MODULE - 3

FLOW THROUGH SOILS

water strongly affects engineering behaviour of most kind of soils and water is an important factor. In most geotechnical engineering problems. Hence it is essential to understand basic principles of flow of water through soil medium

→ Flow of water takes place through inter connected ~~pores~~ pores between soil particles consid-ered in one direction.

* Flow of water through soils may be laminar flow or a turbulent flow.

* In laminar flow, each fluid particle travels along a definite path which never process the path of any other particle.

* In turbulent flow, the paths are irregular and twisting, crossing and re crossing.

* In most soil mechanics problems the flow is considered laminar.

permeability :-

The ease with which the water can flow through ~~soils~~ soil mass is permeable.

The study of sepage of water is important for the following engineering propertie

① Determination of settlement of a Saturated Compressible Soil layer.

② calculation of Seapage through the body of earthdams, and Stability of source.

③ calculation of uplift pressure under hydraullic structures and their Safety against piping.

④ Ground water flow towards walls and drianage of Soils.

DARCY'S LAW (French Scientist H.P.G. Darcy)

The theory of seapage flow in porous media is based on generalization of Darcy's law.

* Darcy's law is stated as " Velocity of

flow in porous media is proportional to the hydraulic gradient. [Flow is laminar]

$$Q = kiA$$

Q = Discharge pert unit time

k = co-efficient of permeability (mm/s)

i = Hydraulic gradient

A = Total cross section area of soil mass perpendicular to direction of flow

Hydraullic gradient (i) :-

It is defined as the loss of head per unit length of flow.

$$i = \frac{\Delta h}{L}$$

co-efficient of permeability :- [k]

co-efficient of permeability of soil describes how easily a liquid will move through a soil hence it is also reffered to as hydraulic conductivity.

~~Thi~~

* This factor can be affected by the viscousity or thickness (fluidity) of a liquid and it's density.

water is also called neutronion fluid or ideal fluid.

	SOIL TYPE	CO-EFFICIENT OF PERMEABILITY (mm/s)
①	Coarse	$10 - 10^{3}$
②	Fine Gravel, coarse and medium sand	$10^{-2} - 10$
③	Fine sand, loose silt	$10^{-4} - 10^{-2}$
④	Dense silt, clayey silt	$10^{-5} - 10^{-4}$
⑤	Silty clay, clay	$10^{-8} - 10^{-5}$

Assumptions of Darcy's law :-

* The flow is laminar i.e flow of fluids is described as laminar if a fluid particle flow follows a definate path and does not cross the path of other particles.
* water in soil are incompressible i.e countinuity equation is assumed to be valid.
* The soil is saturated
* The flow is steady i.e flow condition do not change with time.

VALIDITY OR LIMITATIONS OF DARCY'S LAW

According to Darcy's law, the velocity of flow through soil mass is directly proportional to the hydraulic gradient for laminar flow conditions only.

* In practice one can expect the flow through to be always laminar in the case of fine grained soil deposits because of flow permeability. and hence low velocity of flow.

* However in the case of sands and gravels flow will be laminar up to a certain value of velocity for each deposit and investigations have been carried out to find the limit of application of Darcy's law.

* According to experimental investigations, flow through sands will be laminar and Darcy's law is valid as long as the reynold's number expressed in the form shown below is less than or equal to unity.

$$\frac{V D_a \gamma_w}{\eta g} \leq 1$$

V = velocity of flow in cm/s

D_a = size of particles (avg) in cm

* It is found that the limiting value of Reynold's number taken as one is very approximate as its actual value can have wide variation.

* Depending partly on the characteristic size of particle used in the equation.

* The upper limit of effective size of particles can be taken as 3mm for Darcy's law to be valid.

Factors affecting permeability :-

① Particle size :- Permeability of coarse soil is less than the fine soil.

② Structure of soil mass :-

③ Shape of particles :- Angular particles have less permeability than rounded particles.

④ Void ratio :- For higher void ratio, higher is the co-efficient of permeability or hydraulic conductivity.

⑤ properties of water :-

Unit weight of water and viscousity affects permeability. Viscosity varies with temperature.

⑥ Degree of saturation :- Partially saturated soils have less permeability than fully saturated soils.

⑦ Adsorbed water :- Adsorbed water layer on particles is not free to move and this layer affects permeability.

⑧ Impurities in water :- Impurities in water tend to block or plug the flow passage of water and reduce the permeability of soil.

Determination of coefficient of permeability

IMP

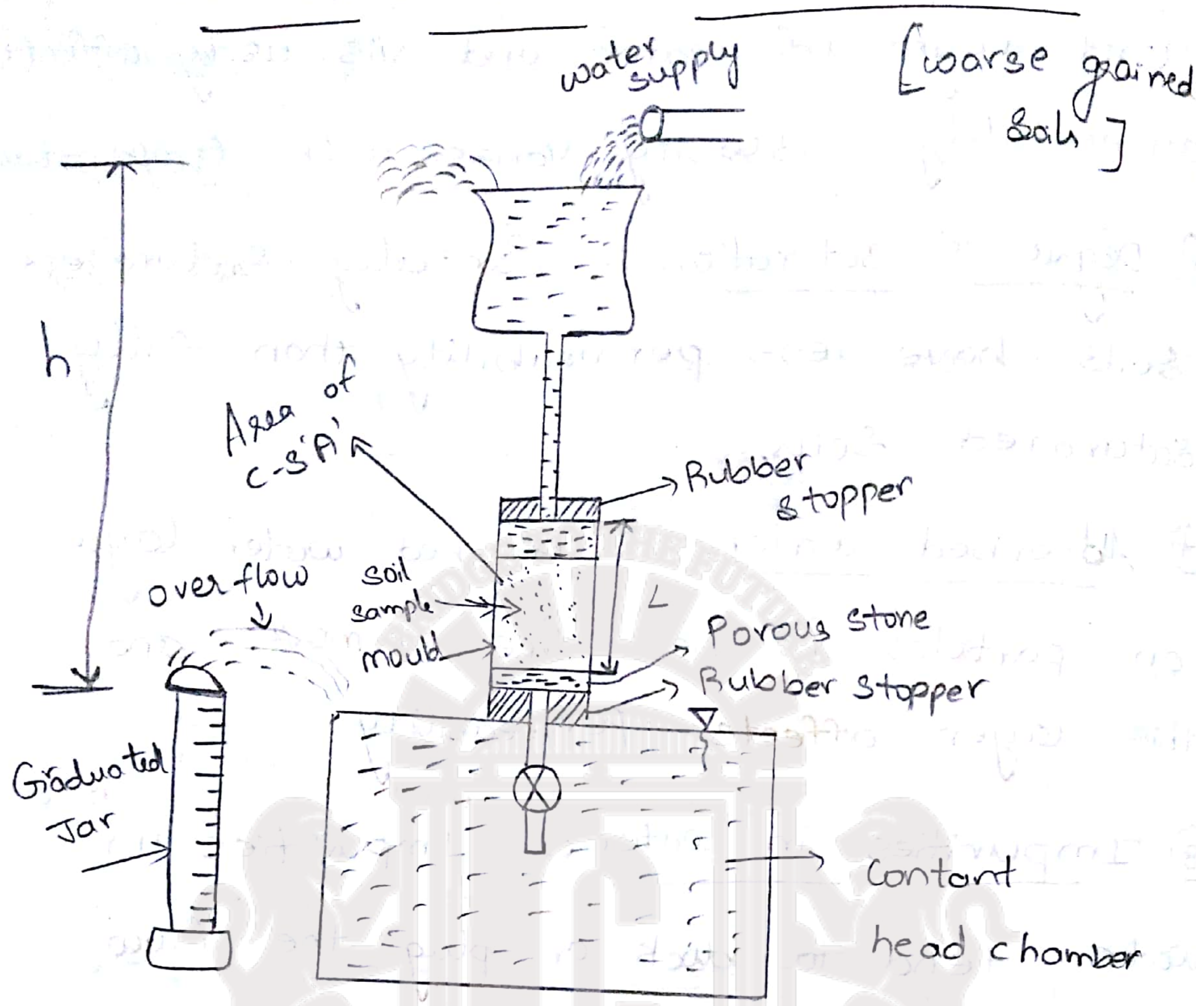
LABORATORY TESTS :-

① Constant Head permeability

② Varying or Falling Head permeability

CONSTAND HEAD PERMEABILITY

[coarse grained Soils]



The constant head test is suitable for more permeable coarse grained soils. The basic laboratory test is shown in figure.

* The soil specimen is placed in cylindrical mould and constant head 'h' of water flowing through the soil is maintained by adjusting the supply.
* The out flow water is collected in a measuring cylinder and the duration of collection period is noted.

* Repeat the procedure of discharge of water through the Soil Sample for a Specific time period three times.

