

## 2.2 Embankment Dam

### 2.2.1 Introduction

Embankment dam is a water impounding structure constructed from fragmental natural materials excavated or obtained close to the dam site. The natural fill materials are placed and compacted without the addition of any binding agent, using high capacity mechanical plant. They rely on their weight to resist the flow of water, just like concrete gravity dams.

Embankment dam derive its strength from position, internal friction and mutual attraction of particles. Relative to concrete dams, embankment dams offer more flexibility; and hence can deform slightly to conform to deflection of the foundation without failure.

Broadly, depending upon the material used during construction, embankment dams are classified in to two:

- 1) *Earth fill Embankments*: if compacted soils, i.e. clays/silts & sands, account for over 50% of the placed volume of material
- 2) *Rock fill Embankment*: if compacted rock particles larger than a man can easily lift, i.e. coarse grained frictional material, accounts for over 50% of the placed volume of materials.

Embankment dam possesses many outstanding merits which could be summarized as follows:

- A. Suitability of the type to different site conditions such as wide valleys, steep sided gorges, etc.
- B. Adaptability to a broad range of foundation condition such as rock and pervious soil formation,
- C. use of natural materials,
- D. Extreme flexibility to accommodate different fill materials,
- E. Highly mechanized and effectively continuous construction process,
- F. Appreciable accommodation of settlement-deformation without risk of serious cracking and possible failure.

The relative disadvantages of the embankment dam are

- A. Inherently susceptible to damage or destruction by overtopping
- B. Necessity of separate spillway structure
- C. Vulnerability to concealed leakage and internal erosion in dam or foundation

#### a. Key elements and appurtenances of Embankment dam

Every embankment dam consists of three basic components plus a number of appurtenances which enable the basic components to function efficiently shown in Figure 2-1.

#### **Foundation:**

The foundation of embankment dam could either be earth or rock material. The foundation provides support resisting both vertical and horizontal loads. It may also resist seepage beneath the embankment

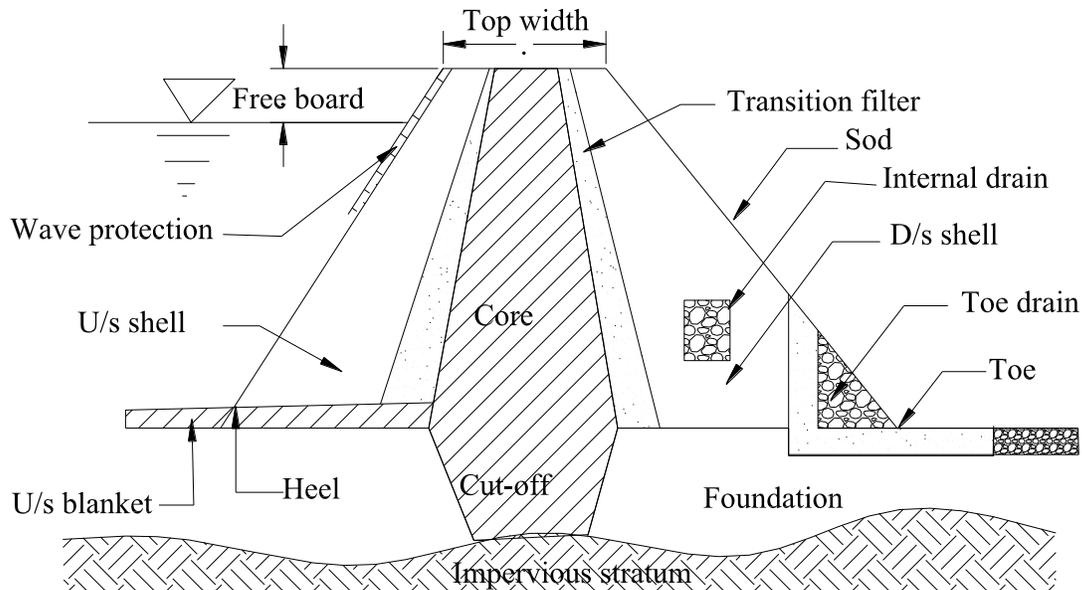


Figure 2-1 Basic components and appurtenances of Embankment dam

### Core or membrane

The primary purpose of the core or membrane is to hold back free water. Depending on the structural requirements of the dam, the core may be placed at the center or upstream from the center, or on the upstream face (in the case of certain rock fill dams)

When the foundation is incapable of resisting under seepage the core is extended down into the foundation to impervious layer. Such an extension of the core is termed cut-off.

### Core material:

Earth, concrete or masonry, steel sheeting, etc. are used as core material. Lack of flexibility of concrete and masonry make them undesirable. An earth core (when suitable material is available) is usually cheaper and more water tight than any other type. Suitability of earth core depends on the character of the available soil.

Table 2-1: Permeability of different soil types

Permeability coefficient	Typical soil	Value as core
2 – 0.002	Sand	Considerable leakage
0.002 – 0.0002	Silty clay	Usable with good control if some leakage is tolerable
0.0002 – 0.000006	Silts	Little leakage if well compacted
$\leq 0.000006$	Silty clay, clay	Impervious

A core should not be composed of silt which tends to swell upon saturation. To avoid swelling tendencies, the elasticity index should not exceed 30.

Typical requirements for core compaction are

90 – 97% of standard proctor maximum, or

87 – 95% of modified proctor maximum.

*Water content:* - as high as possible consistent with the above requirements.

*Core thickness:* - to control erosion and provide good compaction a minimum core thickness in meters is given by

$$b = 6 + 0.1\Delta h \quad (\text{clay})$$

$$b = 6 + 0.3\Delta h \quad (\text{silt})$$

Where:  $\Delta h$  = head difference at that point;

b = core width at that point.

### Shell

The purpose of shell is to provide structural support for the core and to distribute the loads over the foundation. The shell also acts as foundation for most of the appurtenances. Sometimes the core and shell of a dam are constructed of the same material (homogenous dam).

#### Shell (embankment) materials

Availability and strength are the requirements for selection.

**Strength:** - the strength for the upstream side should be that at the inundated condition.

The same strength should be used for the downstream face which is below the maximum phreatic line.

**Permeability:** - high permeability is desirable from the standpoint of pressure buildup during construction and stability during sudden drawdown.

Typical compaction requirements:

95 – 100% of standard proctor maximum;

92 – 97% of modified proctor maximum.

**Slopes:** - shell slopes are based on stability analysis. When the stability is insufficient, improvements are possible by adopting

- Flatter slopes;
- Increasing strength through high density;
- Treatment for weak foundation;
- Drainage of the foundation and embankment.

Table 2-2: Tentative slopes of shoulder for different embankment materials

Soil type	Upstream	Downstream
Gravel, sandy gravel with core	2.5H : 1V	2.0H : 1V
Clean sand with core	3.0H : 1V	2.5H : 1V
Low density silt, micaceous silt	3.5H : 1V	3.0H : 1V
Low plasticity clay	3.0H : 1V	2.5H : 1V

**Composite slopes:** - are used for large dams. They can be found in two ways: a series of straight slopes or a constant slope with berms.

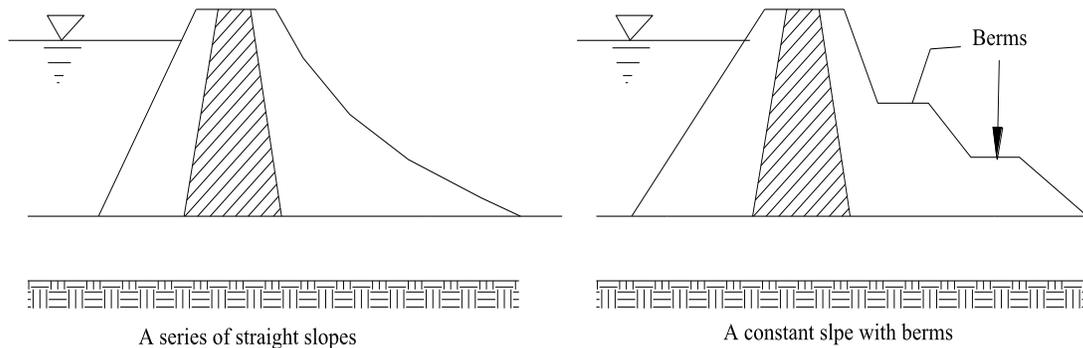


Figure 2-2 composite slopes for shell of embankment dam

A berm is a level surface on the slope that can serve the following purposes

1. Increases slope stability by increasing dam width;
2. Breaks the continuous downstream slope to reduce surface erosion
3. Provides level surface for maintenance operations, roads, etc.

Berm is also used at the bottom of a zone of riprap to provide supporting shoulder.

#### Height of dam:

Required height of an embankment dam is the vertical distance from the foundation to the water surface in the reservoir, when the spillway is discharging at design capacity, plus a free board allowance.

Free Board = maximum wave run-up height + allowance for settlement + allowance for splash

Maximum wave run-up height =  $4h_w/3$

Where:  $h_w$  = effective wave height (with expectancy of 1%)

Table 2-3: Wave run-up to maximum wave height ratio on slopes

Slope	Ratio of run-up to maximum wave height	
	Smooth Surface	Riprap surface
1.5H : 1V	2.5	1.6
1H : 1V	2.0	1.3

Maximum vertical height of run-up = Expected wave height \* appropriate factor from Table 2-3

Settlement allowance: the following may be used as guide.

For foundation: 1% of height of dam

For embankment: 1-2% of height of embankment

Splash allowance could be taken 0.30 – 0.50m.

#### Top Width:

- Should be sufficient to keep the phreatic line with in the dam when the reservoir is full
- Should be sufficient to withstand wave action and earthquake shock

- Has to satisfy secondary requirements such as minimum roadway width.

**Appurtenances**

*Transition filter:* - it is provided between core and shell to prevent migration of the core material into the pores of the shell material. It is particularly needed between clay cores and rock and gravel shells.

The objective of transition filter is to carry away seepage that has passed through the core and cut-off and to prevent stratum of the upper part of the downstream shell.

*Toe drain:* - it helps to prevent sloughing of the downstream face as a result of rain water or seepage saturation. In small dams, the toe drain serves also as internal drain. In large dams with pervious foundation, the toe drain and the internal drain are sometimes combined. Drains need protective filter (inverted filter) to prevent clogging of the drain.

