GRAVITY& EARTHEN DAMS:

- Forces acting on a gravity dam,
- causes of failure of a gravity dam,
- elementary profile and practical profile of a gravity dam,
- limiting height of a low gravity dam,
- stability analysis, drainage galleries.

- Types of Earth dams,
- causes of failure of earth dam,
- criteria for safe design of earth dam,
- seepage through earth dam-graphical method,
- measures for control of seepage.

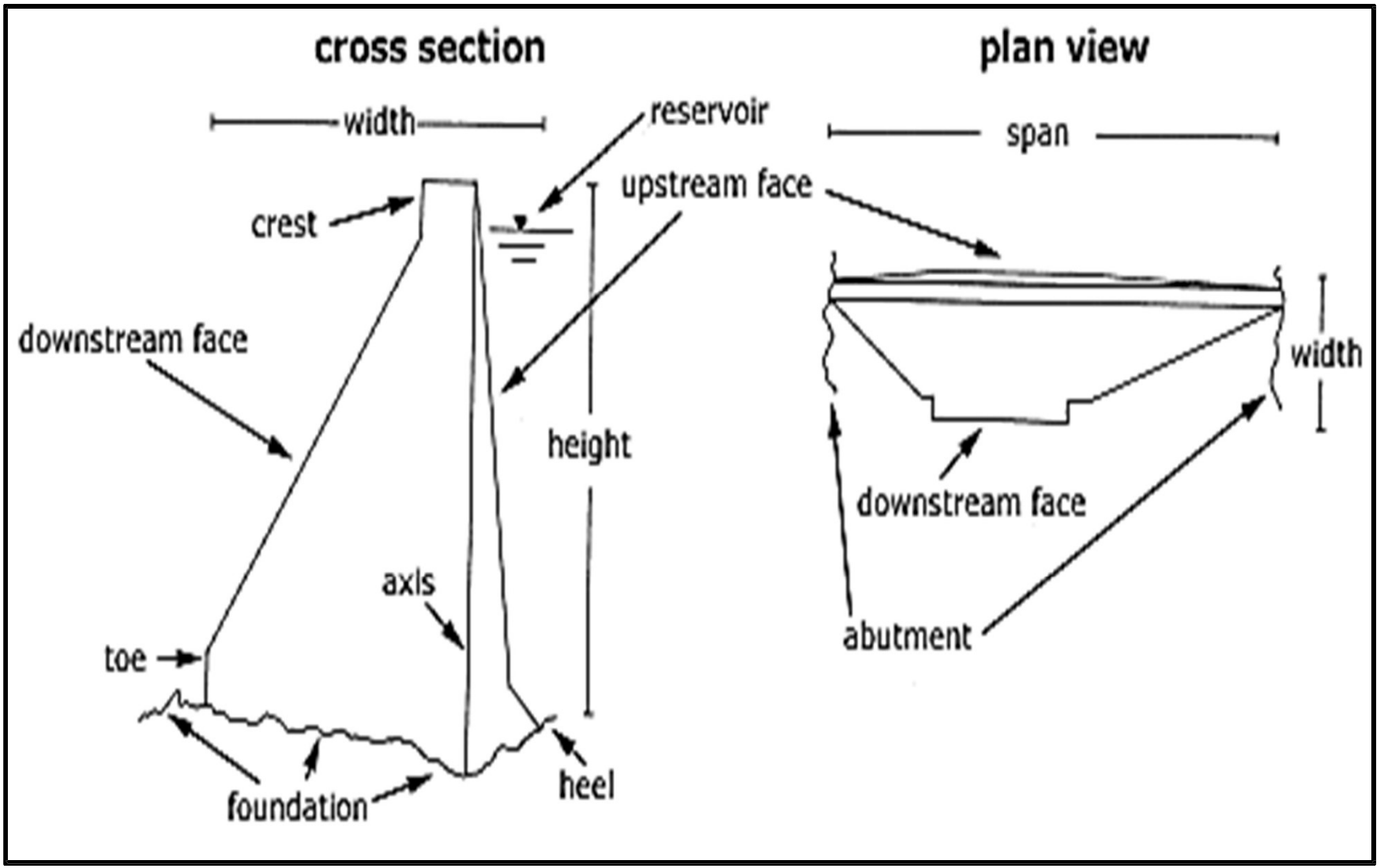
GRAVITY DAMS

Contents:

- **Gravity Dam**
- **Cross Section**
- **Advantages & Disadvantages**
- **Forces Acting on Gravity Dam**

GRAVITY DAMS

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


cross section
plan view
width
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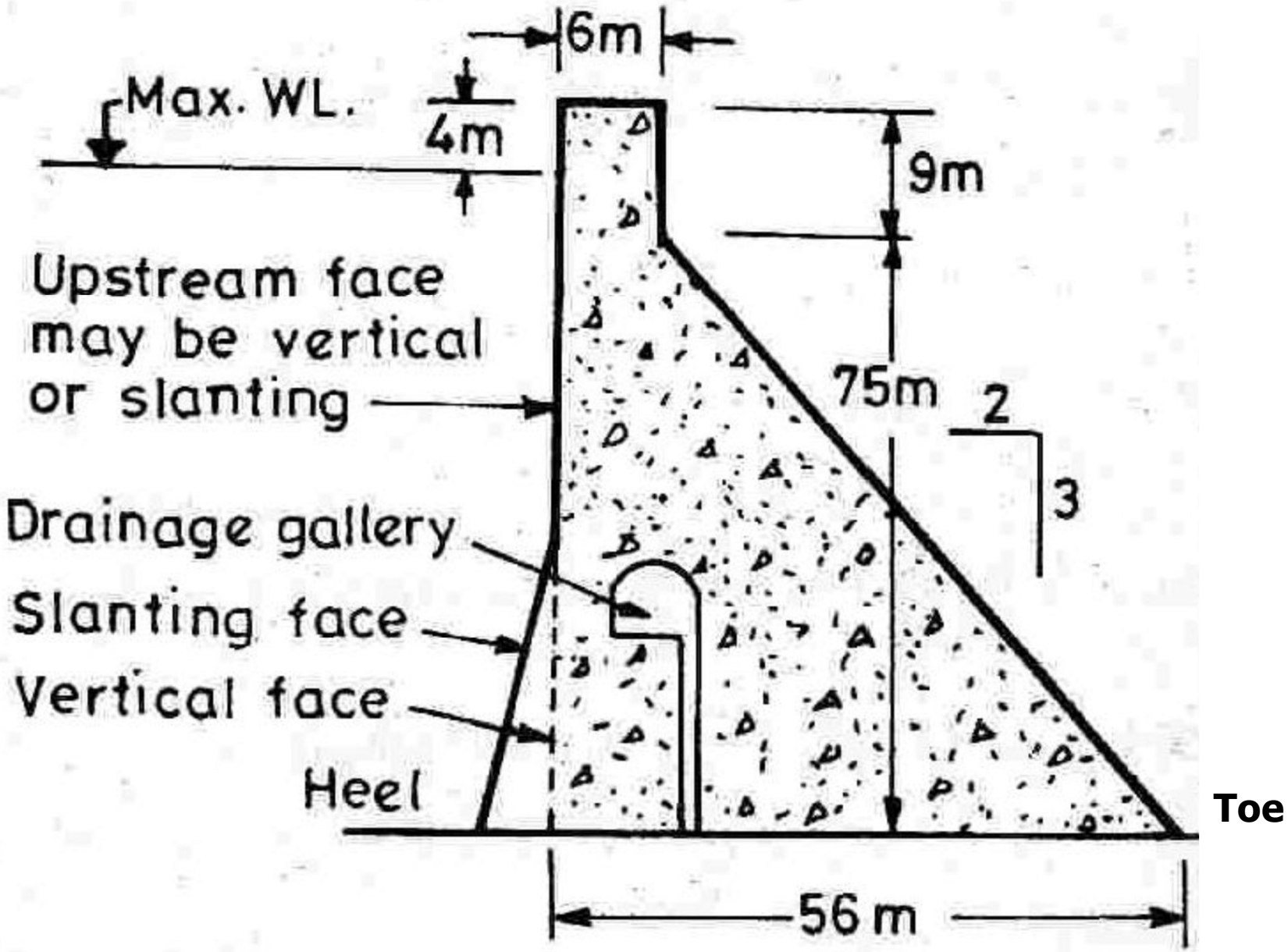
Typical Cross section of Gravity Dam



GRAVITY DAM

- A structure which is designed in a such away that its own weight resists the external forces.
- Advantages of gravity dam:

- Durable and solid.
- Requires little maintenance.
- Constructed of masonry or concrete.
- Constructed on strong natural foundation.
- Straight or slightly curve in shape.

- Strong, stable and durable.
- Suitable across moderately wide valleys and gorges having steep slopes where earth dams, if constructed, might slip.
- Constructed to very great heights.
- Provided good rock foundations are available.
- Well adapted for use as an overflow spillway section where earth dams cannot be used as an overflow section.

Disadvantages

- Great height can be constructed only on sound rock foundations.
- Initial cost of a gravity dam is usually more than that of an earth dam.
- Take a longer time in construction especially when mechanized plants for batching, mixing and transporting concrete are not available.
- Require more skilled labor.

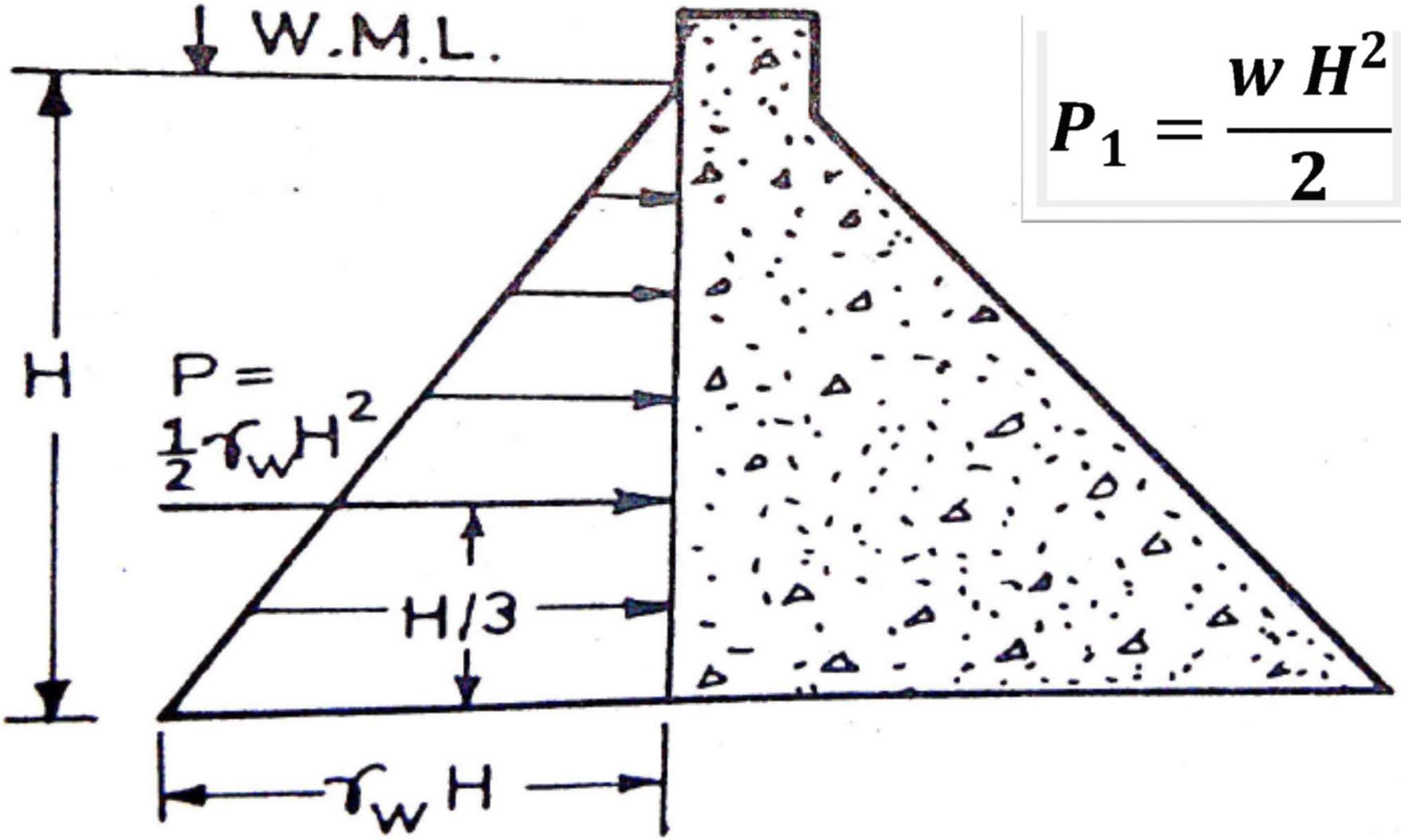
Forces acting on Gravity Dam

1) Water Pressure
2) Self Weight of the Dam
3) Uplift Pressure
4) Silt Pressure
5) Wave Pressure
6) Ice Pressure
7) Earthquake Pressure
8) Wind Pressure

1) Water Pressure

- It is the major external force acting on a dam.
- The water pressure on the upstream face depends on the water surface level in the reservoir and acts horizontally. In case the dam has a batter in the upstream side, the load of water over the batter is also present and acts vertically.

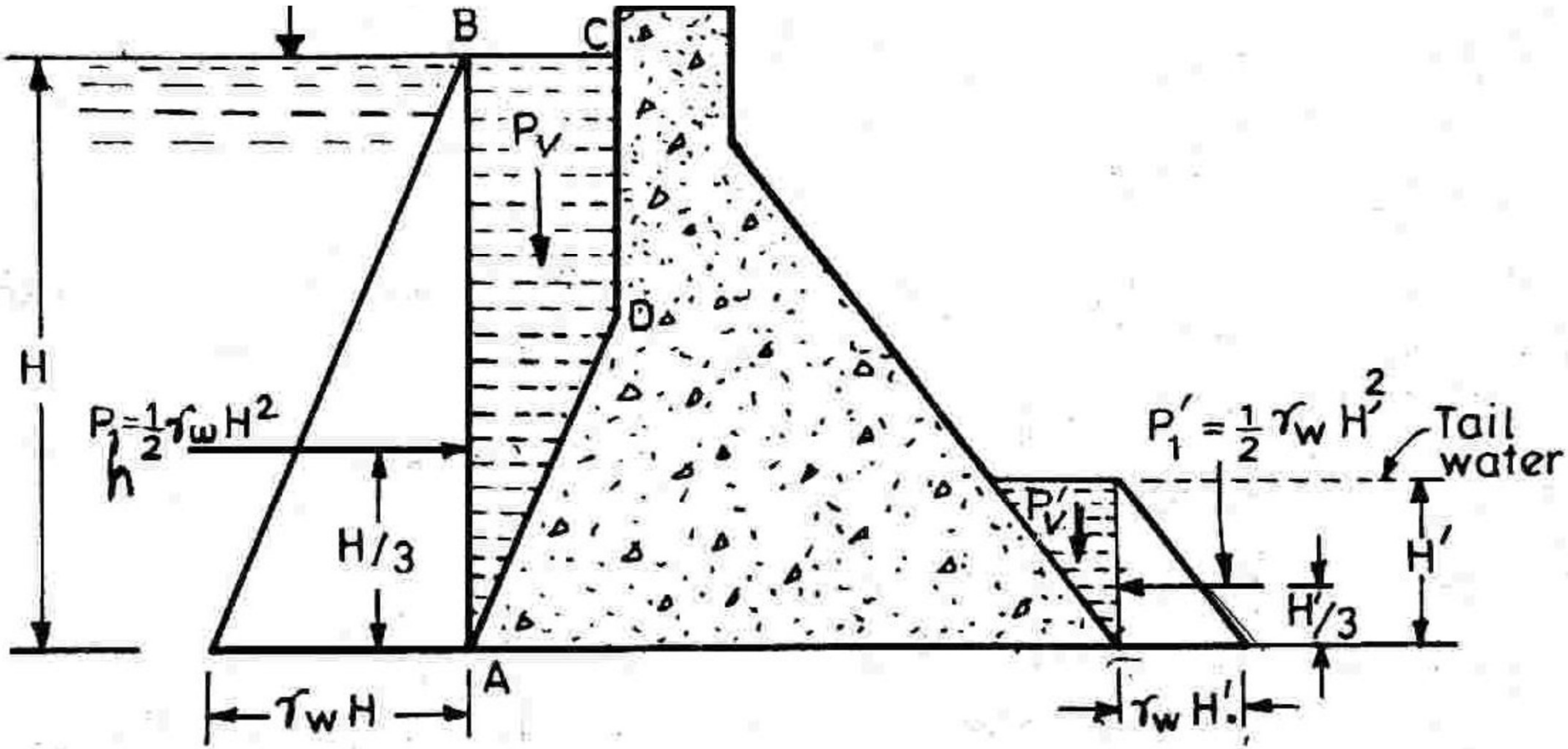
When U/S face is exactly vertical



$$P_1 = \frac{w\,H^2}{2}$$

- **The horizontal water pressure acts at a height of H/3 from base of the dam.** **Unit weight of water, γw = 1000 kg/m3 =** 9.807 kN/m^3

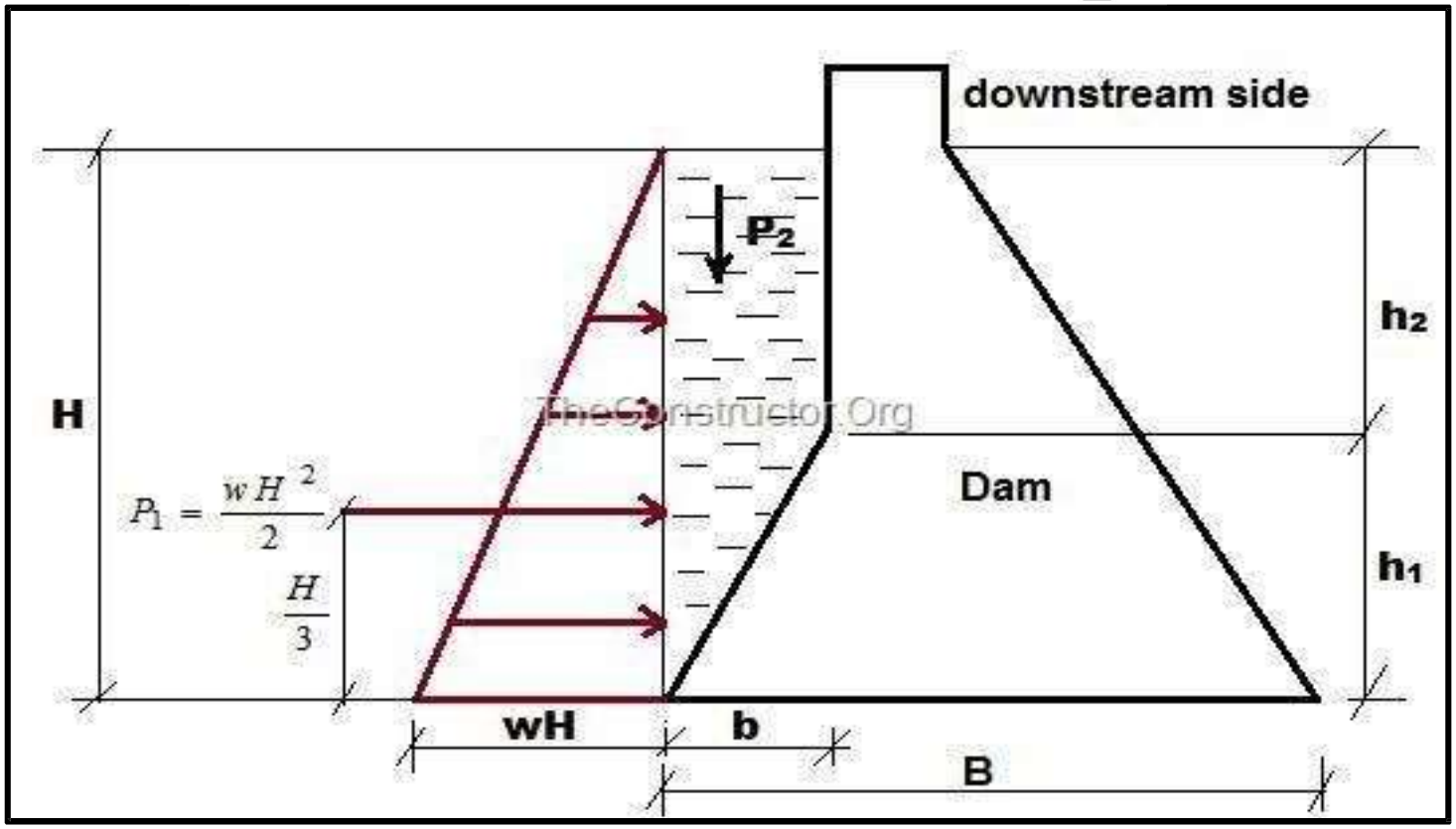
When U/S face is partly vertical and partly inclined.



**Tail Water Pressure: Similarly, the water pressure on the downstream face is due to the tail water and acts horizontally while the weight of water on the downstream face acts vertically.

