Intensity of the earthquake

The intensity of an earthquake at a place is a measure of the strength of shaking during earthquake and is indicated by a number according to the modified mercalle scale or M.S.K. scale of seismic intensities.

(5) ICE pressure:—

The ice pressure is more important for dams constructed in cold countries. The ice formed on the water surface of the reservoir is subjected to expansion and contraction due to temparature variations. The ~~coefficient~~ of Thermal expansion of ice being five times ~~more than~~ that of concrete, the dam face has to resist the force due to expansion of ice. This force acts linearly along the length of the dam. at the reservior level.

(6) Wave pressure:—

Waves are generated on the reservior surface because the wind blowing over it. Wave pressure depends on the height of the waves developed. Wave height may be calculated from the following formula

$$h_w = 0.0322\sqrt{V.F} + 0.763 - 0.271(F)^{1/4} \quad \text{for } F < 32\,km$$

$$h_w = 0.0322\sqrt{V.F} \quad \text{for } F > 32\,km.$$

where h = height of waves in metres between Trough and crest.

V = wind velocity in km per hour

F = Fetch or straight length of water expanse in km.

(7) Silt pressure :—

The river brings debris and silts along with it. The silt load gets deposited to an appreciable extent when dam is constructed. If γ_s' is the submerged unit weight of silt and ϕ is the angle of internal friction and h_s is the height to which the silt is deposited.

The silt pressure is given by $P_s = \frac{1}{2} \gamma_s' h_s^2 \frac{1-\sin\phi}{1+\sin\phi}$

If the upstream face is inclined the vertical weight of silt supported on the slope also act as vertical force.

(8) Wind pressure :—

It is a minor force and need hardly be taken into account for the design of dams. Wind pressure is required to be considered only on that portion of the super structure which is exposed on the action of wind. Normally wind pressure is taken as 1 to 1.5 kN/m^2 for the area exposed to the wind pressure.

(*) Modes of failures : Stability Requirements

Following are the modes of failures of Gravity Dams

(1) overturning (2) sliding (3) compression or crushing (4) Tension

1. **overturning** :— The overturning of the dam section takes place when the resultant force at any section cuts the base of the dam downstream of the toe. In that

case the resultant moment at the Toe becomes clockwise or.-ve). on the other hand if the resultant cuts the base within the body of the dam. There will be no overturning.

For stability requirements the dam must safe against overturning. The factor of safty against overturning is defined as the ratio of the resultant movement (+ve) of the overturning moments i.e

$$F.S = \frac{\Sigma \text{ Righting moments}}{\Sigma \text{ overturning moments}} = \frac{\Sigma M_R}{\Sigma M_o}$$

The factor of safty against overturning should not be less than 1.5.

② Sliding :— A dam will fail in sliding at its base or at any other level if horizontal forces causing sliding are more than the resistant available at that level. The horizontal resistance against sliding may due to friction alone, or due to friction and shear strength of the joint. Shear strength develops at the base if benced foundation are provided and at other joints are carefully bid so that a good bond develops.

The factor of safty against sliding is defined as the ratio of actual coefficient of static friction. the horizontal Joint to the sliding friction.

$$S.F = \tan\theta = \frac{\Sigma H}{\Sigma V}$$

and factor of safty against sliding (F.S.S) is

$$F.S.S = \frac{\mu}{\tan\theta} = \frac{\mu \Sigma V}{\Sigma H}$$

The coefficient of friction μ varies from 0.65 to 0.75.

③ **Compression or crushing**

In order to calculate the normal stress distribution at the base or at any section, let H be the total Horizontal force, V be the Total vertical force, & R be the resultant force cutting the base at an eccentricity e from the centre of the base width b.

The normal stress at any point on the base will be sum of the direct stress and bending stress.

Thus, direct stress $= \frac{V}{b \times 1}$

Bending stress $= \pm \frac{M.y}{I} = \pm \frac{V.e}{1/6\, b^2} = \pm \frac{6.V.e}{b^2}$

Hence the Total normal stress P_n is given by

$$P_n = \frac{V}{b}\left[1 \pm \frac{6e}{b}\right]$$

The positive sign will be used for calculating normal stress at the Toe, since the bending stress will be compressive force and negative sign will be used for calculating normal stress at heel.

Thus, the normal stress at the Toe is

$$(P_n)_{Toe} = \frac{V}{b}\left[1 + \frac{6e}{b}\right]$$

The normal stress at the heel is

$$(P_n)_{heel} = \frac{V}{b}\left[1 - \frac{6e}{b}\right]$$

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(4) Tension: —

The normal stress at heel is

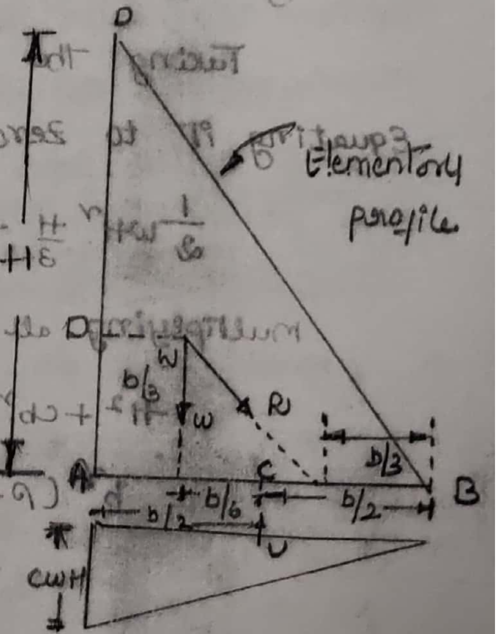
$$(P_n)_{heel} = \frac{V}{b}\left[1 - \frac{6e}{b}\right]$$

It is evident that if $e > b/6$ the normal stress at heel will be -ve or Tensile.

