Unit-2

Permeability

Soil Water :

Water present in soil in any form is termed as soil water.

- Soil water
  - Free water
  - Held water
    - Adsorbed water
      - Hygroscopic water
      - Film water
    - Structural water

I. Free water / Ground water :

It is the subsurface water which fills into voids of the soil continuously upto the GWT level.

Free water moves in the pores of the soil under the influence of gravity. Hence it is also termed as Gravity water.

II. Held water :

Held water is retained in the pores of the soil and it cannot move under the influence of gravitational force.

II 1. Adsorbed water:

The water held by electrochemical forces existing on the soil surface is known as Adsorbed water.

a) Hygroscopic water:

It is the water which is being absorbed by the soil solids from the atmosphere by physical forces of attraction and is held over the surface due to cohesion.

- Water adsorption capacity depends on size of soil solids.
- " " " is more for fine grained soil than coarse grained soil due to more specific surface area.

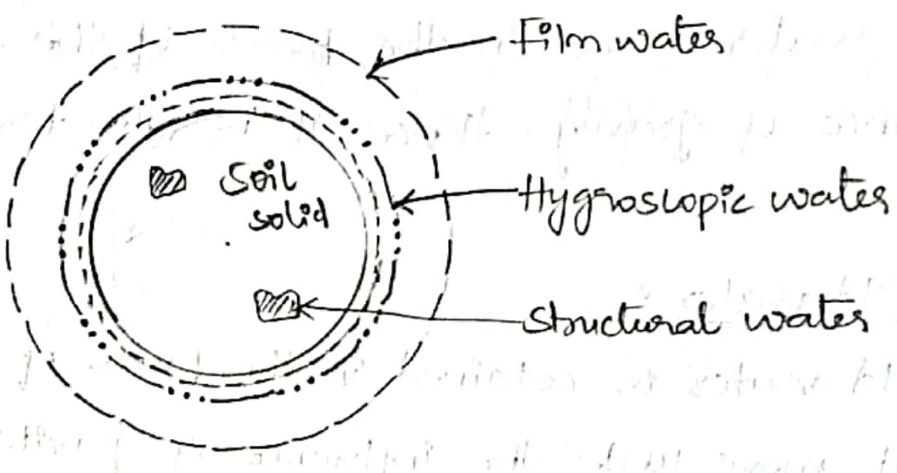
Clay ⇒ 16–17%

Silt ⇒ 6 to 7%

Sand ⇒ <1%

b) Film water:

It is also adsorbed water which is formed due to condensation of aqueous vapour on the layer of the hygroscopic water.



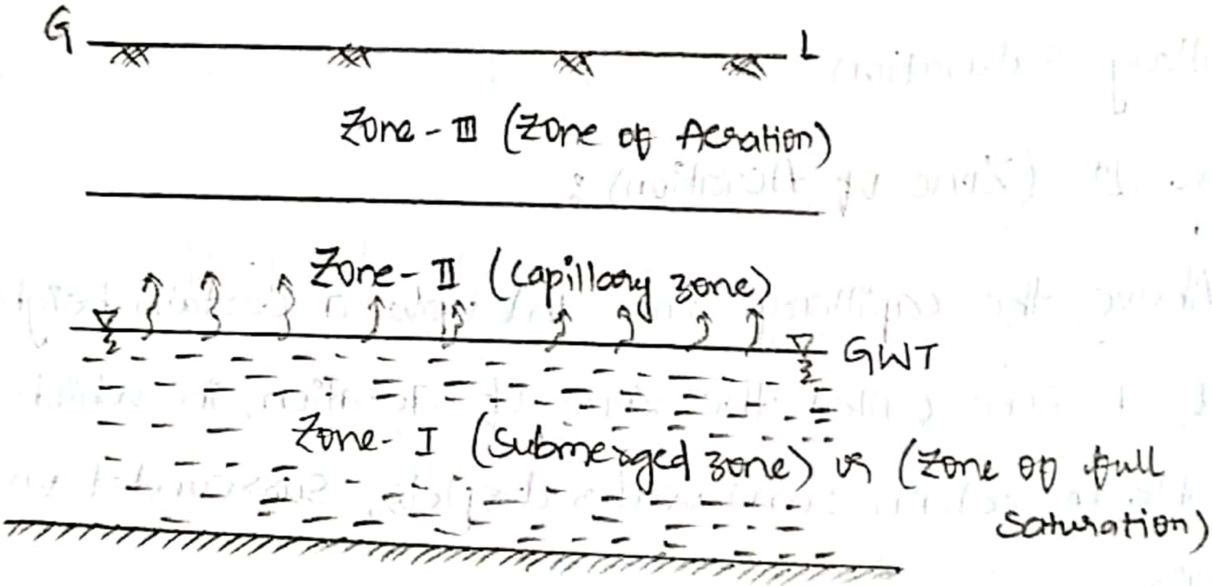
2. Structural water :