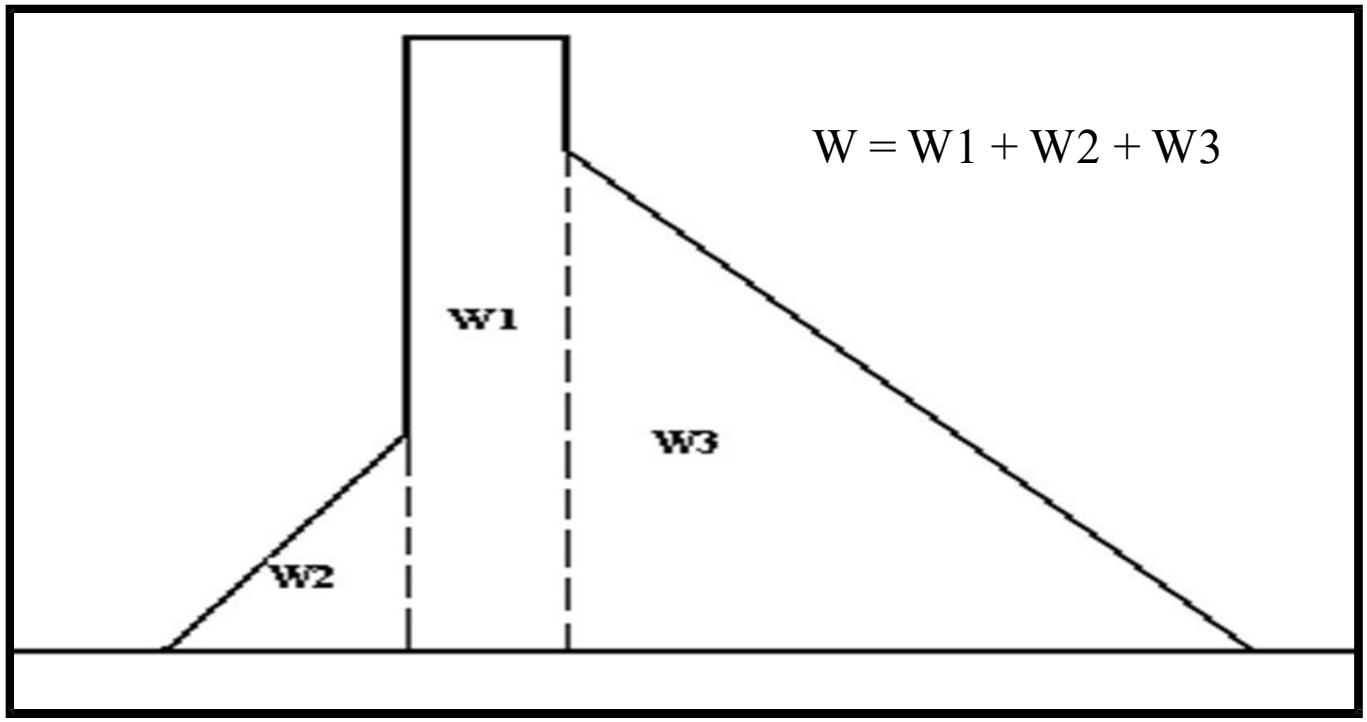
- The horizontal water pressure acts at a height of H/3 from base of the dam, and is given by $$P_1 = \frac{w\,H^2}{2}$$
- The vertical water pressure acts on the length 'b' portion of the base. This vertical pressure is given by

$$P_2 = (w \times h_2 \times b) + (w \times h_1 \times \frac{b}{2})$$



2) Weight of the Dam

- Weight of the dam is the major resisting force.
- Total weight of the dam acts at the center of gravity of its section. Unit length of the dam is considered in the calculation of weight.



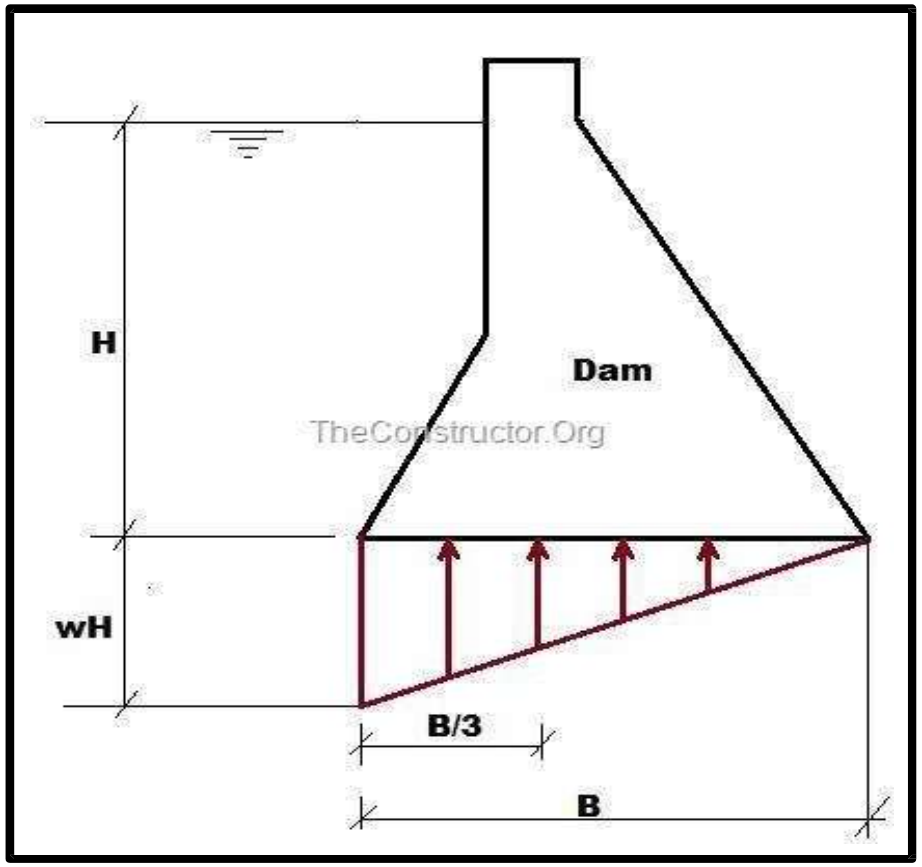
for concrete = 24 kN/m³

for masonary = 23 kN/m³

Weight = Volume per unit length x Density of material

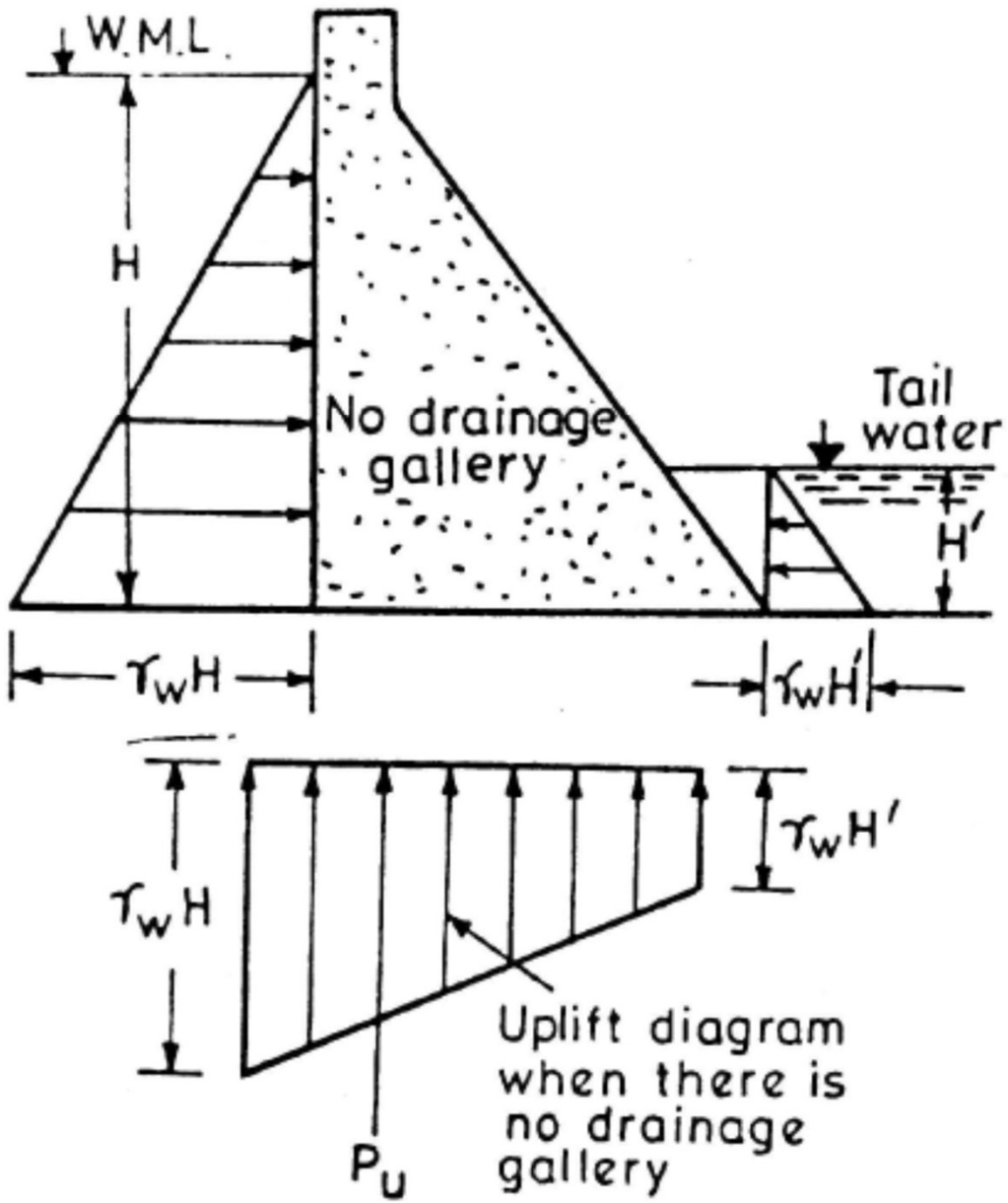
3) Uplift Pressure

- Uplift pressure is the pressure exerted by water as it seeps through the body of the dam or its foundation.
- Seeping water exerts pressure on the base of the dam and it depends upon water head.

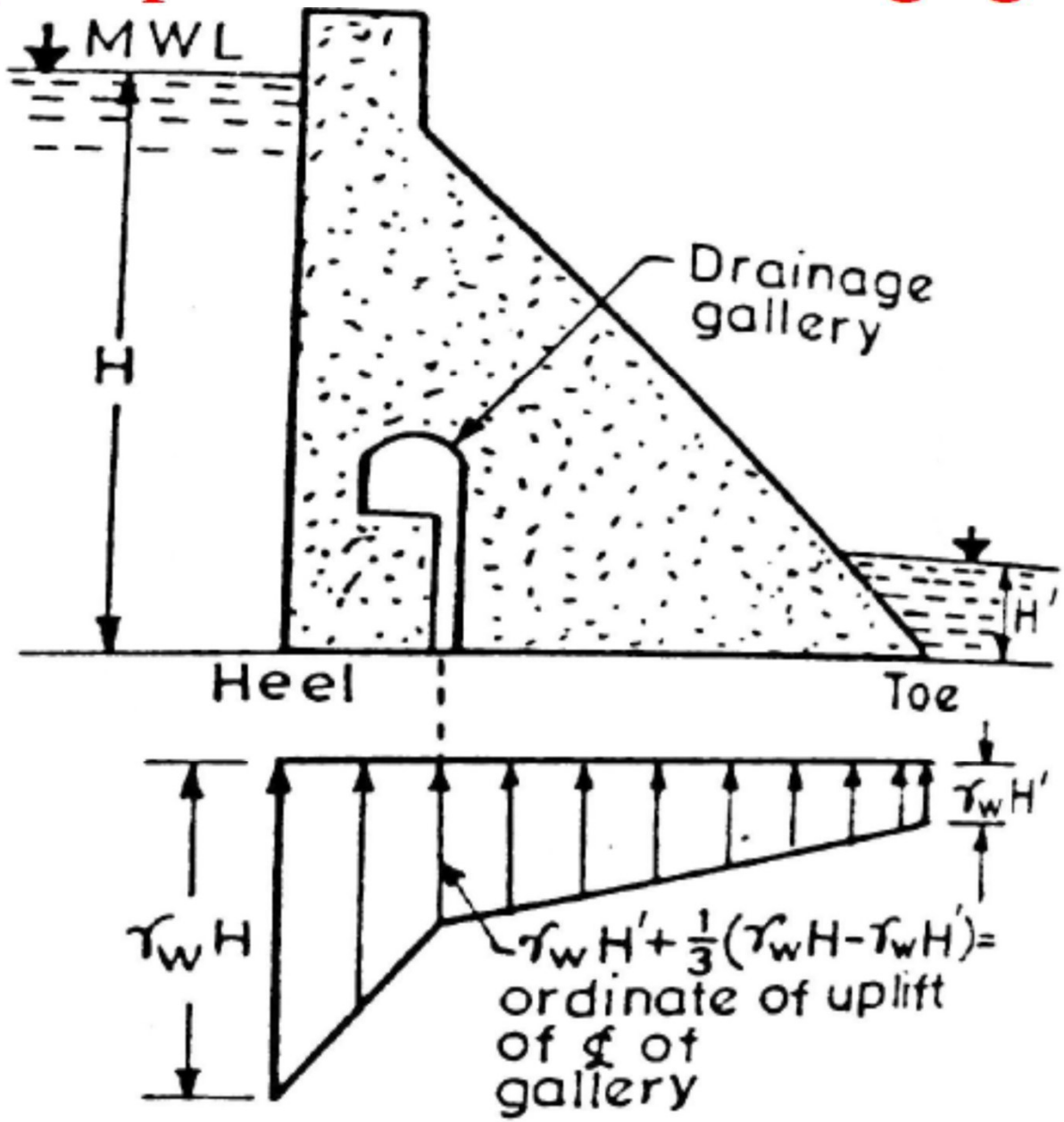


- Uplift pressure is given by $P_u = \frac{w\,H \times B}{2}$

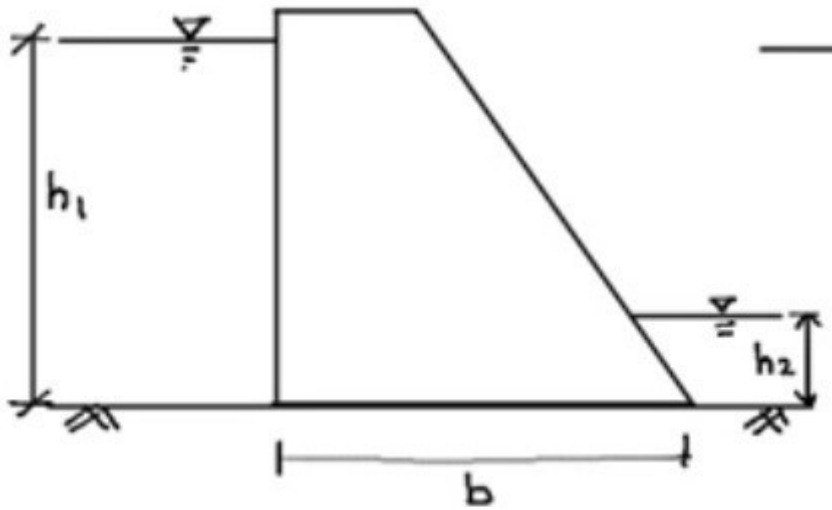
Uplift pressure no drainage gallery

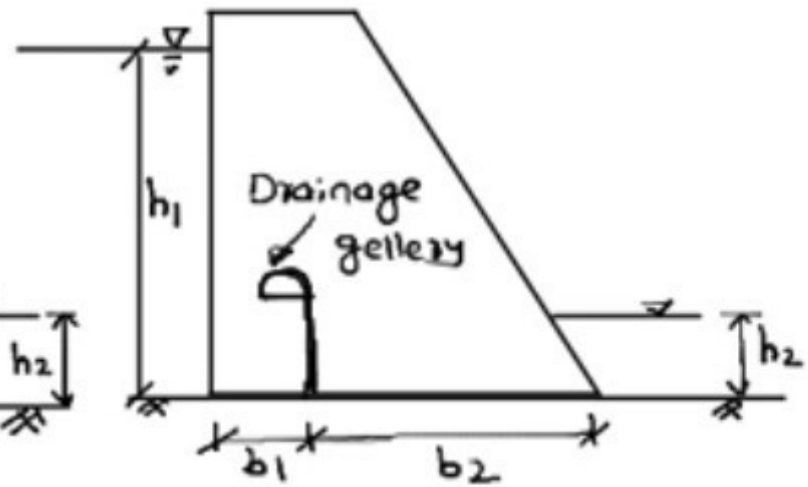


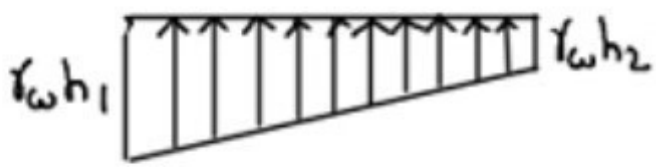
Uplift pressure with drainage gallery



uplift force below the dam.



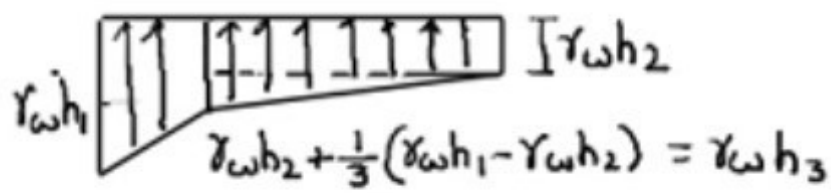




$$U = \frac{1}{2}(\gamma_w h_1 + \gamma_w h_2) \times b$$

$$= \frac{1}{2}\gamma_w b (h_1 + h_2)$$

U = uplift force per m. length



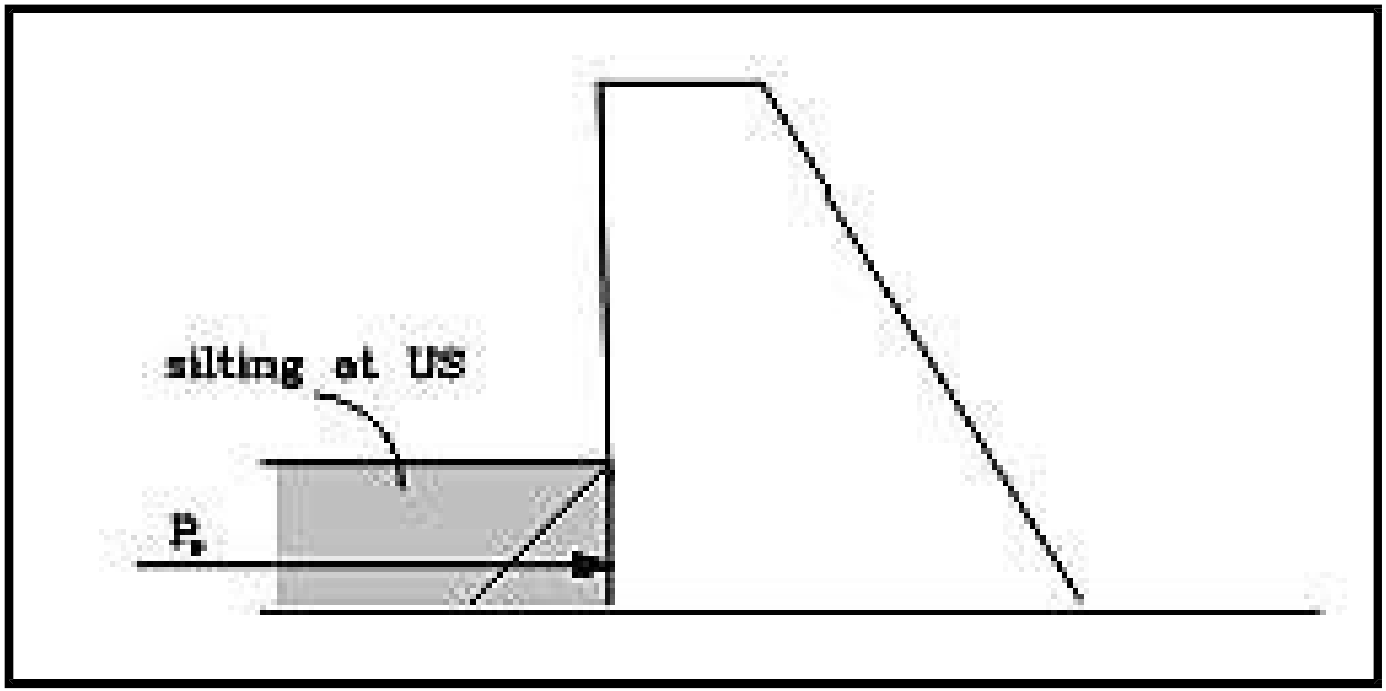
where $h_3 = h_2 + \frac{1}{3}(h_1 - h_2)$

$$U = \frac{1}{2}(\gamma_w h_1 + \gamma_w h_3) b_1 + \frac{1}{2}(\gamma_w h_3 + \gamma_w h_2) b_2$$

$$= \frac{1}{2}\gamma_w \cdot [h_1 + h_3] b_1 + \frac{1}{2}\gamma_w [h_3 + h_2] b_2$$

4) Silt Pressure

- Silt is deposited on the u/s of the dam.
- Sediment deposition in the reservoir results in a force acting horizontally on the upstream face. This force is assumed to have a hydrostatic distribution.
- Silt Pressure is given by $P_s = \frac{\gamma_s H^2 \times K_a}{2}$ acting at H/3 from base.