* From Darcy's law, The total quantity of flow 'Q' in time 't' can be given by

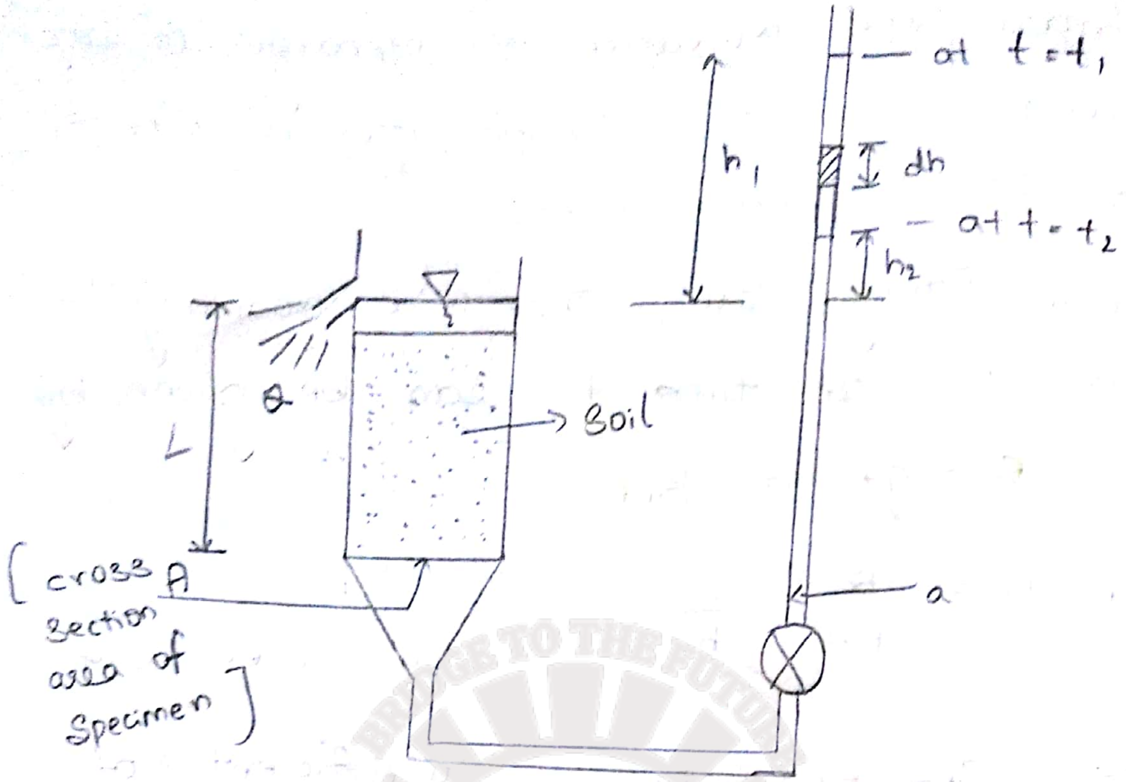
$$Q = qt = kiAt$$

$$\therefore \quad k = \frac{Q}{At} \cdot \frac{L}{h}$$

<u>Note</u> :- The knowledge of co-efficient of permeability or hydraulic conductivity is much essential in solving problems involving yield of water bearing strata, seapage through earthen dams and embackments of canal bank affected by seapage, settlement etc.,

<u>Varying</u>
<u>Variable</u> or <u>falling Head permeability</u>

(Suitable for fine grained soils)



* The falling head permeability test is most suitable for fine grained soil.

* The figure shows the general laboratory arrangement for the test.

* The soil specimen is placed inside the cube or mould and a stand pipe is attached to the top of the specimen.

* water is allowed to flow through the soil specimen.

* The initial head h_1 at time t_1 is recorded and water is allowed to flow through such that the final predetermined head 'h_2' at time 't_2' is recorded.

* 3 or 4 trails are conducted

* The rate of flow of Soil is

$$K = 2.303 \frac{aL}{At} \log_{10}\left(\frac{h_1}{h_2}\right)$$

a = cross Section area of pipe

h_1 = Initial head

h_2 = Final head

$$t = t_2 - t_1$$

A = cross Section area of the Specimen

L = Length of Soil Specimen

Note :- For a constant t, at two instances let water level fall from h_1 to h_2 and h_2 to h_3 then $h_2 = \sqrt{h_1 h_3}$

Need and scope of the experiment :-

1. To estimate ground wate flow
2. To calculate seapage through dams
3. To find out rate of consolidation and settlement of structures
4. To plan the method of lowering the ground water table.
5. To calulate the uplift pressure and piping
6. To design the grouting
7. It is used for soil freezing test
8. To design sewage pits for recharging.

For very fine grained soils, capillarity permeability test is recomended.

Determination of coefficient of permeability

FIELD TESTS

The co-efficient of permeability or hydraulic conductivity of a permeable layer can be determined by pumping from a well at a constant rate and observing the steady state water table in nearby observation wells. The steady state is established when the water levels in the test well and observation wells becomes constant.

* when the water is pumped out from the well the aquifer gets depleted of water and the water table is lowered resulting in a circular dipression in the phreatic surface this is to refer to as drawdown curve or cone of depression.

Aquifer :- A body of permeable rock which can contain or transulate ground water. An aquifer is an underground layer of water bearing permeable rock, rock mn fractures or unconsolidated materials from which ground water can be extracted using the water well.

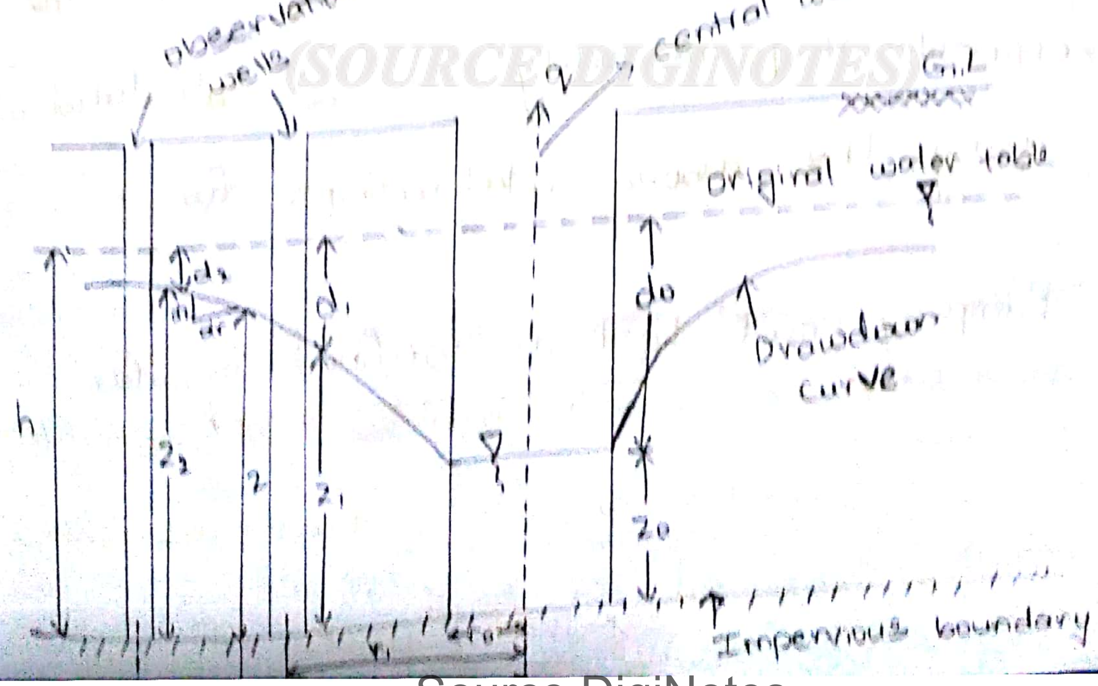
Assumptions made in field permeability test (in-situ)

* The aquifer is homogeneous
* Darcy's law is valid
* The flow is horizontal
* The well penetrates the entire thickness of the aquifer
* Natural ground water regime (confined) remains constant with time

FIELD PERMEABILITY TESTS :-

① Pumping out test in unconfined aquifer

② Pumping out test in confined aquifer



Let R and z be the radial distance and height above the impervious boundary at any point on the draw down curve as shown in figure. At steady state the rate of discharge due to pumping can be expressed as $q = kiA$.

* Hydraulic gradient at any point is given by Dupuit theory.