No Tension should be permitted at any point of the dam under any circumstances for moderately high dams. For no Tension develop, The ecentricity should be less than $b/6$. In other words the resultant should always lie within the middle third.

(*) Elementary profile of Gravity Dam: —

In the absence of any force other than the force due to water an elementary profile will be triangular in section having zero width at the water level at the top, where water pressure is zero and maximum base width b, where maximum water pressure acts. Thus the section of the elementary profile is of the same shape as the hydrostatic pressure distribution diagram. For Reservior empty condition a right angled Traingular profile as shown in fig will provide the maximum profile



Elementory profile

we shall Consider the following forces acting on the elementary profile of a gravity dam.

(1) weight of the dam (W)

$$W = \tfrac{1}{2}\, bH . \rho . w$$

ρ = specific Gravity of dam material ; w = unit weight of water

② water pressure (P)

$P = \frac{1}{2} wH^2$ acting at $\frac{1}{3}H$ from base

③ uplift pressure (U)

$U = \frac{1}{2} c.w.b.H.$

c = uplift pressure intensity Coefficient.

Base width of elementary profile :-

The base width of the elementary profile is to be found under Two criteria.

① Stress Criterion: when the reservior is empty for no Tension to develop. the resultant should act at the inner third point (M_1). For the reservior full Condition for no Tension to develop, the resultant R must pass Through the outer third point (M_2).

Taking the moment of all forces about M_2 and Equating it to zero we get

$$\frac{1}{2} wH^2 . \frac{H}{3} + \frac{1}{2} c.w.b . \frac{b}{3} H - \frac{1}{2} b H \rho w . \frac{b}{3} = 0$$

multiplying all Terms by $\frac{6}{wH}$ we get

$$H^2 + cb^2 - b^2 \rho = 0$$

$$b^2 (\rho - c) = H^2$$

$$b = \frac{H}{\sqrt{\rho - c}}$$

Limiting Height of a Gravity dam :-

The principal stress at the Toe is given by

$$\sigma_1 = wH(\rho - C + 1)$$

In this expression the only variable, changing the value of σ_1 is H. The maximum value of this principal stress should not exceed the allowable stress (f). for the material.

In the limiting case

$$f = \sigma_1 = wH(\rho - C + 1)$$

From which the height H is given by

$$H = \frac{f}{w(\rho - C + 1)}$$

For finding the limiting H. but it is usual not to consider the uplift. Hence putting C = 0 we get.

$$H = \frac{f}{w(\rho + 1)}$$

If the height of the dam is more than that given by Eq. The maximum compressive stress will exceeds the permissible stress

A low gravity dam is the one in which the height defines H is less then that given by so that maximum compressive stress is not greater then the allowable stress. For general case. Taking $w = 9.81\ kN/m^3$ and $\rho = 2.40$. the limiting height in metres is given by

$H = 0.03f$ where f is the allowable stress in kN/m^2

$f = 2940\ kN/m^2$

$$H = \frac{2940}{9.81(2.4+1)} = 88\ m.$$

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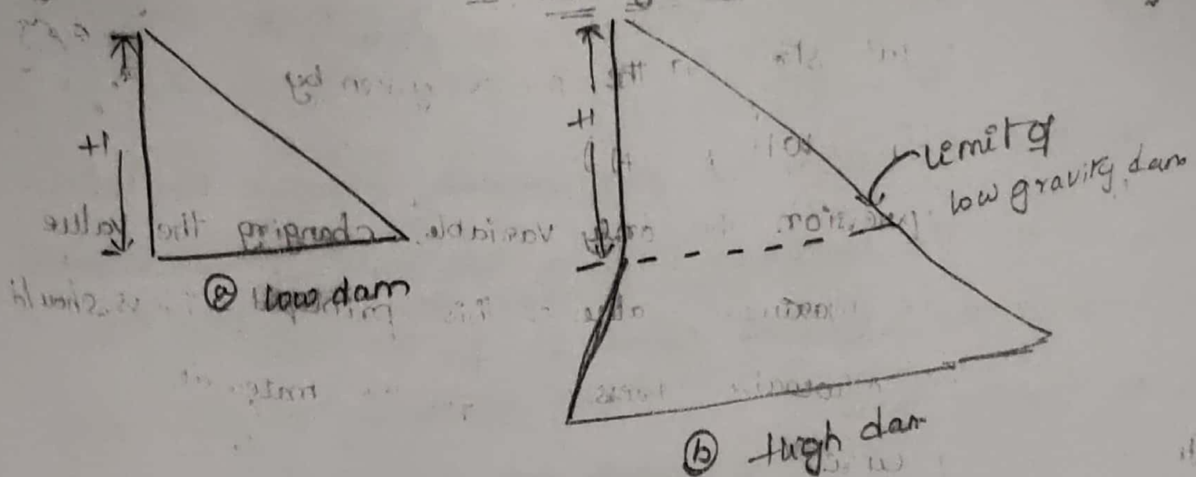


High & Low Gravity Dam.