Case (i) If the upward face of the dam is vertical :-

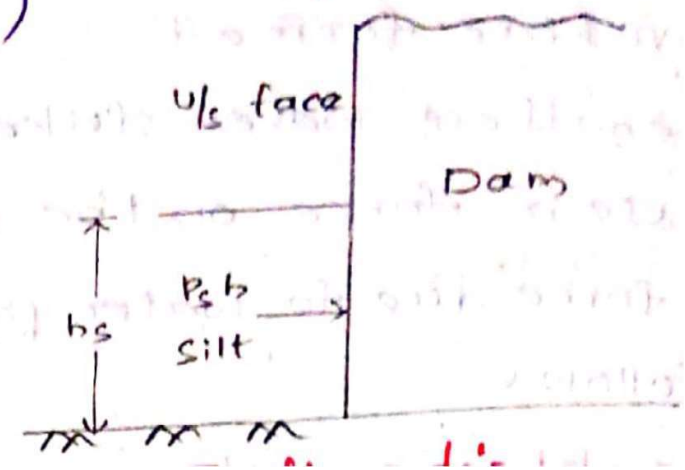
Pressure due to silt in dam $\Big\}$ $P_{sh} = \frac{w_s h_s^2}{2}\left(\frac{1-\sin\phi}{1+\sin\phi}\right)$

Where

w_s = submerged unit wt. of soil

h_s = The depth of silt above the bed

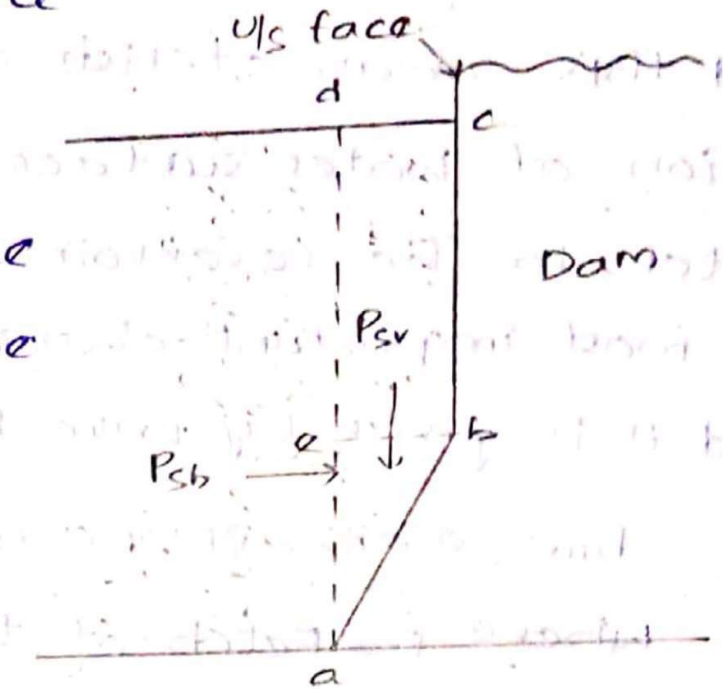
ϕ = Angle of shearing resistance



case (ii) If the upward face of the dam is Inclined:

The horizontal component of total force

$$P_{sh} = \frac{w_s h_s^2}{2}\left(\frac{1-\sin\phi}{1+\sin\phi}\right)$$



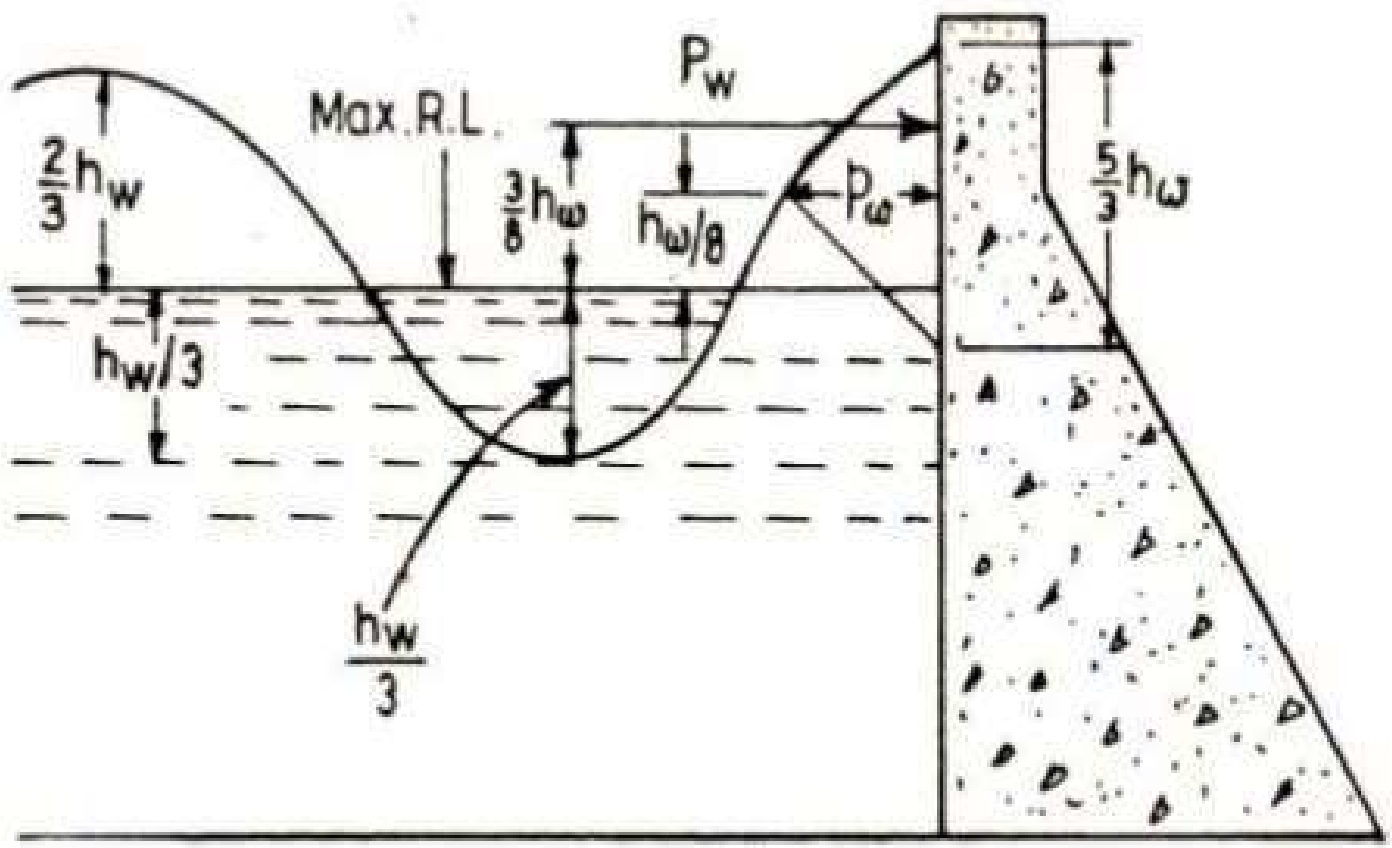
The vertical component of total force

P_{sv} = wt. of submerged wt. of silt in the area abcde over the inclined surface ab

$P_{sv} = w_s \times$ (Area of abcde)

5) Wave Pressure

- Waves are generated on the surface of the reservoir by the blowing winds, which exert a pressure on the upstream side. Wave pressure depends upon wave height, and is given by the equation $P_W = \frac{2.4\,\gamma_w\,h_W}{2} \times \frac{5}{3}\,h_W$ acting at $\frac{3}{8}\,h_W$ above the reservoir surface.



Wave Pressure

- Waves are generated on the surface of the reservoir by the blowing winds, which causes a pressure towards the downstream side. Wave pressure depends upon the wave height. Wave height may be given by the equation
- **$H_w = 0.032 \sqrt{V.F} + 0.763 - 0.271 (F)^{3/4}$ for $F < 32$ Km And**
- **$H_w = 0.032 \sqrt{V.F}$ for $F > 32$ Km**
- Where h_w= height of waves from top of crest and bottom of trough in metre.
- V= Wind velocity in Km/ hr

- F= Fetch or Straight length of water expanse in Km.
- The maximum Pressure Intensity due to wave action may be given by
- **pw= 2.4 γ_w hw** and acts at $h_w/8$ metres above the still water surface.
- The Pressure distribution may be assumed to be Triangle, of height 5 hw/3
- Hence, the total force due to wave action (P_w)
- $P_w = ½ (2.4 \gamma_w h_w) . 5 hw/3$
- $P_w = 2. \gamma w . h_w^2$
- This force acts at a distance of 3/8 h_w above the reservoir surface.

6) Ice Pressure

- The Ice Pressure which may be formed on the surface of the reservoir in cold countries, may sometimes melt and expand. The dam face has then to resist the thrust exerted by the expanding Ice. This force acts linearly along the length of the dam and at the reservoir level. The magnitude of this force varies from **250 to 1500 KN/ m^2** depending upon the temperature variations. On an average, a value of **500 KN/ m^2** may be allowed under ordinary conditions.

IS: 6512-1984 recommends 250 kN/m^2 applied to the face of dam over the anticipated area of contact of ice with the face of the dam.

7) Earthquake Pressure

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

8. Earthquake coarses :-

1. If the dam is located in the seismic regions, it should be designed for earthquake coarses.
2. As per IS 1893- 1975 India has divided into 5 seismic zones depending on the severity.
3. The earthquake acceleration is expressed as $\alpha \cdot g$

   where α = seismic coefficient

   g = acceleration due to gravity.

Seismic coefficient is divided into

1. Horizontal Seismic coefficient, α_h
2. Vertical Seismic Coefficient, $\alpha_v = .75\ \alpha_h$

α_h can be determined by one of the two methods

1. Seismic Coefficient Method < 100m height of the dam
2. Response Spectrum Method > 100m height of the dam

Seismic coefficient is divided into

1. Horizontal Seismic coefficient, α_h
2. Vertical Seismic Coefficient, $\alpha_v = .75\ \alpha_h$

α_h can be determined by one of the two methods

1. Seismic Coefficient Method $<$ 100m height of the dam
2. Response Spectrum Method $>$ 100m height of the dam

8) Wind Pressure

- It is a minor force acting on dam
- Acts on Superstructure of the dam
- Normally, wind pressure is taken as 1 to 1.5 kN/m^2

Causes of failure of a Gravity Dam

A gravity dam may fail in following modes:

1. **Overturning of dam about the toe**
2. **Sliding – shear failure of gravity dam**
3. **Compression – by crushing of the gravity dam**
4. **Tension – by development of tensile forces which results in the crack in gravity dam.**

1) Overturning

- If the moments of the destabilizing forces (such as water pressure on the upstream face and uplift) about the toe of the dam exceed those of the stabilizing forces (mainly the weight of the dam), the dam can overturn.
- If the resultant force cuts the base within the body of dam there will be no overturning.
- For safety against overturning.

1) Overturning

- The horizontal forces act against the gravity dam causes overturning moments. To resist this, resisting moments are generated by the self-weight of the dam.
- If the resultant of all the forces acting on a dam at any of its sections, passes through / outside the toe, the dam will rotate and overturn about the toe. This is called overturning failure of gravity dam. If the resultant force cuts the base within the body of dam there will be no overturning.
- The ratio of the resisting moments about toe to the overturning moments about toe is called the factor of safety against overturning. Its value generally varies between 2 and 3.
- FOS = sum of resisting moments / sum of overturning moments
- F.O.S.= $\frac{\Sigma M_R}{\Sigma M_O} \geq 1.5$

2) Sliding (Shear Failure)

- When the net horizontal forces acting on gravity dam at the base / any plane in the dam exceeds the frictional resistance (produced between body of the dam and foundation), The failure occurs is known as sliding failure of gravity dam.
- For safety against sliding

$$F.S.S = \mu \times \frac{\Sigma V}{\Sigma H} > 1$$

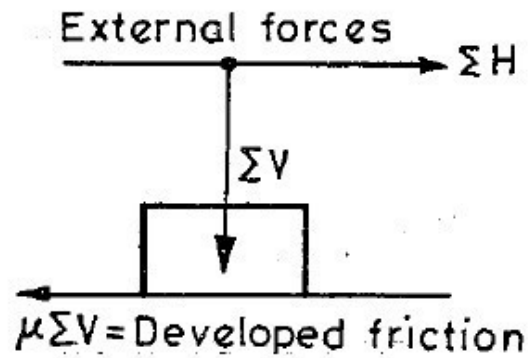
Where

μ = coefficient of static earth pressure

= 0.65 to 0.75

2) Sliding (Shear Failure) cond...

The friction developed between two surfaces is equal to $\mu\Sigma V$., (Fig. 19.9) where ΣV is the algebraic sum of all the vertical forces whether upward or downward, and μ is the coefficient of friction between the two surfaces. In order that no sliding takes place, the external horizontal forces (ΣH) must be less than the shear resistance $\mu \cdot \Sigma V$.



$$\Sigma H < \mu\Sigma V$$

$$\frac{\mu\Sigma V}{\Sigma H} > 1$$

$\frac{\mu \cdot \Sigma V}{\Sigma H}$ represents nothing but the factor of safety against sliding, which must be greater than unity.

$$\therefore \text{ F.S.S. (Factor of safety against sliding)} = \frac{\mu \cdot \Sigma V}{\Sigma H}$$

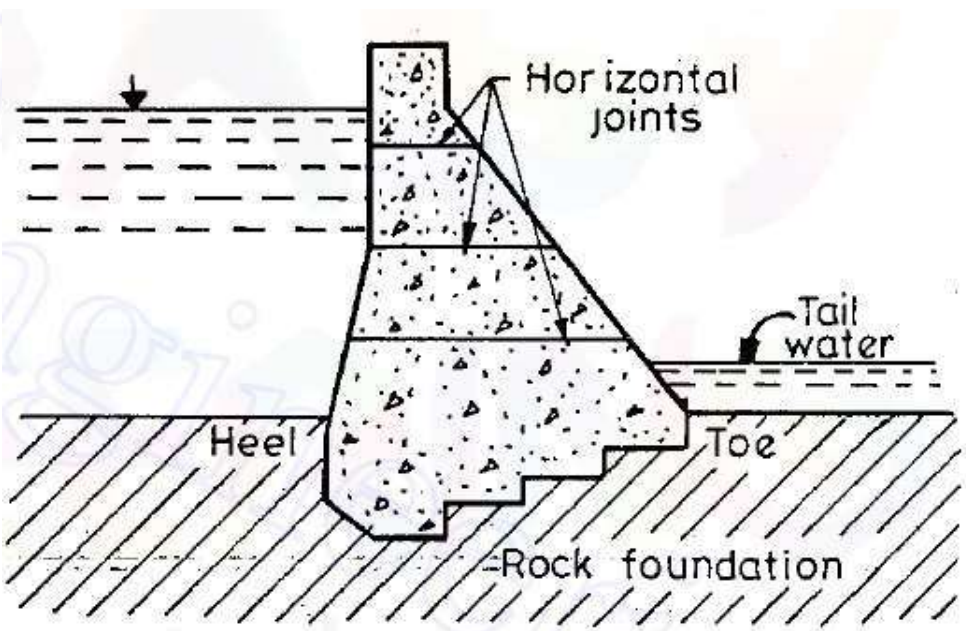
In low dams, the safety against sliding should be checked only for friction, but in high dams, for economical precise designs, the shear strength of the joint, which is an additional shear resistance, must also be considered. If this shear resistance of the joint is also considered, then the equation for factor of safety against sliding which is measured by shear friction factor (S.F.F.) becomes

$$\text{S.F.F.} = \frac{\mu \Sigma V + B \cdot q}{\Sigma H} \qquad \text{...(19.16)}$$

where B = width of the dam at the joint,

q = Average shear strength of the joint which varies from about 1400 kN/m^2 (14 kg/cm^2) for poor rocks to about 4000 kN/m^2 (40 kg/cm^2) for good rocks.

The value of μ generally varies from 0.65 to 0.75.



3) Compression or Crushing

(2) **Compression or crushing.** A dam may fail by the failure of its materials, *i.e.* the compressive stresses produced may exceed the allowable stresses, and the dam-material may get crushed. The vertical direct stress distribution at the base is given by the equation :

$$p = \text{Direct stress} + \text{Bending stress.}$$

$$\therefore \quad p_{\substack{max \\ min}} = \frac{\Sigma V}{B} \pm \frac{M}{I} y = \frac{\Sigma V}{B} \pm \frac{\Sigma V \cdot e}{B^2/6} = \frac{\Sigma V}{B}\left[1 \pm \frac{6e}{B}\right]$$

or

$$p_{\substack{max \\ min}} = \frac{\Sigma V}{B}\left[1 \pm \frac{6e}{B}\right] \qquad \text{...(19.15)}$$

where e = Eccentricity of the resultant force from the centre of the base.

ΣV = Total vertical force.

B = Base width.

The positive sign will be used for calculating normal stress at the toe, since the bending stress will be compressive there, and negative sign will be used for calculating normal stress at the heel. Thus, the normal stress at the toe is

$$p_{n,toe} = \frac{\Sigma V}{b}\left(1 + \frac{6e}{b}\right)$$

and the normal stress at the heel is

$$p_{n,heel} = \frac{\Sigma V}{b}\left(1 - \frac{6e}{b}\right)$$

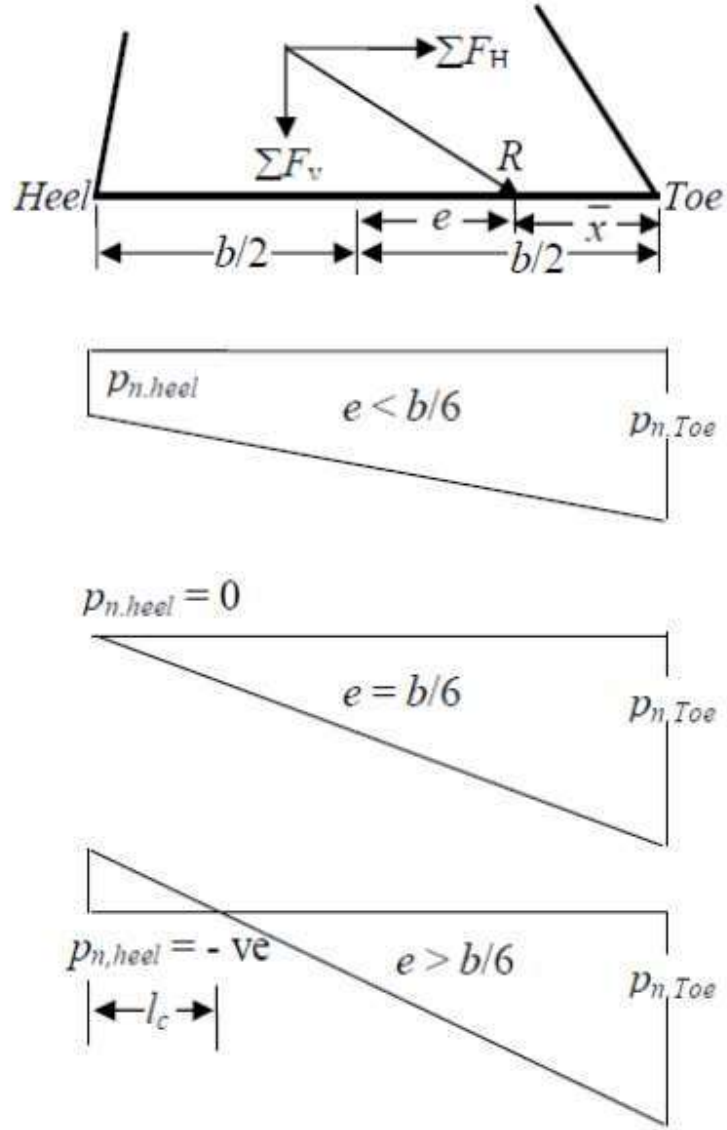


Fig. shows the normal stress distributions for a general case when the pressure at both toe and heel are compressive. Evidently, the maximum compressive stress occurs at the toe and for safety, this should not be greater than the allowable compressive stresses both for the dam and foundation materials. When the eccentricity e is equal to $b/6$ we get

$$p_{n,toe} = \frac{2\Sigma V}{b}; \qquad p_{n,heel} = 0$$

4) Tension

(3) **Tension.** Masonry and concrete gravity dams are usually designed in such a way that no tension is developed anywhere, because these materials cannot withstand sus- ering.net d tensile stresses. If subjected to such stresses, these materials may finally crack. However, for achieving economy in designs of very high gravity dams, certain amount of tension may be permitted under severest loading condition. This may be permitted because of the fact that such worst loading conditions shall occur only momentarily for a little time and would neither last long nor occur frequently. The maximum permissible tensile stress for high concrete gravity dams, under worst leadings, may be taken as 500 kN/m^2 (5 kg/cm^2).

4) Tension cond..

In order to ensure that no tension is developed anywhere, we must ensure that p_{min} is at the most equal to zero.

Since $$p_{\frac{max}{min}} = \frac{\Sigma V}{B}\left[1 \pm \frac{6e}{B}\right] \quad ...(19.15)$$

$$p_{min} = \frac{\Sigma V}{B}\left[1 - \frac{6e}{B}\right]$$

If $$p_{min} = 0,$$

$$\frac{\Sigma V}{B}\left[1 - \frac{6e}{B}\right] = 0$$

or $$1 - \frac{6e}{B} = 0$$

or $$e = \frac{B}{6}.$$

Hence, maximum value of eccentricity that can be permitted on either side of the centre is equal to $\frac{B}{6}$; which leads to the famous statement : *the resultant must lie within the middle third.*

19.4.1. Principal and Shear Stresses. The vertical stress intensity, p_{max} or p_{min} determined from the equation (19.15) is not the maximum direct stress produced anywhere in the dam. The maximum normal stress will, in fact, be the major principal stress that will be generated on the major principal plane. When the reservoir is full, the vertical direct stress [given by equation (19.15), and represented by p_v in future] is maximum at the toe as the resultant is nearer to the toe. To study the principal stresses that will develop near the toe, let us consider a small element *ABC* [See Fig. 19.11 (*a*) and (*b*)] near the toe of

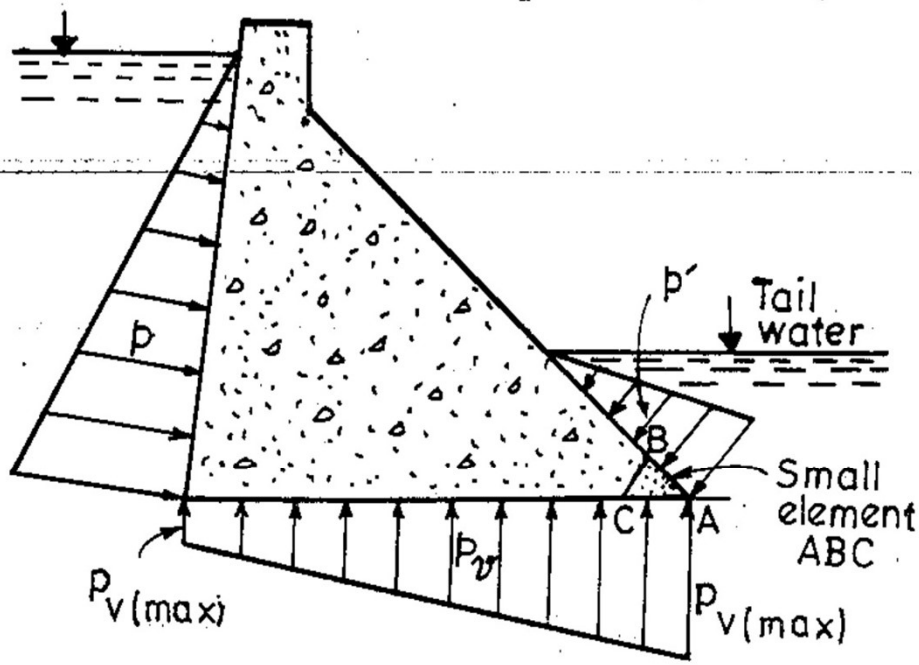


Fig. 19.11 (*a*)

the dam. The element is so small that the stress intensities may be assumed to be uniform on its faces.