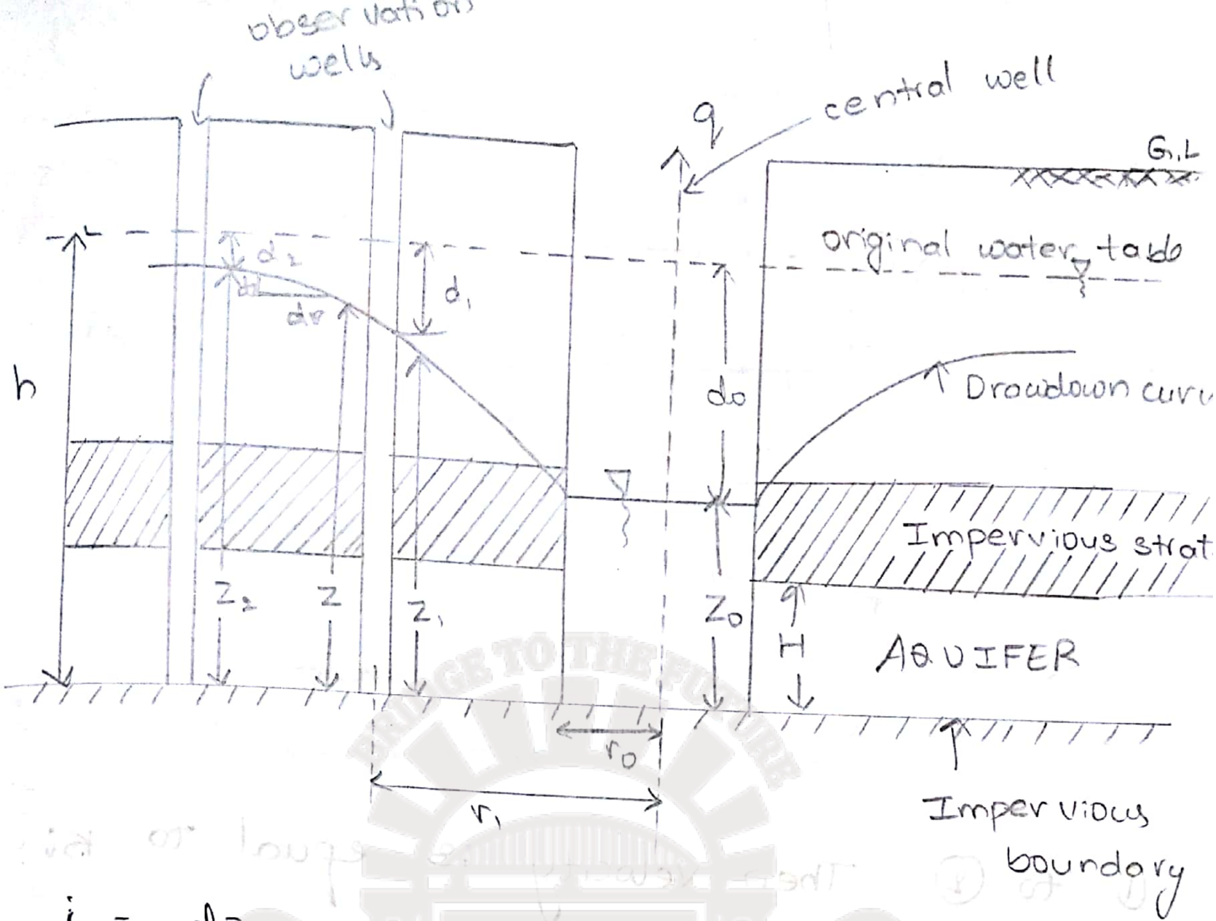
$$i = \frac{dz}{dr}$$

$$z_1 = (h - d_1) \quad , \quad z_2 = (h - d_2)$$

$$k = \frac{2.303\, q}{\pi[(d_1 - d_2)(2h - d_1 - d_2)]} \left[\log_{10} \frac{r_2}{r_1}\right]$$

If the values of r_1, r_2 and z_1, z_2 and 'q' are known from field measurements, the co-efficient of permeability can be calculated by using the above relationships for 'k'.

(2) Pumping out test in confined aquifer



$$i = \frac{dz}{dr}, \quad z_1 = (h - d_1), \quad z_2 = (h - d_2)$$

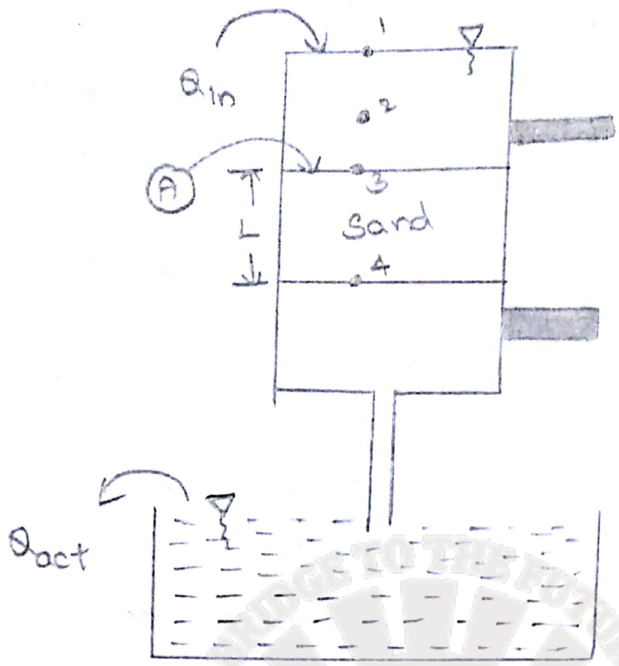
$$K = \frac{q}{2.7283 \times H\,(d_1 - d_2)}\left[\log \frac{r_2}{r_1}\right]$$

The values of $r_1 r_2$, z_1 z_2 and q are known from field measurements, the co-efficient of permeability can be calculated by using above relationship for K.

Seepage velocity and superficial velocity

Consider flow of water through soil medium of length 'L' and cross sectional area 'A' as shown in figure.

If 'v' is the velocity of downward moment of drop of water from position



① to ② . Then velocity is equal to Ki.

∴ K can be interpretened as the approach velocity or Superficial velocity for unit hydraulic gradient. i.e, $K = \frac{V}{(i=1)} = K$

Drop of water flow from position ③ to ④ at faster rate than it thus from positions ① to ② because the average area of flow through the soil is smaller. The actual velocity of water flowing through the voids is termed as seapage velocity.

By the principle of continuity, the velocity of approach 'V' may be related to the seapage velocity or average effective velocity of flow V_s, by equating Q_{in}

and Q_{out}.

$$Q = VA = V_s A_s$$

$$V_s = V \frac{A}{A_v} = V \frac{AL}{A_v L} = V \frac{V}{V_v}$$

$$V_s = \frac{V}{n} = \frac{1+e}{e} V = \frac{ki}{n}$$

$$= \frac{1+e}{e} ki$$

A_v = Area of pores

V = total volume of soil

V_v = volume of voids

e = void ratio

n = porosity

Thus seapage velocity is the superficial velocity divided by the porosity. Above equations indicates that the seapage velocity is also proportional to the hydraullic gradient.

$$V_s = \frac{k}{n} i = \frac{(1+e)}{e} ki$$

$$= k_p i$$

$$V_s \propto k_p$$

k_p - coefficient of percolation

$$k_p = \frac{k}{n} \frac{(1+e)}{e} k$$

<u>co-efficient of percolation</u> :- It is the ratio of coefficient of permeability to the porosity.

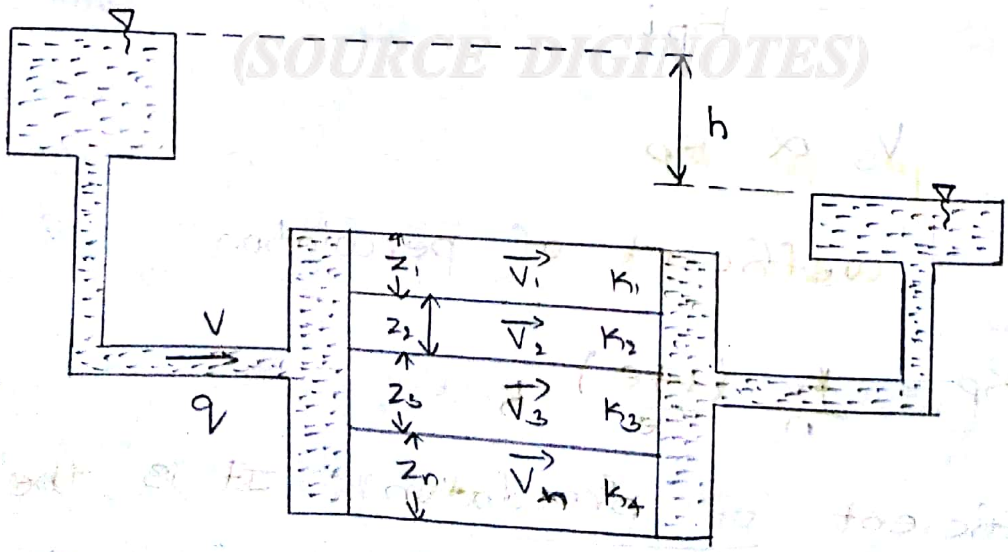
PERMEABILITY OF STRATISFIED SOILS

In nature soil mass may consists of several layers deposited one above the other. Their bedding planes may be horizontal, inclined or vertical. Each layer assumed to be homogeneous and isotropic, as it's own value of permeability.

The average permeability of the whole deposit will depend upon the direction of flow with relation to the direction of bedding plane. We shall consider the cases of flow.

1. Parallel to the bedding planes
2. Perpendicular to the bedding planes

① Average permeability parallel to the bedding planes



FLOW PARALLEL TO BENDING PLANE

Let $z_1, z_2, \ldots z_n$ is the thickness of the layers

$k_1, k_2, \ldots k_n$ = permeability of layers

For flow to be parallel to the bedding planes, the hydraulic gradient 'i' will be the same for all the layers. However since $V = ki$ and since k is different, the velocity of flow will be different in different layers.

Let k_x = average permeability of soil deposit parallel to the bedding plane.