*Riprap:* - required to cover the upstream/downstream face.

Normally riprap extended from above the maximum water level to just below the minimum.

*Sod:* - required on the downstream face to prevent rain wash.

For economic reasons, the material available at the particular site has to be employed as much as possible for the construction of the earth dam and the quantity of imported material should be minimized.

*Internal drains:* - they are essential in large dams where the d/s shell is not so pervious.

### 2.2.2 Types of Embankment dam

The materials available locally control the size and configuration of the dam. Many small embankment dams are built entirely of a single type of material such as stream alluvium, weathered bedrock, or glacial till. These are *homogeneous* dams, constructed more or less of uniform natural material as shown in Figure .

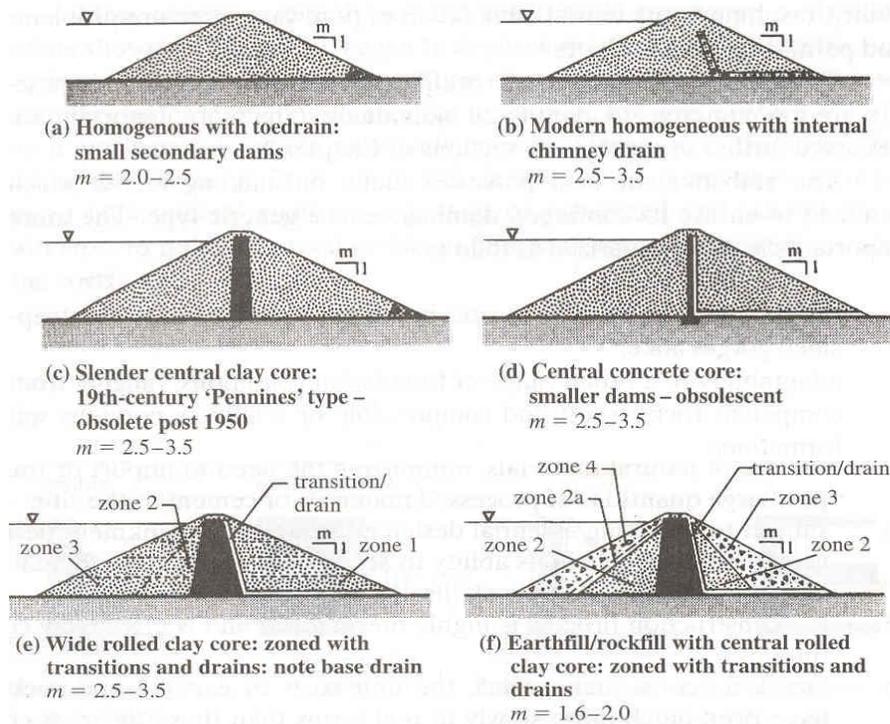


Figure 2.3 Principal variants of earth fill embankment dams (Values of  $m$  are examples)

The central core earth fill profile, shown in Figure (c) and (d), is the most common for larger embankment dams. Larger embankment dams are also zoned and constructed of a variety of materials Figure (f), either extracted from different local sources or prepared by mechanical or hydraulic separation of source material into fractions with different properties.

An important element in a zoned dam is an impermeable blanket or core which usually consists of clayey materials obtained locally. In locations where naturally impermeable materials are unavailable the dams are built of rock or earth-rock aggregates as shown in Figure 2-, and the impermeable layers of reinforced concrete, asphalt concrete, or riveted sheet steel are placed on the upstream face of the dam.

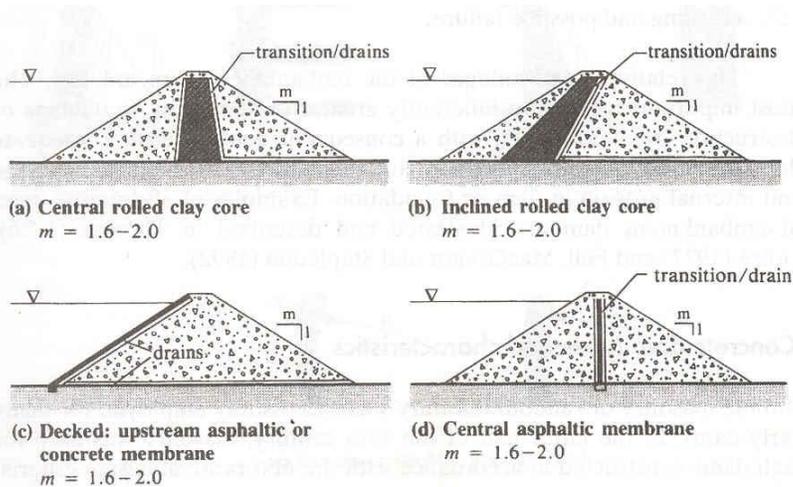


Figure 2-4 Principal variants of rock fill embankment dams (values of  $m$  are examples)

Selection of the optimum type of embankments for a specific location is determined largely by the nature and availability of different fill materials in sufficient quantity.

The primary loads acting on an embankment do not differ in principle from those applicable to gravity dams. There are, however, the conceptual differences there referred to with regard to the water load which is exerted inside the upstream shoulder fill. Self weight load, similarly a distributed internal body load, is significant with respect to stability and internal stress for the embankment and for a compressible soil foundation.

Because of such differences, embankment dam analysis is less formalized and is carried out quite differently from concrete dam analysis.

### 2.2.3 Causes of Failure of Embankment dams

Embankment dams, like any other engineering structure, may fail due to improper design, faulty constructions, lack of maintenance, etc. Generally, causes of failure are grouped into three classes: Hydraulic failure, Seepage failure and Structural failure.

**Hydraulic failures:** About 40% of earth dam failures have been attributed to these causes due to;

1. *Overtopping.* Occurs when the design flood is less than the coming flood. Spillway and outlet capacity must be sufficient to prevent overtopping. Freeboard should also be sufficient to prevent overtopping by wave action.
2. *Erosion of upstream faces.* Wind waves of water developed due to wind near the top water try to notch-out the soil from u/s face and may even sometimes cause the slip of the u/s slope.(upstream slope pitching or rip rap should be applied.)
3. *Erosion of downstream face by gully formation.* Heavy rains falling directly over d/s face and the erosive action of the moving water may lead to the formation of gullies on the d/s face, ultimately leading to the dam failure.
4. *Cracking due to frost action.* Frost in the upper portion of dam may cause heaving of soil with dangerous seepage. Consequently failure. Provide an additional free board allowance up to a maximum of say 1.5m may be provided.

**Seepage failure:** controlled seepage or limited uniform seepage is inevitable in all embankments and it does not produce any harm. However, uncontrolled or concentrated seepage through the dam or the foundation may lead to piping\* or sloughing† and the subsequent failure of the dam.

**Structural failure:** about 25% of the dam failures have been attributed to structural failures. Structural failures are generally caused by shear failures, causing slides.

Causes of failure as categorized based on time of occurrence

During construction

- Unstable slop
- Heavy rainfall that washes the d/s face
- Weak foundation

After construction

- Failure of u/s face due to sudden drawdown
- Failure of d/s when the reservoir is full
- Overtopping
- Seepage failure.

#### 2.2.4 Design features

Some of the more important features that should be considered in the design of embankment dams are:

1. *Zoning of shoulder-fills:* the permeability of successive zones should increase toward the outer slopes, materials with a high degree of inherent stability being used to enclose and support the less stable impervious core and filter.
2. *Spillway location:* geotechnical and hydraulic design considerations require that to minimize the risk of damage to the dam under flood conditions the spillway and discharge channel are kept clear of the embankment.
3. *Freeboard:* is the difference between maximum reservoir level and minimum crest level of the dam. The provision necessary for long-term settlement within the overall minimum freeboard is determined by the height of dam and the depth of compressible foundation at any section.

The overall minimum freeboard from spillway sill to dam crest should be at least 1.5m on the smallest reservoir embankment, and it will be very much greater for larger embankments and/or reservoir.

The minimum height of freeboard for wave action is, generally,  $1.5h_w$

$$h_w = 0.032\sqrt{v.F} + 0.763 - 0.271\sqrt[4]{F} \quad \text{For } F < 32km$$

$$h_w = 0.032\sqrt{v.F} \quad \text{For } F > 32km$$

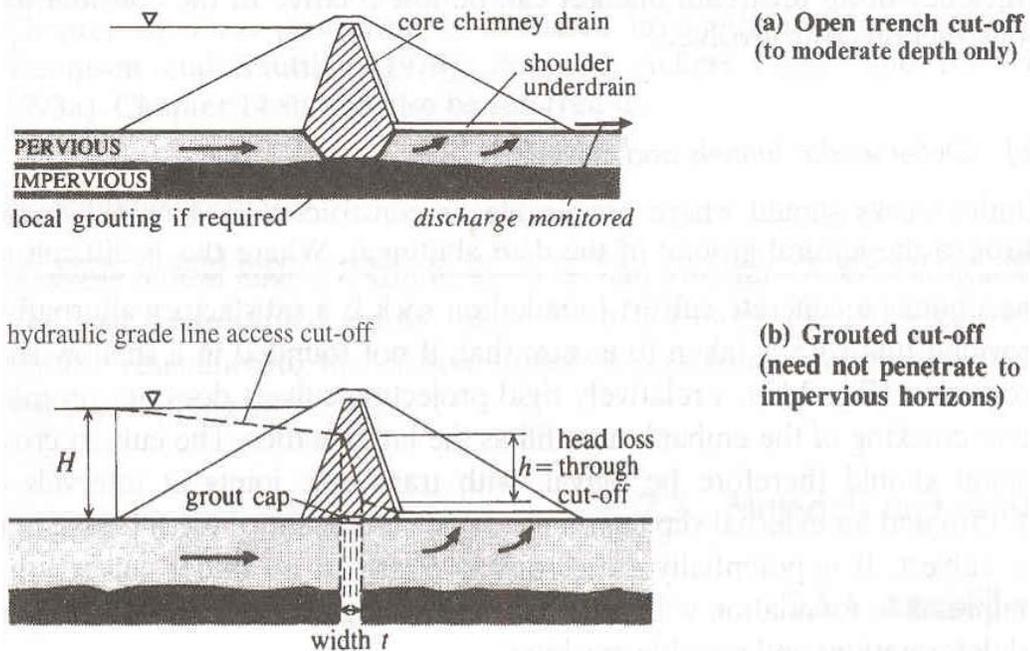
Where;  $v$  is wind velocity (km/hr)

\* The progressive erosion and subsequent removal of soil grains from within the body of the dam or the foundation of the dam

† The progressive removal of soil from the wet d/s face.