Let the downstream face of the dam be inclined at an angle α to the vertical.

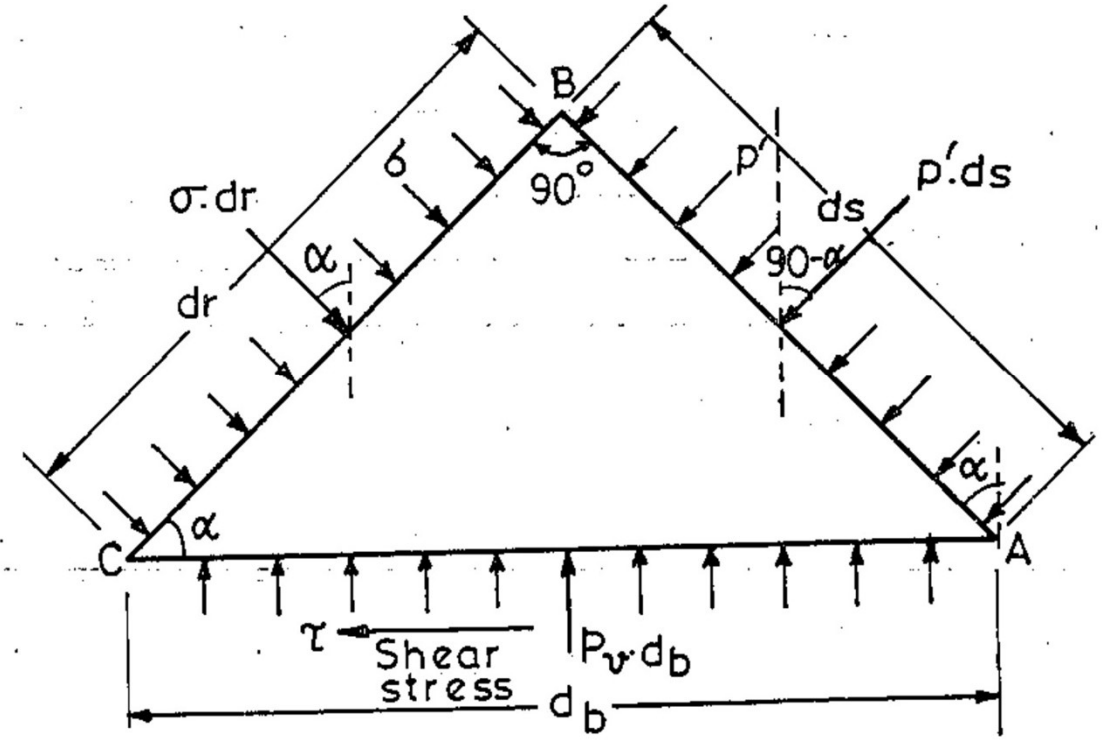


Fig. 19.11. (*b*) Enlarged view of small element *ABC* of Fig. 19.11 (*a*).

This face of the dam will act as a principal plane because the water pressure p' acts at right angles to the face, and also there is no shear stress acting on this plane. Since the principal planes are at right angles to each other ; the plane *BC* drawn at right angles to the face *AB* will be the second principal plane. Let the stress acting on this plane be σ.

Let ds, dr and db be the lengths of AB, BC and CA respectively. p' is the intensity of water pressure on face AB and p_v is the intensity of vertical pressure on face AC, and σ is the intensity of normal stress (principal stress) on face BC. Considering unit length of the dam, the forces acting on the faces AB, BC and CA are $p'\,ds$, $\sigma\,dr$ and $p_v \cdot db$ respectively.

Resolving all the forces in the vertical direction, we get

$$p' \cdot ds \cdot \sin\alpha + \sigma \cdot dr \cdot \cos\alpha = p_v \cdot db,$$

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

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

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

or $$p' \cdot \sin^2 a + \sigma \cdot \cos^2\alpha = p_v.$$

or $$\sigma = \frac{p_v - p' \cdot \sin^2\alpha}{\cos^2\alpha}$$

or $$\boxed{\sigma = p_v \cdot \sec^2 \alpha - p' \tan^2 \alpha} \quad \text{...(19.17)}$$

For σ to be maximum, p' should be zero, *i.e.* when there is no tail water ; then in such a case

$$\sigma = p_v \cdot \sec^2 \alpha \quad \text{...[19.17 (a)]}$$

Since $\sec^2 \alpha$ is always more than 1, it follows, that σ will be more than p_v. *This value of normal stress, which is the maximum produced anywhere in the body of the dam, must be calculated and should not be allowed to exceed the maximum allowable compressive stress of dam material.*

If the hydrodynamic pressure (p_e') exerted by the tail water during an earthquake moving towards the reservoir is also considered, then the net pressure on the face AB will be ($p' - p_e'$), because the effect of this earthquake will be to reduce the tail water pressure.

The principal stress (σ) can then be given by

or
$$\boxed{\sigma_{at\ toe} = p_v \cdot \sec^2 \alpha - (p' - p_e') \tan^2 \alpha} \qquad ...(19.18)$$

The equation for σ, derived above for the element at the toe is also applicable to the element at the heel. The equation at the heel is, therefore, given as :

$$\sigma_1 = \sigma_{at\ heel} = p_v \cdot \sec^2 \phi - (p + p_e) \tan^2 \phi \qquad ...(19.19)$$

where ϕ is the angle which the u/s face makes with vertical.

But at the heel, the pressure of water p is always more than σ, and hence, p will be the minor principal stress at the heel.

However, if u/s face is vertical, $\phi = 0$ and $\sigma 1 = Pv$

Shear stress on the horizontal plane near the toe. A shear stress τ will act on the face CA on which the vertical stress is acting. Resolving all the forces [Fig. 19.11 (b)] in the horizontal direction, we get

$$\sigma \cdot dr \sin\alpha - p' \cdot ds \cdot \cos\alpha = \tau_0 \cdot db$$

or $$\sigma \cdot (db \cdot \cos\alpha) \sin\alpha - p' (db \cdot \sin\alpha) \cos\alpha = \tau_0 \cdot db$$

or $$\sigma \cdot \sin\alpha \cos\alpha - p' \sin\alpha \cos\alpha = \tau_0$$

or $$\tau_0 = (\sigma - p') \sin\alpha \cos\alpha$$

Substituting the value of σ from equation (19.17), we get

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

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

or $$= [(p_v - p') \sec^2\alpha \cdot \sin\alpha \cdot \cos\alpha]$$

or $$\boxed{\tau = (p_v - p') \tan\alpha} \qquad ...(19.20)$$

Neglecting tail water, shear stress is given by

$$\tau_0 = p_v \cdot \tan\alpha \qquad ...[19.20\ (a)]$$

If the effect of hydrodynamic pressure produced by an earthquake moving towards the reservoir, is also considered, the equation for shear stress on a horizontal plane near the toe becomes,

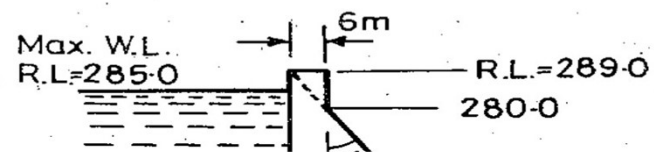
$$\tau_0 = \left[p_v - (p' - p_e')\right] \tan \alpha \qquad \text{...(19.21)}$$

Similarly, shear stress at heel

$$= \tau_{0\,(heel)} = \left[p_v - (p + p_e)\right] \tan \phi$$

–ve sign shows that the direction is reversed.

Example 19.1. *Fig. 19.12 (a) shows the section of a gravity dam (non-overflow portion)*



However, if u/s face is vertical, φ = 0 and τ = 0

STABILITY ANALYSIS

The stability of a gravity dam can be approximately and easily analysed by two dimensional gravity method and can be precisely analysed by three dimensional methods such as slab analogy method, trial load twist method, or by experimental studies on models. Two dimensional gravity method is discussed below :

19.5. Gravity Method or Two Dimensional Stability Analysis

The preliminary analysis of all gravity dams can be made easily by isolating a typical cross-section of the dam of a unit width. This section is assumed to behave independently of the adjoining sections. In other words, the dam is considered to be made up of a number of cantilevers of unit width each, which act independently of each other. This assumption of independent functioning of each section, disregards the beam action in the dam as a whole.

If the vertical transverse joints of the dam are not grouted or keyed together, this assumption is nearly true. Hence, for wide *U*-shaped valleys, where transverse joints are not generally grouted, this assumption is nearly satisfied. But for narrow V- shaped valleys, where the transverse joints are generally keyed together and the entire length of the dam acts monolithically as a single body, this assumption may involve appreciable errors. In such cases, preliminary designs may be done by gravity method and precise final designs may be carried out by any of the available three dimensional methods.

The description of the three dimensional methods is beyond the scope of this book, and only the two dimensional analysis has been used for the design of gravity dams in this chapter.

Assumptions. The various assumptions made in the two dimensional designs of gravity dams are summarised below :

(*i*) The dam is considered to be composed of a number of cantilevers, each of which is 1 m thick and each of which acts independent of the other.

(*ii*) No loads are transferred to the abutments by beam action.

(*iii*) The foundation and the dam behave as a single unit ; the joint being perfect.

(*iv*) The materials in the foundation and body of the dam are isotropic and homogeneous.

(*v*) The stresses developed in the foundation and body of the dam are within elastic limits.

(*vi*) No movements of the foundations are caused due to transference of loads.

(*vii*) Small openings made in the body of the dam do not affect the general distribution of stresses and they only produce local effects as per St. Venant's principle.

Procedure. Two dimensional analysis can be carried out analytically or graphically.

(*a*) *Analytical Method.* The stability of the dam can be analysed in the following steps :

(*i*) Consider unit length of the dam.

(*ii*) Work out the magnitude and directions of all the vertical forces acting on the dam and their algebraic sum, *i.e.* ΣV.

(*iii*) Similarly, work out all the horizontal forces and their algebraic sum, *i.e.* ΣH.

(*iv*) Determine the lever arm of all these forces about the toe.

(*v*) Determine the moments of all these forces about the toe and find out the algebraic sum of all those moments, *i.e.* ΣM.

(*vi*) Find out the location of the resultant force by determining its distance from the toe,

$$\bar{x} = \frac{\Sigma M}{\Sigma V}.$$

(*xii*) Find out the eccentricity (e) of the resultant (R) using $e = \frac{B}{2} - \bar{x}$. It must be less than $B/6$ in order to ensure that no tension is developed anywhere in the dam.

(*viii*) Determine the vertical stresses at the toe and heel using Eq. (19.15), *i.e.*

$$p_v = \frac{\Sigma V}{B}\left[1 \pm \frac{6e}{B}\right] \quad ...(19.15)$$

Sometimes stresses are found by ignoring uplift.

(*ix*) Determine the maximum normal stresses, *i.e.* principal stresses at the toe and the heel using equations (19.18) to (19.20). They should not exceed the maximum allowable values. The crushing strength of concrete varies between 1500 to 3000 kN/m^2 depending upon its grade M15 to M3O.

(*x*) Determine the factor of safety against overturning as equal to $\frac{\Sigma \text{ Stabilising moment } (+)}{\Sigma \text{ Disturbing moment } (-)}$; +ve sign is used for anti-clockwise moments and –ve sign is used for clockwise moments.

(*xi*) Determine the factor of safety against sliding, using Sliding factor = $\frac{\mu \Sigma V}{\Sigma H}$.

$$\text{Shear friction factor (S.F.F.)} = \frac{\mu \Sigma V + bq}{\Sigma H}.$$

Sliding factor must be greater than unity and S.F.F. must be greater than 3 to 5. The analysis should be carried out for reservoir full case as well as for reservoir empty case. The entire procedure has been illustrated in example 19.2.

(*b*) *Graphical method.* In the graphical method, the entire dam section is divided into a number of horizontal sections at some suitable intervals, particularly at the places where the slope changes, as shown in Fig. 19.13. For each section, the sum of the vertical forces (ΣV) and the sum of all the horizontal forces (ΣH) acting above that particular section, are worked out and the resultant force (R) is drawn, graphi-

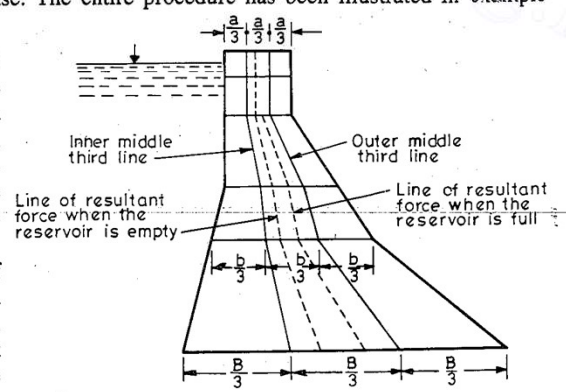


Fig. 19.13

cally. This is done for each section and a line joining all the points where the individual resultants cut the individual sections, is drawn. This line represents the resultant force and should lie within the middle third, for no tension to develop. The procedure should be carried out for reservoir full case as well as for reservoir empty case. The resultant in both cases must show non-development of tension in the dam body.

19.6. Elementary Profile of a Gravity Dam

The elementary profile of a dam, subjected only to the external water pressure on the upstream side, will be a right-angled triangle, having zero width at the water level and a base width (B) at bottom *i.e.*, the point where the maximum hydrostatic water pressure acts. In other words, the shape of such a profile is similar to the shape of the hydrostatic pressure distribution (Fig. 19.14).

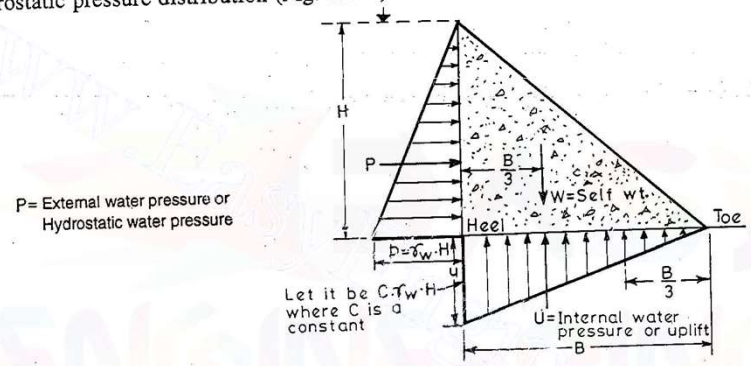


Fig. 19.14

When the reservoir is empty, the only single force acting on it is the self-weight (W) of the dam and it acts at a distance $B/3$ from the heel. This is the maximum possible innermost position of the resultant for no tension to develop. Hence, such a line of action of W is the most ideal, as it gives the maximum possible stabilising moment about the toe without causing tension at toe, when the reservoir is empty. *The vertical stress distribution at the base, when the reservoir is empty,* is given as :

$$P_{max/min} = \frac{\Sigma V}{B}\left[1 \pm \frac{6e}{B}\right]$$

Here $\Sigma V = W$

$$e = \frac{B}{6}.$$

$$\therefore \qquad p_{max/min} = \frac{W}{B}\left[1 \pm \frac{6 \cdot B}{B \cdot 6}\right]$$

or $$p_{max} = \frac{2W}{B}$$

and $$p_{min} = 0.$$

Hence, the maximum vertical stress equal to $\frac{2W}{B}$ will act at the heel ($\because$ the resultant is nearer the heel) and the vertical stress at toe will be zero.

When the reservoir is full, the base width is governed by :

(*i*) The resultant of all the forces, *i.e.* P, W and U (Fig. 19.14) passes through the outer most middle third point (*i.e.* lower middle third point).

(*ii*) The dam is safe in sliding.

(*i*) For the 1st condition to be satisfied, we proceed as follows : Taking moments of all the forces (Fig. 19.14) about the lower middle third point (*i.e.* the point through which resultant is passing), we get

$$W\left(\frac{B}{3}\right) - U\left(\frac{B}{3}\right) - P\frac{H}{3} = R \times 0$$

or
$$(W - U)\frac{B}{3} - P\frac{H}{3} = 0$$

But
$$W = \frac{1}{2} \times B \times H \times 1 \times S_c \times \gamma_w$$

where S_c = Sp. gravity of concrete, *i.e.* that of the material of the dam.

γ_w = unit wt. of water = 9.81 kN/m^3

Let the uplift at the heel be $C \cdot \gamma_w \cdot H$., where C is a constant which according to U.S.B.R. recommendation is taken equal to 1.0 in calculation and will be equal to zero when no uplift is considered.

$\therefore$
$$U = \left(\frac{1}{2} C \cdot \gamma_w \cdot H\right) B$$

and
$$P = \frac{1}{2}\gamma_w \cdot H \cdot H = \frac{\gamma_w H^2}{2}$$

$\therefore$ Equation $(W - U)\frac{B}{3} - P\frac{H}{3} = 0$, becomes

$$\left[\frac{1}{2} B \cdot H \cdot S_c \cdot \gamma_w - \frac{1}{2} C \cdot \gamma_w \cdot H \cdot B\right]\frac{B}{3} - \frac{\gamma_w H^2}{2} \cdot \frac{H}{3} = 0$$

or
$$\frac{B}{3} \times \frac{1}{2} B \cdot H \cdot \gamma_w \cdot [S_c - C] = \frac{\gamma_w H^3}{6}$$

or
$$B^2 (S_c - C) = H^2$$

or
$$\boxed{B = \frac{H}{\sqrt{S_c - C}}} \qquad ...(19.22)$$

Hence, if B is taken equal to or greater than $\frac{H}{\sqrt{S_c - C}}$, no tension will be developed at the heel with full reservoir,

when
$$C = 1$$

$$B = \frac{H}{\sqrt{S_c - 1}} \qquad ...[19.22\ (a)]$$

If uplift is not considered, $B = \frac{H}{\sqrt{S_c}}$ $(\because C = 0)$ ...[19.22 (*b*)]

(*ii*) For the II condition (*i.e.* dam is safe in sliding) to be satisfied ; the frictional resistance $\mu\Sigma V$ or $\mu(W-U)$ should be equal to or more than the horizontal forces $\Sigma H = P$.

or $$\mu(W-U) \geq P$$

or $$\mu\left(\frac{1}{2}BH \cdot S_c \cdot \gamma_w - \frac{1}{2}C \cdot \gamma_w \cdot H \cdot B.\right) \geq \frac{\gamma_w H^2}{2}$$

or $$\mu(S_c - C)\frac{1}{2} \cdot B \cdot H \cdot \gamma_w \geq \frac{\gamma_w H^2}{2}$$

or $$\mu(S_c - C)B \geq H$$

or $$B \geq \frac{H}{\mu(S_c - C)}$$

Under limiting condition

or $$\boxed{B = \frac{H}{\mu(S_c - C)}} \quad ...(19.23)$$

If $C = 1$ $$B = \frac{H}{\mu(S_c - 1)} \quad ...[19.23(a)]$$

If $C = 0$, *i.e.* no uplift is considered, then

$$B \geq \frac{H}{\mu S_c} \quad ...[19.23(b)]$$

The value of B chosen should be greater of the two values given by Equations (19.22) and (19.23).

Using $S_c = 2.4$ and $\mu = 0.7$ and $C = 0$, we get

$$B \text{ (by Equation 19.22)} = \frac{H}{\sqrt{2.4 - 0}} = \frac{H}{\sqrt{2.4}}$$

$$B \text{ (by Equation 19.23)} = \frac{H}{0.7 \times (2.4 - 0)} = \frac{H}{0.7 \times 2.4} = \frac{H}{1.68}$$

But $\frac{H}{1.68}$ is less than $\frac{H}{\sqrt{2.4}}$

$\therefore$ For all practical purposes, the base width may be taken as $\frac{H}{\sqrt{S_c}}$

The vertical stress distribution when reservoir is full is given as :

$$p_{max/min} = \frac{\Sigma V}{B}\left[1 \pm \frac{6e}{B}\right]$$

where $\Sigma V = W - U$

$$= \left(\frac{1}{2}B \cdot H \cdot 1 \cdot S_c \cdot \gamma_w - \frac{1}{2}C \cdot \gamma_w \cdot H \cdot B\right)$$

$$= \frac{1}{2}B \cdot \gamma_w \cdot H \cdot [S_c - C]$$

$$e = \frac{B}{6}$$

$$\therefore \quad p_{max/min} = \frac{\frac{1}{2} \cdot B \cdot \gamma_w \cdot H (S_c - C)}{B} \left[1 \pm \frac{6B}{6B}\right]$$

maximum stress will occur at toe, because the resultant is near the toe.