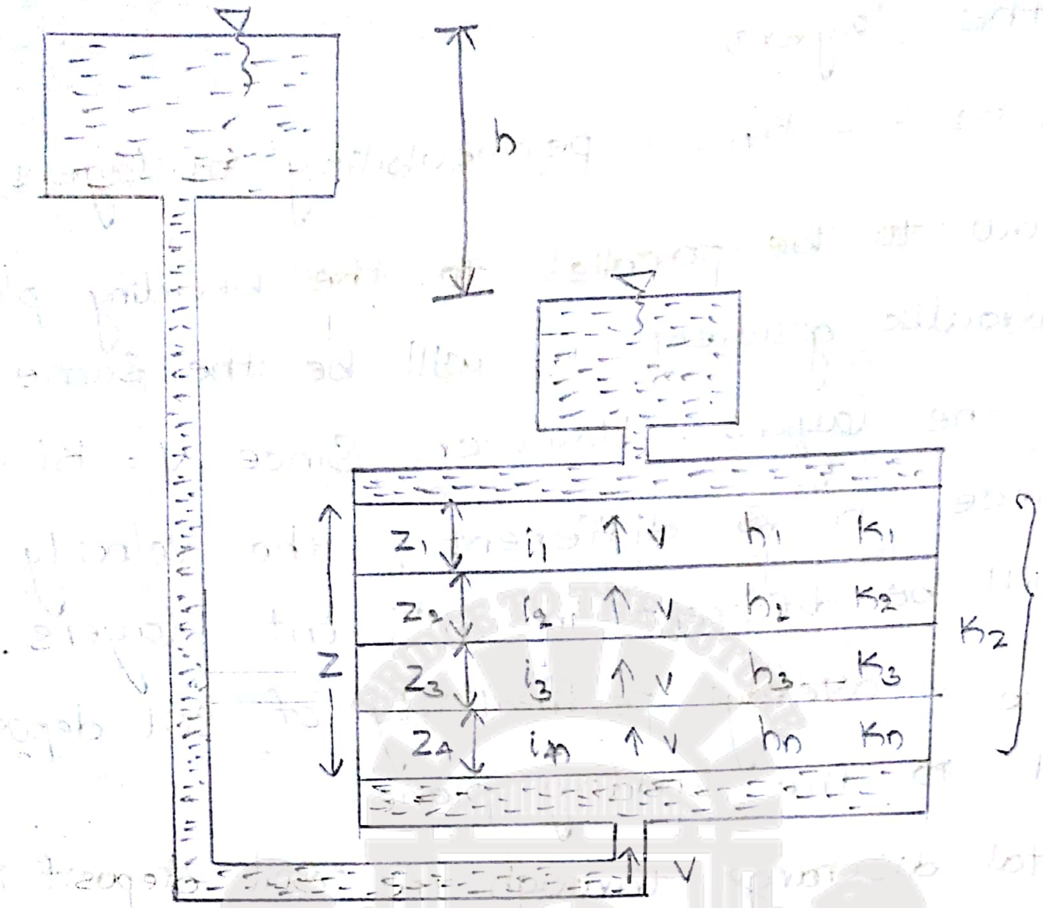
Total discharge through the soil deposit is equal to sum of discharge through the individual layers.

$$q = q_1 + q_2 + \ldots q_n$$

$$q = k_x i z = k_1 i z_1 + k_2 i z_2 + \ldots k_1 z_n$$

$$\boxed{k_x = \frac{k_1 z_1 + k_2 z_2 + \ldots k_n z_n}{z}}$$

② Average permeability Perpendicular to the bedding planes



In this case the velocity of flow and hence the unit discharge will be the same through each layer. However the hydraulic gradient and hence the head loss through each layer will be different. Denoting the head loss through the layers $h_1, h_2 + --- h_n$

$$h = h_1, h_2 + -- h_n$$

h = total head $\qquad i = \frac{h}{Z}$

$$h_1 = i_1 z_1$$

$$h_2 = i_2 z_2$$

$$h_n = i_n z_n$$

$h = i_1 z_1 + i_2 z_2 + \ldots i_n z_n$

$h_1 + h_2 + \ldots h_n$

k_z = Average permeability perpendicular to the bedding plane

$$V = k_z (i) = k_z \left(\frac{h}{z}\right)$$

(or) $$h = \frac{Vz}{k_z}$$

Also, $i_1 = \frac{V}{k_1}$, $i_2 = \frac{V}{k_2}$

$$i_n = \frac{V}{k_n}$$

$$\frac{Vz}{k_z} = \frac{Vz_1}{k_1} + \frac{Vz_2}{k_2} + \frac{Vz_n}{k_n}$$

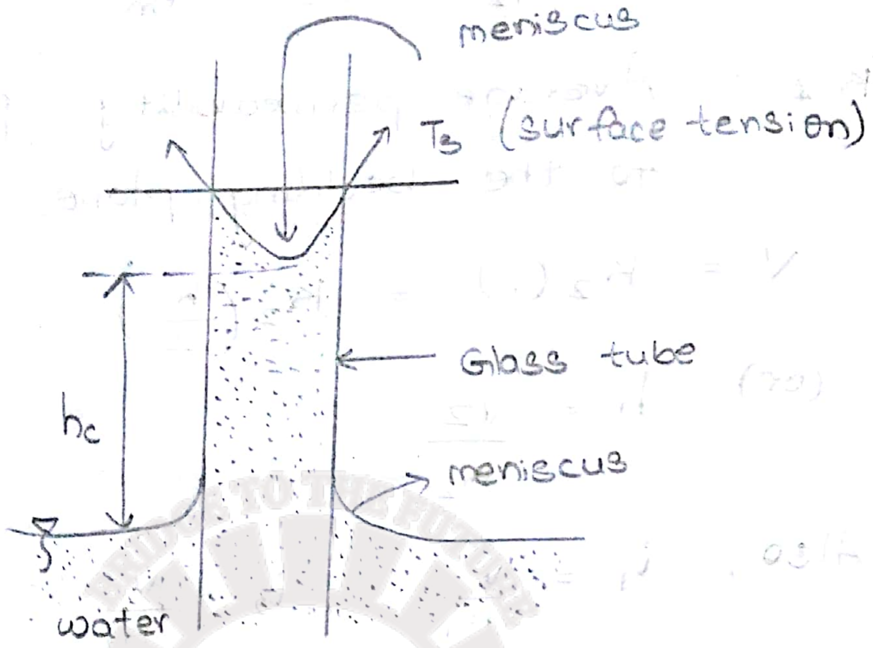
$$k_z = \left(\frac{z}{\frac{z_1}{k_1} + \frac{z_2}{k_2} + \ldots \frac{z_n}{k_n}}\right)$$

Average permeability parallel to bedding plane k_x is greater than average permeability perpendicular to the bedding plane.

$$\boxed{k_x > k_z}$$

1 > 2

CAPILLARY PHENOMENA :



Capillary water in Soil

capillary rise results from the combined actions of Surface tension and intermolecular forces between liquid and soil.

The rise of water in soils above the ground water table is comparable to the rise of water into capillary tubes placed in a Source of water but the void Spaces in a Soil are irregular in Size and shape as they inter connect in all directions.

* The pressure on the water table level is Zero, any water above this level must have a negative pressure.

* In soils a negative core pressure increases, the effective stresses and varies with a degree of saturation

capillary rise in glass tube :-

For pure water and glass tube of very small diameter capillary rise is given by

$$h_c = \frac{4T_s}{d\gamma_w}$$

T_s = Surface tension

γ_w = unit weight of water

d = diameter of capillary tube

h_c = capillary rise in tube

The maximum negative pore pressure is

$$u = h_c \gamma_w = \frac{4T_s}{d}$$

The surface tension T_s of water at 20°c

$$= 75 \times 10^{-8} \text{ KN/cm}$$

* The pressure is zero when the soil voids are filled with air and is negative when the voids are partly filled with water and air.

[In which case the surface tension forces operate to achieve a suction effect and the shear strength of the soil is increased

SEAPAGE ANALYSIS

Two-dimensional flow

Flow is two dimensional if it can be assumed that the flow parameters vary in the direction of flow in one direction at right angles to this direction.

Fluid motion is said to be a two-dimensional flow when the flow velocity at every point is parallel to a fixed plane. The velocity on any point on a given normal to the fixed plane should be constant.