$F$  is fetch or straight length of water expansion in km

4. *Foundation seepage control*: seepage flows and pressure within the foundation are controlled by cut-offs and by drainage. *Cut-offs* is impervious barriers which function as extensions of the embankments core into foundation. The cut-offs are generally two types:
- Fully penetrating cut-off: penetrate to impervious strata
  - Partially penetrating cut-off: terminate where the head loss across the cut-off is sufficient to effect the required degree of control



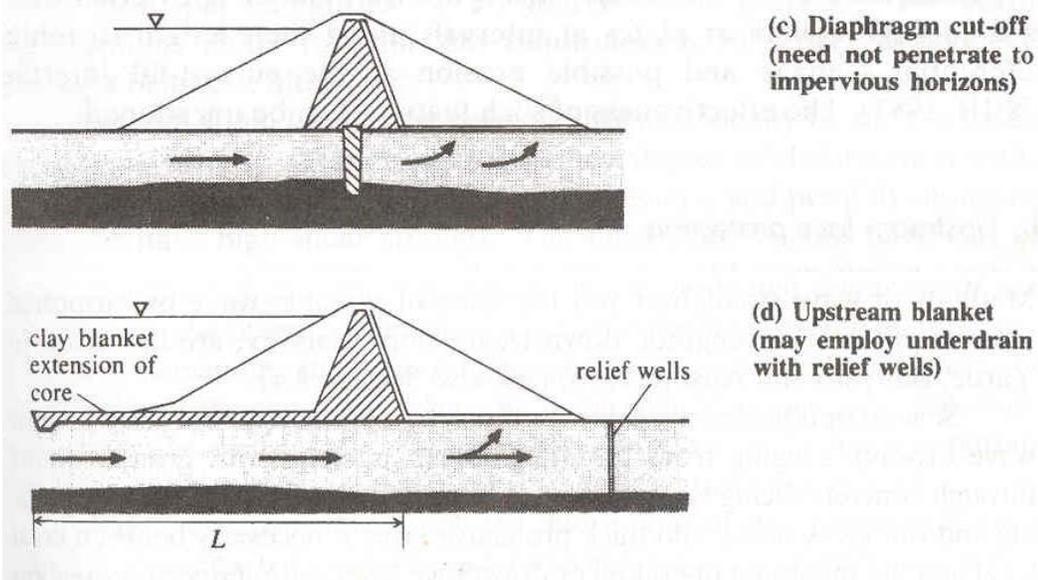


Figure 2-5 Cut-offs and control of under seepage

5. *Outlet works (tunnels and culverts)*: outlet works should where practicable be constructed as a tunnel driven through the natural ground of the dam abutments. Where this is difficult or uneconomical a concrete culvert founded on rock is a satisfactory alternative.
6. *Upstream face protection*: several options are available for protection of the upstream face against wave erosion, ranging from traditional stone pitching with grouted joints through concrete facing slabs to the use of concrete block work, rock armoring and riprap.
7. *Embankments crest*: the top width of larger earthen dam should be sufficient to keep the seepage line well within the dam, when reservoir is full. The crest should have a width of not less than 5m, and should carry a surfaced and well-drained access road. The top width (W) of the earth dam can be selected as per the following recommendation:

$$W = \frac{H}{5} + 3 \quad , \text{ For very low dams} \quad 4.1$$

$$W = 0.55\sqrt{H} + 0.2H \quad , \text{ For dams lower than 30m} \quad 4.2$$

$$W = 1.65(H + 1.5)^{\frac{1}{3}} \quad , \text{ For dams higher than 30m} \quad 4.3$$

Where: H is the height of the dam.

### 2.2.5 Criteria for Safe Design of Earth Dam

An earth dam must be safe and stable during phases of construction and operation of the reservoir. The practical criteria for the design of earth dams may be stated briefly as follows.

1. No overtopping during occurrence of the inflow design flood.
  - a. appropriate design flood
  - b. Adequate spillway
  - c. Sufficient outlet works
  - d. Sufficient free board
2. No seepage failure
  - a. Phreatic (seepage) line should exit the dam body safely without sloughing downstream face.
  - b. Seepage through the body of the dam, foundation and abutments should be controlled by adapting suitable measures.
  - c. The dam and foundation should be safe against piping failure.
  - d. There should be no opportunity for free passage of water from U/S to D/S both through the dam and foundation.
3. No Structural failure
  - a) Safe U/S & D/S slope during construction
  - b) Safe U/S slope during sudden draw down condition.
  - c) Safe D/S slope during steady seepage condition
  - d) Foundation shear stress within the safe limits.
  - e) Earth quake resistant dam
4. Proper slope protection against wind & rain drop erosion.
5. Proper drainage
6. Economic section

### 2.2.6 Seepage analysis

Seepage occurs through the body of all earthen dams and also through their pervious foundation. The phreatic surface of the seepage regime, i.e. line within the dam section below which there is positive hydrostatic pressures in the dam, must be kept well clear of the downstream face to avoid high pore water pressures which may promote slope instability.

The amount of seepage can be easily computed from the flow net, which consists of two sets of curves, known as '*Equipotential line*' and '*stream lines*', mutually perpendicular to each other. For homogeneous embankments dam, discharge per unit width ( $q$ ) of the dam passing through a flow net is described as:

$$q = kH \frac{N_f}{N_d}$$

Where:  $H$  is the head differential.

$N_f$  is number of stream lines.

$N_d$  is number of equipotential lines.

#### Determination of Phreatic Lines

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

- i. The divide line between the dry (or moist) and submerged soil.

- ii. The top stream line and hence, helps us in drawing the flow net.
- iii. To ensure that the phreatic line doesn't cut the downstream face of the dam, which is extremely necessary for preventing softening of the dam.

#### A. Homogeneous dam section with horizontal filter

It has been found that the seepage line is pushed down by the filter and it is very nearly parabolic except near its junction with u/s face. Since the u/s face of the dam (i.e. GB in Figure 2-7) becomes an equipotential line when fully covered with water, the seepage line shall be perpendicular to the face near its junction point B.

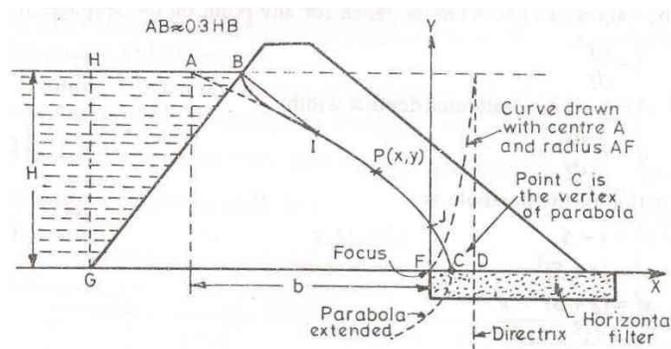


Figure 2-6 Seepage through homogenous dam section with horizontal filter

#### Equation of the base parabola

Let a base parabola with focus at F is drawn and produced so as to intersect the water surface at a point A as shown in fig 2.7. Taking the focus (F) as the origin, equation of the parabola  $p(x, y)$  can be written as

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

Where;  $FD$  is the distance of the focus from the directrix, called focal distance and is represented by  $S$ .

Hence the equation of the parabola of the seepage line becomes:

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

Location of A is approximately  $0.3HB$  horizontal distance upstream from point B according to Cassagrande. Where,  $H$  is the projection of the point G on the water surface.

If the horizontal distance between the already determined point A and the focus (F) is taken as say  $b$ , then  $(b, H)$  represents the coordinates of the point A on the parabola. And hence;

$$\begin{aligned}\sqrt{b^2 + H^2} &= b + S \\ S &= \sqrt{b^2 + H^2} - b\end{aligned}$$

The center point (C) of  $FD$  will then be the vertex of the parabola. When  $x = 0$ ,  $y = S$ . Hence the vertical ordinate  $FJ$  at F will be equal to  $S$ . Knowing the points A, C, and J and

working out a few more points from the equation, the parabola can be easily drawn and corrected for the curve BI, so as to get the seepage line BIJC.

The amount of seepage can also be calculated easily from the equation of the seepage line as derived below.

Darcy's law is defined as,  $q = KiA$ . When steady conditions have reached, the discharge crossing any vertical plane across the dam section (unit width) will be the same. Hence, the value  $i$  and  $A$  can be taken for any point on the seepage line

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

$$A = y * 1$$

$$q = K \frac{dy}{dx} y$$

But from the equation of the parabola,

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

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

$$q = KS$$

Example:

An earth dam made of a homogeneous material has a horizontal filter and other parameters as shown in the figure. Determine the phreatic line and the seepage quantity through the body of the dam.