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

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

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

The principal stress near the toe (σ) which is the maximum normal stress in the dam, is given by Equation (19.17)

$$\sigma = p_v \sec^2 \alpha - p' \tan^2 \alpha$$

when there is no tail water *i.e.*, $p' = 0$

$$\sigma = p_v \sec^2 \alpha$$

σ at toe, with full reservoir in elementary profile

$$= \gamma_w H (S_c - C) \sec^2 \alpha$$

$$= \gamma_w H (S_c - C) [1 + \tan^2 \alpha]$$

$$= \gamma_w H (S_c - C) \left[1 + \frac{B^2}{H^2}\right]$$

But $B = \dfrac{H}{\sqrt{S_c - C}}$ from Eq. (19.22)

or $\dfrac{B^2}{H^2} = \dfrac{1}{S_c - C}$

$$\therefore \quad \sigma = \gamma_w H (S_c - C) \left[1 + \frac{1}{S_c - C}\right]$$

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

when $C = 1$, $S_c = 2.4$

$$\sigma = \gamma_w H \left(\frac{2.4 - 1 + 1}{2.4 - 1}\right) = \frac{2.4}{1.4} \gamma_w H$$

$$= \mathbf{1.71\, \gamma_w H.}$$

The shear stress τ_0 at a horizontal plane near the toe is given by the equation (19.20) as :

$$\tau_0 = (p_v - p') \tan \alpha$$

If $p' = 0$

$$\tau_0 = p_v \tan \alpha$$

But $p_v = \gamma_w H (S_c - C)$ from Eq. (19.24)

$$\therefore \quad \tau_0 = \gamma_w H (S_c - C) \tan \alpha$$

or $$\tau_0 = \gamma_w \cdot H (S_c - C) \frac{B}{H}$$

$$= \gamma_w H (S_c - C) \frac{1}{\sqrt{S_c - C}}$$

or $$\boxed{\tau_0 = \gamma_w H \sqrt{S_c - C}} \quad ...(19.26)$$

19.7. High and Low Gravity Dams

The principal stress calculated for an elementary profile is given by Equation (19.25), *i.e.* $\sigma = \gamma_w H (S_c - C + 1)$. The value of principal stress calculated above varies only with H, as all other factors are fixed.

To avoid dam failure by crushing, the value of σ should be less than or at the most equal to the maximum allowable compressive stress of dam material. If f represents the allowable stress of the dam material, then the maximum height (H_{max}) which can be obtained in an elementary profile, without exceeding the allowable compressive stresses of the dam material, is given as :

$$f = \gamma_w H (S_c - C + 1)$$

or $$H = \frac{f}{\gamma_w (S_c - C + 1)}$$

The lowest value of H will be obtained when $C = 0$, *i.e.* when uplift is neglected. Hence, for determining the limiting height and to be on a safer side, uplift is neglected.

H_{max} *i.e.* maximum possible height is given as :

$$H_{max} = \frac{f}{\gamma_w (S_c + 1)} \quad ...(19.27)$$

Hence, if the height of a dam having an elementary profile of a triangle, is more than that given by the Equation (19.27), the maximum compressive stress generated will exceed the allowable value. In order to keep it safe within limits, extra slopes on the upstream as well as on the downstream, below the limiting height will have to be given, as shown in Fig. 19.15.

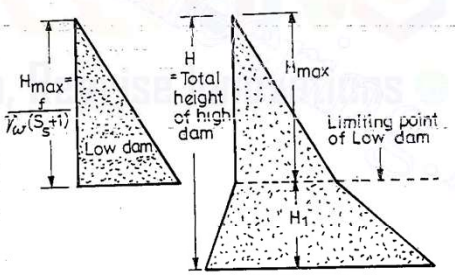


Fig. 19.15

This limiting height (H_{max}) given by Equation (19.27), draws a dividing line between a low gravity dam and a high gravity dram, which are purely technical terms to differentiate between them.

Hence, a low gravity dam is the one whose height is less than that given by Equation (19.27). If the height of the dam is more than this, it is known as a high gravity dam.

The limiting height of a low concrete gravity dam, constructed in concrete having strength equal to 3000 kN/m^2 is thus given :

$$H_{max} = \frac{f}{\gamma_w (S_c + 1)}$$

where $\gamma_w = 9.81 \text{ kN/m}^3$

$S_c = 2.4$

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

$$\therefore \quad H_{max} = \frac{f}{\gamma_w (S_c + 1)} = \frac{3000}{9.81\,(2.4+1)} = \mathbf{90\ m}$$

19.8. Profile of a Dam from Practical Considerations

The elementary profile of a gravity dam, (*i.e.* a triangle with maximum water surface at apex) is only a theoretical profile. Certain changes will have to be made in this profile in order to cater to the practical needs. These needs are : (*i*) providing a straight top width, for a road construction over the top of the dam ; (*ii*) providing a free-board above the top water surface, so that water may not spill over the top of the dam due to wave action, etc.

The additions of these two provisions, will cause the resultant force to shift towards the heel. The resultant force, when the reservoir is empty, was earlier passing through the inner middle third point. This will, therefore, shift more towards the heel, crossing the inner middle third point and consequently, tension will be developed at the toe. In order to avoid the development of this tension, some masonry or concrete will have to be added to the upstream side, as shown in Fig. 19.16, which shows the typical section along with the possible dimensions that can be adopted for a low gravity dam section. It should, however, be checked for stability analysis.

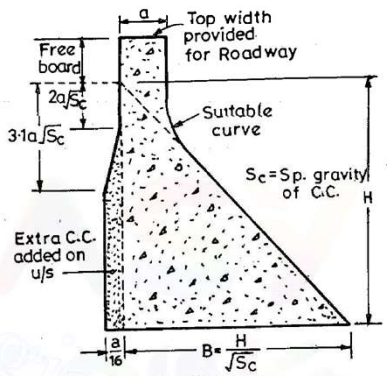


Fig. 19.16.
Typical section of a low gravity dam.

19.9. Design Considerations and Fixing the Section of a Dam

The free-board and top width for roadway should be selected as follows :

(*i*) **Freeboard.** The margin between the maximum reservoir level and top of the dam is known as freeboard. This must be provided in order to avoid the possibility of water spilling over the dam top due to wave action. This can also help as a safety for unforeseen floods, higher than the designed flood.

The freeboard is generally provided equal to $\frac{3}{2}h_w$. where h_w is given by Equations (19.11) and (19.12). However, these days, a *free-board equal to 4 to 5% of the dam height is provided.*

(*ii*) **Top width.** The effects produced by the addition of top width at the apex of an elementary dam profile, and their remedies are explained below :

In Fig. 19.17, let *AEF* be the triangular profile of dam of height H_1. Let the element *ABQA* be added at the apex for providing a top width *a* for a road construction. Let M_1 and M_2 be the inner third and outer third points on base. Thus, AM_1 and AM_2 are the inner third and outer third lines. The weight of the element (W_1) will act through the *e.g.* of this triangle, *i.e.* along *C.M.* Let *CM* and AM_1 cross at *H*, and *CM* and AM_2 cross at *K*.

Galleries in Gravity Dams

- **Galleries are the horizontal or sloping openings or passages left in the body of the dam.**
- They may run longitudinally (i.e. parallel to dam axis) or traversely (i.e. normal to the dam axis) and are provided at various elevations. **All the galleries are interconnected** by steeply sloping passages or by vertical shafts fitted **with stairs or mechanical lifts.**

Function and types of galleries in Dams

(i) Foundation Gallery

- **A gallery provided in a dam may serve one particular purpose or more than one purpose**. For example, a gallery provided near the rock foundation, serves **to drain off the water** which percolates through the foundations. This gallery is called a **foundation gallery or a drainage gallery.**

Function and types of galleries in Dams

- It runs longitudinally and is quite near to the upstream face of the dam. Drain holes are drilled from the floors of this gallery after the foundation grouting has been completed. **Seepages is collected through these drain holes**.
- **Besides draining off seepage water, it may be helpful for drilling and grouting of the foundations**, when this can not be done from the surface of the dam.

Function and types of galleries in Dams

Inspection Galleries

- **The water which seeps through the body of the dam is collected by means of a system of galleries provided at various elevations** and interconnected by vertical shafts, etc. **All these galleries, besides draining off seepage water, serves inspection purpose.**
- They provide access to the interior of the dam and are, therefore, called **inspection purposes. They generally serve other purposes along with this purpose.**

Function and types of galleries in Dams

The main functions are summarized below:

- They **intercept** and drain off the **water seeping through the dam body.**
- They **provide access to dam interior** for **observing and controlling the behavior of the dam.**
- They **provide enough space for carrying pipes**, etc. during **artificial cooling of concrete**

Function and types of galleries in Dams

- **They provide access to all the outlets and spillway gates, valves, etc**. by housing their electrical and mechanical controls. **All these gates, valves, etc, can hence be easily controlled by men, from inside the dam itself.**
- **They provide space for drilling and grouting of the foundations**, then it cannot be done from the surface of the dam. **Generally, the foundation gallery is used for this purpose.**

GALLERIES

A gallery is an opening within a dam that provides access into or through the dam. These may run either longitudinally or transversely and may be either horizontal or inclined. The following are the common types and uses of galleries:

(i)Drainage galleries provide a drainage way for water percolating through the upstream face or seeping through the foundation.

(ii)Grouting galleries provide space for drilling and grouting the foundation.

(iii)Inspection galleries provide access to the interior of the structure for observing its behaviour after completion.

(iv) Gate galleries (or chambers or vaults) provide access to, and room for, such mechanical and electrical equipment as are used for the operation of gates in spillways and outlet works.

(v)Cable galleries provide access through the dam for control cables and/or power cables and related equipment.

(vi) Visitors' galleries provide access routes for visitors.

Other galleries may be needed in a particular dam to meet special requirements, such as the artificial cooling of concrete blocks, the grouting of contraction joints, and so on.

EARTHEN DAMS

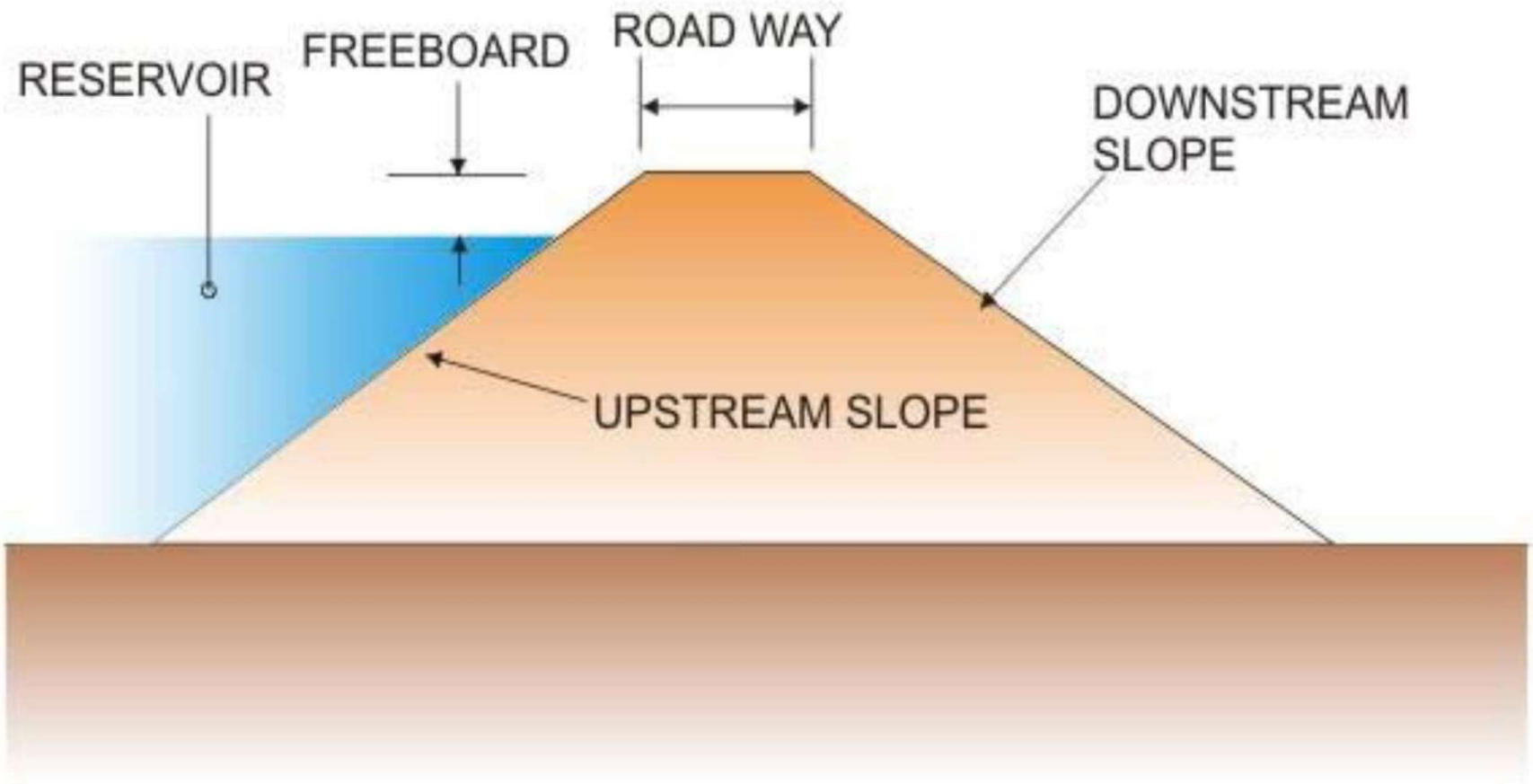


FIGURE 1. General shape of an embankment dam

INTRODUCTION

- An embankment dam, as defined earlier, is one that is built of natural materials. In its simplest and oldest form, the embankment dam was constructed with low-permeability soils to a nominally homogeneous profile.
- The section featured neither internal drainage nor a cutoff to restrict seepage flow through the foundation. Dams of this type proved vulnerable associated with uncontrolled seepage, but there was little progress in design prior to the nineteenth century.

INTRODUCTION

It was then increasingly recognized that, in principle, larger embankment dams required two component elements.

1. An impervious water-retaining element or **core** of very low permeability of soil, for example, soft clay or a heavily remoulded 'puddle' clay, and
2. **Supporting shoulders** of coarser earthfill(or of rockfill), to provide structural stability

- The top crest is kept wide so as to accommodate roadway In order to check the **seepage through** the body of the dam, a number of variations are provided. For earthen embankment dams, these range from the following types:

1. Homogeneous dam with toe drain
2. Homogeneous dam with horizontal blanket
3. Homogeneous dam with chimney drain and horizontal blanket
4. Zoned dam with central vertical core & toe drain
5. Zoned dam with central vertical core, chimney filter and horizontal blanket
6. Zoned dam with inclined core, chimney filter and horizontal blanket

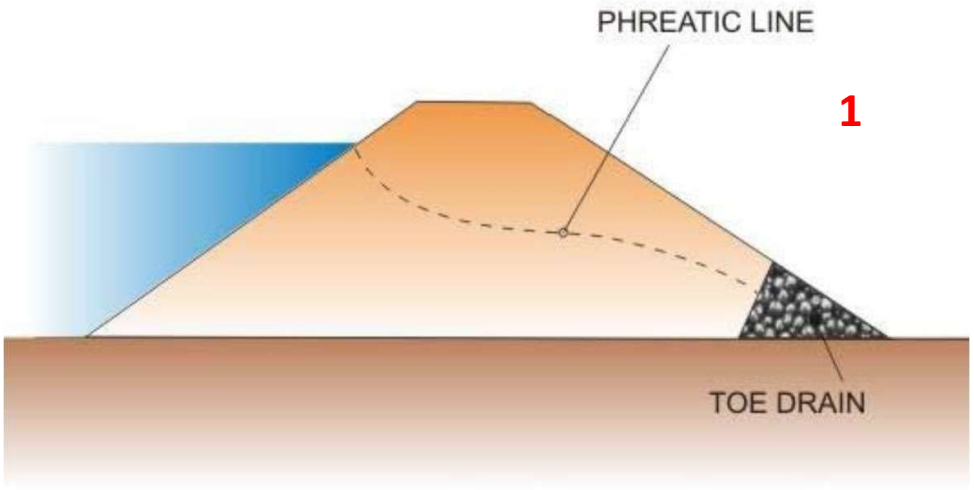


FIGURE 2. Homogeneons earthen embankment dam with toe drain

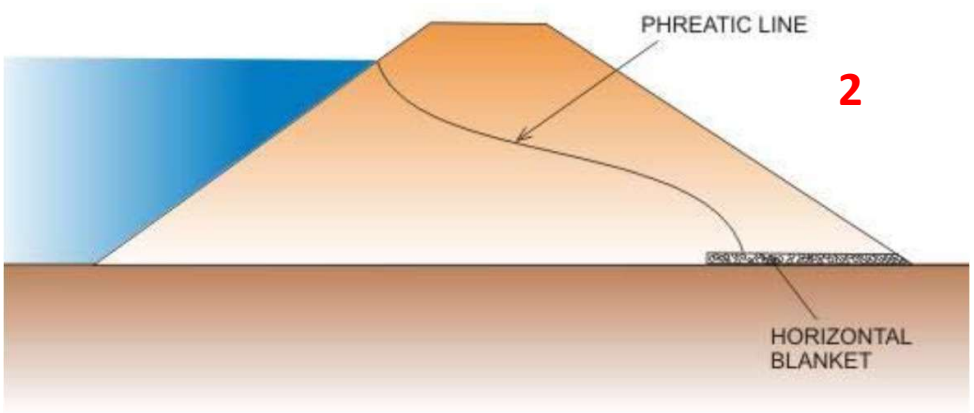


FIGURE 3.Homogeneous dam with horizontal blanket

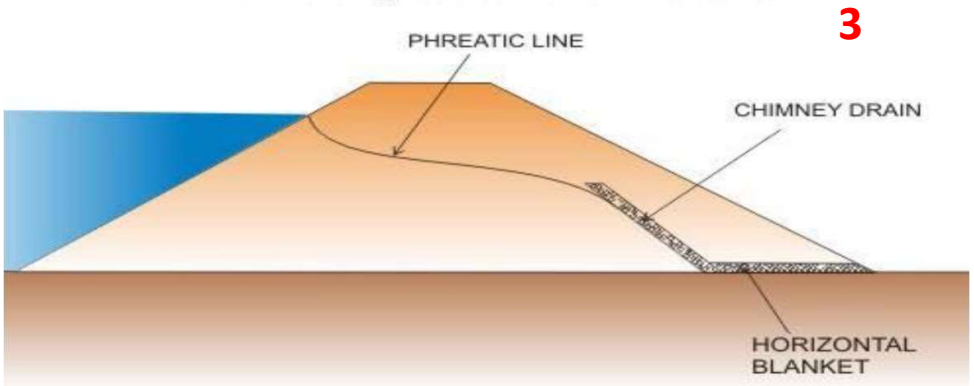
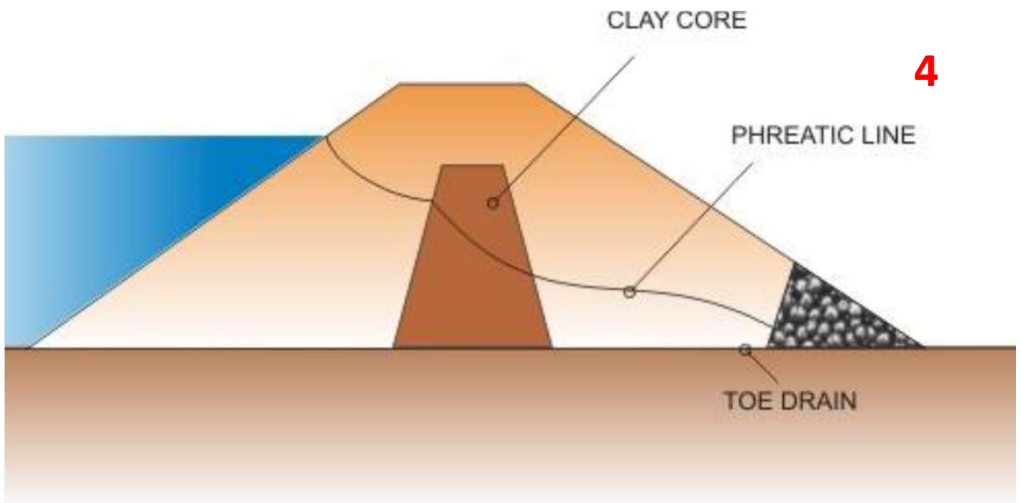
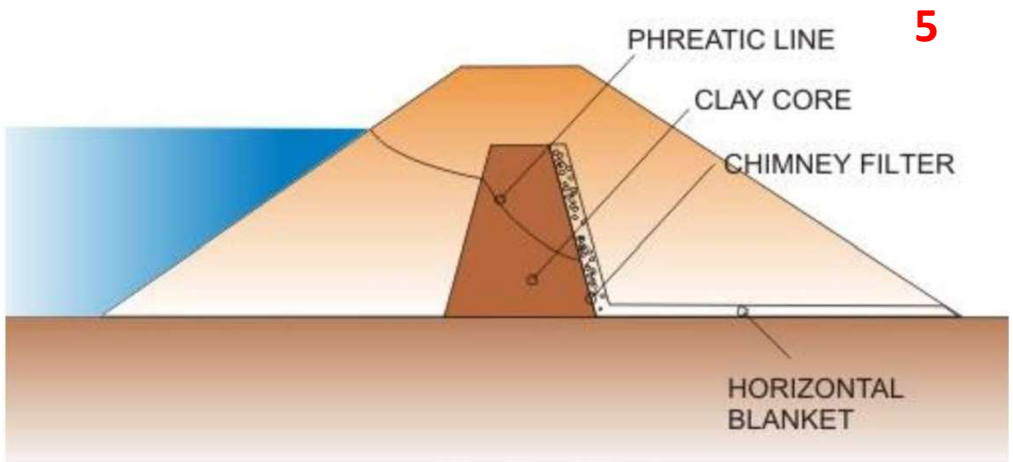
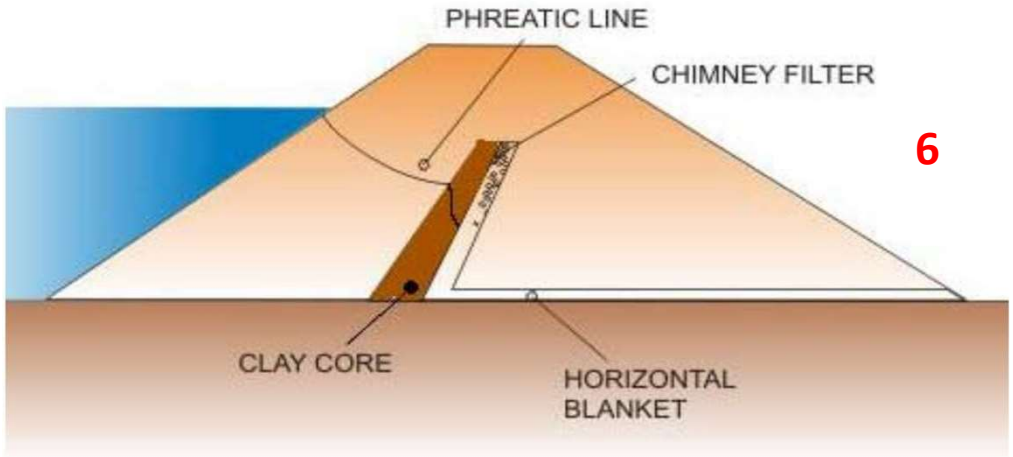


FIGURE 4. Homogeneous dam with inclined chimney drain connected to horizontal blanket







Embankment dam types

EARTHEN DAM

Earth dams have been built since the early days of civilization. They are constructed mainly from earth (or soil). Fig..