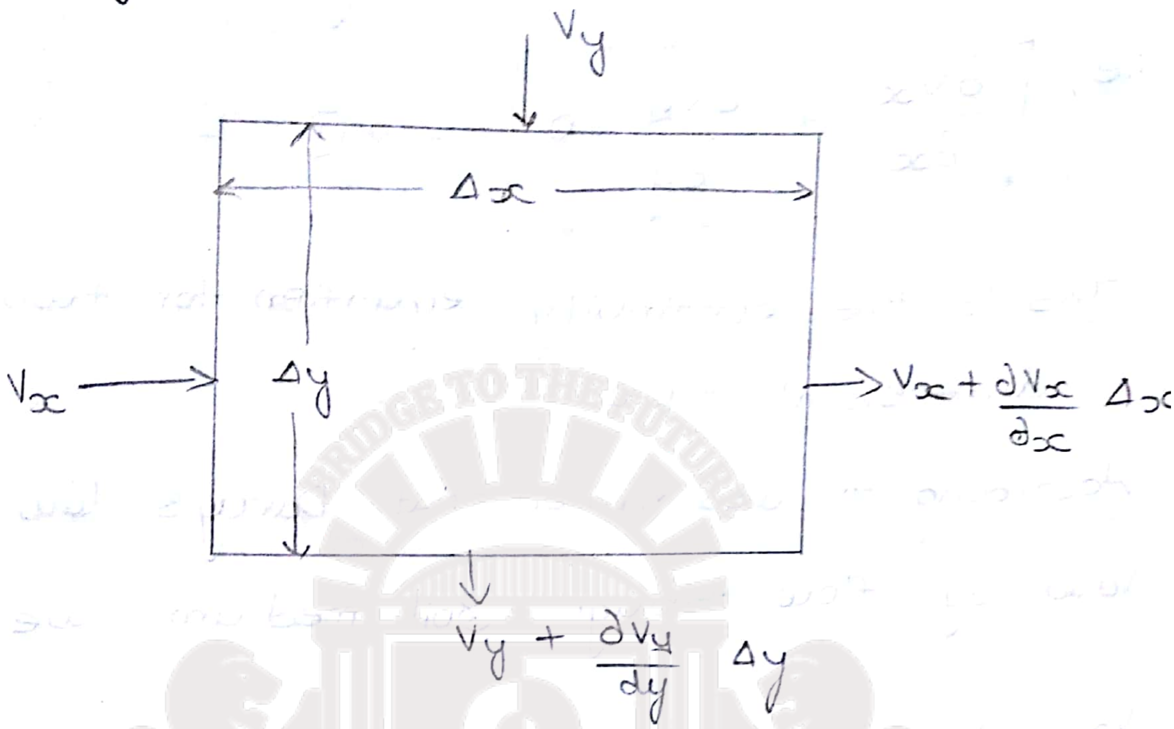
LAPLACE EQUATION

Two-dimensional flow

Assumptions:-

* Laplace equation governs the flow of a fluid through a porous material.
* Flow is two dimensional
* water and soil are incompressible
* Soil is isotrophic and homogeneous
* The soil is fully saturated

* The flow is steady that is flow conditi[ons] do not change with time.
* Darcy's law is valid.



consider an element of size Δx, Δy and of unit thickness perpendicular to the plane of figure. Let V_x and V_y be the velocity components at entry in x and y direction, then the corresponding velocity components at exit will be $\left(V_x + \frac{\partial V_x}{\partial x} \Delta x\right)$ and $\left(V_y + \frac{\partial V_y}{\partial y} \cdot \Delta y\right)$. According to the assumption that quantity of water entering an element is equal to the quantity of water leaving the element in any given time.

we have,

$$V_x(\Delta y \cdot 1) + V_y(\Delta x \cdot 1) = \left(V_x + \frac{\partial V_x}{\partial x} \cdot \Delta x\right)(\Delta y \cdot 1) + \left(V_y + \frac{\partial V_y}{\partial y} \Delta y\right)(\Delta x \cdot 1)$$

i.e, $\boxed{\frac{\partial V_x}{\partial x} + \frac{\partial V_y}{\partial y} = 0} \rightarrow ①$ -

This is the continuity equation for two dimensional flow.

According to assumption that Darcy's law is valid by flow through soil medium, we have

$$V_x = K_x \cdot i_x = K_x \frac{\partial h}{\partial x}$$

$$V_y = K_y \cdot i_y = K_y \frac{\partial h}{\partial y}$$

K_x, K_y = coefficient of permeability in x and y directions

By substituting in eq^n ①, we have

$$\left[K_x + \frac{\partial^2 h}{\partial x^2}\right] + \left[K_y + \frac{\partial^2 h}{\partial y^2}\right] = 0$$

For isotropic soil medium,

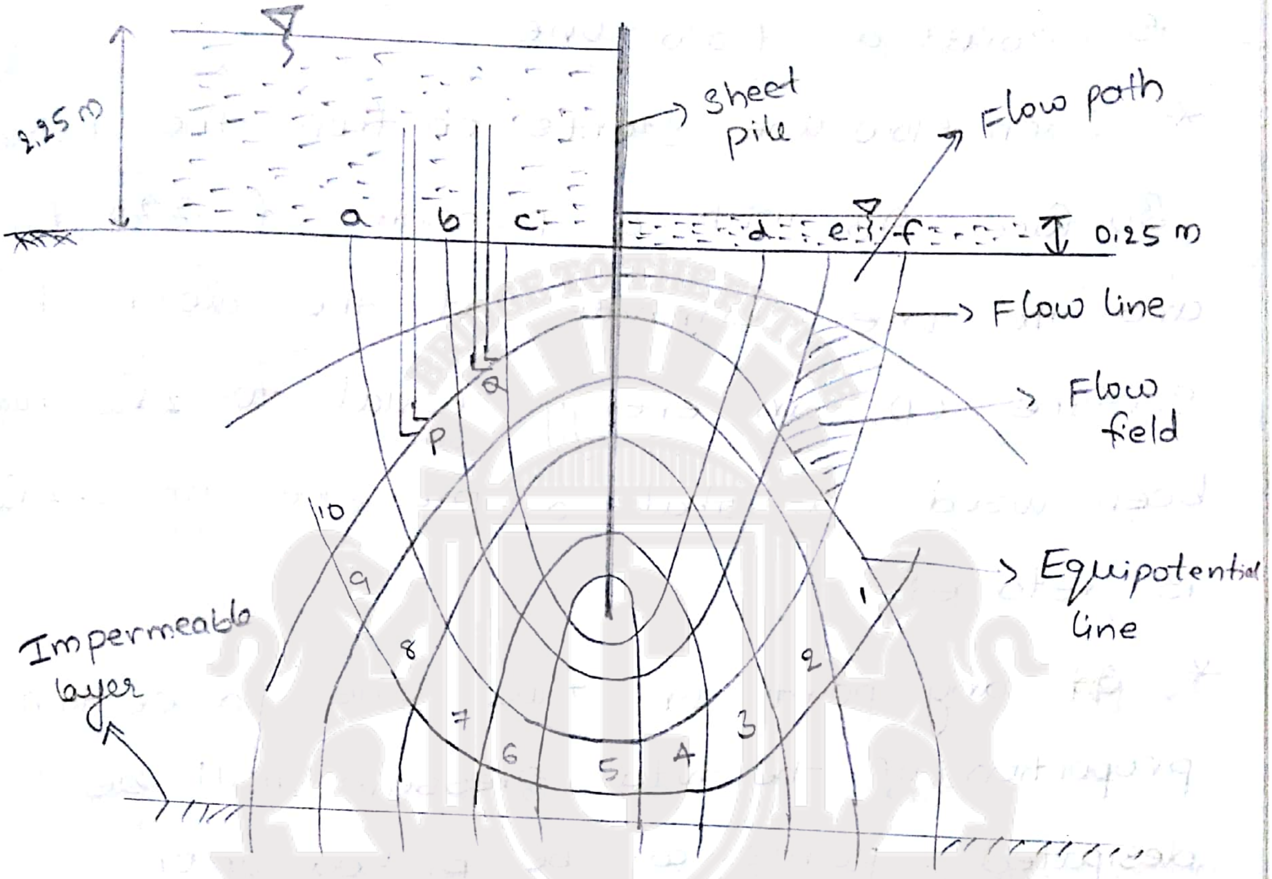
$$K_x = K_y = K$$

So the above equation reduces to form

$$\boxed{\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} = 0}$$

This is the laplace equation for two dimensional flow.

FLOW NETS



Above figure shows sheet pipe driven to a depth of 3m. into pervious sand underlaid by impermeable stratum. on oneside of Sheet pipe the depth of water is 2.25 m and on the other side, the depth of water is 0.25 m. Thus the headloss due to Seapage due to soil is 2m between the two sides of the sheet pipe wall under this unbalanced head Sepage flow takes place

* A particle of water enters sand at some

point on upstream, those goes below the tip of the sheet pipe and ends up at some point on downstream the flow path assumed by the particle of water in previous medium is called a flow line

* Each flow line starts on from the upstream Surface AB with a pressure of 2.25 γ_w and the time terminates at the down side Surface CD, an energy equal to 2 γ_w have been losed so that a pressure of 0.25 γ_w is retained