It is the water which is combined chemically to the crystal structure of the soil mineral.

• This water cannot be separated or removed under normal engineering activities.

Capillary Rise or Capillarity in Soils

Rain water percolates into the ground water under the influence of gravity and gets stored in the soil pores over an impervious stratum, in the form of ground water reservoir.



The upper surface of the zone of full saturation of the soil is called water table. or phreatic surface.

Soil water can exist in 3 zones as shown below:

Zone - I (Submerged zone) : This zone exist below the GWT. This zone is under submerged condition which is completely filled with water.

Zone – II (Capillarity Zone) :

If gravity was the only force, acting on the Percolating water and taking it downward, then the soil above the water table would be completely dry. But it is not so in actual practice. Soil in this zone is completely saturated upto some height above the water table. This phenomenon of rising water in soil is known as Capillarity in soils.

The water which is lifted by forces above the free water surface level / GWT is known as Capillary water. This water is present in the suspension in the voids of soil and fills it upon a certain distance above the GWT known as Zone of Capillary Saturation.

Zone – III (Zone of Aeration) :

Above the capillary zone and upto a certain height, there exist a zone called the zone of aeration, in which the soils is able to retain small water droplets, surrounded on all sides by air.

Flow of water through soils (or) Permeability:

Soils are Permeable due to the existence of interconnected voids through which water can flow from Points of high energy to low energy.

The property of a soil which permits flow of water [or any other Liquid] through it, is called the Permeability.

[or]

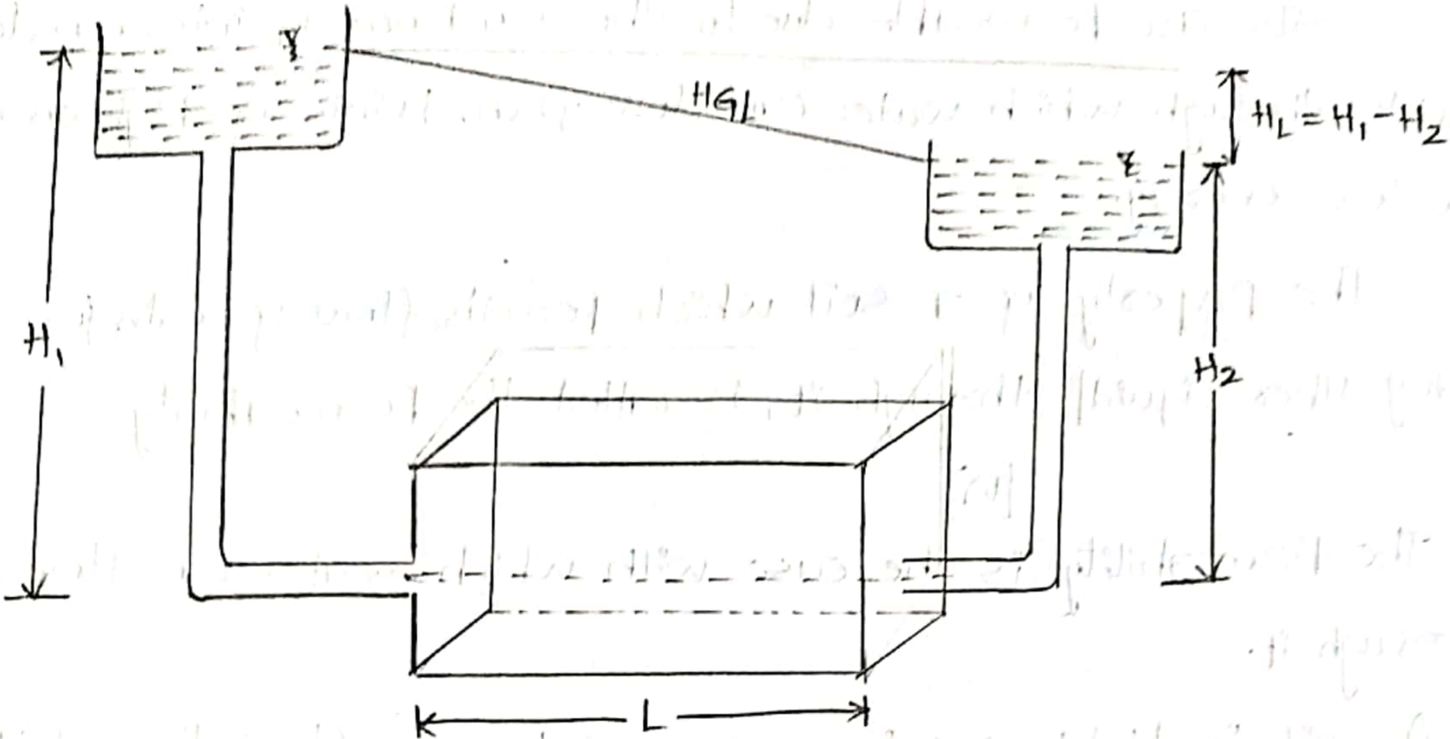
The Permeability is the ease with which water can flow through it.