⊛ Drainage Galleries :—

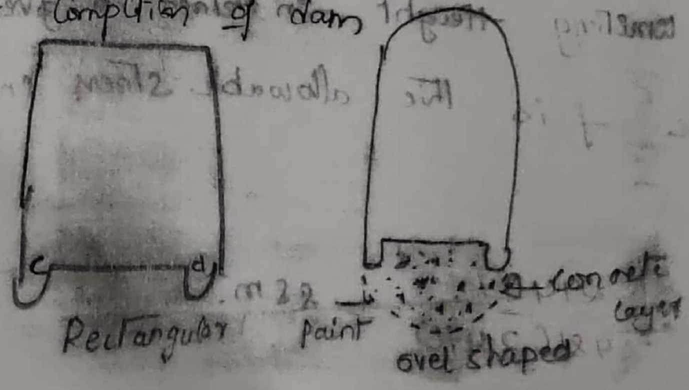
A gallery is a formed opening left in a dam. This may run in Transverse or longitudinal direction and may run horizontally or on a slope. The shape and size varies from dam to dam. and is generally governed by the fluctuations it has to perform.

Following are the purpose for which a gallery is formed on a Gravity dam

① To provide drainage of the dam section. Some amount of water constantly seeps through the upstream face of the dam

2. To provide facilities for drilling and grouting operations for foundations etc. Drilling for drains is generally resorted to clean them if they are clogged.

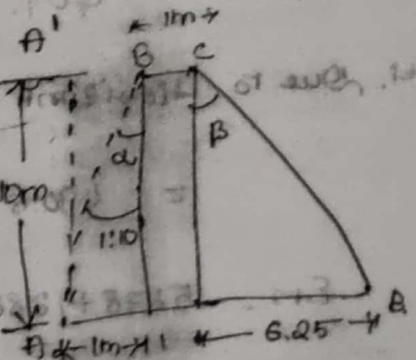
③ To provide space for header and return pipes for post cooling of concrete and grouting the longitudinal Joints after completion of dam



3rd unit Gravity dams

1) A masonry dam 10m high is Trapezoidal in section with a top width of 1m and bottom width 8.25m. The face is exposed to water has a batter of 1:10. Test the stability of dam. Find the principal stress at the Toe and heel of the dam. Assume unit weight of masonry as 22.4 KN/m². w for water = 9.81 KN/m³ and permissible shear stress of Joint = 1400 KN/m².

Sol Consider, 1m length of the dam



1 vertical forces :—

a. Self weight of the dam

$$\frac{a+b}{2} \times H \times \text{unit length} \times 22.4$$

$$= \frac{8.25+1}{2} \times 10 \times 1 \times 22.4$$

$$= 1036 KN$$

(b) weight of water in Column $AA'B$

$$= \frac{10 \times 1}{2} \times 9.81 = 49.05 KN.$$

(c) uplift force $= \frac{1}{2} BHCw$

$$= \frac{1}{2} \times 8.25 \times 10 \times 9.81 = 404.66 KN$$

$$EV = 1036 + 49.05 - 404.66 = 680.39 KN$$

(2) Horizontal water pressure :—

$$EH = \frac{wh^2}{2} = \frac{9.81 \times 10^2}{2} = 490.05 KN.$$

Moment due to various forces @ Toe :—

(a) Due to self weight of Dam

Moment $= \left(\left(\frac{1}{2} \times 1 \times 10 \times 22.4\right) \times 7.2\right.$

$+ (1 \times 10 \times 22.4)(6.25 + 0.5) + \frac{1}{2} \times 6.25 \times 10 \times 22.4)(\frac{2}{3} 6.25)$

$= 5278$ KN-m

2. Due to column of water in AA'B

Moment $= \frac{1}{2} \times 10 \times 1 \times 9.81 \times (8.25 - 1/3) = 388.31$ KN-m

3. Due to uplift force

Moment $= 404.66 \times 2/3 \times 8.25 = 2225.63$ KN.m

4. Due to Horizontal water pressure.

$= 490.5 \times 10/3 = 1635$ kN/m.

$\Sigma M = -5278 + 388.31 - 2225.63 + 1635$

$= 1805.68$ KN-m

Calculation of factor of safety

Factor of safety against overturning

$= \frac{\Sigma M_R}{\Sigma M_O} = \frac{(+) M}{-(M)} = \frac{5668.31}{3860.63} = 1.47 < 1.5$ unsafe

Factor of safety against overturning

$= \frac{\mu \Sigma V}{\Sigma H} \quad \mu = 0.75. \quad = \frac{0.75 \times 680.39}{490.5} = 1.04 > 1$

Shear friction factor $= \frac{\mu \Sigma V + bS}{\Sigma H} = \frac{0.75 \times 680.39 + 8.25 \times 1400}{490.5}$

$= 24.6$

Calculation of Stress :—

The resultant acts at a distance x from toe.