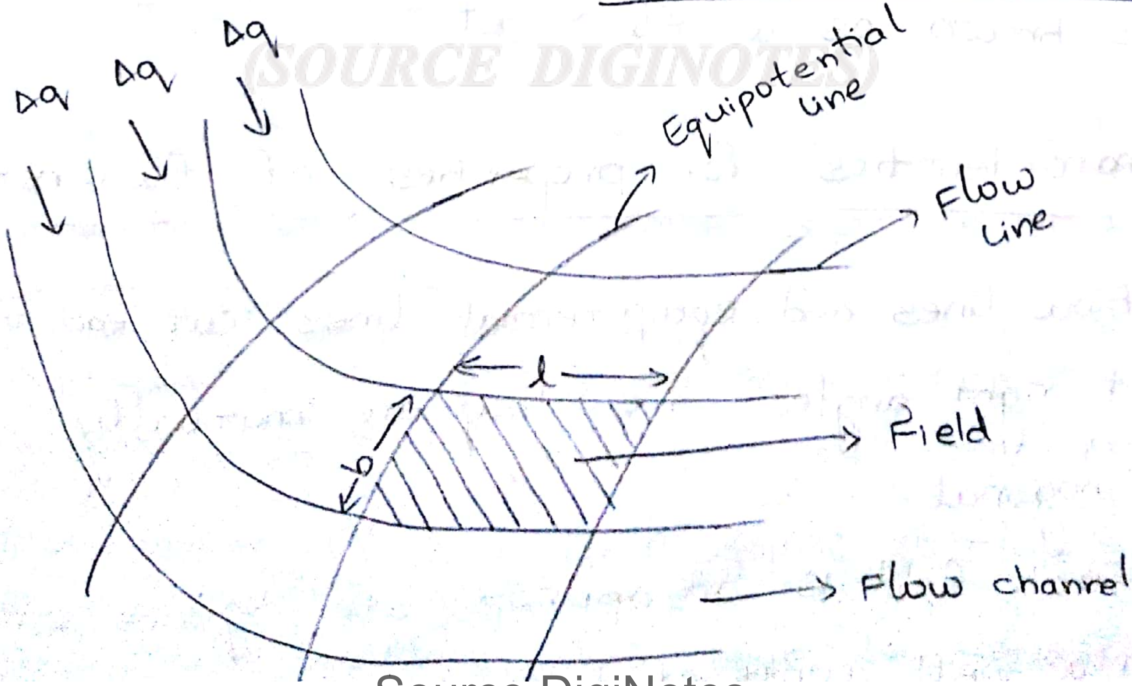
* At any point on flow line, a certain proportion of the total pressure will be desipated. points can be picked upon different flow lines such as AB, CD etc where the head desipated during flow upto those point is equal to a given fraction of total head loss of 2 m during the entire flow. if Such points are joined, the line so obtained is called an equipotential line. on any equipatential line, the head available to cause the flow is same.

* If peizometers are inserted into the soil at different points along an equipotential

line, water would rise to the Same elevation in all these piezometer at point P and Q on the equipotential line 10. It can be seen that the peizometric heads are different. It is the total head that remains the same.

* The two Sides of curves namely, the flow lines and the equipotential lines form a flow net which is a graphical representation of how the hydraullic energy is desipated as a water flow through a pervious medium. The hydraullic gradient between two Successive equipotential line such as 10 and 9 is the head lost betwe these lines divided by the corresponding length of flow.

portion of a flow net



Flow channel:- The space between any two adjucent flow lines is called flow channel.

Field :- The space enclosed between two adjacent flow lines and two successive equipotential lines is called a field.

Equipotential line :- It is an imaginary line in a field of flow such that the total head is the same for all points on the line and therefore the direction of flow is perpendicular to the line at all points. or

An equipotential line is a contour or line joining points of equal potential or head

Flow net :- A grid obtained by drawing a series of stream lines and equipotential lines is known as a flow net.

characteristics for properties of flow net:-

① Flow lines and equipotential lines cut each other at right angles. i.e they are mutually orthogonal.

② Each field is an approximate square and in a well constructed flow net one should be able to draw a circle in a field touching all the four Sides.

(*) In a homogeneous soil, every transition in the shapes of the two types of curves will be smooth being either elleptical or parabolic in shape. (*) The rate of flow through each channel is Same.

(*) The Same potential drop occurs between two Successive equipotential lines.

Uses of flownet :-

1. Estimation of seapage losses from reservoirs
2. Determination of uplift pressures below dams
3. checking the possibility of piping beneath dams
4. pore-water pressure determination.

1. Estimation of seapage losses from reservoirs:

It is possible to use the flow net in the transformed space to calculate the flow underneath the dam.

2. Determination of uplift pressures below dams:

From the flow net, the pressure head at any point at the base of the dam can be determined. The uplift pressure distribution along the base can be drawn and then Summed up

3. checking the possibility of piping beneath dams

At the toe of a dam when the upward exit hydraulic gradient approaches unity, boiling conditions can occur leading to errosion in soil and consequent piping.

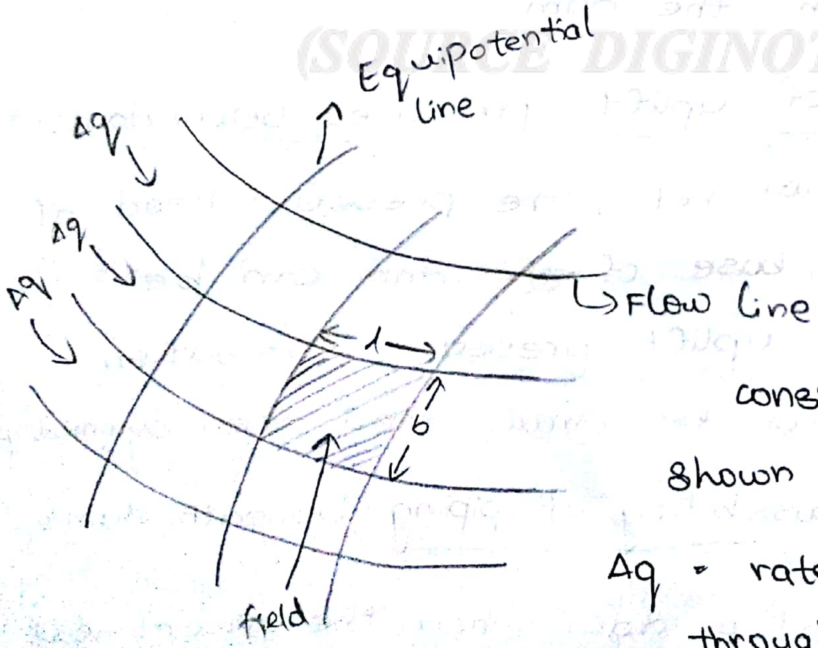
many dams on soil foundations have failed because of a sudden formation of a piped shape discharge channel. As the stored water rushes out, the channel widens and catastropic failure results. This is also referred to as piping failure

④ pore - water pressure determination :-

pore water pressure at a given point due to seapage through an earth dam.

(i) Quantity of Seapage
(ii) Seapage pressure at a point
(iii) Hydrostatic pressure at a point
(iv) Excit gradient

(i) Determination of quantity of Seapage :-



l = length
b = breadth
l = b = 1

consider the field as shown in the flow net

Δq = rate of discharge through each flow channel

Δh = head drop per field

$$\Delta h = \frac{H}{N_d}$$

H = Total head causing the flow

N_d = Number of potential drops in the entire flow net

N_f = Number of flow channels for the complete flownet.

Applying Darcy's law, we have

$$\Delta q = K \cdot \frac{\Delta h}{l} (b \times 1)$$

$$= K \cdot \frac{H}{N_d} \cdot \frac{b}{l}$$

For flow through entire flow net

$$q = \Delta q \cdot N_f$$

$$q = K \cdot \frac{H}{N_d} \cdot N_f \cdot \frac{b}{l}$$

$$\boxed{q = K \cdot H \left(\frac{N_f}{N_d}\right)} \quad [\because b = 1,\ l = 1]$$

This equation is used for finding the discharge

(ii) Determination of Seepage pressure at a point :-

Seepage pressure at a point, P_s is given by

$$\boxed{P_s = h \gamma_w}$$

h = Total head at that point

$h - (H - n.\Delta h)$

H = Total head causing flow

n = No of potential drops upto the point under consideration

Δh = potential head drop per field

$= \frac{H}{Nd}$

i) Determination of Hydrostatic pressure at a point

The hydrostatic head at a point is given by,

$h_w = h - z$

z = datum head at that point

h = Total head at that point

$= (H - n.\Delta h)$

H = Total head causing flow

n = No of potential drops upto the point under consideration

Δh = potential head drop per field $= \frac{H}{Nd}$

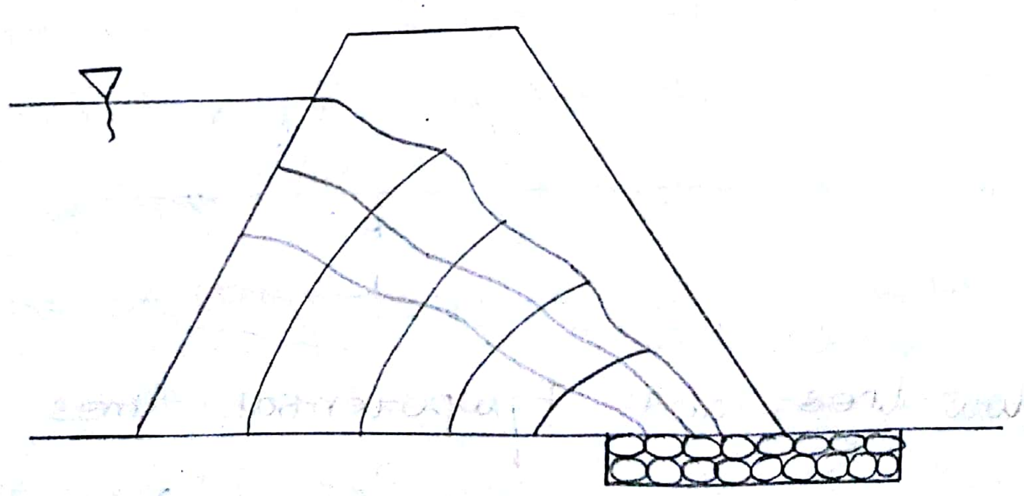
Seapage through body of homogeneous earth dam

The study of the problem of Seepage through body of earth dam involves determining the location and shape of phreatic line. The phreatic line is the top boundary flow line of the zone of Seapage in Section of earth dam.