- A soil is highly pervious when water can flow through it easily. In an impervious soil, the permeability is very low and water cannot easily flow through it.
- A completely impervious soil does not Permit the water to flow through it. However, such completely impervious soils do not exist in nature, as all the soils are pervious to some degree. A soil is termed as impervious when the permeability is extremely low.

Importance of Permeability:

1. It is essential in a number of soil engineering problems, such as settlement of buildings, seepage through & below the earth structures
2. It is also required in the design of filters used to prevent piping in hydraulic structures.

Darcy's Law :



The flow of free water through soil is governed by Darcy's law. In 1856, Darcy demonstrated experimentally that for Laminar flow in a homogeneous soil, the velocity of flow (v) is given by $v \propto i$

$$V = ki$$

Velocity of flow is also known as Discharge velocity.

K = Coefficient of Permeability [m/sec, cm/sec]

i = Hydraulic Gradient $= \dfrac{\text{Seepage Head}}{\text{Length}}$

$$i = \frac{H_1 - H_2}{L} = \frac{H_L}{L}$$

Discharge (q) = Velocity of flow (v) x Total c/s area of soil (A)

$$q = VA = KiA$$

If Hydraulic Gradient = 1, the Coefficient of Permeability is equal to the velocity of flow.

Type of soil	Permeability [cm/sec]
Gravel	>1
Sand	$1-10^{-3}$
Silt	$10^{-3}-10^{-7}$
Clay	$<10^{-7}$

Factors Affecting Permeability of Soils:

1. **Particle Size:**

The Coefficient of permeability of a soil is proportional to the square of the particle size (D).

$$k \propto d^2$$

- Coarse grained soil has more Permeability than that of fine grained soil.

2. **Shape of Particles:**

The Permeability of a soil depends upon the shape of Particles. Angular particles have greater specific surface area as compared with the rounded particles.

$$k \propto \frac{1}{S_s}$$

★ Angular particles are Less ~~more~~ Permeable than rounded particles.

3. Void Ratio :

Greater the void Ratio, the higher the value of the Coefficient of Permeability.

4. Properties of water :

The Coefficient of permeability is directly proportional to the unit weight of water (γ_w) and is inversely proportional to its viscosity (μ).

5. Structure of soil mass :

Stratified/Layered soil deposits have greater permeability parallel to the plane of stratification than that perpendicular to this plane. Permeability of a soil deposit also depends upon shrinkage cracks, joints etc.

6. Degree of Saturation :/Entrapped soil :

If the soil is not fully saturated, it contains air pockets formed due to entrapped air. which causes blockage for the passage of water, hence lower will be the permeability.

7. Adsorbed water :

The fine grained soils have a layer of adsorbed water strongly attached to their surface. This adsorbed water layer is not free to move under gravity. It causes an obstruction to flow of water in the pores and hence reduces the permeability of soils.