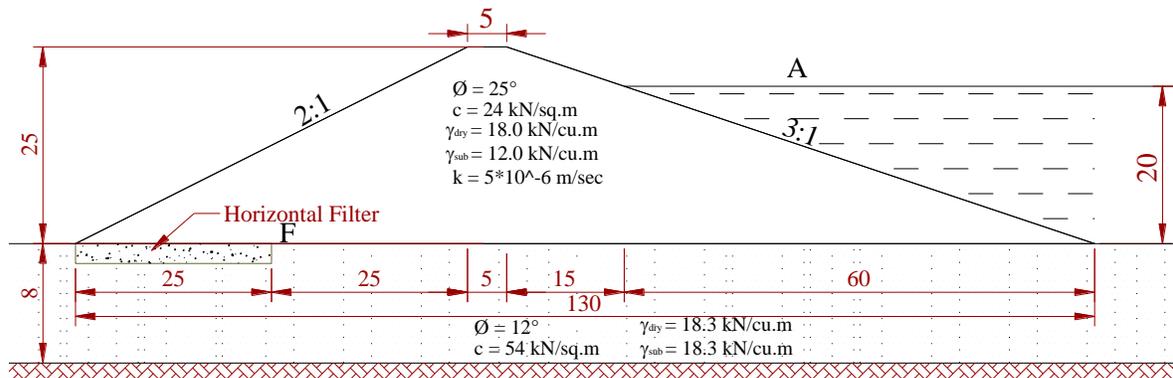


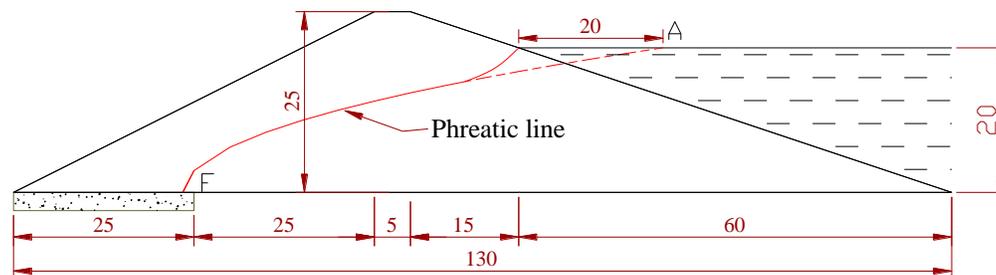
Figure 2-7a Section of a homogeneous earth dam

For the origin of the Cartesian co-ordinate system at the face of the filter (point F), the equation of the parabola of the seepage line can be expressed as:

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

At point A,  $x = 65\text{m}$ , and  $y = 20\text{m}$ . Inserting into the parabola equation,  $S = 3.07\text{m}$ . Working out a few more points from the equation, the parabola can be easily drawn and corrected for the curve at the upstream face of the dam, so as to get the seepage line.

x	-1.51	0	10	15	25	30	40	45	55	65
$y^2$	0	9.06	69.26	99.36	159.56	189.66	249.86	279.96	340.16	400.36
y	0	3.01	8.32	9.97	12.63	13.77	15.81	16.73	18.44	20.01



The amount of seepage flow is

$$\begin{aligned}
 Q &= kS \\
 &= 5 * 10^{-6} * 3.07 \\
 &= 15.35 * 10^{-6} \text{m}^3/\text{sec per meter width of dam}
 \end{aligned}$$

### B. Homogeneous dam section without horizontal filter

The focus (F) of the parabola will be the lowest point of the downstream slope as shown in Figure . The base parabola BIJC will cut the downstream slope at J and extend beyond the dam toe up to the point C i.e. the vertex of the parabola.

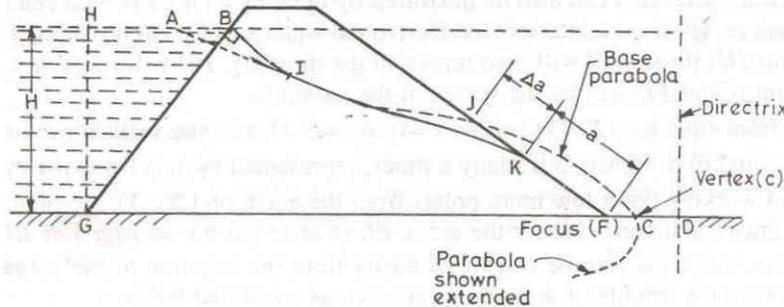


Figure 2.7B Homogeneous dam section without filter

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

$\alpha^\ddagger$ in degrees	$\frac{\Delta a}{a + \Delta a}$
30°	0.36
60°	0.32
90°	0.26
120°	0.18
135°	0.14
150°	0.10
180°	0.0

Where  $\alpha$  is the angle which the discharge face makes with the horizontal.  $a$  and  $\Delta a$  can be connected by the general equation;

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

#### Example

An earth dam made of a homogeneous material has the coefficient of permeability  $K = 5 \times 10^{-4}$  cm/ sec and the other parameters are as shown in the Figure 2-. Determine the phreatic line and the seepage quantity through the body of the dam.

$\ddagger$   $\alpha$  will be equal to 180° for a horizontal filter case and it will be less than 90° when no drainage is provided.

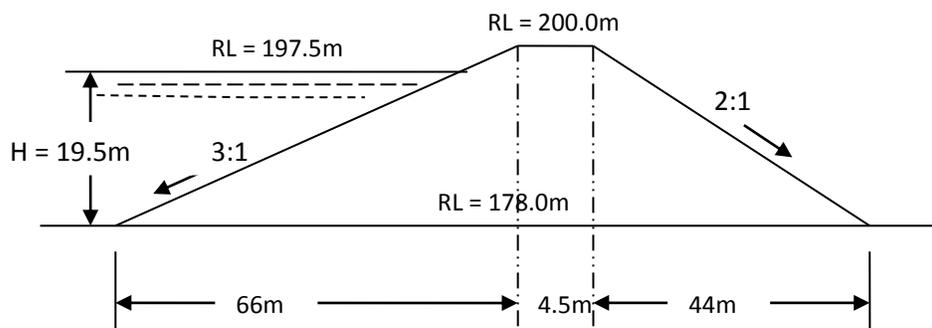


Figure 2-7C Body of homogeneous earth dam

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

Case (a) when  $\alpha < 30^\circ$

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

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

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

$Q = K \frac{dy}{dx} \quad A = K \frac{dy}{dx} y$

Integrating

$Qx = K \frac{y^2}{2} + C$

At pt A At  $x = b, y = H$

$C = Qb - K \frac{H^2}{2}$

$Qx = K \frac{y^2}{2} + Qb - K \frac{H^2}{2}$

$Q = \frac{K}{2(x-b)} (y^2 - H^2)$  (1)

At pt J  $y = a \sin \alpha$

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

$Q = K a \sin \alpha \tan \alpha$  (2)

Equating 1 & 2

$K a \sin \alpha \tan \alpha = \frac{K}{2(x-b)} (y^2 - H^2)$

At J  $y = a \sin \alpha$  &  $x = a \cos \alpha$

Solving for a

$a = \frac{b}{\cos \alpha} - \sqrt{\frac{b^2}{\cos^2 \alpha} - \frac{h^2}{\sin^2 \alpha}}$  (3)

Once a is known from 3, discharge per unit length at JJ' is found by 2,  $Q = k * a * \sin \alpha * \tan \alpha$ .

**Case (b). When  $\alpha$  lies between  $30^\circ$  and  $60^\circ$**

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

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

**Casagrande Solution for a, when  $30^\circ < \alpha < 60^\circ$**

Casagrande suggested that in this case the hydraulic gradient should be taken as  $(dy/dl)$  instead of  $(dy/dx)$ , where l is the distance measured along the phreatic line.

$$i = \frac{dy}{dl}$$

$$Q = K y \frac{dy}{dl}$$

Integrating

$$Q l = K \frac{y^2}{2} + C$$

At pt A At  $l = L, y = H$

$$C = QL - K \frac{H^2}{2}$$

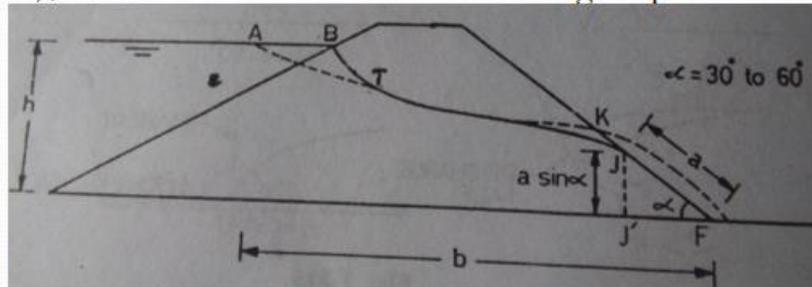
$$Ql = K \frac{y^2}{2} + QL - K \frac{H^2}{2}$$

$$Q = \frac{K}{2(l-L)} (y^2 - H^2)$$

At pt J

$$\frac{dy}{dl} = \sin \alpha$$

$$Q = K a \sin^2 \alpha$$



Equating 1 & 2

$$K a \sin^2 \alpha = \frac{K}{2(l-L)} (y^2 - H^2)$$

At J  $y = a \sin \alpha$  &  $l = a$

For  $\alpha < 60^\circ$ , assuming  $L = \sqrt{b^2 + H^2}$

Solving for a

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

Once a is known from 3, discharge per unit length is found by 2,  $Q = k * a * \sin^2 \alpha$ .

## 2.8 Characteristics of Phreatic line (Seepage line)

Based on the above discussions, the characteristics of the phreatic line may be summarized below:

1. At the entry point, the phreatic line must be normal to the upstream face since the upstream face is a 100% equipotential line. For other entry condition (Fig.2.8), the phreatic line starts tangentially with the water surface.

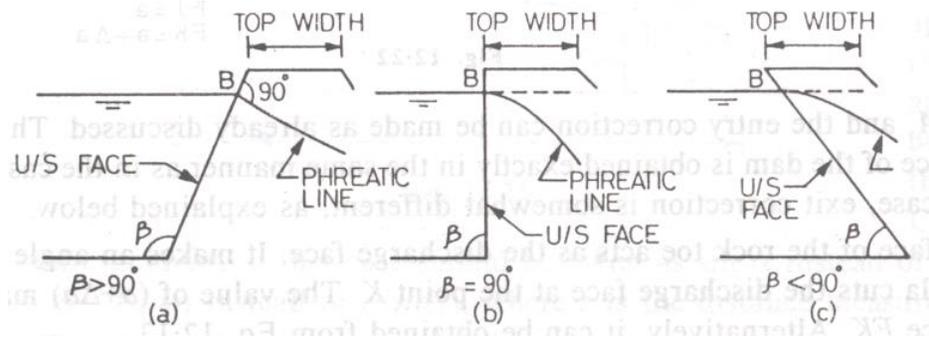


Fig 2.8: Entry conditions of phreatic line

2. The pressure along the phreatic line is atmospheric. Hence the only change in the head along it is due to drop in the elevation of various points on it. Due to this, the successive equipotential lines will meet it at equal vertical intervals.
3. The focus of the base parabola lies at the break out point of the bottom flow line, where the flow emerges out from relatively impervious medium to a highly pervious medium.
4. When horizontal filter or drainage toe is provided, the phreatic line would tend to emerge vertically.
5. In the absence of any filter, the seepage line will cut the downstream slope at some point above the base. The location of this point, and the phreatic line itself, is not dependent on the permeability or any other property, so long as the dam is homogeneous. The geometry of the dam alone decides these.
6. The presence of pervious foundation below the dam does not influence the position of phreatic line.
7. In the case of a zoned dam with central impervious core, the effect of outer shells can be neglected altogether. The focus of the base parabola will be located at the downstream. Toe of the core (Fig. 2.8)

### 2.2.7 Stability analysis

Three considerations govern the design of an earth embankment.