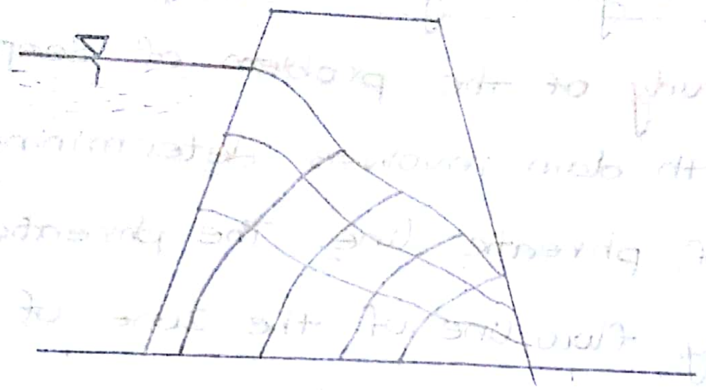
pheatic line can be located by

1. Graphical method [casagrande's Graphical method]
2. Analytical method
3. Experimental method

It is useful to note that there will be positive hydrostatic pressure where as on the phreatic line, it is equal to atmospheric pressure. Once the phreatic line is located the flow net can be easily located.



with toe filter



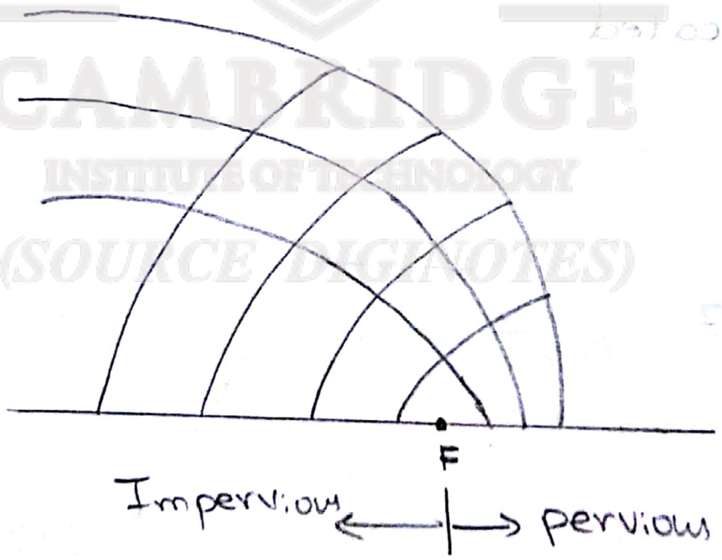
Flow through earth dam without toe filter

Casagrande's Graphical method

In casagrande method, to obtain a phreatic line the base parabola is corrected at entry and exit ends. it suit the boundary conditions

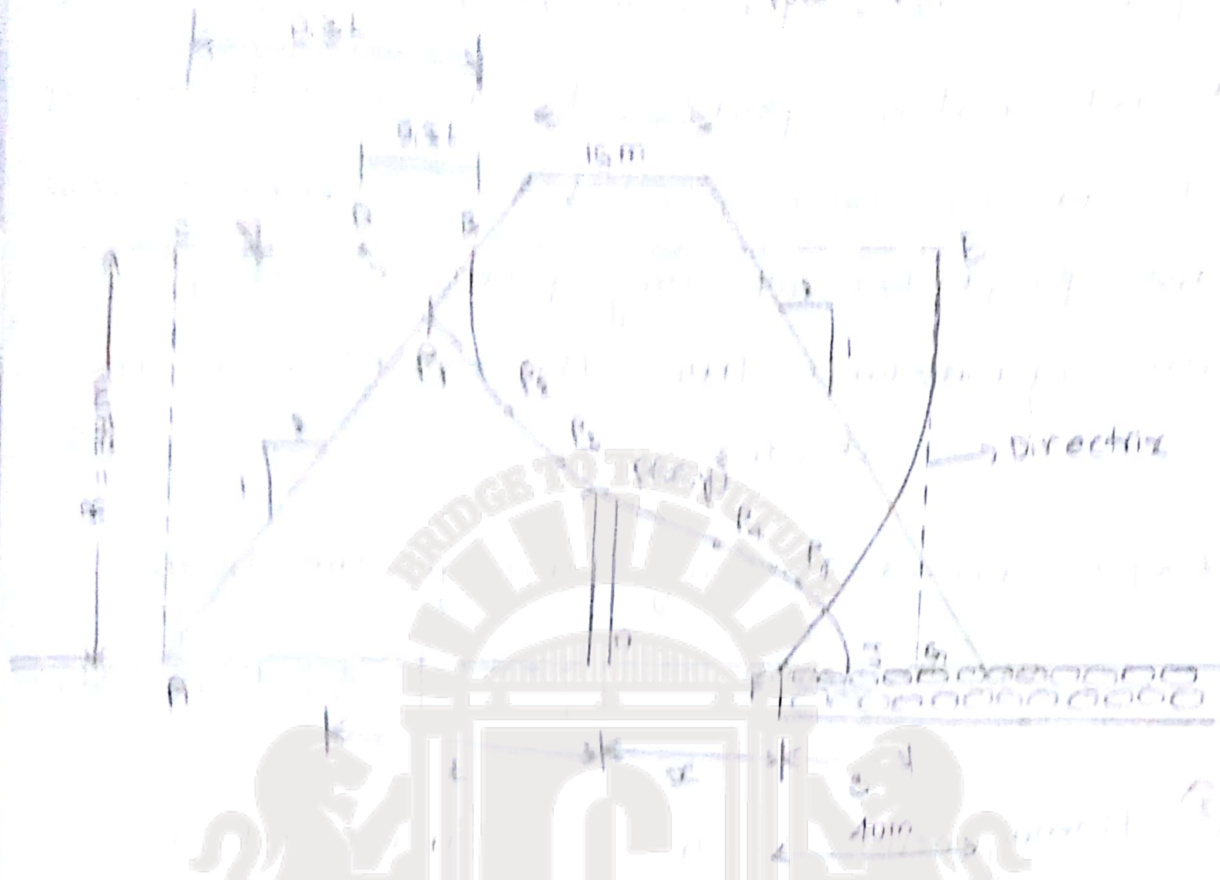
① Homogeneous Earth dam with horizontal filter.

② Homogeneous earth dam without filter.



Flow lines and Equipotential lines as confocal parabolae (F)

(i) Seepage through homogeneous earth dam with horizontal toe filter



A graphical procedure for obtaining phreatic line consists of the following steps and illustrate

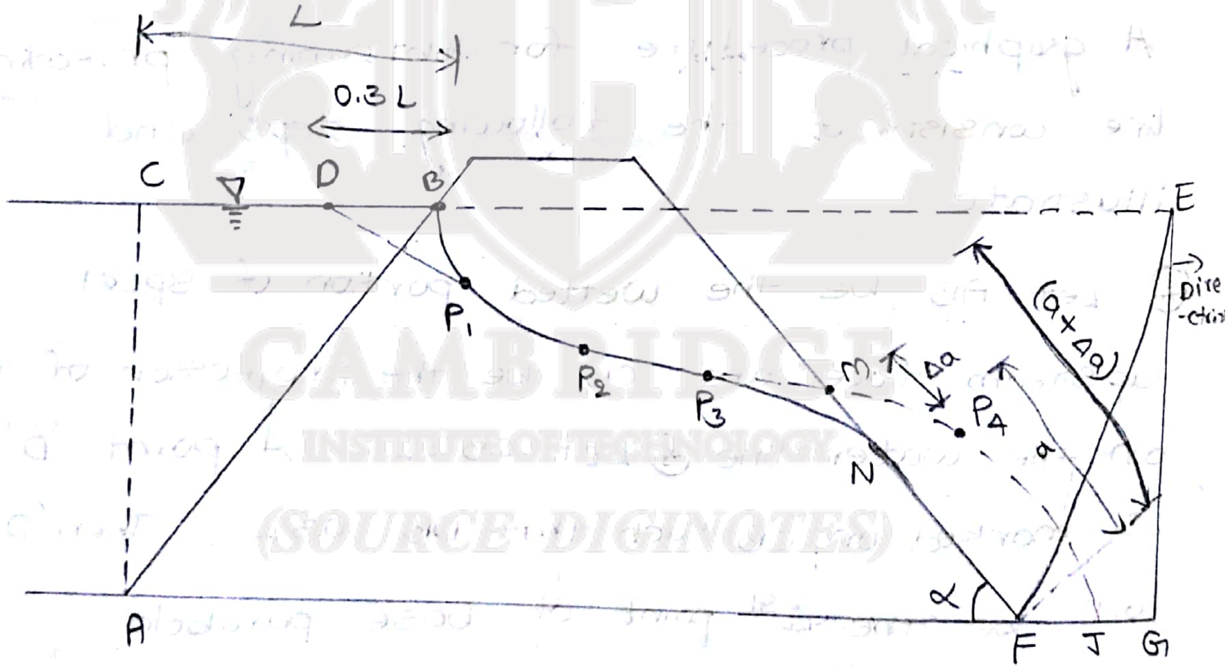
(i) Let AB be the wetted portion of upstream face and CB be the projection of AB on the water line (ii) Let CB = L. A point 'D' is marked on CB such that DB = 0.3L. Then 'D' will be the 1st point of base parabola.