8. Impurities in water :

Any foreign matter in water has a tendency to plug the flow passage and reduce the effective voids and hence the Permeability of soils.

$$K \propto d^2, \quad K \propto \frac{1}{S_s}, \quad K \propto e, \quad K \propto \frac{\gamma}{\mu}, \quad K \propto \frac{1}{\text{Air}},$$

$$K \propto \frac{1}{\text{Impurities}}, \quad K \propto \frac{1}{\text{Adsorbed water}}, \quad K$$

Laboratory Determination of Coefficient of Permeability

The Coefficient of Permeability of a soil sample can be determined by the following methods

1) Constant-head Permeability
2) Falling/Variable-head Permeability

1. Constant-head Permeability test : [IS : 2720 (Part 17)]

The Coefficient of Permeability of a relatively more Permeable soil can be determined in a laboratory by the Constant head Permeability test.

$$q = kiA$$

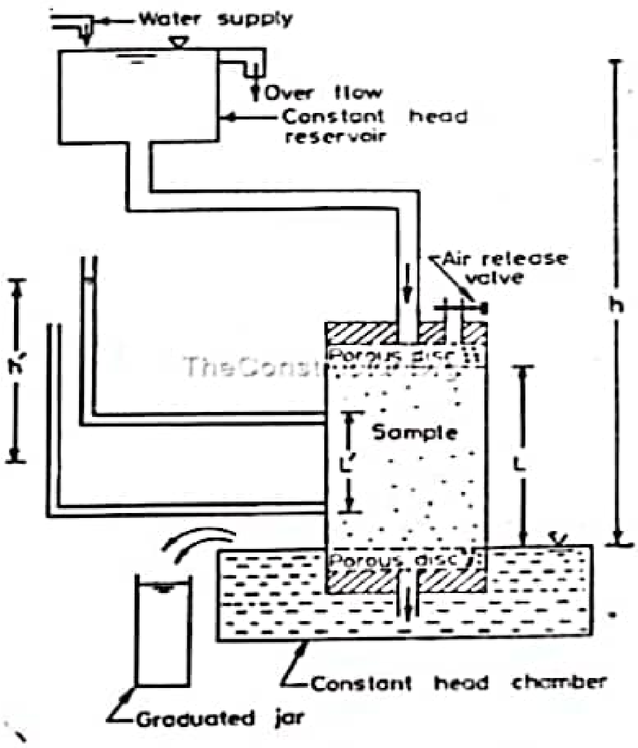
$$K = \frac{q}{i \cdot A} \Rightarrow K = \frac{qL}{hA} \quad \left(\because i = \frac{h}{L}\right)$$

$$K = \frac{QL}{thA} \quad \left[\because q = \frac{Q}{t}\right]$$

Procedure:

1. Select a representative soil mass of about 2.5 kg properly mixed.
2. Fill the soil into the mould and compact it to the required dry density by making use of a suitable compacting device.
3. Set the assembly as shown in figure after saturating the porous stones.
4. The water supply is properly adjusted to maintain constant head.
5. Open the valve and saturate the sample by allowing water to flow through for a sufficiently long time to remove all air-bubbles.
6. When the whole setup is ready for the test, open the valve, allow the water to flow through the sample collect water in a graduated jar starting simultaneously a stopwatch. Note the time to collect a certain quantity of water Q.
7. Repeat the test three times and determine the average of Q for the same time interval t.
8. Measure the head h, length of sample L, and calculate the cross-sectional area A of the sample.
9. Calculate k by making use of equation

Diagram:



Constant head Permeameter

Observation & Calculations:

Length of Soil sample, L = __________

Diameter of Soil sample, D = __________

Area of soil sample, A = __________

SL No.	Volume of water collected, V (cc)	Time, t (sec)	q=V/t	Head over the sample, H (cm)	k = (qL)/(AH) (cm/sec)
1		20			
2		2			
3		20			

q = Volume of water collected divided by time

$$q = \frac{V}{t} \quad \frac{cm^3}{sec} \qquad t = \text{Constant } (20 \text{ sec})$$

L = Length of soil sample (cm)

A = Area of " " (cm^2)

H = Head over the sample (cm)

k = Coefficient of permeability of soil

∴ This test is suitable for clean sand and gravel.