- i. Side slopes must be stable;
- ii. Dimensions must be sufficient to control seepage;
- iii. Base width must be long enough to distribute weight of dam over sufficient area to prevent overstress in the foundation.

An earthen embankment usually fails because of the sliding of a large soil mass along a curved surface.

#### Stability of side slopes of earth dam

*Forms of side slope failure:*

**Toe failure:** - most likely to occur when the slopes are relatively steep or when the soil below the toe of the slope is strong.

**Base failure:** - occurs when the slopes are flat or when the soil below the toe is relatively weak.

**Face or slope failure:** - occurs only when there is a relatively weak zone in the upper part of the slope or when there is a very strong stratum above the toe level.

The method used for examining the stability of slopes of earthen embankments is called the *Swedish Slip Circle Method* or the *Slices Method*. It assumes the condition of plane strain with failure along a cylindrical arc. The location of the Centre of the possible failure arc is assumed. The earth mass is divided into a number of vertical segments called slices as shown in figure 2.6.  $O$  is the center and  $r$  is the radius of the possible failure.

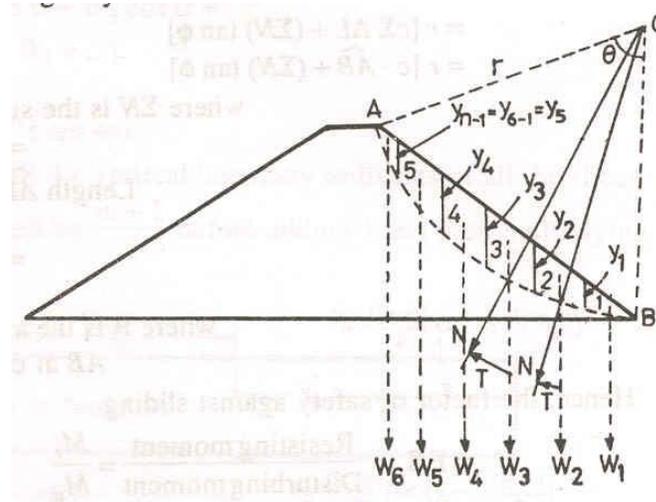


Figure 2-9.1 possible slip surface in Earth fill dam

The side slopes of earth dam will be stable if the soil mass is not dislodged from the slopes. However, the soil mass in an earthen dam is subjected to forces which tend to cause movement or sliding of the soil mass. These forces are known as actuating, driving or disturbing forces which mainly consists of gravity forces. The movement or sliding of the soil mass in an earthen dam is resisted by the resisting or stability forces which are provided mainly by the shearing strength of the soil.

The stability of the side slope of an earthen dam is thus analyzed by assuming a surface slippage within the soil mass and by determining the resisting and the disturbing forces acting on this surface and the moments of these forces about the center of rotation  $n$ , and then factor of safety against sliding is calculated.

The forces acting on the slices are:

1. The self-weight  $W$  of the slice acting vertically downward through the center of gravity.
2. The cohesive forces acting tangentially opposite to the direction of probable slippage

3. The soil reaction across the arc. When the soil mass is about to slide, the soil reaction will act at an angle  $\phi$  (the angle of internal friction of the soil) to the normal i.e. radial direction
4. The soil reaction on the two vertical sides of the slice exercised by the adjacent slices on the right and left respectively.
5. Pore pressures at the base of the arc, and left and right side of the slice.

Usually it is assumed that the soil reactions on the two vertical sides of the slice cancel each other and so also the pore pressures on the two sides balance each other

The disturbing force is the component of weight of slice in tangential direction i.e.,

$$T = W \sin \alpha,$$

Where:  $\alpha$  is the angle which the slope makes with the horizontal.

The total disturbing forces will be summation of disturbing forces for all slices;

$$\sum T = T_1 + T_2 + T_3 + \dots$$

The total disturbing moments over the sliding surface will be equal to

$$M_d = \sum T_i r_i = r \sum T_i$$

The magnitude of shear strength developed in each slice will depend upon the normal components of that slice. Its magnitude will be:

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

Where;  $c$  is the unit cohesion of the soil

$\Delta L$  is curved length of the slice

$\phi$  is the angle of internal friction

$N$  is equal to  $W \cos \alpha$

The total resisting force will be summation of resisting forces for all slices;

$$\begin{aligned} &= \sum c\Delta L + \sum N \tan \phi \\ &= c \sum \Delta L + (\sum N) \tan \phi \end{aligned}$$

The total resisting moment over the entire sliding surface will be equal to

$$M_r = r(c \sum \Delta L + (\sum N) \tan \phi)$$

Hence the factor of safety against sliding

$$FS = \frac{M_r}{M_d} = \frac{c \sum \Delta L + \tan \phi \sum N}{\sum T}$$

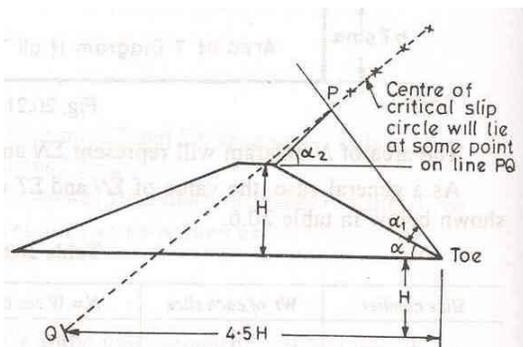
For determining the stability of the proposed side slope of an earth dam it is necessary to find the least factor of safety which may occur on any of the possible surfaces of slippage or slip circles. Slip circle which yields the least factor of safety is the most critical and hence it is known as critical surface slippage or critical slip circle. For locating the critical surface of slippage, it is necessary to try several different surfaces of slippage as one trial gives the value of factor of safety for that arc only.

For preliminary analysis 4 to 5 slices may be sufficient; however, 10 to 15 slices are considered in general. It is not necessary for the analysis to make all the slices of equal width, but for the sake of convenience it is customary to have slices of equal width.

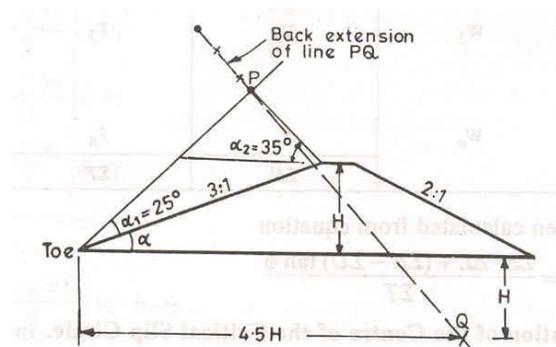
In order to reduce the number of trials, Fellenius has suggested a method of drawing a line, representing the locus of the critical slip circle. The determination of this line PQ is shown in Figure . The point P is obtained with the help of directional angles  $\alpha_1$  and  $\alpha_2$  as shown in **Error! Reference source not found.**

Table 2-4 Slope and respective directional angle

Slope	Directional angles	
	$\alpha_1$ in degrees	$\alpha_2$ in degrees
1:1	27.5	37
2:1	25	35
3:1	25	35
4:1	25	35
5:1	25	35



a. downstream slope



b. upstream slope

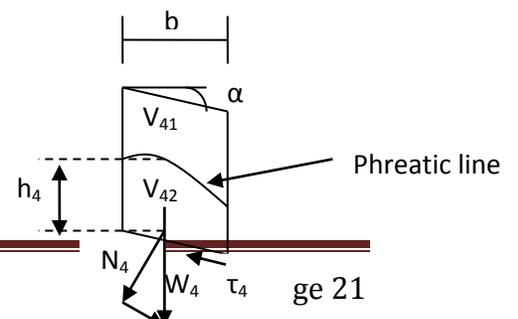
Figure 2.9.2 locus of critical circle

Design parameters to be employed in stability analysis may be summarized as follows:

**a. Stability of downstream slope during steady seepage (reservoir full)**

The most critical condition for the d/s slope occurs when the reservoir is full and the seepage is taking place at full rate.

The seepage water below the phreatic line exerts a pore pressure on the soil mass which lies below the phreatic line, see Figure 2.9.3.



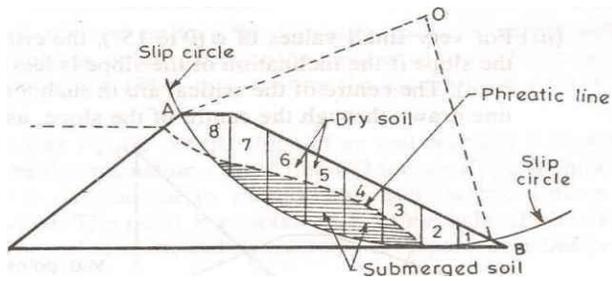


Figure 2.9.3 stability of downstream slope during steady seepage

Consider slice number 4 in Figure , the weight of the slice is defined as;

$$W_4 = \gamma_{dry} * V_{41} + \gamma_{sat} * V_{42}$$

$\alpha_4$  read from the scaled drawing of the earth fill dam. And tangential component of  $W_4$  is defined as, which is shear stress developed at failure plane,

$$T_4 = W_4 \sin \alpha_4$$

The pore pressure for slice 4 is represented by the piezometric head  $h_4$ . Hence pore water pressure is

$$U_{w4} = \gamma_w h_4$$

Shear strength developed for the slice is quantified from two soil parameters, apparent cohesion  $c$  and angle of shearing resistance  $\phi$ . Shear strength at failure plane is defined as

$$\tau_4 = cL_4 + (W_4 \cos \alpha_4 - \gamma_w h_4 L_4) \tan \phi$$

$$\text{Where; } L_4 \text{ is } \frac{b}{\cos \alpha_4}$$

The factor of safety of slide 4 is

$$FS_4 = \frac{T_4}{\tau_4} = \frac{cL_4 + (W_4 \cos \alpha_4 - \gamma_w h_4 L_4) \tan \phi}{W_4 \sin \alpha_4}$$

The factor of safety for the entire circle is then given by the equation

$$FS = \frac{\sum c_i L_i + \sum (W_i \cos \alpha_i - \gamma_w h_i L_i) \tan \phi}{\sum W_i \sin \alpha_i}$$

### ***b. Stability of Upstream Slope during sudden drawdown***

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

The tangential components of the saturated soil lying over the arc will create a disturbing force; while the normal component minus the pore pressure shall supply the shear strength of the soil.