$x = \frac{\Sigma M}{\Sigma V} = \frac{1805.68}{680.39} = 2.65$m

~~Calculation of~~

Its distance from Centre is $e = b/2 - \bar{x}$

$$= \frac{8.25}{2} - 2.65 = 1.475\ m$$

Compressive stress at Toe is

$$P_n = \frac{\Sigma V}{b}\left[1+\frac{6e}{b}\right] = \frac{680.39}{8.25} + \left[\frac{1+6(1.475)}{8.25}\right]$$

$$= 167.8\ kN/m^2\ (+ve)$$

Compressive stress at heel

$$\frac{\Sigma V}{b}\left(1-\frac{6e}{b}\right) = \frac{680.39}{8.25}\left[1-\frac{6(1.475)}{8.25}\right]$$

$$= -2.9\ kN/m^2 \quad \text{ie Tension}$$

From fig. :-

$$\tan\alpha = \frac{1}{10} \qquad \tan\beta = \frac{6.25}{10}$$

$$\sec\alpha = \sqrt{1.01} \qquad \sec\beta = \sqrt{1.391}$$

Principle stress at Toe of dam $= P_n \sec^2\beta = 167.8 \times 1.391$

$$= 233.41\ kN/m^2$$

Principle stress at heel

$$P_n \sec^2\alpha - P \tan^2\alpha = -2.91 \times 1.01 - (0.01)$$

$$= -2.91 \times 1.01 - 9.81 \times 10 \times 0.01 = -3.91\ kN/m^2$$

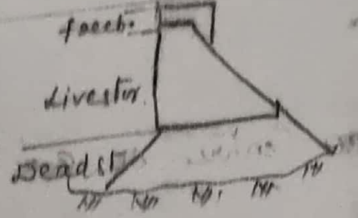
shear stress at Toe $= \tau = P_n \tan\beta = \frac{167.8 \times 6.25}{10}$

$$= 104.9\ kN/m^2$$

shear stress at heel $= -[P_n - P]\tan\alpha$

$$= -(-2.91 - 9.81) \times 1/10 = 10.1\ kN/m^2$$

General Terms.



1. High flood level:- The maximum water Level that attains during flood is called high flood Level. The dam and spillway sections are designed to withstand the water pressure at this level is called as "Maximum water Level".

2. Full Reservior level :— It is also Called as full Tank Level (F.T.L). It is the Level upto which the water is stored. obviously the crest of the spillway is fixed at this Level.

3. Free Board:- To prevent the over topping of the dam during peak floods a sufficient margin is provided between the maximum water Level in the reservior and top of the dam.

① Gross free Board :- It is the difference of level b/w F.R.L and Top of the dam.

② Net free Board:- It is the difference of Level b/w M.W.L and top of the dam.

④ Dead storage:- It is the part of the stored water in the reservior Basin which is not available for use and hence Termed as Dead storage. The Capacity of dead storage is so fixed that it Can allow the silting for about 100 years without reduction in the effective storage.

⑤ Live storage It is also Called the available or Effective storage. It is difference b/w. the Gross storage and Dead storage

Live storage = Gross storage — Dead storage

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EARTH DAMS

Earth dams have been built since the early days of civilisation. Today as in the past, the earth dam constitutes to be the most common type of dams since it is generally built of locally available materials in their natural state, with a minimum of processing.

Depending upon the method of construction earth dams can be divided into two categories.

① Rolled fill dam ② Hydraulic filled dam.

In the rolled fill dam, the embankment is constructed in successive, mechanically compacted layers. The suitable material are transported from the borrow pits to the construction site by earth moving machinery.

In hydraulic filled dam, the material are excavated, transported and placed by hydraulic methods. Flumes are laid at a suitable falling gradient along the outer edge of the embankment.

Rolled fill dams can further be divided into

① Homogeneous embankment type

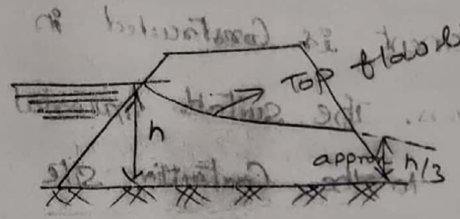
② Zoned embankment type

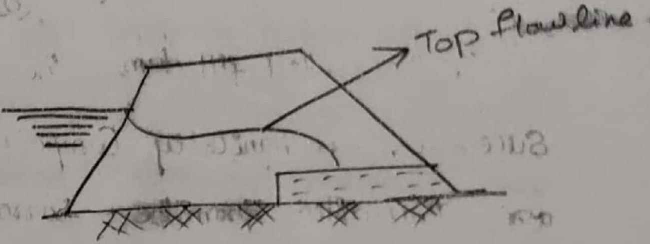
③ Diaphragm embankment type.

1. Homogeneous embankment type :—

A purely homogeneous earth dam is composed of single kind of material. Homogeneous dams have been built since the earlyst times and are used today, whenever only one type of material is economically available. However they are only for low to moderate heights. A purely homogeneous section has been replaced by a modified homogeneous section.

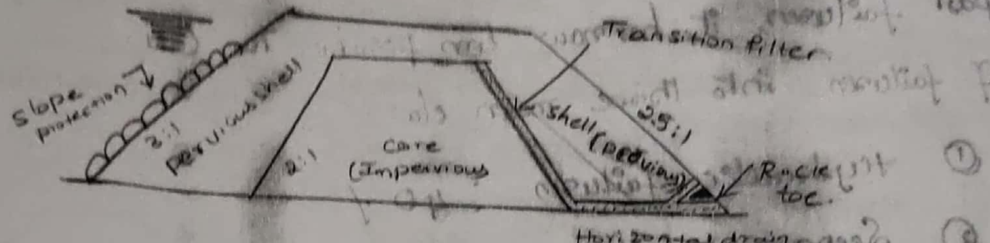
Homogeneous dams are usually composed of impervious or semi impervious soils to provide an adequate barrier. The upstream slope has to be flatter to make it safe during the sudden drawdown condition. Many successful embankments have been built to relatively pervious sands and sand gravel mixture.





