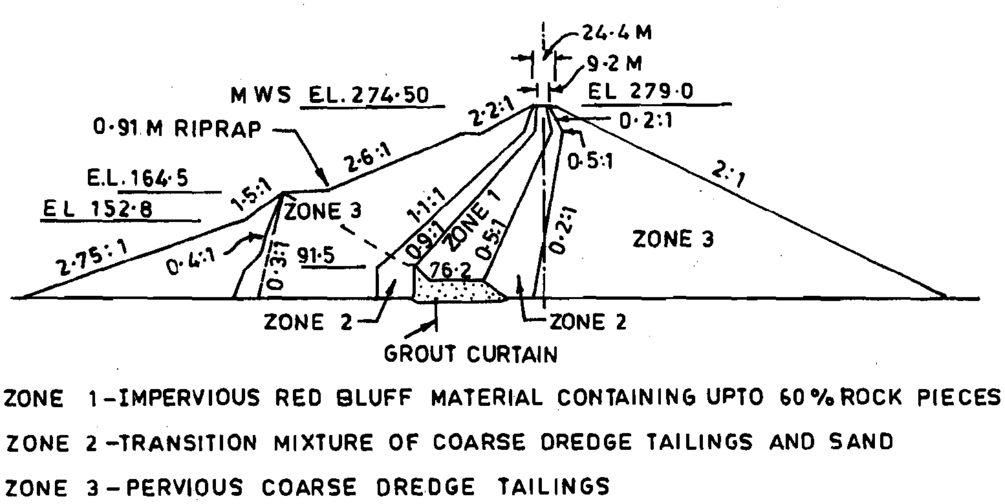


Figure 3.10 : Oroville Dam

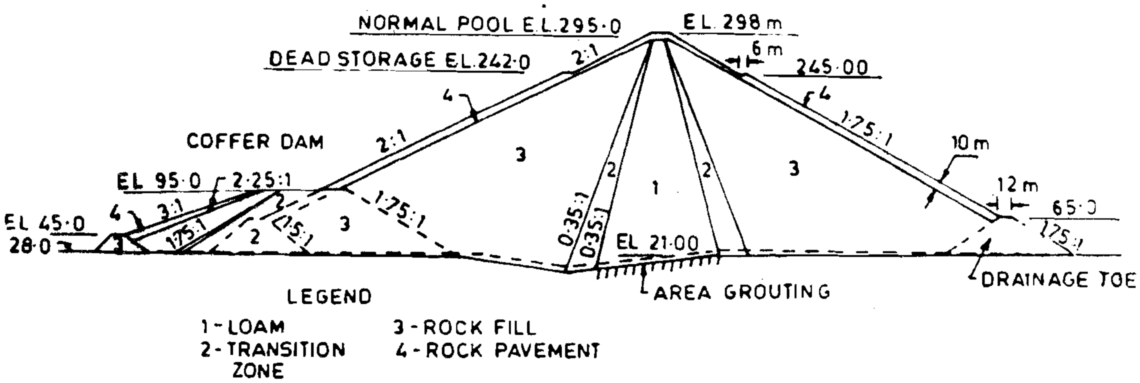


Figure 3.11 : Nurek Dam Section (USSR)

both sides of the core so that core particles may not migrate into the rockfill. For stability analysis, surfaces of failure consisting of two or three planes are more appropriate than circular arcs. The method of analysis is called the "wedge method" and is beyond the scope of this unit.

Method of Rock Placement

In the earlier construction practice the rock was dumped at site from large heights and this was usually done from buckets travelling on cableways strung across the valley, but sometimes from the sides of the valley itself. The idea was to cause the rock to break its corners and roll into a stable position. Jets of water were played on the rock as it was falling for the same purpose.

A technique of placing rockfill in 1 - 2 m thick layers, more or less like earth, is gradually becoming more popular. Smaller, well graded rock is used in this case. The layers are compacted by vibratory rollers or caterpillar tractors. Water is commonly used, but not always.

Settlement of Rockfill Dams

The main problem of rockfill dams is their settlement resulting in danger of cracking of the membrane. The settlement results from crushing of rock corners and readjustment of rock under load. The settlement may be roughly estimated by

$$S = 0.001\, H^{3/2}$$

where,

S = settlement (m), and

H = dam height (m).

During construction all effort should be made to obtain a compact mass. The placing of the membrane should be delayed to enable settlement under rock load to be substantially completed. Subsequent settlement under water load is, however, unavoidable.

SAQ 7

i) What are the main types of rockfill dams?

ii) What materials are used in a rockfill dam?

iii) What are the foundation requirements of a rockfill dam?

iv) Describe a membrane type rockfill dam.

v) What materials are used as membranes in rockfill dam?

vi) Describe a rockfill dam with an earth core.

vii) What is the method of placing rock in a rockfill dam?

viii) Why does a rockfill dam settle? How can it be restricted?