Table 2-5: General format of computation

Slice#	W	T	N	U	I	UI	N'=N-ul	tanΦ	N'tanΦ	Cl	N'tanΦ+cl
1											
...											
n											
Σ T										Σ (N'tanΦ+cl)	

The factor of safety is finally obtained from the equation

$$FS = \frac{\sum c_i L_i + \sum N' \tan \phi}{\sum T'}$$

Where:  $N'$  represents normal components on submerged density

$T'$  represents tangential components on saturated unit weight of the soil

### 2.9.1 Stability of earth dam against horizontal shear developed at the base of the dam

*Approximate method for checking the stability of u/s and d/s slopes under steady seepage from consideration of horizontal shear at base*

#### 1. Stability of u/s slope during sudden drawdown

It is based on the simple principle that a horizontal shear force  $P_u$  is exerted by the saturated soil. The resistance to this force  $R_u$  is provided by the shear resistance developed at the base of the soil mass, contained within the u/s triangular shoulder GMN of Fig.

Considering unit length of the dam, the horizontal force  $P_u$  is

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

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

Where;

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

$$R_u = cB_u + W \tan \phi = cB_u + \left( \gamma_{sub} \frac{1}{2} B_u h \right) \tan \phi$$

Where;  $W$  is the weight of the u/s triangular shoulder of the dam

The factor of safety against can be easily calculated, using

$$FS = \frac{R_u}{P_u}$$

It should be more than 1.5.

#### 2. Stability of d/s slope under steady seepage

It is based on the consideration of horizontal shear at base under the d/s slope of the dam. The horizontal shear force  $P_d$  is given by, referring Fig;

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

Where;

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

Shear resistance  $R_d$  of d/s slope portion of the dam is given by;

$$R_d = cB_d + W \tan \phi = cB_u + (\gamma_{dry} A_1 + \gamma_{sub} A_2) \tan \phi$$

The downstream profile RTS of the downstream slope portion of the dam has an area  $A_1$  and of dry soil above seepage line and the area of submerged soil say  $A_2$  below the seepage line.

The entire weight  $W$  may be calculated on the basis of submerged soil as it will be on a still safer side. In that case,

$$W = \gamma_{sub} B_d h$$

The factor of safety against shear can be easily determined as;

$$FS = \frac{R_d}{P_d}$$

### 2.10 Foundation analysis

Foundation stress in earth dams are not usually critical except when the foundation material consists of unconsolidated clay or silt with low shearing strength.

Consider a dam on homogeneous, unconsolidated earth foundation of thickness  $t$

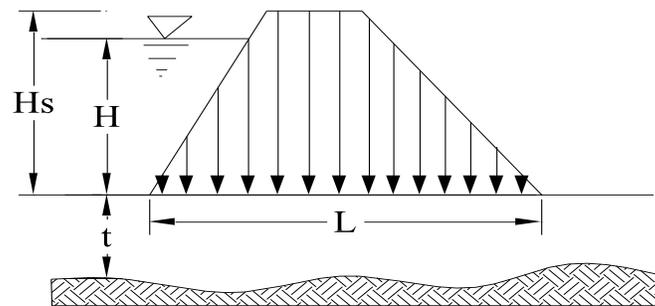


Figure 2.9.4 Homogenous embankment dam with pervious foundation of thickness  $t$

The downward force exerted on the foundation at the center of the dam tends to squeeze the foundation material from under the dam. But shear stress develops in the foundation resisting this action. Assuming the foundation loading to vary as indicated above, Leo Jugenson suggested the following maximum stresses:

$$\text{If } t > L, \tau_{\max} = 0.256 \gamma_f H_s \quad ; \quad \text{Where } \gamma_f = \text{specific weight of fill}$$

Usually  $t < L$

$$\text{If } t < L/10, \tau_{\max} = \gamma_f H_s t/L$$

$$\text{Shear strength} = S_s = c + \sigma \tan\Phi$$

The factor of safety against overstress is

$$FS = S_s / \tau_{\max}$$

A minimum value of  $FS = 1.5$  is recommended.

### Example:

Design the embankment dam shown in Figure 2- used as an example for analysis of seepage flow. Detail all the necessary procedures and important consideration in the process.

### Design Solution

The stability design process starts by determining the phreatic line profile which is done before. The critical slipage circle is then drawn by following the suggestion made by Fellinus. Here a single slipage circle is considered for illustration and four slices were considered for both upstream and downstream slope failure. The geometric information's were then determined as shown below.

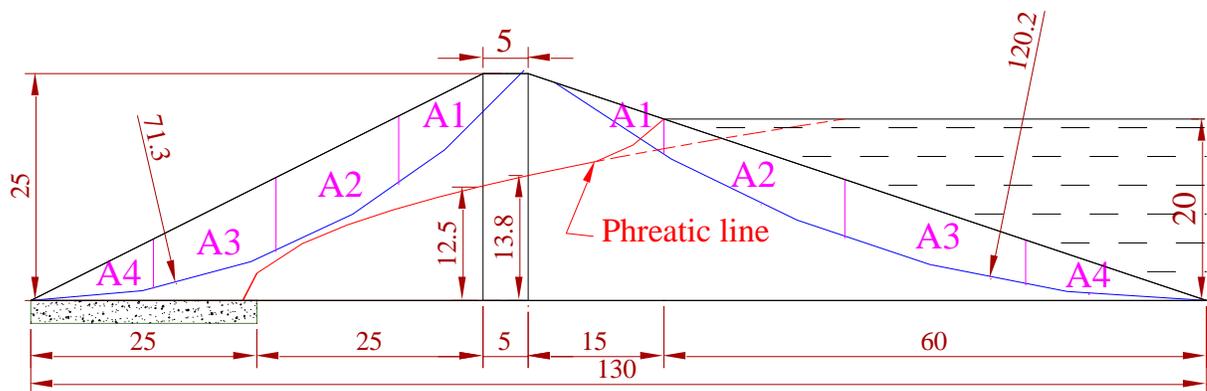


Fig. Sample of failure circle, slices and related measurements of the earth dam section

Geometric properties of slices

Slice	Upstream				Downstream		
	Area(m <sup>2</sup> )	$\alpha$ (deg)	$h_w$ (m)	L (m)	Area (m <sup>2</sup> )	$\alpha$ (deg)	L (m)
A1	25.97	32	0	14.5	68.58	40	18
A2	111.68	24	6.1	22	111.48	28	15.4
A3	120.83	15	6.2	20.7	96.1	17	14.2
A4	55.52	4	3.6	20.1	40.11	7	13.7

Area in relation with phreatic line.

Area (m <sup>2</sup> )	Dam	U/s shoulder	D/s shoulder
Under seepage line	1102.08	838.1	221

(saturated)			
Dry portion	584.92	99.4	404
Total	1687	937.5	625

To assess the overall stability of the dam considering 1m length,

Item	Dam		U/s shoulder		D/s shoulder	
	Area(m <sup>2</sup> )	Weight(kN)	Area(m <sup>2</sup> )	Weight(kN)	Area(m <sup>2</sup> )	Weight(kN)
Under seepage line	1102.08	13224.96	838.1	10057.2	221	2652
Dry portion	584.92	10528.56	99.4	1789.2	404	7272
Total	1687	23753.52	937.5	11846.4	625	9924

The stability design then proceeds by first considering the entire embankment and its interaction with the foundation.

**Shear resistance** of the dam at the base(R)

$$R = C + W \tan\Phi$$

Where: C = total cohesive resistance of the soil at the base

$$= c \cdot B \cdot 1 = 24 \cdot 130 \cdot 1$$

$$= 3120 \text{ kN}$$

$$W \tan\phi = 23753 \cdot \tan 25^\circ$$

$$= 11076 \text{ kN}$$

$$R = 3120 + 11076$$

$$= 14196 \text{ kN}$$

Horizontal force due to hydrostatic pressure of water

$$P = \frac{1}{2} \gamma_w h^2 = \frac{1}{2} \cdot 10 \cdot 20^2$$

$$= 2000 \text{ kN}$$

Factor of safety against failure due to horizontal shear at the base

$$FS = R/P = 7.1 > 1.5$$

Safe!

Check **stresses** in the foundation

$$t = 8 \text{ m} < L/10 = 130/10 = 13,$$

Hence,

$$\tau_{\max} = \gamma_f H_s t/L$$

$$= 18.3 \cdot 20 \cdot 8/130$$

$$= 22.52 \text{ kN/m}^2$$

$$\text{Shear strength} = S_s = c + \sigma \tan\Phi$$

$$= c + W/L \tan\phi$$

$$= 54 + 23754/130 \cdot \tan 12^\circ$$

$$= 92.8 \text{ kN/m}^2$$

The factor of safety against overstress is

$$FS = S_s / \tau_{\max} = 92.8/22.52 = 4.12 > 1.5$$

Safe!