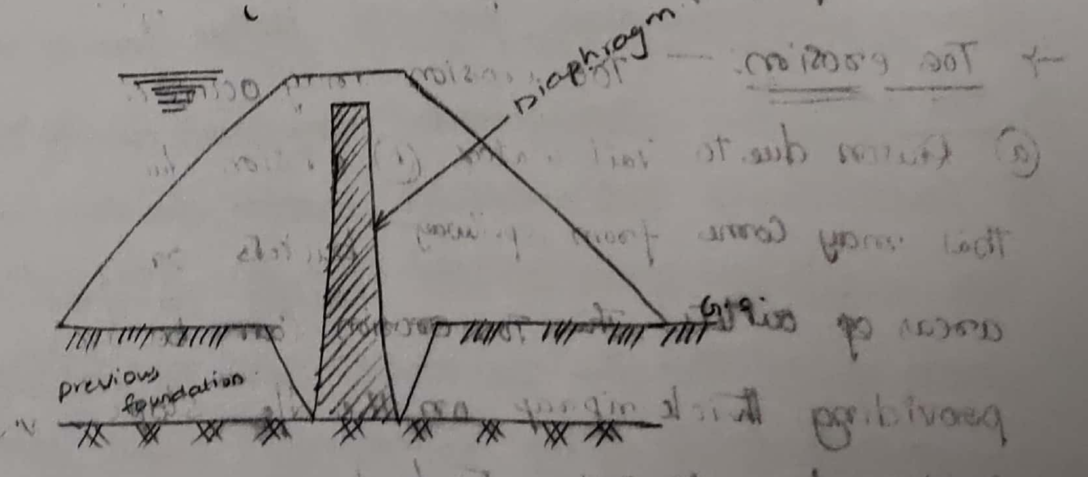
② Zoned embankment Type:

Zoned embankment Type earth dam is the one in which the dam is made up of more than one material. The most common of rolled dam section is that in which a central impervious core is flanked by zones of material considerably more pervious. A suitable drainage system in the form of a horizontal drain or a rock toe is also provided at the d/s side. If a variety of soils are readily available, the choice of type of earthfill dam should always be zoned embankment type because the inherent advantages will lead to economical in the cost of construction



(3) Diaphragm embankment Type:-

This is a modification over the the homogeneous embankment Type, in which the bulk of the embankment is constructed is constructed of pervious material and a thin diaphram of impermeable material is provided to check the seepage. The diaphram may be impervious soil, cement concrete bituminous concrete or any other material, and may be placed at the centre of the section as a central vertical core or at the upstream face as blanket. However the distinction between a diaphragm type and zoned type must be clearly known.



Causes of failures of earth dam

On the basis of investigation reports on most of the past failures, it has now been possible to catagorise the types of failures into three main classes.

1. Hydraulic failures : 40 %.
2. Seepage failures : 30 %.
3. ~~overtop~~ Structural failures : 30 %.

Hydralic failures: — Hydraulic failures include the following
① overtopping ② wave erosion ③ Toe erosion ④ Gullying

→ Overtopping: — The earth dam may get overtopped if the design flood is underestimated or, if the spillway is of insufficient capacity. Faulty operation of spillway gates etc. may also some times lead to overtopping. Insufficient free board or Settlement of foundation and embankment may also lead to overtopping

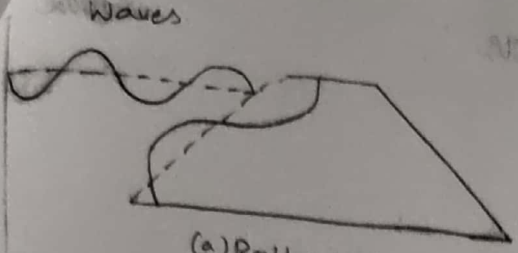
→ Wave erosion. — Fig shows the wave erosion. The effects of wave is to notch out earth from the upstream slope in absence of proper slope protection in the form of riprap

→ Toe erosion: — Toe erosion may occur due to two reasons ⓐ Reason due to Tail water ⓑ erosion due to cross currents that may come from spillway buckets or from exist areas of outlets. The Toe erosion can be avoided by providing thick riprap on the d/s slope, upto a height slightey above the Tail water level.

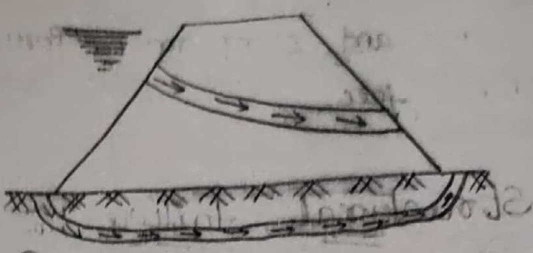
→ Gullying: — downstream slope may fail due to the formation of gullies by heavy downpour. To eliminate failure due to gullying proper berms, Turfing and good drainage.