3.10 SUMMARY

Earth and rockfill dams are economical structures if adequate quantities of the appropriate materials are available. The various types of earth dams depends on the quantities of materials available. The stability of the dam section has to be analysed for the worst combination of loads. The methods of analysis also depends upon the materials used in the dam section. The quality control during construction has to be good to avoid problems with these structures arising from careless workmanship. All these aspects would be clear to you by the end of this unit.

3.11 KEY WORDS

Cohesive Soils : Soils having only cohesion and no friction, such as clays.

C- ϕ Soils : Soils having both cohesion and friction between soil particles.

Conduit Leakage : Conduits are sometimes embedded in the body of an earth dam. Seepage may occur along the outside of the conduit which develops into piping or leaks through the conduit may cause seepage which may also lead to piping.

Diaphragm Dam : A dam in which a thin diaphragm of impervious material such as clay or cement concrete is placed in the central or on the upstream face of the dam. The diaphragm may be vertical or inclined.

Foundation Requirements : Rockfill dams are founded on rock foundation but not as strong as that required for a concrete or a masonry dam.

Foundation Slides : Slides caused in the foundation due to seams of weathered rocks, shales, or soft clay.

Freeboard : It is the required allowance in the dam height against water splashing on the dam top due to effect of wind.

Homogeneous Dam : A dam constructed with a single material.

Hydraulic Failures : Failures of an earth dam may be attributed to hydraulic failures, that is, to surface erosion due to overtopping or rainfall or scouring due to wave action.

Leakage through Embankments : Leakage through embankment lead to piping failures.

Location of Critical Slip Circle : The slip circle having the lowest factor of safety against failure is the critical slip circle. Various methods are available to locate the critical slip circle depending on the type of material used in the dam and that occurring in the foundation.

Material for Rockfill Dams : They include igneous, metamorphic and sedimentary rocks of large and small sizes to make up the bulk of the dam section.

Membranes : They are impervious materials laid on the upstream face of a rockfill dam to reduce seepage through the rockfill dam section.

Methods of Rock Placement : Rock is placed in the rockfill dam so as to minimise the effort of placement. Various methods are adopted.

Pervious Foundations : Foundations consisting of sands and gravels or cavities and fissures allow profuse seepage from the reservoir.

Rockfill Dams with Earth Cores : They are similar to earth dams with a freely draining shell.

Seepage Failures : Failures of earth dams due to excessive seepage through the foundation or the embankment are termed seepage failures.

Settlement of Rockfill Dams	:	The upstream membrane may crack due to the settlement of a rockfill dam and this is the most serious problem with such dams.
Slides in Embankment	:	When the slope of embankment is too steep for the strength of the soil, slides may take place.
Sloughing	:	The downstream portion of the dam may slump due to erosion leaving a relatively steep face which slumps again.
Stability Analysis	:	The checking of the dam section for safety against all destabilising forces is stability analysis.
Structural Failures	:	They are due to slides in the foundation or in the embankment.
Top Width	:	It is the width of the dam at the top required for a roadway and the parapet walls.
Zoned Dams	:	Embankment dams formed of a number of different materials with transition zones between different material in contact are zoned dams.

3.12 ANSWERS TO SAQs

SAQ 1

a) Homogeneous dam, zoned dam and diaphragm dam.

SAQ 2

Hydraulic failures; seepage failures; and structural failures.

FURTHER READINGS

1) Asawa, G.L. (1993): *Irrigation Engineering.* Wiley Eastern Ltd, New Delhi, 568p.

2) Golze, A.R. (ed) (1977): *Handbook of Dam Engineering.* Van Nostrand Reinhold Co., New York, 793 p.

3) Linsley, R.K.; Franzini, J.B. (1964): *Water Resources Engineering.* McGraw-Hill Book Co, New York, 2nd ed, 690 p.

4) Mutreja, K.N. (1990): *Applied Hydrology.* Tata McGraw-Hill Publishing Co. Ltd., New Delhi, 959p.

5) Singh, Bharat (1988): *Fundamentals of Irrigation Engineering.* Nem Chand & Bros., Roorkee, 8th ed, 608 p.

6) USBR (1970): *Design of Small Dams.* Oxford & IBH Publishing Co., Calcutta, 611 p.

7) Varshney, R.S.; Gupta, S.C.; Gupta, R. (1972): *Theory and Design of Irrigation Structures.* Nem Chand & Bros., Roorkee, 858p.

8) Yoganarasimhan, G.N. (1984): *Reservoir Operation Studies.* National Institute of Hydrology, Jal Vigyan Bhavan, Roorkee, 79p.