*Stability of u/s and d/s slopes against sliding shear.*

Upstream slope (under sudden drawdown):

Considering unit length of the dam, the horizontal force  $P_u$  is

$$\begin{aligned}
 \gamma_1 &= \frac{\gamma_{sub} h_1 + \gamma_{dry} (h - h_1)}{h} \\
 &= \frac{12 * 13.8 + 18(25 - 13.8)}{25} \\
 &= 14.7 \\
 P_u &= \left[ \frac{\gamma_1 h^2}{2} \tan^2 \left( 45 - \frac{\phi}{2} \right) + \gamma_w \frac{h_1^2}{2} \right] \\
 &= \left[ \frac{14.7 * 25^2}{2} \tan^2 \left( 45 - \frac{25}{2} \right) + 10 \frac{13.8^2}{2} \right] \\
 &= 2816.6
 \end{aligned}$$

Shear resistance  $R_u$  of upstream slope portion of the dam developed at base GN is given by,

$$R_u = cB_u + W \tan \phi = 54 * 75 + 11846.4 * 0.47 = 9574.1$$

Where;  $W$  is the weight of the upstream triangular shoulder of the dam.

The factor of safety against shear can be easily calculated,

$$FS = \frac{R_u}{P_u} = \frac{9574.1}{2816.6} = 3.4 > 1.5 \quad \text{Safe!}$$

It has been known that the maximum intensity of shear stress occurs at a distance  $0.6B_u$  (where  $B_u$  is the base length of the upstream shoulder) from the heel and is equal to 1.4 times the average shear intensity.

$$\begin{aligned}
 \text{Hence, maximum shear stress induced } (\tau_{max}) &= 1.4(P_u/B_u) \\
 &= 1.4 (2816.6/75) \\
 &= 52.6
 \end{aligned}$$

The unit shear resistance developed at the same point is

$$\begin{aligned}
 \tau_f &= c + 0.6h\gamma_{sub}\tan\phi \\
 &= 24 + 0.6 * 25 * 12 * \tan 25^\circ \\
 &= 107.9
 \end{aligned}$$

FS at the point of maximum shear should be greater than unity.

$$FS = \tau_f / \tau_{max} = 107.9 / 52.6 = 2.0 > 1 \quad \text{Safe!}$$

For the downstream shoulder, similarly,

$$\begin{aligned}
 \gamma_1 &= \frac{12 * 12.5 + 18(25 - 12.5)}{25} = 15 \\
 P_u &= \left[ \frac{15 * 25^2}{2} \tan^2 \left( 45 - \frac{25}{2} \right) + 10 \frac{12.5^2}{2} \right] = 2683.7 \\
 R_u &= cB_u + W \tan \phi = 54 * 60 + 9924 * 0.47 = 7904.3
 \end{aligned}$$

$$FS = \frac{R_u}{P_u} = \frac{7904.3}{2683.7} = 2.9 > 1.5$$

Safe!

$$\begin{aligned}\text{Maximum shear stress induced } (\tau_{\max}) &= 1.4(P_u/B_d) \\ &= 1.4 (2683.7/50) \\ &= 75.1\end{aligned}$$

The unit shear resistance developed at the same point is

$$\begin{aligned}\tau_f &= c + 0.6h\gamma_{\text{sub}}\tan\phi \\ &= 24 + 0.6 * 25 * 12 * \tan 25^\circ \\ &= 107.9\end{aligned}$$

FS at the point of maximum shear should be greater than unity.

$$FS = \tau_f / \tau_{\max} = 107.9 / 75.1 = 1.44 > 1$$

Safe!

Analysis of upstream and downstream slopes by Swedish Circle method:

Upstream slope

Slice	Area	□	W	T	N	U	L	UL	N'=N-ul	tanΦ	N'tanΦ	Cl	N'tanΦ+cl
A1	25.97	32	467.46	247.72	396.43	0	14.5	0	396.43	0.47	186.32	348	534.32
A2	111.68	24	1340.16	545.09	1224.3	61	22	1342	-117.7	0.47	-55.32	528	472.68
A3	120.83	15	1449.96	375.28	1400.55	62	20.7	1283.4	117.15	0.47	55.06	496.8	551.861
A4	55.52	4	666.24	46.47	664.62	36	20.1	723.6	-58.98	0.47	-27.72	482.4	454.68
			Σ T	1214.56								Σ (N'tanΦ+cl)	2013.54

$$FS = \frac{\sum c_i L_i + \sum N' \tan \phi}{\sum T'} = \frac{2013.54}{1214.56} = 1.66 > 1.5$$

Safe!

Downstream slope

Slice	Area	□	W	T	N	l	N'=N-ul	tanΦ	N'tanΦ	Cl	N'tanΦ+cl		
A1	68.58	40	1234.44	793.48	945.64	18	945.64	0.47	444.45	432	876.45		
A2	111.48	28	2006.64	942.06	1771.76	15.4	1771.76	0.47	832.73	369.6	1202.33		
A3	96.1	17	1729.8	505.74	1654.22	14.2	1654.22	0.47	777.48	340.8	1118.28		
A4	40.11	7	721.98	87.99	716.6	13.7	716.6	0.47	336.8	328.8	665.6		
			2329.27									Σ (N'tanΦ+cl)	3862.66

$$FS = \frac{3862.66}{2329.27} = 1.66 > 1.5$$

Safe!

### 2.11 Internal drainage system

#### General

Purpose of drainage:

1. To reduce the pore pressure thereby increasing the stability of the dam
2. To prevent piping so that soil particles are not carried away from the embankment.

A drainage system consists of two components.

- a. the protective filter
- b. the conduit which collects and disposes of the seepage.

Protective filter: - serves to allow free drainage and to prevent erosion.

It is provided between

Riprap and embankment

Core and embankment

#### Embankment and drains

Experiments by Terzaghi, Bertram and others have shown that a filter need only hold the coarse 15% of the grain size. These coarse particles  $D_{85}$  and over, will collect over the filter opening bridging over it and trapping finer particles.

Size of filter holes,  $D_f \leq D_{85}$  ( of the soil being filtered)

From tests, the following criteria are established.

$D_{15}(\text{filter}) \leq 4 \text{ to } 5 D_{85}(\text{soil})$ ; to satisfy prevention of migration of soil particles.

$D_{15}(\text{filter}) \geq 4 \text{ to } 5 D_{15}(\text{soil})$ ; for free drainage.

Filter gradation curve should be smooth and parallel to the soil being filtered. When the soil is gap graded recompute and re-plot the grain size distribution using only the fraction finer than the break as representing the entire soil; apply the filter criteria to this distribution

#### Filter thickness:

Thin filter is desirable to minimize flow resistance. Practical considerations, however, put minimum sizes as shown in Table-2.6.

Table-2.6: Representative mean filter thickness

Filter	Thickness for given head, cm		
	0 – 25 m	25 – 50 m	50 – 100 m
Fine sand	15	30	45
Coarse sand	25.5	45	60
Gravel	30	60	75

For every fine grained soil a multiple layered filter is necessary. Each successive layer is designed to fit the  $D_{15}$  and  $D_{85}$  of the finer layer it must filter. The last layer must fit the openings of the drain conduit which carries the water away.

Example:

Determine the size of the bed material for the embankment dam with the grain size shown below.

Grain size	Riprap	Dam
D <sub>15</sub>	70 mm	0.3 mm
D <sub>85</sub>	130 mm	2.0 mm

### Drain Conduit:

Function: to collect water from the filter and carry it away with as little head as possible.

Quantity of Flow: Estimated from the seepage analysis. A factor of safety of 5 is not uncommon.

Simplest conduit: uniform coarse fragmental material (coarse sand, gravel, crushed rock, etc.)

A properly designed filter must surround the drain. For high discharges or when suitable crushed rock is not available, pipe conduits with perforated and flexible joints are employed.

### Types of drain

*Trench drain:*

Trench drain is used for intercepting seepage through homogeneous foundations and those containing horizontal pervious strata or seams

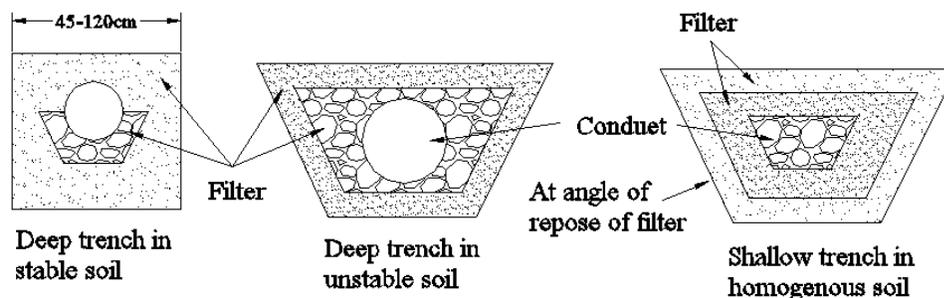


Figure 2-11.1 Trench Drain

*Mound drain:* used when the need for embankment drainage exceeds that of the foundation.

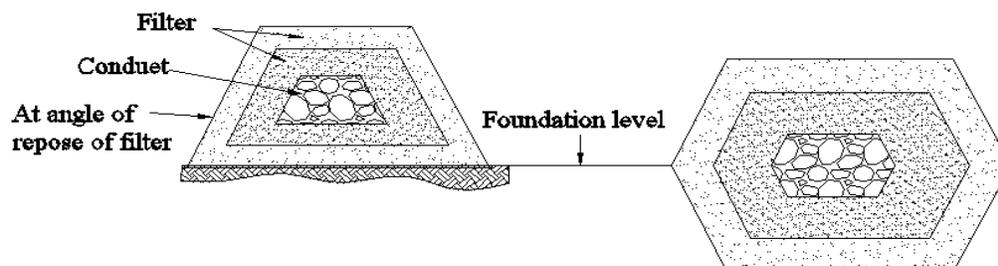


Figure 2.11.2 Mound drain

Position- when there is cut-off, the drain is placed immediately downstream from the cut-off to relieve any pressure build up.

When there is no cut-off, the position depends on the seepage analysis of dam and foundation.

Upstream location

Increases stability at the cost of increased seepage and cost of conduit.

Minimum distance from downstream toe to provide substantial increase in stability is 1/3 of the base width. Maximum distance is 2/3 of base width, from downstream toe.

*Blanket drain:*

This is horizontal drain placed on top of foundation.