Wave erosion

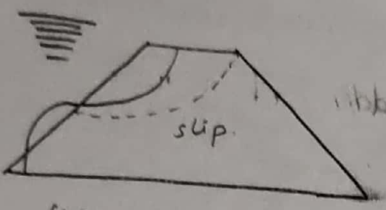
Piping Failures



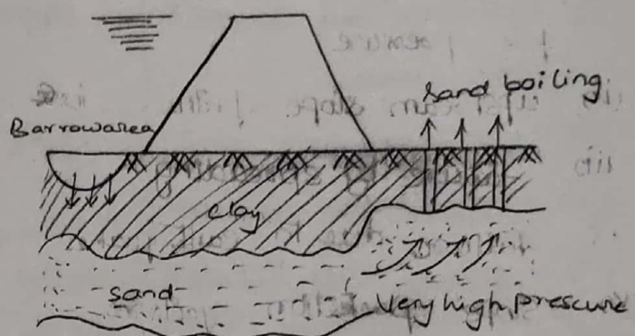
(a) Roller action



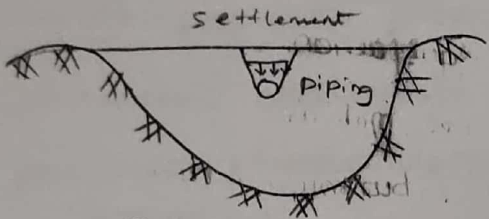
(a) piping (or) seepage erosion through dam and through foundation.



(b) upstream slip



(b) Piping due to ~~excess~~ exit gradient and due to removal of u/s blanket ~~near the~~ dam



(c) settlement due to piping

2 Seepage failures :—

Seepage failures may be due to (a) piping , (b) sloughing.

1. piping — The seepage of water through the body and foundation of the earth dam may lead to piping or progressive erosion of concentrated leaks, causing a large number of catastophic failures. water seeping through the earth dam may have four bad effects. (i) seepage water generates erosive forces which dislodge particles from soil structure.

(ii) The flow with its associated pore pressure.

(2) sloughing :—

Failure due to progressive sloughing or ravelling is closely related to piping. under the full reservior condition. the downstream toe remains saturated and may erode, producing a small slump or miniature slide. This miniature slide always leaves a steep face, which becomes saturated by seepage from the reservior and slump again. forming a slightly higher and more unstable face.

③ Structural failure :—

(i) upstream and downstream slope failures due to construction pore pressure.

(ii) upstream slope failure due to sudden drawdown.

(iii) failure by spreading

(iv) failure due to earthquake.

(v) slope protection failures.

(vi) Foundation slide : spontaneous liquifaction.

(vii) Damage caused by water soluble materials.

(viii) Failure due to damage caused by burrowing animals.

✱ Stability Analysis

① Stability of downstream slope during steady seepage.

② Stability of upstream slope during sudden drawdown.

①. Stability of downstream slope during steady seepage :—

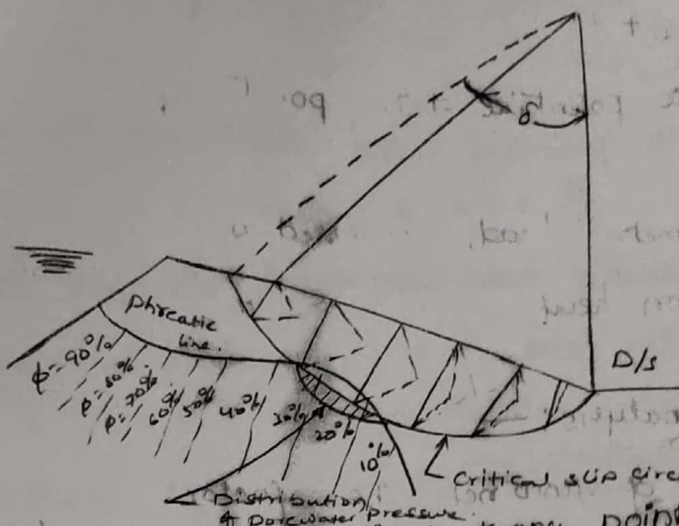
critical condition for d/s slope occurs when slope occurs when the reservior is full and percolation is at its The direction of seepage forces trend to decrease stability.

In other words, the pore water pressure (U) acting on the soil mass below the saturated line reduces the effective stress responsible for mobilising shearing resistance.

The factor of safty in this case is given by

$$F.S = \frac{c\hat{L} + \tan\phi\, \Sigma (N - U)}{\Sigma T}$$

when EU is the Total pore pressure on slip surface



The pore water pressure at any point is represented by the piezometric head (hw) at that point. Thus the variations of pore water pressure along a likely slip surface is obtained by measuring at each of its intersection with equipotential line the vertical height from that intersection to the level at which the equipotential line cuts the pheretic line.

② Stability of upstream slope During Sudden Drawdown

ⓐ solution with the help of flow net and pressure net.

For the U/s slope, steady seepage does not represent the critical state, because the seepage pressure then acts inwards from this slope and tends to increase the stability on the upstream side. For the U/s slope the critical condition is when the reservoir is suddenly emptied without allowing any appreciable change in the water level within the saturated mass soil. This state is known as sudden drawdown.

The magnitude and distribution of pore water pressure on a likely slip surface is estimated from a pressure net which is developed from the flow net.