2. Falling head/variable head permeability test :

For relatively less permeable soils, the quantity of water collected in the graduated jar of the constant-head permeability test is very small and cannot be measured accurately. For such soils, the variable-head permeability is used.

The time required for the water level to fall from a known initial head (h_1) to a known final head (h_2) is determined.

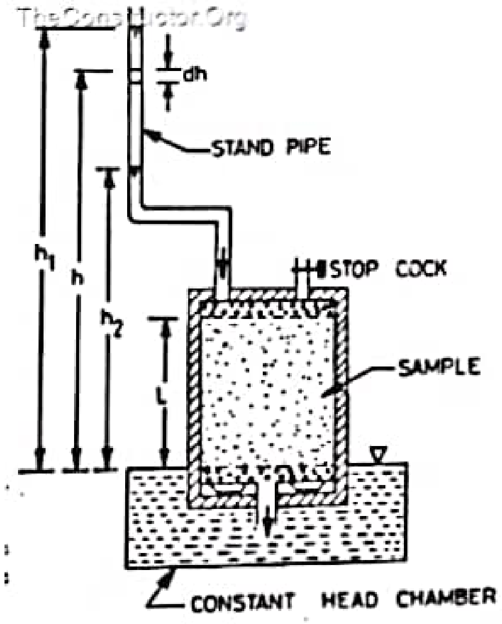
Let us consider the instant when the head is h. For the small time dt, the head falls by dh. Let the discharge through the sample is q.

$$\frac{dh}{dt} = \frac{q}{a} \Rightarrow a \cdot dh = -q \cdot dt$$

Procedure:

1. Connect the specimen to the selected stand-pipe through the top inlet.
2. Open the bottom outlet and record the time interval required for the water level to fall from a known initial head to a known final head as measured above the center of the outlet.
3. Refill the stand-pipe with water and repeat the test till three successive observations give nearly same time interval; the time intervals being recorded for the drop in head from the same initial to final values, as in the first determination.
4. Alternatively, after selecting the suitable initial and final heads, h_1, and h_2, respectively, observe the time intervals for the head to fall from h_1 to (h_1*h_2)^0.5, and similarly from (h_1*h_2)^0.5 to h_2.
5. The time intervals should be the same; otherwise, the observation shall be repeated after refilling the stand-pipe.

Diagram:



Falling head permeameter

Observation & Calculations:

Area of stand pipe, a (cm^2) =

Length of soil specimen, L (cm) =

Cross sectional area of soil specimen, A (cm^2) =

SL No.	Initial head, h_1 (cm)	Final head, h_2 (cm)	Time interval, t (sec)	$k = \frac{2.303 * aL * (\log 10\ (h1/h2))}{A * t}$ (cm/sec)
1				
2				
3				

$$a\,dh = -(A \times K \times i)\,dt$$

$$a\,dh = -A \cdot k \times \frac{h}{L}\,dt$$

$$\frac{A \cdot k\,dt}{aL} = -\frac{dh}{h}$$

$$\frac{AK}{aL}\int_{t_1}^{t_2} dt = -\int_{h_1}^{h_2} \frac{1}{h} \cdot dh = \log h_1 - \log h_2$$

$$\frac{AK}{aL}(t_2 - t_1) = \log_e (h_1/h_2)$$

$$k = \frac{aL}{At} \log_e (h_1/h_2) \qquad t = t_2 - t_1$$

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

The following data were collected in a constant head permeability test :

Internal dia. of permeameter = 7.5 cm

Head lost over a sample length of 18 cm = 24.7 cm

Quantity of water collected in 60 sec = 626 mL

Porosity of the soil sample = 44 %

Calculate the coefficient of permeability of the soil, Also

Given data,

$$D = 7.5\,cm,\quad L = 18\,cm,\quad H = 24.7\,cm$$

$$L = 18\,cm,\quad t = 60\,sec,\quad V = 626\,mL$$

$$A = \frac{\pi}{4}D^2 = \frac{\pi}{4}\times 7.5^2 = 44.18\,cm^2$$

$$Q = KiA$$

$$K = \frac{q}{iA} = \frac{V}{tiA} = \frac{626\ cc}{60\times \frac{H}{L}\times A}$$