(iii) The begining point 'F' of the filter will be the focus of the base parabola with 'D' as centre and DF as radius an arc is drawn to cut water line produced at 'E' through 'E' vertical line EG is drawn EG will be the ... of the base parabola.

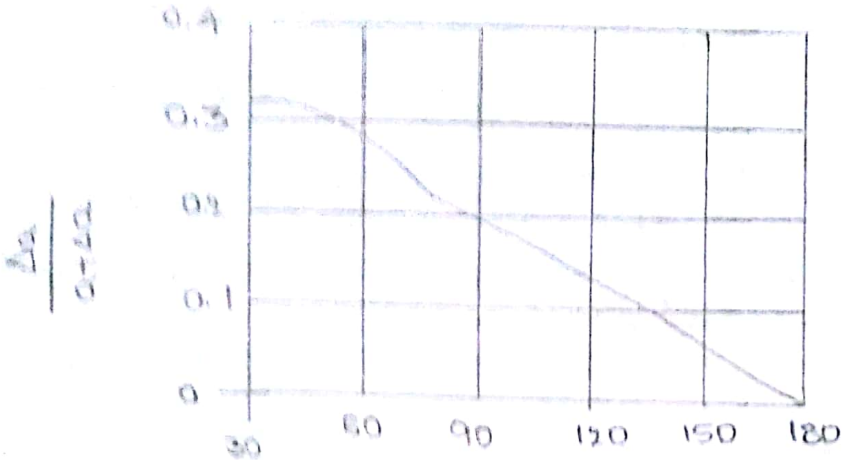
* A midpoint 'J' of FG will be the last point of base parabola in the body of dam
* Intermediate points P_1, P_2 and P_3, P_4 and P_5 on the base parabola are obtained from the properties at any point on the parabola are equidistance from the focus and the diretrix.

Expression of discharge through the body of the dam $\boxed{q = KS}$ S = focal distance

② Homogeneous Earth dam without filter



The portion NF is called discharge phase

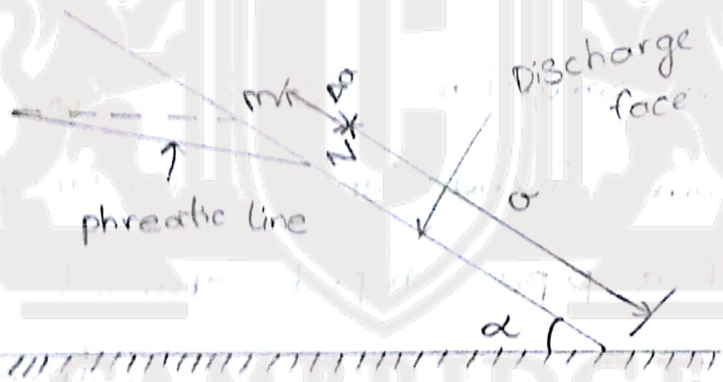


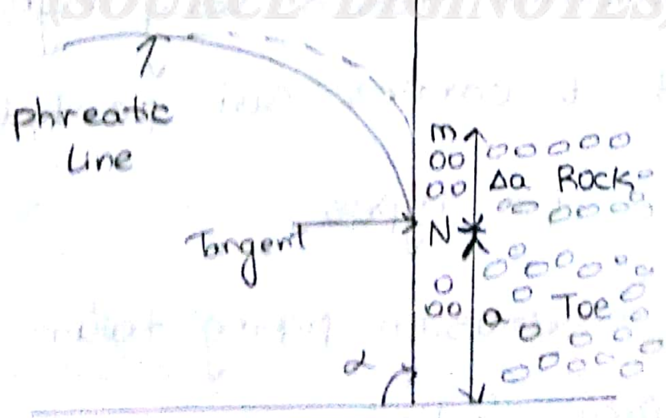
Relation between α and $\frac{\Delta a}{a+\Delta a}$

Angle of inclination α in degrees

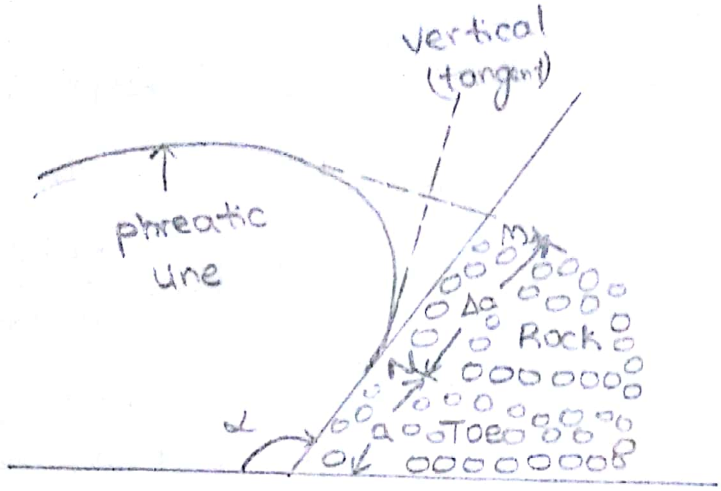
Variation of lower end of phreatic line for different inclinations of discharge face

(i)

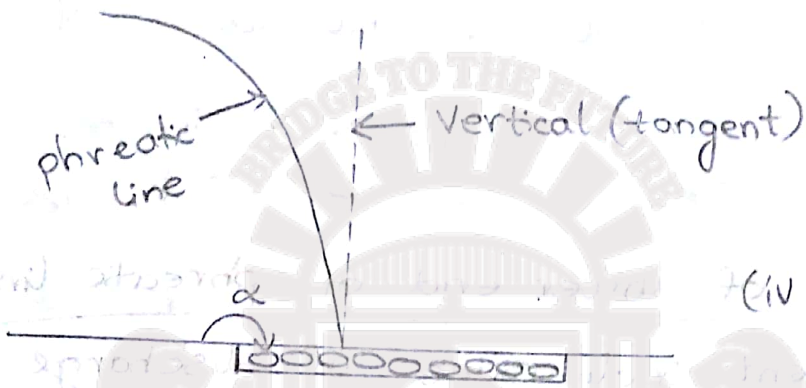




(ii) $\alpha = 90^\circ$



(iii) $\alpha > 90^\circ$



(iv) $\alpha = 180^\circ$

PIPING :-

Hydraullic structures such as Weirs and dams pervious foundations sometimes failed by formation of a pipe shaped channel in its foundation known as piping failure

The failure occurs when water flowing through a foundation has a very high hydraullic gradient and it carries soil particles in it.

There are two types

1. Backward - erosion piping failure
2. Heave piping failure

Geostatic Stress :- Stress caused due to the Self weight of Soil

Effective Stress :- Stress due to self weight and external loading which leads to compression which again leads to decrease in void ratio and increase in frictional resistance

Neutral Stress or pore pressure

Stress due to water present in core which tries to expel soil particles with same pressure in all the directions.

Total stress (σ) = Effective stress ($\bar{\sigma}$) + Neutral stress or pore pressure (u)

$$\boxed{\sigma = \bar{\sigma} + u}$$

Impact of effective stress in construction of Structures

Effective stress plays an important role in

1. Settlement of Soil
2. Shear Strength of Soil

1. Settlement of Soil

The phenomenon of gradual reduction in soil due to expulsion of water from soil force is called consolidation or compression or Settlement of soil.

* As the effective stress strength increases, void ratio decreases.

② Shear strength

Many geotechnical problems required an assessment of shear strength (a) structural foundation (b) Earth slope (c) Highway pavement

(a) Structural Foundation :-

Load from structure transfered to ground through foundation this produces shear stress and compressive stress

* If shear stress is produce is more than the shear strength of soil which comes the structure to collapse.

(b) Earth slope :-

on a sloping ground, gravity produces shear stresses in soil of these stresses exceed the shear ~~triangle~~ strength, a land slide occurs.

(c) High way pavement :-

wheel loads from ~~the~~ vehicles are transferred to pavement to the ground. These loads produce shear stress which causes shear failure

Quick Sand phenomenon :- [Quick condition]

In the case of upward flow of water through a soil mass, the seapage pressure acts in the upward direction causing reduction in effective stress. In case of submerged soil mass, the upward seapage pressure may become equal to downward pressure due to submerged weight of soil at a certain level.

When this happens in the case of cohension less soil, the soil at that level ~~uses~~ looses all its strength as the effective stress becomes zero.

$$C_f = c + \sigma' \tan \phi$$
$$= 0 + 0 (\tan \phi)$$
$$= 0$$

$C_f \rightarrow$ shear strength

$c \rightarrow$ Cohesion

$\sigma' \rightarrow$ Effective stress

$\phi \rightarrow$ angle of internal friction

Because of this, soil particles have tendency to be carried away by flowing water. This phenomenon of lifting of soil particles by flowing water is quick sand phenomenon.