ROCKFILL DAM

Rockfill dams are constructed mainly from rockfill or pieces of rocks. Fig. 2.1(1

They require somewhat stronger foundations as compared to earth dams,

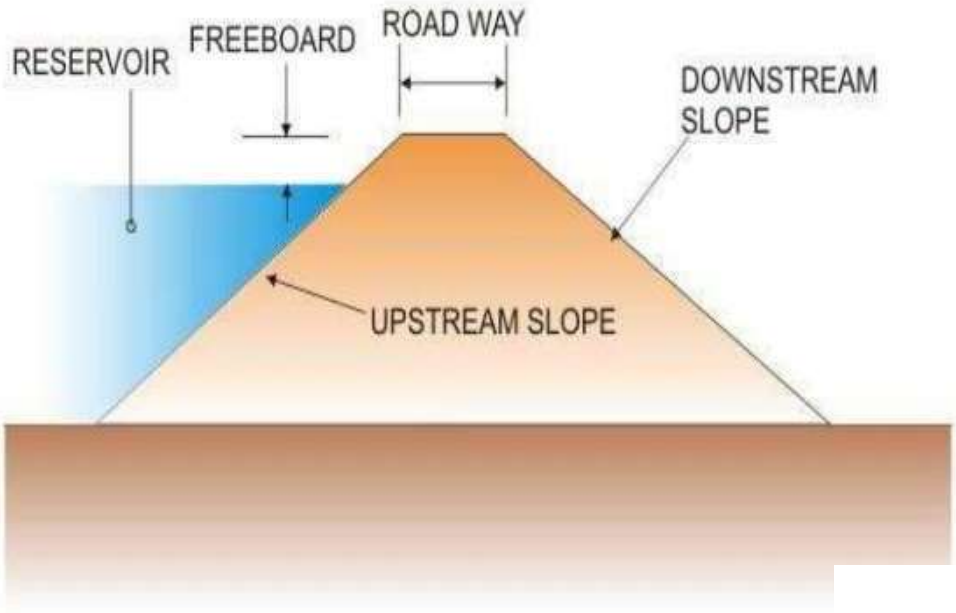


FIGURE 1. General shape of an embankment dam

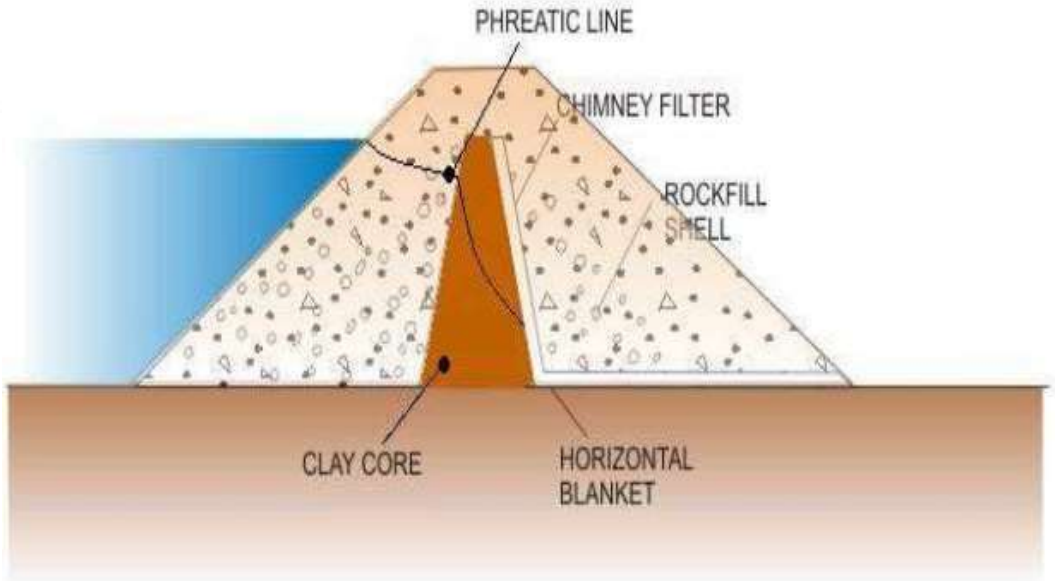


FIGURE 8. Rockfill dam with vertical clay core, chimney filter and horizontal blanket.

which is a composite earth and rockfill type embankment dam antf it is 335 m high. In India, **Tehri dam across river Bhagirathi is the highest dam which is also a** compositie earth and rockfill dam type embankment dam and it is *261 m high.*

MODULE -2- EARTH DAMS

Introduction

An **embankment dam** is a large artificial dam. It is typically created by the placement and compaction of a complex semi-plastic mound of various compositions of soil, sand, clay, or rock. It has a semi-pervious waterproof natural covering for its surface and a dense, impervious core. This makes such a dam impervious to surface or seepage erosion. Such a dam is composed of fragmented independent material particles. The friction and interaction of particles binds the particles together into a stable mass rather than by the use of a cementing substance.

Embankment dams come in two types: the **earth-filled dam** (also called an earthen dam or terrain dam) made of compacted earth, and the **rock-filled dam**. A cross-section of an embankment dam shows a shape like a bank, or hill. Most have a central section or core composed of an impermeable material to stop water from seeping through the dam. The core can be of clay, concrete, or asphalt concrete. This dam type is a good choice for sites with wide valleys. They can be built on hard rock or softer soils. For a rock-fill dam, rock-fill is blasted using explosives to break the rock. Additionally, the rock pieces may need to be crushed into smaller grades to get the right range of size for use in an embankment dam.
The building of a dam and the filling of the reservoir behind it places a new weight on the floor and sides of a valley. The stress of the water increases linearly with its depth. Water also pushes against the upstream face of the dam, a nonrigid structure that under stress behaves semiplastically, and causes greater need for adjustment (flexibility) near the base of the dam than at shallower water levels. Thus the stress level of the dam must be calculated in advance of building to ensure that its break level threshold is not exceeded.

Causes of failure of earth dams

Stability and Failure of Earth Filled Dams
Failure of earth dams may be:

1. Hydraulic Failure
2. Seepage Failure
3. Structural Failure

1. Hydraulic Failure:

1. Overtopping of dams
2. Erosion of the Upstream Surface
3. Erosion of the Downstream Surface
4. Erosion of the Downstream toe

i. Overtopping of dams:

This type of dam is made up of only one type of material. Usually porous materials is used. These dams are easy and cheap to construct but cannot be used to make multipurpose large dams. For large multipurpose dams zoned type method is used. Over topping failures result from the erosive action of water on the embankment. Erosion is due to un-controlled flow of water over, around, and adjacent to the dam. Earth embankments are not designed to be over-topped and therefore are particularly susceptible to erosion. Once erosion has begun during over- topping, it is almost impossible to stop. A well vegetated earth embankment may withstand limited over topping if its crest is level and water flows over the crest and down the face as an evenly distributed sheet without becoming concentrated. The owner should closely monitor the reservoir pool level during severe storms.

ii. Erosion of the Upstream Surface:

Here zones of different materials are made.
Shell is used to give support and stability to the structure of dam. It is made of coarse materials and is pervious in nature.
Core is used to make the dam water tight and to reduce the seepage. Fine material is used here. Used in large dams.

iii. Erosion of the Downstream Surface:

Due to rainfall, snow and winds the downstream surface of the dam also erodes. By providing a section of coarse materials here, this erosion can be reduced or prevented.

2. Seepage Failure:

All earth dams have seepage resulting from water permeating slowly through the dam and its foundation. Seepage must be controlled in both velocity and quantity. If uncontrolled, it can progressively erode soil from the embankment or its foundation, resulting in rapid failure of the dam. Erosion of the soil begins at the downstream side of the embankment, either in the dam proper or the foundation, progressively works toward the reservoir, and eventually develops a direct connection to the reservoir. This phenomenon is known as "piping." Piping action can be recognized by an increased seepage flow rate, the discharge of muddy or discolored water, sinkholes on or near the embankment, or a whirlpool in the reservoir. Once a whirlpool (eddy) is observed on the reservoir surface, complete failure of the dam will probably follow in a matter of minutes. As with over topping, fully developed piping is virtually impossible to control and will likely cause failure. Seepage can cause slope failure by creating high pressures in the soil pores or by saturating the slope. The pressure of seepage within an embankment is difficult to

determine without proper instrumentation. A slope which becomes saturated and develops slides may be showing signs of excessive seepage pressure.

Seepage failure of the dams is of the following types

1. Piping through the dam
2. Piping through the foundation
3. Conduit Leakage

1. **Piping thorugh the dam:** There are two kinds of forces acting on the downstream face of the dam:

1. Weight of the material
2. Seepage Force

If the seepage force exceeds the weight of the material the water washes away the soil from the plate and creates a hole in the ground. This hole deepens as more and more mateial is taken away from it and extends longitudinally, making a pipe hole called "Piping in the dam".

3. Structural Failure:

Structural failures can occur in either the embankment or the appurtenances. Structural failure of a spillway, lake drain, or other appurtenance may lead to failure of the embankment. Cracking, settlement, and slides are the more common signs of structural failure of embankments. Large cracks in either an appurtenance or the embankment, major settlement, and major slides will require emergency measures to ensure safety, especially if these problems occur suddenly. If this type of situation occurs, the lake level should be lowered, the appropriate state and local authorities notified, and professional advice sought. If the observer is uncertain as to the seriousness of the problem, the Division of Water should be contacted immediately. The three types of failure previously described are often interrelated in a complex manner. For example, uncontrolled seepage may weaken the soil and lead to a structural failure. A structural failure may shorten the seepage path and lead to a piping failure. Surface erosion may result in structural failure.

Failure of downstream face during steady seepage conditions
1. Failure of upstream face during sudden draw down
2. Failure due to sliding of foundation
3. damage due to burrowing animals
4. Failure of dam due to earthquake

1.Usually upper part of the dam is dry and the lower is saturated with water which gives rise to pore water pressure within the voids. Dam body is saturated - All pores / voids are filled with water, pore water pressure is induced. Effective pressure reduces and shear strength of soil decreases

2. When water is suddenly withdrawn or in other words if the level of water in the reservoir reduces suddenly, the soil on the upstream face of the dam body may be highly saturated and has pore water pressure that tries to destabilise the dam and if this force is high enough, it can fail the dam.

3.If the shear strength of the soil on which the foundation is built is weak though the foundation itself may be strong but due to weakness of the soil foundation may slide on the sides and in some cases the foundation itself may be not able to resist the shear force that may have increased from normal due to any reason.

4.Burrowing animals - Small animals living in the holes and pits may have dug theri holes anywhere in te dam body which may widen with the passage of time and can be dangerous.

5. Earthquake

Minor defects such as cracks in the embankment may be the first visual sign of a major problem which could lead to failure of the structure. The seriousness of all deficiencies should be evaluated by someone experienced in dam design and construction. A qualified professional engineer can recommend appropriate permanent remedial measures.

Preliminary section of Earthen Dam

The various components of an earthen dam are shown in Fig.

1. **Shell, Upstream Fill, Downstream Fill or Shoulder:** These components of the earthen dam are constructed with pervious or semi-pervious materials upstream or downstream of the core. The upstream fill is called the upstream shell and the downstream portion is the downstream shell.
2. **Upstream Blanket**: It is a layer of impervious material laid on the upstream side of an earthen dam where the substratum is pervious, to reduce seepage and increase the path of flow. The blanket decreases both the seepage flow and excess pressure on the downstream side of the dam. A natural blanket is a cover of naturally occurring soil material of low permeability.
3. **Drainage Filter**: It is a blanket of pervious material constructed at the foundation to the downstream side of an earthen dam, to permit the discharge of seepage and minimize the possibility of piping failure.
4. **Cutoff Wall or Cutoff**: It is a wall, collar or other structure intended to reduce percolation of water through porous strata. It is provided in or on the foundations.
5. **Riprap**: Broken stones or rock pieces are placed on the slopes of embankment particularly the upstream side for protecting the slope against the action of water, mainly wave action and erosion.
6. **Core Wall, Membrane or Core**: It is a centrally provided fairly impervious wall in the dam. It checks the flow of water through the dam section. It may be of compacted puddled clay, masonry, or concrete built inside the dam.

7. **Toe Drain**: It is a drain constructed at the downstream slope of an earthen dam to collect and drain away the seepage water collected by the drain filters.
8. **Transition Filter**: It is a component of an earthen dam section which is provided with core and consists of an intermediate grade of material placed between the core and the shells to serve as a filter and prevent lateral movement of fine material from the core.

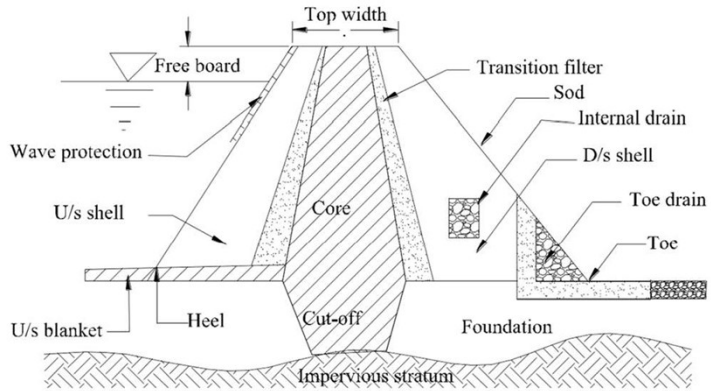


Fig.. Cross-section of an Earthen Dam with Various Components. (Source: Michael and Ojha, 2012)

Advantages

1. Design procedures are straightforward and easy.
2. Local natural materials are used.
3. Comparatively small establishment and equipment are required.
4. Earth fill dams resist settlement and movement better than more rigid structures and can be more suitable for areas where earth movements are common.

Disadvantages

1. An earthen embankment is easily damaged or destroyed by water flowing on, over or against it. Thus, a spillway and adequate upstream protection are essential for any earthen dam.
2. Designing and constructing adequate spillways is usually the most technically difficult part of any dam building work. Any site with a poor quality spillway should not be used.
3. If it is not adequately compacted during construction, the dam will have weak structure prone to seepage.
4. Earthen dams require continual maintenance to prevent erosion, tree growth, subsidence, animal and insect damage and seepage.

Types of Earthen Dam

1. Based on the method of construction:

(a)Rolled Fill Earthen Dams: In this type of dams, successive layers of moistened or damp soils are placed one above the other. Each layer not exceeding 20 cm in thickness is properly consolidated at optimum moisture content maintained by sprinkling water. It is compacted by a mechanical roller and only then the next layer is laid.

(b) Hydraulic Fill Earthen Dam: In this type of dams, the construction, excavation and transportation of the earth are done by hydraulic methods. Outer edges of the embankments are kept slightly higher than the middle portion of each layer. During construction, a mixture of excavated materials in slurry condition is pumped and discharged at the edges. This slurry of excavated materials and water consists of coarse and fine materials. When it is discharged near the outer edges, the coarser materials settle first at the edges, while the finer materials move to the middle and settle there. Fine particles are deposited in the central portion to form a water tight central core. In this method, compaction is not required.

2. Based on the mechanical characteristics of earth materials used in making the section of dam:

(a) **Homogeneous Earthen Dams:** It is composed of one kind of material (excluding slope protection). The material used must be sufficiently impervious to provide an adequate water barrier, and the slopes must be moderately flat for stability and ease of maintenance (Fig.).

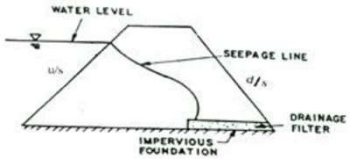


Fig. Homogenous Earthen Dam. (Source: Michael and Ojha, 2012)

(b) **Zoned Earthen Dams**: It contains a central impervious core, surrounded by zones of more pervious material, called shells. These pervious zones or shells support and protect the impervious core (Fig).

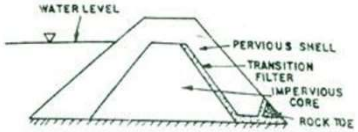


Fig.. Zoned Earthen Dam. (Source: Michael and Ojha, 2012)

(c) **Diaphragm Earthen Dam**: This type of dam (Fig. 11.4) is a modified form of homogenous dam which is constructed with pervious materials, with a thin impervious diaphragm in the central part to prevent seepage of water. The thin impervious diaphragm may be made of impervious clayey soil, cement concrete or masonry or any impervious material. The diaphragm can be constructed in the central portion or on the upstream face of the dam. The main difference in zoned and diaphragm type of dams depends on the thickness of the impervious core or diaphragm. The thickness of the diaphragm is not more than 10 m.

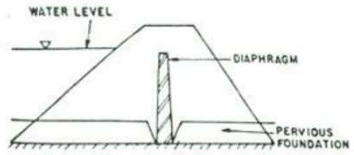


Fig.. Diaphragm Earthen Dam. (Source: Michael and Ojha, 2012)

Design Criteria

Following main design criteria may be laid down for the safety of an earth dam:

1. To prevent hydraulic failures the dam must be so designed that erosion of the embankment is prevented. For this purpose, the following steps should be followed:

(a) Spillway capacity is sufficient to pass the peak flow.

(b) Overtopping by wave action at maximum water level is prevented.

(c)The original height of structure is sufficient to maintain the minimum safe freeboard after settlement has occurred.

(d) Erosion of the embankment due to wave action and surface runoff does not occur.

(e) The crest should be wide enough to withstand wave action and earthquake shock.

2. To prevent the failures due to seepage:

(a) Quantity of seepage water through the dam section and foundation should be limited.

(b) The seepage line should be well within the downstream face of the dam to prevent sloughing.

(c)Seepage water through the dam or foundation should not remove any particle or in other words cause piping.

(d)There should not be any leakage of water from the upstream to the downstream face. Such leakage may occur through conduits, at joints between earth and concrete sections or through holes made by aquatic animals.

3. To prevent structural failures:

(a)The upstream and downstream slopes of the embankment should be stable under all loading conditions to which they may be subjected including earthquake.

(b) The foundation shear stresses should be within the permissible limits of shear strength of the material.

Design of Earthen Dam

The preliminary design of earthen dam is done on the basis of past experiences. For designing purpose several parameters, given below should be considered.

1. Top Width
2. Free Board
3. Settlement Allowance
4. Casing or Outer Shell
5. Cut-off Trench
6. Downstream Drainage System

1. Top Width: Minimum top width (W) should be such that it can enhance the practicability and protect it against the wave action and earth wave shocks. Sometimes it is also used for transportation purposes. It depends upon the height of the earthen dam and can be calculated as follows:

$$W = \frac{H}{5} + 3 \quad \text{(for very low dam)}$$

$$W = 0.55\sqrt{H} + 0.2H \quad (H \leq 30)$$

$$W = 1.65\sqrt[3]{H + 1.5} \quad (H \geq 30)$$

where H = the height of the dam (m), for Indian conditions it should not be less than 6 m.

Free board: It is the vertical distance between the top of the dam and the full supply level of the reservoir or the added height. It acts as a safety measure for the dam against high flow condition that is waves and runoff from storms greater than the design frequency from overtopping the embankment. The Recommended values of free board for different heights of earthen dams, given by U.S.B.R., are given in Table.

Table . Recommended Values of Free Board given by U.S.B.R.

Nature of spillway	Height of dam	Free board
Free	Any	Minimum 2 m and maximum 3 m over the maximum flood level
Controlled	< 60 m	2.5 m above the top of the gate
Controlled	> 60 m	3 m above the top of the gate

If fetch length or exposure is given then the free board can also be calculated by Hawksley's formula:

$$h_w = 0.014 D_m^{0.5}$$

where, = wave height (m); D_m = fetch or exposure (m).

2. Settlement Allowance: It is the result of the settlement of the fill and foundation material resulting in the decrease of dam storage. It depends upon the type of fill material and the method and speed of construction. It varies from 10% of design height for hand compacted to 5% for machine compacted earthfill.

3.Casing or Outer Shell: Its main function is to provide stability and protection to the core. Depending upon the upstream and downstream slopes, a recommendation for the casing and outer shell slopes for different types of soils given by Terzaghi is presented in Table 1.