The pressure net gives lines of equal piezometric heads (h_w) expressed in terms of percentage of Total hydraulic head

$$\phi = h_w + y$$

where ϕ = hydraulic potential at the point expressed as percentage of total heads

h_w = piezometric head, expressed as a percentage of Total Head

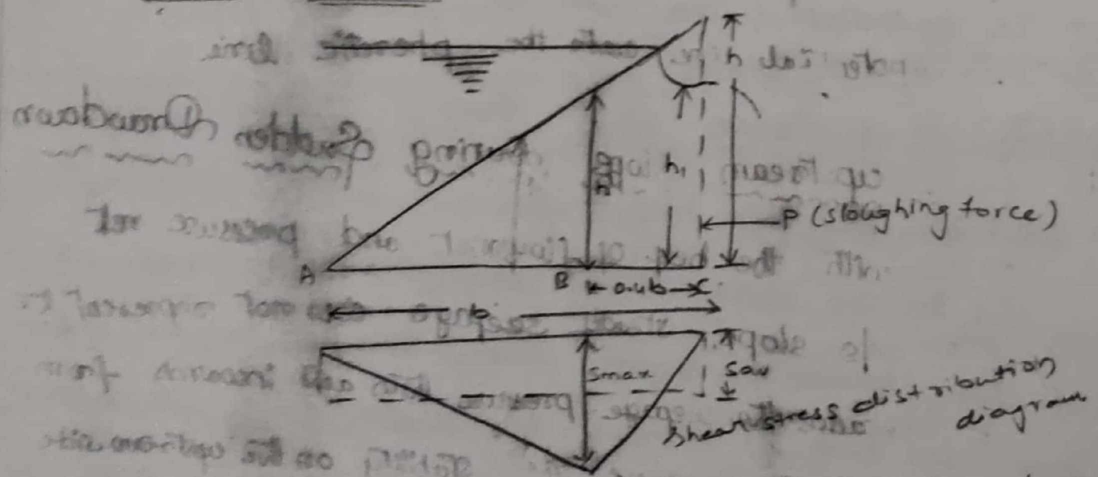
y = elevation head " " " " "

(b) Approximate Analysis :—

In the absence of flow net the factor of safety can be calculated from the expression

$$F.S = \frac{C\hat{L} + \tan\phi \, \Sigma N'}{\Sigma T}$$

(c) Analysis for sloughing of upstream slope During sudden drawdown



Let γ' = submerged unit weight of the material in u/s portion of dam

γ_s = saturated unit weight " " "

b = horizontal length of the slope AD

P = Total Horizontal shear or sloughing force on upstream force of the dam

S_{av} = Average unit shear over upstream part of dam

S_{max} = Maximum unit shear over upstream part of the dam

h, h_1, h' = heights as marked

Then the Horizontal shear force is given by

$$P = \frac{\gamma_s h^2}{2} \tan^2(45 - \frac{\phi}{2}) + \frac{\gamma_w h_1^2}{2}$$

$$S_{av} = \frac{P}{b}$$

$$S_{max} = 2 S_{av} = \frac{2P}{b}$$

The maximum shear ocurs at a distance of 0.4b from the u/s point as shown.

R = shearing Resistance = $N_e \tan\phi + C.b$

where $N_e = \gamma' \cdot A$

A = area of u/s slope = $\frac{1}{2} b h$

$$\text{Factor safety} = \frac{R}{P}$$

This should be greater then 2

In order to find the factor of safty with respect to maximum shear force

$$S_{max} = 2 S_{av} = \frac{2P}{b}$$

S_r = Shearing resistant at B = $h' \gamma' \tan\phi + c$

$$= 0.6 h \gamma' \tan\phi + c$$

F.S at the point of maximum shear = $\frac{S_r}{S_{max}}$

This should be greater then 1.

(*) Seepage Control Measures

The Seepage Control measures are necessitated to prevent adverse effects of water percolating through embankment and its foundation. The following devices are used for seepage control through earth dam (i)

(a) Embankment Seepage Controlling

1. Toe filter
2. Horizontal drainage filter

③ protective filter d/s of the Toe
④ d/c coarse section
⑤ chimney drain extending upword into embankments

④ **Foundation Seepage Control.**

⑥ Impervious cutoff
⑦ upstream pervious Blanket.
8 d/s seepage Berms
9. Drainage Trenches
10. Relief wells

① **Toe filter:—**

Rock Toe keeps the phreatic line well within the Section and also ficilitates drainage. Its height Generally kept equal to 30 to 40%. of reservior head.

② **Horizontal Drainage filter:—**

The horizontal drainage filter may extended from 25 to 100% of the distance from downstream Toe to the Centreline of the dam. The horizontal filter Serves the following purpose. (i) It keeps the phreatic line well within the embankment (ii) It gives greater leakage because of shorter seepage path. (iii) It also provide drainage for foundation.

Seepage Control Measures

The seepage control measures are necessitated to prevent adverse effects of water percolating through embankment and its foundation. The following devices are used for seepage control through earth dam:

a) Embankment seepage control:-

1) Toe filter
2) Horizontal drainage filter.
3) protective filter d/s of the toe
4) D/s coarse section (embankment zoning)
5) Chimney drain extending upward into embankments

b) Foundation seepage control:-

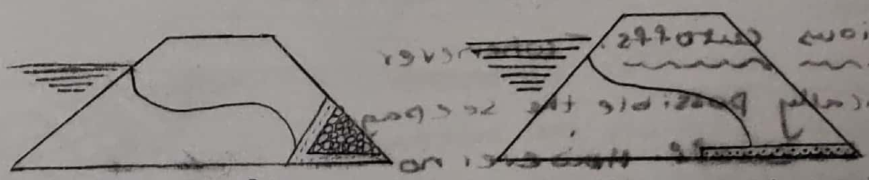
6) Impervious cutoff
7) upstream impervious blanket
8) D/s seepage berms
9) Drainage trenches
10) Relief wells.