To intercept water from vertical fissures in the foundation;

To lower the seepage line in the embankment

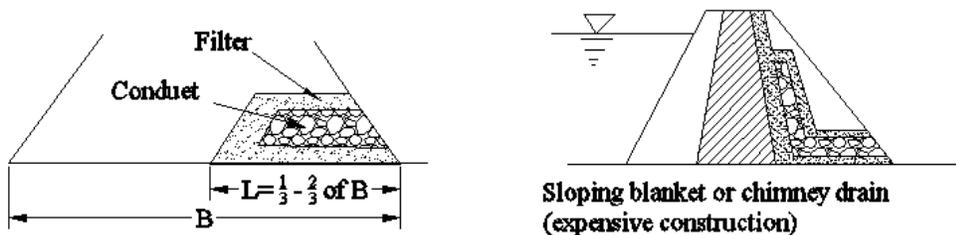


Figure 2.11.3 Blanket drain

*Riprap:*

Riprap is required on the upstream slope and the downstream slope below the tail water level. An estimation of the required weight of rock pieces required for riprap is given by Hudson as

$$W \geq (H^2 \rho_{st} \tan \alpha) / (3.2 \Delta^2)$$

Where: the factor 3.2 is for smooth quarry stone

$\rho_{st}$  = density of rock

$\alpha$  = angle of slope

$\Delta = (\rho_{st} - \rho_w) / \rho_w$

$\rho_w$  = density of water

Types of riprap

Dumped riprap: consists of angular broken rock dumped from truck and spread.

Hand-placed riprap: consists of more or less prismatic stone placed on end to form rough pavements.

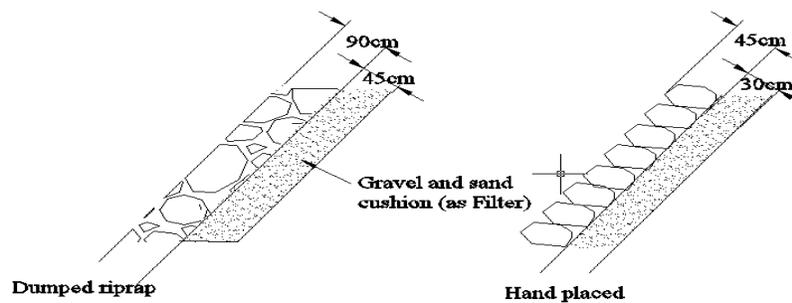


Figure 2.11.4 Types of riprap

### 2.3. River Diversion

River diversion is a technique of diverting river water away from downstream part into the canal or tunnel or to particular confined side.

River diversion takes place for two purposes:

- For construction purpose
- For water use purpose

#### *River Diversion construction purpose*

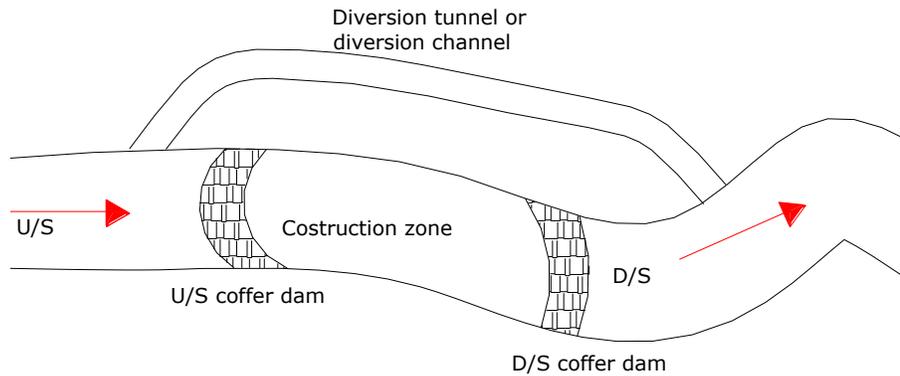
During construction of gravity or embankment dam, concreting of dam or placing of earthen material cannot be done in water. Therefore, before starting construction of major dam, the river water has to be diverted away from the dam site so that the construction can be done in dry condition.

**Coffer Dam:** usually earthen embankment (for economic purpose) is constructed on the U/S and D/S of the dam site to isolate the construction area which is to be kept dry.

- ☞ The diversion tunnel or canal takes off U/S of the coffer dam on the U/S and then joins on the D/S of the coffer dam in the D/S side.
- ☞ Some water usually seeps into the construction area, which has to be pumped out to keep the area dry.

The diversion of river water can be accomplished in either of the following ways:

#### **I. Provision of diversion tunnel or Channel**

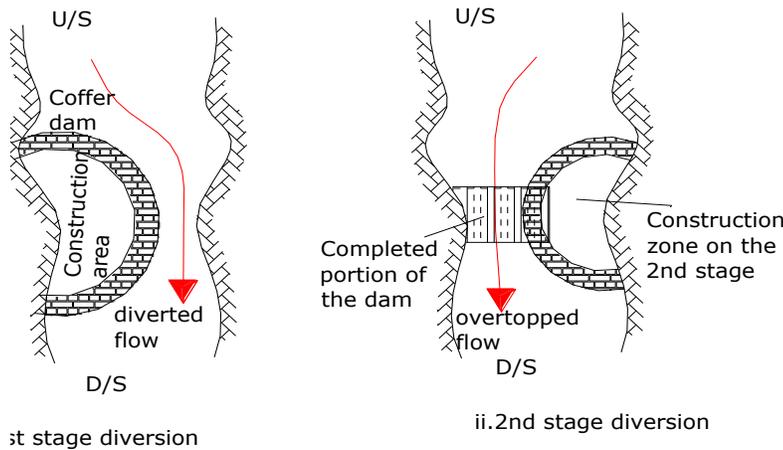


Used during construction of both concrete & embankment dams

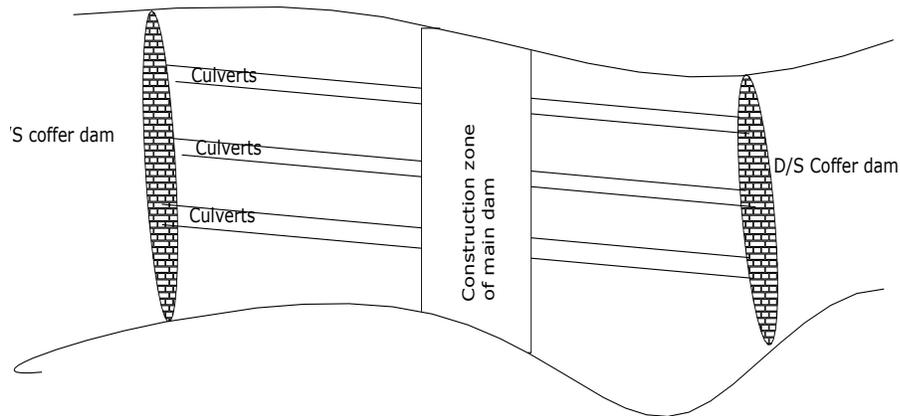
**II. Two stage construction**

Instead of diverting the river into a tunnel or channel, it is some times more economical to have two stage construction of gravity dam. In such a case, the flow is, first of all, diverted and confined to one side of the channel by constructing semi circular coffer dam. Then construction progresses in free water zone. On the next stage the diverted water is allowed to overtop or to pass through outlet on the side of already constructed portion of the dam.

This method is suitable only during construction of concrete dam.



**III. River diversion through the construction site** (through culverts in the body of the dam) Example: Gilgel Gibe Hydroelectric project



### *Choice of design flood*

Flood occurs when the flow depth  $h$  is larger than the limit depth  $h_L$  and the corresponding limit discharge  $Q_L$  can be predicted. If the limit discharge  $Q_L$  has a return period of  $n$  years, the probability of flooding within one year is

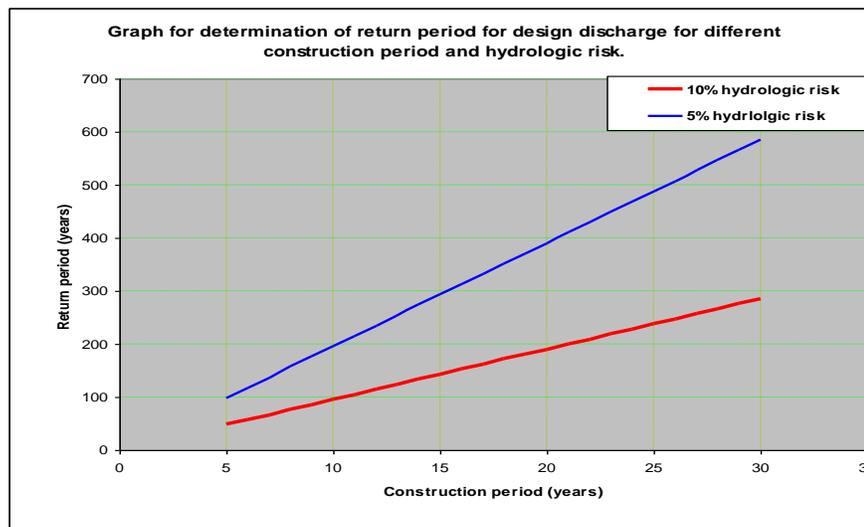
$$P_1 = n^{-1}$$

And for  $m$  years of construction

$$P_m = 1 - (1 - n^{-1})^m$$

This probability is also referred to as the hydrologic risk of the construction site. For  $n \gg m$ ,  $P_m = m/n$ . For instance if the risk of flooding is limited say to 10%, the design flood has a return period of  $n = 10m$ . Accordingly, the design discharge  $Q_D = Q_L = Q_n$  where  $n = 10m$ .

Generally for different construction period and hydrologic risk of 5% & 10% the value of return period for the design discharge is determined from the following graph.



### *Diversion by culvert/tunnel and coffer dam*

Tunnels are usually confined to steep side rock valleys. They have the advantage of not interfering with foundation excavation and dam construction work. Culvert under embankment dams may be preferred in flat valleys or where the rock is too poor for efficient tunneling, but tunneling appears to be the most frequent choice.

River diversion through tunnels is the commonest practice in the construction of high head dams. An economic and engineering study of coffer dam height versus tunnel size may be done to establish the most economic combination for the maximum diversion discharge. It is also possible to plot a chart to see the tunnel –to-coffer dam cost alternative and to select the most economical combination.