Table. Recommended Slopes of Earthen Dam (Sources: S.K. Garg, 2008)

Sl. No.	Types of material	u/s slope	d/s slope
1.	Homogenous well graded material	$2\frac{1}{2}:1$	$2:1$
2.	Homogenous coarse silt	$3:1$	$2\frac{1}{2}:1$
3.	Homogenous silty clay or clay a) Height less than15 m b) Height more than 15 m	 $2\frac{1}{2}:1$ $3:1$	 $2:1$ $2\frac{1}{2}:1$
4.	Sand or sand and gravel with clay core	$3:1$	$2\frac{1}{2}:1$
5.	Sand or sand and gravel with R.C. core wall	$2\frac{1}{2}:1$	$2:1$

Cutoff Trench: It is provided to reduce the seepage through the foundation and also to reduce the piping in the dam. It should be aligned in a way that its central line should be within the upstream face of the impervious core. Its depth should be more than 1 m. Bottom width of cutoff trench (B) is calculated as:

$$B = h - d$$

where h = reservoir head above the ground surface (m); and d = depth of cutoff trench below the ground surface (m).

4. Downstream Drainage System: It is performed by providing the filter material in the earthen dam which is more pervious than the rest of the fill material. It reduces the pore water pressure thus adding stability to the dam.

Three types of drains used for this purpose are:

a) Toe Drains
b) Horizontal Blanket
c) Chimney Drains.

Determination of parametric line by Casagrande's method

Phreatic Line in Earth Dam

Phreatic line is also known as seepage line or saturation line. It is defined as an imaginary line within a dam section, below which there is a positive hydrostatic pressure and above it there is a negative hydrostatic pressure. The hydrostatic pressure represents atmospheric pressure which is equal to zero on the face of phreatic line. Above the phreatic line, there is capillary zone, also called as capillary fringe, in which the hydrostatic pressure is negative. The flow of seepage water, below the phreatic line, reduces the effective weight of the soil; as a result shear strength of a soil is reduced due to increased intergranular pressure in earth fill material.

1. **Derivation of Phreatic Line with Filter**

In this case, before going directly for derivation, the important features of phreatic line must be known. From the experimental evidence, it has been found that, the seepage line is pushed down by the toe filter

and is very close to parabolic shape except at the junction point of the upstream face. The upstream face of the dam represents 100% equipotential line when it is covered by the water; under this condition the seepage line should be drawn perpendicular to this face at the junction point.

Casagrande method is used for deriving the phreatic line (Fig); the procedure is described as follows:

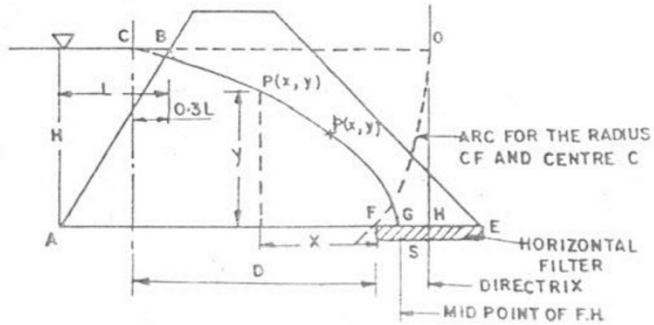


Fig.. Derivation of phreatic line in earth dam. (Source: Suresh, 2002)

a)Let the phreatic line is assumed to be a base parabola with its focus at point *F*, *i.e.* at the starting point of the filter, *FE*.

b)AB is the upstream face of the earth dam and *L* is the horizontal projection of face *AB* on the water surface. Measure the distance *BC* equal to 0.3*L*. Count point *C* as a starting point of base of parabola.

c)For deciding the position of directrix of the parabola, the principle which states that every point of the parabola is at equidistance from the focus as well as directrix. Hence, considering *C* as a center and *CF* as radius, an arc is drawn which cut the horizontal line *CB* at point O. Since *CO*= *CF*, hence vertical line *OH* will be directrix of parabola.

d) The last point G of the parabola will fall at the middle of the points *F* and *H*.

e)The intermediate points of parabola are located on the principle that their distance from the focus and directrix are the same. Here to locate the point *P* as an intermediate point, a vertical line *DP* is drawn at any distance *x* from the *F*. Now considering the distance *DH* as radius with *F* as a center, an arc is drawn which cuts the vertical *DP* at point P.

f)Now all there obtained points are joined by free hand to get the base parabola. However, this needs to be corrected at the entry point, for the feature that phreatic line must be started from the point *B* only, not from *C*. It should be sketched perpendicular to the upstream face *AB*, as it is 100 percent equipotential line. Now phreatic line is sketched by free hand in such way that, it should be perpendicular to face *AB* and meets to rest of the points of the parabola tangentially. In addition, the base parabola should also be met perpendicular, to the downstream face of the dam at point*G*.

13.3 Equation of Parabola

The equation of base parabola can be derived from its basic properties *i.e.* the distance of any point *P(x, y)* on the parabola from its focus is the same as the distance of the point *P(x, y)* from directrix.

Thus we have,

$$PF = DH$$

$$(x^2 + y^2)^{1/2} = DF + FH = x + s$$

Where, s = focal distance (*FH)*
From equation,

$$x^2 + y^2 = x^2 + s^2 + 2xs$$

$$y^2 = s^2 + 2xs \qquad 1$$

This is the desired equation of base parabola.
For deriving the expression of discharge (q) for the earth dam equipped with horizontal filter, the Darcy's law is used. According to which, the discharge (q) through vertical section *PD*, is equal to:

$$q = kiA = k.\frac{\partial y}{\partial x}(y \times 1) \qquad 2$$

Partial differentiation of Eqn.2, resulted

$$\frac{\partial y}{\partial x} = \frac{s}{(2xs + s^2)^{\frac{1}{2}}} \qquad 3$$

Substituting the value of in Eqn. 3, the rate of seepage flow through the dam is given by:

$$q = k\,\frac{s}{(2xs + s^2)^{\frac{1}{2}}} \times (2xs + s^2)^{\frac{1}{2}}$$

or, $$q = k \times s$$

This is the expression for computing the rate of seepage discharge through the body of earthen dam, in terms of focal distance s. The distance s can be determined either graphically or analytically. Considering C as co-ordinate, the value of s can be obtained as:
From Eqn: 1

$$s = \sqrt{x^2 + y^2} - x$$

At point C, $x = D$ and $y = H$

Therefore, $$s = \sqrt{D^2 + H^2} - D$$

Thus, $$q = k \times s$$

$$q = k\,[(D^2 + H^2)^{1/2} - D]$$

By using this equation, if the value of coefficient of permeability (k) and focal distance (s) are known, the discharge (q) can be calculated. This gives an accurate value of seepage rate and is applicable to such dams, which are provided with horizontal drainage (filter) system but can also be used for other types of dam section.

2. Phreatic Line in Earthen Dam without Filter

The position of phreatic line in an earth dam without filter can be determined using the same manner, as in previous case i.e. with a filter. In this case, the focal point (F) of the parabola will be the lowest point of the downstream slope (Fig. 2). The base of the parabola BJC cut at a point Jon downstream slope and is extended beyond the limit of the dam, as indicated by dotted line, but the seepage line should be emerged at point K, tangential to downstream face. In this way, the phreatic line should be shifted to the point K from J. The distance KF is known as discharge face, which always remains under saturation condition. The correction JK (say) by which the base of parabola need to be shifted downward, can be determined by graphical and analytical methods.

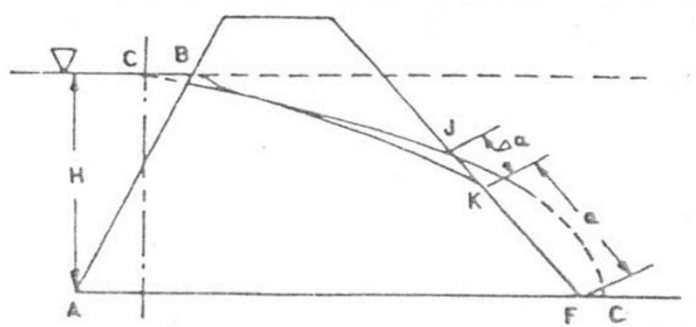


Fig. 2. Phreatic line without filter.
(Source: Suresh, 2002)

1. Graphical Method

Casagrande has given a general solution to determine the value of for various degrees of inclination of the discharge face. The inclination angle may be more than 90°, especially in case of rock fill dam.

Let, if a is the slope angle of the discharge face with the horizontal is known, and then various values of $\frac{\Delta a}{a+\Delta a}$ corresponding to a are given by Casagrande (Table).

$$a + \Delta a = JF \text{ (From Fig.}$$

Here, *JF* indicates the distance of the focus from the point, where base of parabola cuts downstream face. The values of and can be obtained by Eqn and Table.

Table. Values of $\frac{\Delta a}{a+\Delta a}$ for various slope angles (a)

Slope angle α (in degree)	$\frac{\Delta a}{a+\Delta a}$	Remarks
30	0.36	**Note**: Intermediate values of $\frac{\Delta a}{a+\Delta a}$ can be computed by interpolation method
60	0.32	
90	0.26	
120	0.18	
135	0.14	
150	0.10	
180	0	

Estimation of seepage

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.

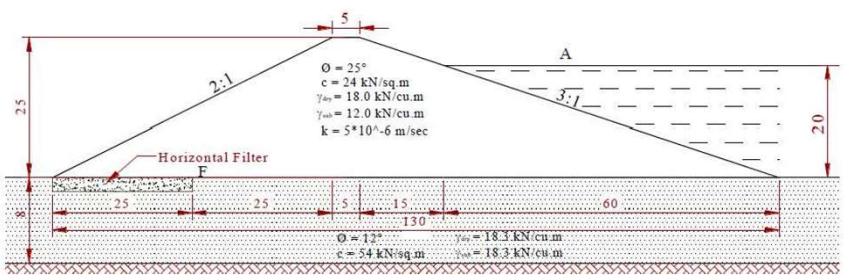


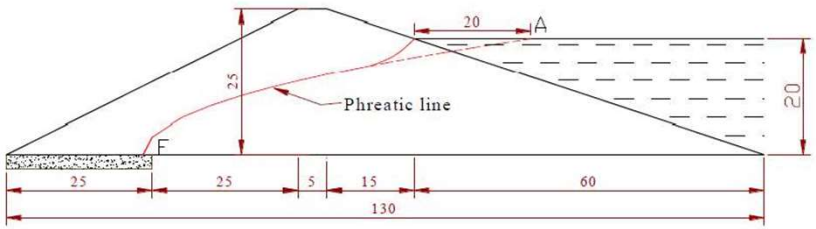
Figure 5 Section of a homogenous earth dam

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 = 65m, and y = 20m. Inserting into the parabola equation, S = 3.07m. 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

Q = kS

$= 5 * 10^{-6} * 3.07$

$= 15.35 * 10^{-6}\ m^3/sec$ per meter width of dam

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 5-8. 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.

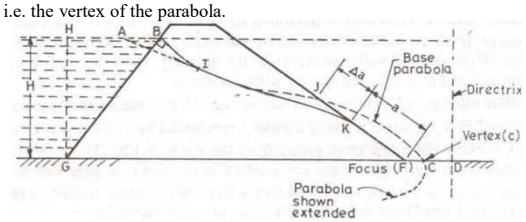


Figure Homogeneous dam section without filter

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 Δa) 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

α is the angle which the discharge face makes with the horizontal. a and Δa can be connected by the general equation;

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

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

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

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

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

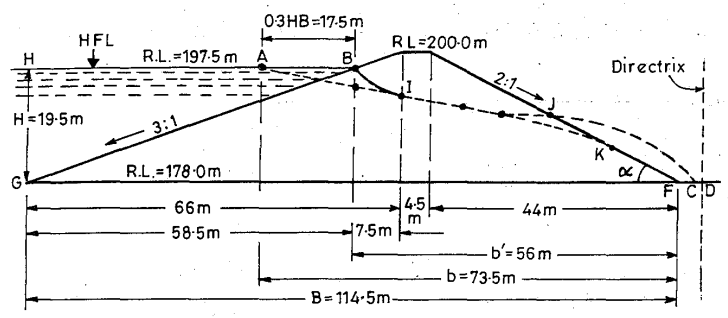


Fig. 20.17

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

$$AB \approx 0.3\, HB$$

or

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

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

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

or

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

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

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

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

or

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

Table 20.4

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

where $b' = 56$ m

$H = 19.5$ m

$\sin 26.56° = 0.447$

$\cos 26.54° = 0.894.$

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

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

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

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

Number of potential drops = *10*

Number of flow channels = *4.*

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

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

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

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

$H = 22 - 2 = 20$ m.

$N_f = 4$

$N_d = 10$

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

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

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

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

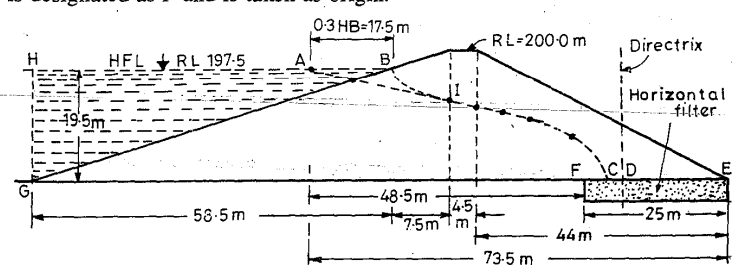


Fig. 20.18

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

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

will now be for the point A.

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

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

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

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

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

At point F, $\quad x=0$

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

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

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

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

Table 20.5

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

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

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

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

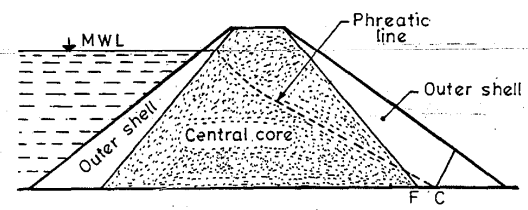


Fig. 20.19

20.13. Stability of Earthern Slopes

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

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

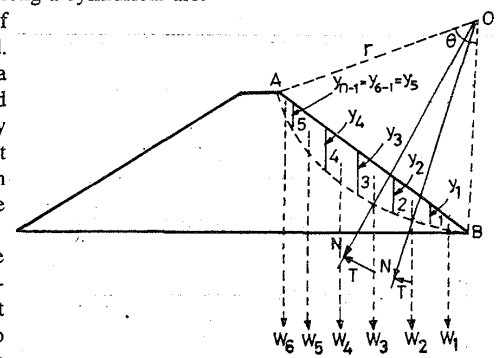


Fig. 20.20

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

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

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

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

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

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

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

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

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

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

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

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

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

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

The total resisting moment over the entire arc AB

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

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

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

where ΣN is the sum of all the normal components.

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

Length $\widehat{AB}$ of slip circle

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

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

Hence, the factor of safety against sliding

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

or

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

or

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

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

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

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

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

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

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

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

.....

.....

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

where n is the total number of slices.

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

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

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

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

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

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

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

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

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

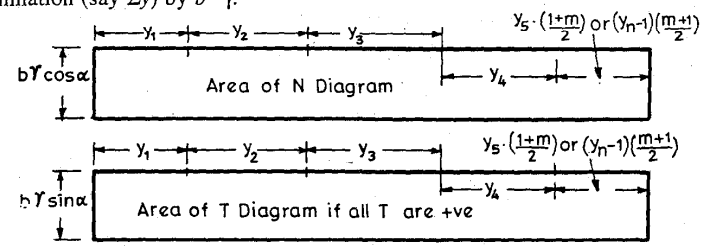


Fig. 20.21

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

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

Table 20.6

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

The F.S. is then calculated from equation

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

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

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

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

Table 20.7

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

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

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

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

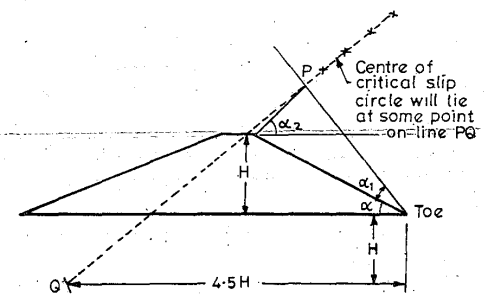


Fig. 20.22 (a). Locus of critical circle for d/s slope.

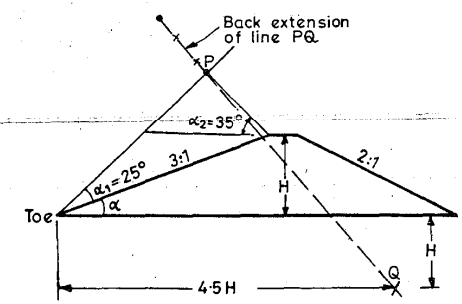


Fig. 20.22 (b) Locus of critical circle for u/s slope.

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

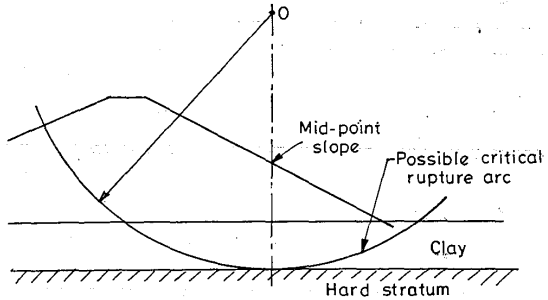


Fig. 20.23.

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

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

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

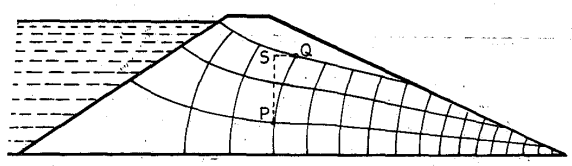


Fig. 20.24

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

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

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

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

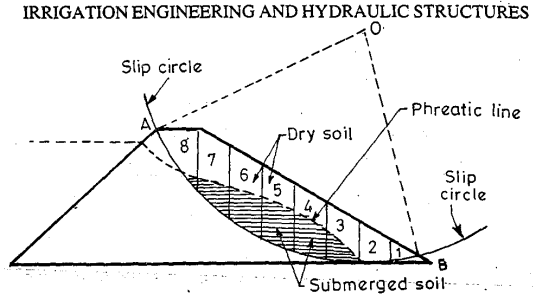


Fig. 20.25 (*a*)

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

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

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

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

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

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

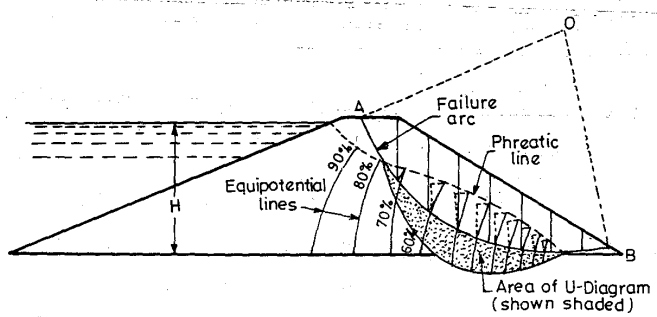


Fig. 20.25 (*b*)

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

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

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

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

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

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

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

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

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

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

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

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

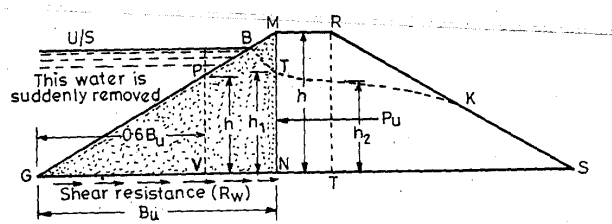


Fig. 20.26

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

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

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

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

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

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

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

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

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

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

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

In such a case,

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

or

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

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

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

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

It should be more than 1.5.

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

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

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

∴ Maximum shear stress induced

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

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

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

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

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

F.S. at the point of the maximum shear

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

It should be more than 1.

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

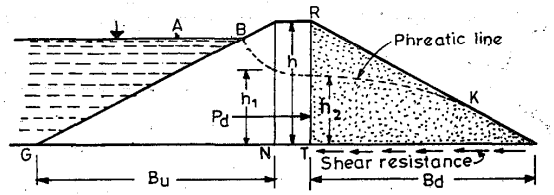


Fig. 20.27

With reference to Fig. 20.27.

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

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

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

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

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

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

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

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

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

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

or

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

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

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

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

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

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

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

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

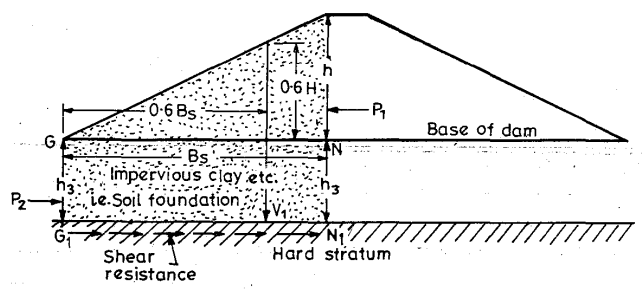


Fig. 20.28

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

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

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

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

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

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

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

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

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

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

Now the average shear stress at the base of the slope

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

where B_s is the base width below the slope.

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

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

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

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

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

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

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

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

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

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

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

The average shear resistance

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

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

This should be greater than 1.5.

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

The unit shear resistance at this point V_1

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

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

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

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

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

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

This should be greater than unity.

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

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

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

Solution.

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

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

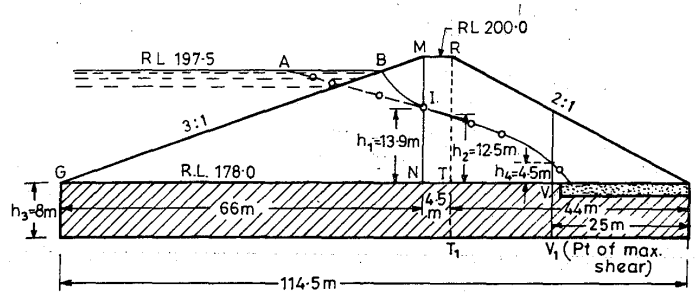


Fig. 20.29 (*a*)

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

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

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

Now

Weight of the dry portion of the dam section

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

Weight of the submerged portion of the dam section

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

Total weight of dam (called average weight)

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

Shear resistance of the dam at the base

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

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

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

B = Total base width = 114.5 m

$W \tan \phi = 19{,}180 \tan 25°$

$\therefore$ Shear resistance at base,

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

Horizontal force = Horizontal pressure of water.