A brief description of the these measures will now follow:

① Toe filter

Rock toe keeps the phreatic line well within the section and also facilitates drainage. Its height is generally kept equal to 30 to 40% of reservoir head. The gradation of various materials and layers should be satisfy filter criteria.

DRAINAGE FILTERS



② Horizontal drainage filter:-

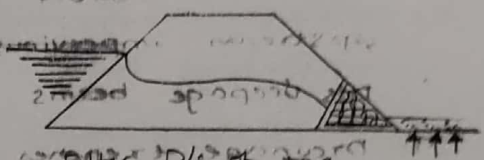
The Horizontal drainage filter may extended from 25 to 100% of the distance from downstream toe to the centre line of the dam. The horizontal filter serves the following purposes:

① It keeps the phreatic line well within the embankment.
② It gives greater leakage because of shorter seepage path.
③ It also provides drainage for foundation.
④ It accelerates consolidation.

Where a high degree of stratification of embankment may occur, the rock-fill toe has an advantage over drainage blanket in that it intercepts more layers of the embankment.

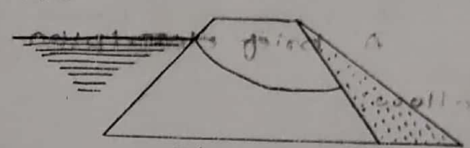
3) Filter downstream of the toe:- The provision of such filter provides additional weight and thus makes the upward flow more safe.

4) Downward Stream Coarse:-



Filter D/s of toe

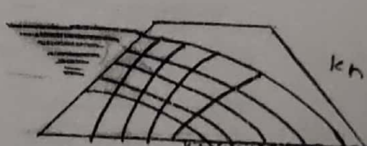
This also intercepts the flow through the embankment, and makes the d/s slope safe against piping. It is also an earth-quake resistance Measure.



D/s coarse section ①

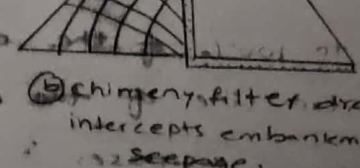
5) chimney drain:- when there is high degree of embankment stratification, the horizontal permeability is greater than the degree of embankment vertical, Resulting in greater horizontal spread of seepage. A correctly built vertical drain can completely intercept embankment seepage, regardless of the degree of stratification of embankment. It is also earthquake resistant.

6) Impervious cutoffs:- whenever economically possible the seepage should be cutoff. However no cutoff is absolutely impervious and the reduction of seepage is relative matter. The various cutoffs may consist of 1) cutoff trenches
2) Grout curtains
3) sheet pile cutoffs



$k_h = 9k_v$

ⓐ Horizontal filter drain Relatively ineffective



$k_h = 9k_v$

ⓑ chimney filter drain intercepts embankment seepage.

Effectiveness of chimney drain

It may be impossible or extremely expensive to continue the cutoffs to an impervious stratum. In such a case, only partial cutoff is provided.

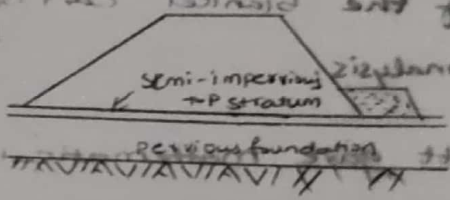
For Homogeneous foundation material, the effectiveness of partial cutoff is limited. since 90% depth of seepage cutoff reduces the discharge only by about 64% and 50% depth by 25%

⑦ Down stream seepage berms:-

Berms can be used to control seepage effeciently where the d/s top stratum is relatively thin and uniform or where no d/s top stratum is present.

They serve two purposes:

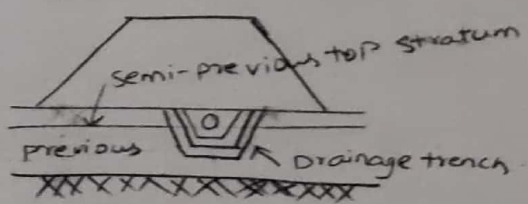
① They provide additional weight to resist uplift pressure beneath the top stratum and.



② They afford the some protection against possible sloughing of d/s slope of the dam as a result of seepage.

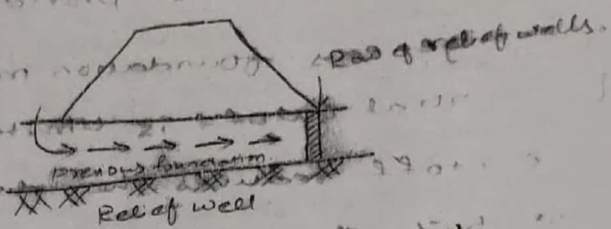
⑧ Drainage trenches:-

They are provided when top stratum is thin and previous stratum is also shallow.



⑨ Relief wells:-

The preliminary purpose of relief well is to reduce the substratum uplift pressure d/s of the dam which otherwise would cause formation of sand boils and possibly sub-surfaces piping. They intercept the seepage and control the outlet for seepage. They were fired used by U.S crops of engineers.



10) Upstream impervious blanket:-

Impervious upstream or riverside blankets overlaying a pervious foundation are effective in reducing the quality of seepage. They also, to some extent, reduce uplift pressure and escape gradients downstream of land side. A part of reservoir head is dissipated through the blanket. The length of the blanket can be determined by the Benett's analysis.

Benett gave mathematical solution for the performance of U/s impervious blanket