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

Factor of safety against failure due to horizontal shear at base

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

The unit shear resistance developed at this point

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

$\therefore$ Area of submerged soil

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

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

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

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

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

Average shear induced at base

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

Maximum shear stress induced

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

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

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

This unit shear resistance developed at this point

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

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

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

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

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

Average compressive stress on foundation soil

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

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

Area of section of dam

$$= 1{,}409 \text{ sq. m (calculated earlier)}$$

Dry weight of dam section

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

Average compressive stress at base

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

Shear stress induced at base

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

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

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

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

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

where $h = 22$ m

$h_3 = 8$ m.

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

ϕ_1 is given by equation (20.41) as :

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

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

or $\quad \tan\phi_1 = 0.312$

or $\quad \phi_1 = 17.3°$

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

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

Average shear stress induced at base of d/s slope

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

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

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

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

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

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

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

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

Unit shear resistance τ_{f_2} at point T_1

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

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

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

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

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

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

Over all F.S. against shear

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

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

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

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

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

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

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

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

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

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

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

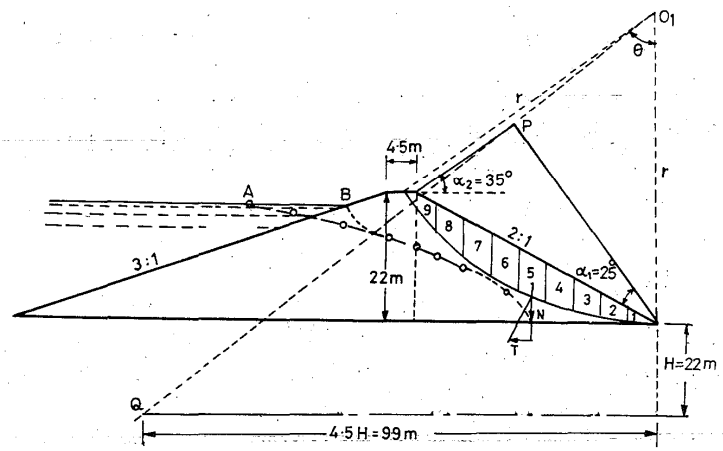


Fig. 20.29 (*b*)

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

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

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

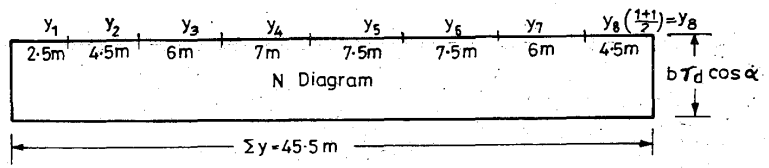


Fig. 20.30 (*a*). *N*-diagram.

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

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

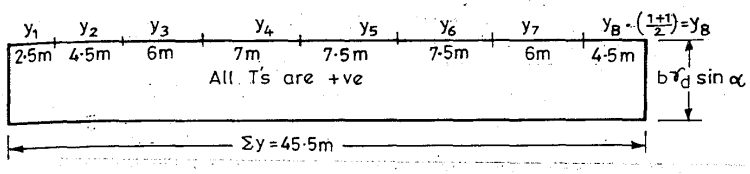


Fig. 20.30 (b). T-diagram.

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

Net F.S. = 1.15 Ans. < 1.5 (∴ Unsafe)

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

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

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

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

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

SEEPAGE CONTROL IN EARTH DAMS

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

20.14. Seepage Control Through Embankments

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

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

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

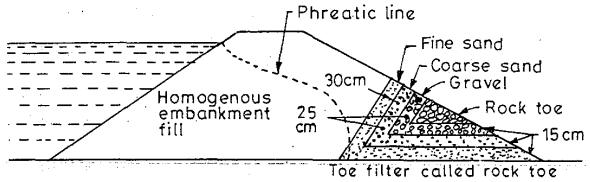


Fig. 20.31 (*a*). Rock Toe.

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

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

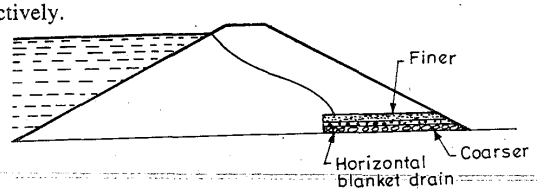


Fig. 20.31 (*b*). Horizontal Filter.

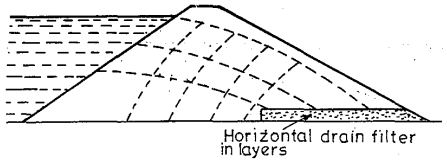


Fig. 20.31 (*c*). Inefficient 'Horizontal drain' in stratified embankments.

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

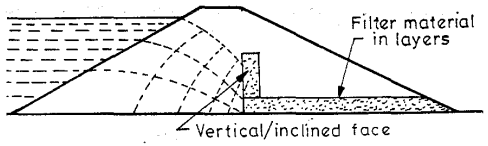


Fig. 20.31 (*d*). 'Chimney Drain' in Stratified Embankments.

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

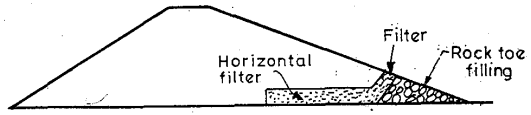


Fig. 20.31 (*e*). Horizontal filter combined with rock toe.

20.15. Seepage Control Through Foundations

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

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

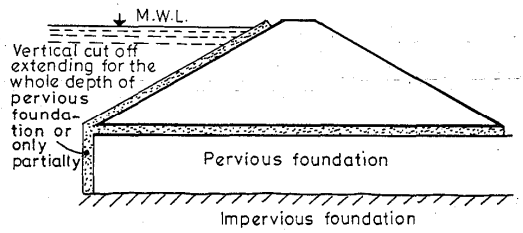


Fig. 20.32

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

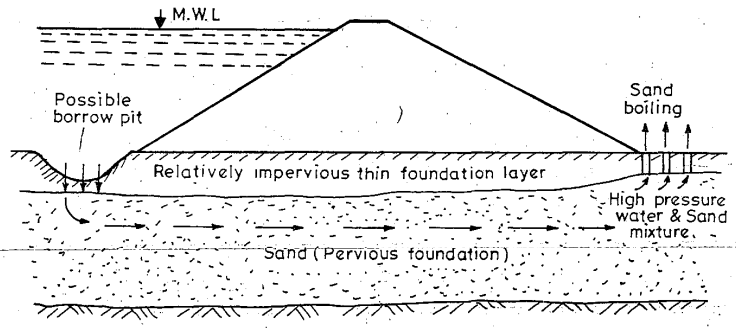


Fig. 20.33 (*a*). Sand Boiling Phenomenon.

Such a possibility, can be controlled by constructing relief wells or drain trenches through the upper impervious layer, as shown in Fig. 20.33 (*b*) and (*c*), so as to permit escape of seeping water. The possibility of sand boiling may also be controlled by

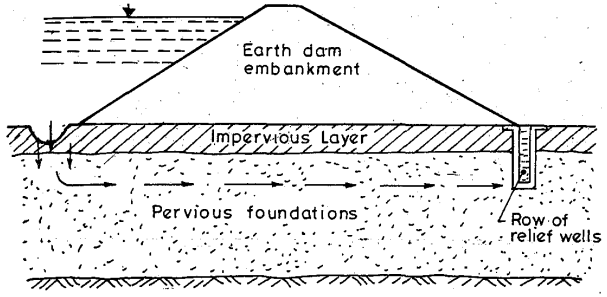


Fig. 20.33 (*b*). Provision of Relief Wells.

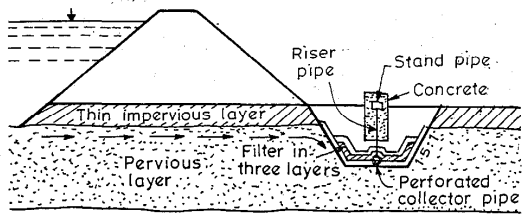


Fig. 20.33 (*c*). Enlarged View of Drain Trench.

providing d/s berms beyond the toe of the dam as shown in Fig. 20.33 (*d*). The weight of the overlying material, in such a case, is sufficient to resist the upward pressure and thus preventing the possibility of sand boiling. The provision of such berms, also protects the d/s toe from possible sloughing due to seepage.

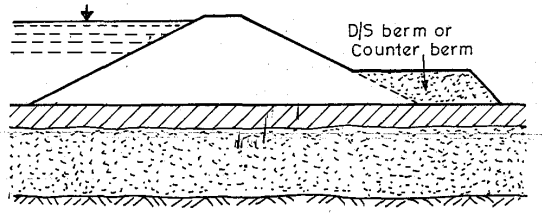


Fig. 20.33 (*d*). Provision of d/s Berms.

20.16. Design of Filters

The drainage filters must be designed in such a way that neither the embankment nor the foundation material can penetrate and clog the filters. The permeability or size of filter material should also be sufficient to carry the anticipated flow with an ample margin of safety. A rational approach to the design of filters has been provided by Terzaghi. According to him, the following filter criteria should be satisfied.

$$\frac{D_{15} \text{ of filter}}{D_{85} \text{ of base materials}} < 4 \text{ to } 5 < \frac{D_{15} \text{ of filter}}{D_{15} \text{ of base material}} \qquad \text{...(20.50)}$$

The embankment soil or the foundation soil surrounding the filter, is known as base material.

When the ratio of D_{15} of filter to D_{85} of base material does not exceed 4 to 5, base material is prevented from passing through the pores of the filter. Similarly, when the ratio of D_{15} of filter to D_{15} of base material is more than 5 (between 5 to 40), the seepage forces within the filter are controlled up to permissible small magnitudes.

Multilayered filters (generally 3 layers) consisting of materials of increasing permeabilities from the bottom to top are, many a times, provided and are known as **inverted filters.** These filters are costly and should be avoided where possible. The minimum total thickness of filter is 1 m. However, if sufficient quantities of filter material are available at reasonable costs, thicker layers of filter may be provided. The thicker the layer, the greater the permissible deviation from the filter requirements.

20.17. Slope Protection

20.17.1. Protection of Upstream Slope. The upstream slope of the earth dam is protected against the erosive action of waves by stone pitching or by stone dumping, as shown in Fig. 20.34. The thickness of the dumped rock should be about I metre and should be placed over a gravel filter of about 0.3 m thickness. The filter prevents the washing of fines from the dam into the rip-rap. The provision of such a dumped rip-rap has been found to be most effective and has been found to fail only in 5% cases.

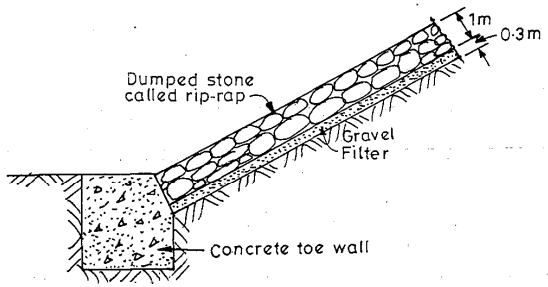


Fig. 20.34

The stone pitching, *i.e.* the hand packed rip rap requires a lesser thickness and may prove more economical if suitable rock is available only in limited quantity. However, when provided in smaller thickness (*i.e.* single layer), it is more susceptible to damage and has been founded to fail in about 30% of cases.

Concrete slabs may also be laid over the u/s slope of the earth dam. When such slabs are constructed, they must be laid over a filter and weep holes should be provided so as to permit escape of water when the reservoir is drawn down. If the filter is not provided, the fines from the embankment may get washed away from the joints creating hollows beneath the slab and causing the consequent cracking and failure of the slab under its own weight. Concrete slab protections have been found to fail in about 36% cases, mainly because of non-providing of filter below them.

20.17.2. Protection of Downstream Slope. The downstream slope of the earthen dam is protected against the erosive action of waves upto and slightly above the water depth, in a similar manner as is explained above for u/s slope.

Moreover, the d/s slope should be protected against the erosive action of rain and its run-off by providing horizontal berms at suitable intervals say about 15 m or so (Fig. 20.5) so as to intercept the rain water and discharge it safely. Attempts should also be made so as to grass and plant the d/s slopes, soon after their construction.

20.18. Rockfill Dams

Rockfill dams have characteristics lying somewhere between the characteristics of gravity dams and those of earthen dams. In other words, they are less flexible than earthen dams and more flexible, then gravity dams. Their foundation requirements are not as strict and rigid as are required for gravity dams. But the foundation requirements

are more rigid than those for earthen dams which can be constructed almost on any types of foundations. The steeper slopes are used in rockfill dams and hence, the base width is quite less. The smaller base width and the possibility of large scale seepage restricts the foundation requirements of such dams.

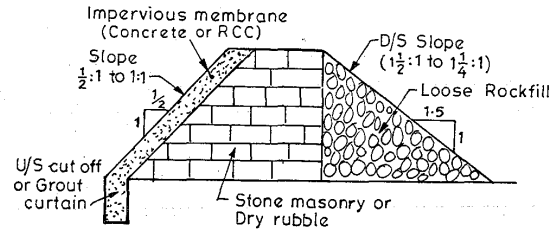


Fig. 20.35. Section of a rockfill dam.

A typical section of a rockfill dams is shown in Fig. 20.35. It essentially consists of an impervious membrane and embankment supporting the membrane. The embankment is divided into two portions. The u/s portion is made of stone masonry or dry rubble, and the d/s portion is made of loose rockfill. The u/s portion embankment supports the membrane and the water load : while the d/s portion supports the u/s embankment, membrane and waterload.

The impervious membrane is usually of concrete, with expansion joints at suitable intervals (say 10 m or so). The expansion joints may be filled with suitable bitumen filler to minimise leakage. Sometimes, R.C.C. membranes are provided with sufficient horizontal and vertical reinforcements, without any expansion joints. The slab thickness is generally less at the top and is more near the dam bottom. The usual slab thickness is between 15 to 50 cm, depending upon the design.

The upstream face containing the membrance, is sloped at a slope varying from say $\frac{1}{2}$: 1 ($\frac{1}{2}H$: 1V) in low dams up to height of about 60 m or so, to 1 : 1 or $1\frac{1}{2}$: 1 in high dams. The d/s slope of all the rockfill dams is kept at about 1.3 : 1, which represents the angle of repose of the rockfill.

Rockfill dams are generally cheaper than concrete dams and can be constructed rapidly if proper rock is available. The rock must be strong enough to withstand high intensity leadings even when wet. The size of loose rock may vary from small stones to 3 m or so. Rockfill dams are very useful in seismic regions, as they provide high resistance to seismic forces because of their flexible character. However, rockfill dams are liable to large settlements, which may lead to cracking of concrete membrane. Repairs for the membrane are, therefore, undertaken from time to time, as and when the necessity arises.

PROBLEMS

1. (*a*) What are 'earthen dams' and under what circumstances are they preferred ?

(*b*) Enumerate the different types of earthen dams, and draw neat sketches showing each type.

[**Hint** : See article 20.2]

(*c*) Enumerate the two different methods which are adopted for constructing earthen dams. Which of these methods would you prefer and why ? [**Hint** : See article 20.3, preferring rolled fill method]

2. (*a*) What is meant by 'pore water pressure' : and what is its significance in the design of earthen dams?

(*b*) What are 'rockfill dams' and what are their advantages over earthen dams. Draw a neat sketch showing the cross section of a rockfill dam.

3. (*a*) Explain in detail the various forces causing instability in a gravity dam.

(*Madras University, 1975*)

(*b*) Draw a section of an earth dam of 20 m height indicating the various parts of the dam.

(*Madras University, 1975*)

4. (*a*) What are the different types of earth dams that are usually adopted. State where each type is adopted ?

(*b*) What are the causes of failures of earth dam ? (*Madras University, 1974*)

5. What are the precautions that you would take while constructing an earth dam ?

Explain the Swedish slip circle method of analysing the stability of an earth dam slopes.

(*Madras University, 1976*)

6. (*a*) Draw a neat section of an earth dam for the following site :

(*b*) Both silty clay and coarse sand are available at site. Hard stratum is available at about 5 m below the natural ground.

(*c*) Explain briefly how the stability of earthen slopes are checked by the slip circle method.

(*Madras University, 1973*)

7. (*a*) Differentiate between 'Rigid dams' and 'Non-rigid dams' giving examples of each type.

(*b*) Explain with neat sketches how you would carry out the stability analysis of an earth dam.

(*U.P.S.C., 1974*)

8. (*a*) Differentiate between 'horizontal' and 'vertical' piping in earth dams. Suggest permanent measures to check vertical piping.

(*b*) Show with the help of simple outline sketches suitable broad designs of earth dams for different available materials and the governing geology of the site.

Note. At least four different designs should be given.

9. (*a*) Give a suitable design for a 50 m high dam for site where both clay silt and sand gravel are available in plenty and where foundation is pervious to a depth of 10 m. Assume suitable data. Give reasons favouring the suggested design.

10. (*a*) Briefly discuss the checks that are required to be made to investigate the stability of an earthen dam. (*U.P.S.C., 1975*)

(*b*) An earthen dam has to be constructed to store a maximum depth of 12 m of water over river bed consisting of coarse sand and gravel up to a depth of 3 m below river bed followed thereafter by bard and sound rock. Clayee soil is available in plenty in the vicinity of the river. Draw and detail a suitable section of the dam at the river bed.

11. Explain how the following parameters affect design of an earth dam :

(*i*) optimum moisture content ;

(*ii*) C and ϕ value of soil ; permeability of soil

(*iii*) sudden draw-down of the reservoir.

Illustrate with neat sketches the following parts of an earthen dam and state their functions briefly:

(*i*) Rock toe ;

(*ii*) horizontal drainage blanket ;

(*iii*) cut-off ;

(*iv*) Rip-rap.

12. Enumerate and explain by neat sketches the different ways by which the earthen dams may fail. Also suggest suitable precautions that should be undertaken to avoid each type of failure.

13. (*a*) Explain the meaning and importance of equipotential lines' and 'stream lines' in connection with seepage analysis of earthen dams.

(*b*) Derive an expression for calculating the seepage discharge through earthen dam bodies made of isotropic soils. What correction factor is required to be multiplied with this expression, if the dam soil is non-isotropic.

14. (*a*) Define and explain the term 'phreatic line' in earthen dams.

(*b*) How would you procced to determine the phreatic line through homogeneous earthen dams provided :

(*i*) with a horizontal filter ;

(*ii*) Without a horizontal filter.

15. (*a*) Explain and elaborate the importance of 'seepage' through earthen dams.

(*b*) What precautions and remedial measures would you undertake to control the 'seepage' through

(*i*) earthen dam body ;

(*ii*) through the dam foundation.

16. Write short notes on any five of the followings :

(*i*) Rock toe

(*ii*) Climney drain

(*iii*) Relief wells

(*iv*) Slope protection in earthen dams

(*v*) Rock fill dams

(*vi*) Consolidation of earthen dams

(*vii*) Pore pressure and its significance in relation to earthen dam construction

(*viii*) 'Seepage failures' of earthen dams

(*ix*) Rational design of drainage filters for earthen dams.

17. A section of a homogeneous earth dam is shown in Fig. 20.36. Calculate the seepage discharge per metre length, through the body of the dam. The coefficient of permeability of the dam material may be taken as 8×10^{-5} m/sec.

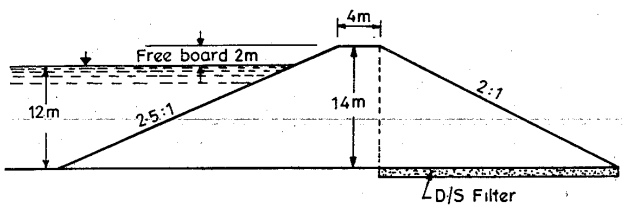


Fig. 20.36

[**Ans.** 2.88×10^{-1} cumecs/m]

18. An earthen dam made of homogeneous material has the following data :

(A) Hydraulic data of dam

Level of top of dam	= 210 m.
Level of deepest river bed	= 192 m.
HFL of reservoir	= 208 m.
Width of top of dam	= 10 m.
Upstream slope	= 3 : 1
Downstream slope	= 2.5 : 1
Length of horizontal filter from d/s toe inwards	= 16 m.

(B) Properties of the material of the dam

Dry density	= 18 kN/m^3
Saturation density	= 21 kN/m^3
Average angle of friction	= 30°
Average cohesion	= 16 kN/m^2.

(C) The foundation soil consists of 4 m thick layer of soil having the following properties :

Average unit weight $= 17\ kN/m^3$

Average cohesion $= 54\ kN/m^2$.

Average angle of internal friction $= 7°$

Check the dam section for the following :

(*i*) Sloughing of u/s slope during sudden drawdown.

(*ii*) Stability of foundation against shear.

[**Hint.** Follow example 20.4]

19. In order to determine the factor of safety of the d/s slope during steady seepage, the section of a homogeneous earth dam was drawn to scale of 1 cm = 10 m ; and the following results were obtained on a trial slip circle.

Area of N-diagram = 12.15 sq cm.

Area of T-diagram = 6.50 sq cm.

Area of U-diagram = 4.02 sq cm.

Length of arc = 11.60 cm.

The dam material has the following properties :

Effective angle of internal friction $= 26°$

Unit of cohesion $= 19.62\ m^2$

Unit weight of soil $= 19.62\ m^2$

Determine the factor of safety of the slope. [**Ans.** F.S. 1.21]