$$= \frac{626}{60\times\frac{24.7}{18}\times 44.18}$$

$$K = 0.172\ cm/sec$$

2. In a falling head permeability test, the head causing fall was initially 90 cm and it drops 6 cm in 15 minutes. How much time is reqd for the head to fall to 45 cm

$$h_1 = 90\,cm,\quad h_2 = 90-6 = 84\,cm,\quad t_1 = 15\,min.$$

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

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

$$\frac{2.303\,aL}{KA} = \frac{t}{\log_{10}\left(\frac{h_1}{h_2}\right)}$$

$$\frac{2.303\,aL}{KA} = \frac{15}{\log_{10}\left(\frac{90}{84}\right)}$$

$$\frac{2.303\,aL}{KA} = \frac{15}{\log_{10}\left(\frac{90}{84}\right)}$$

$$= 500.61 \text{ min.}$$

Next head falls from 90 cm – 45 cm

$$T = \frac{2.303\,aL}{KA} \log_{10}\left(\frac{h_1}{h_3}\right)$$

$$= 500.61 \log_{10}\left(\frac{90}{45}\right)$$

$$T = 150.7 \text{ min}$$

3. The falling head permeability test was conducted on a soil sample of 4 cm diameter and 18 cm length. The head fell from 1 m to 0.4 m in 20 min. If the c/s area of the stand pipe was 1 cm², determine the coefficient of permeability

$D = 4\,cm$, $a = 1\,cm^2$, $L = 18\,cm$, $h_1 = 1m$, $h_2 = 0.4m$

$t = 20 \times 60 = 1200 sec$

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

$$= 2.303 \times \frac{1 \times 18}{\frac{\pi}{4} \times 4^2 \times 1200} \cdot \log_{10}\left(\frac{1}{0.4}\right)$$

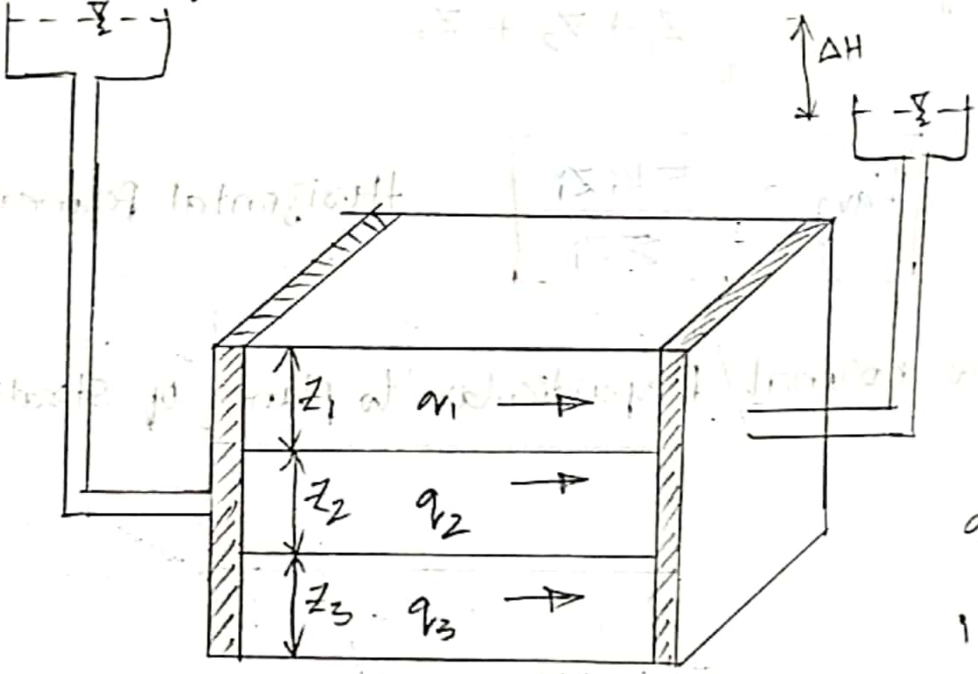
$$k = 0.0109 \text{ cm/sec}$$

Permeability of Layered Soils

A stratified or layered soil deposit consists of number of soil layers having different permeabilities.

Case i) Flow parallel to planes of stratification

Let us consider a deposit consisting of 2 or 3 horizontal layers of soil of thickness Z_1, Z_2, Z_3 as shown in figure.



$q = q_1 + q_2 + q_3$

$i = i_1 = i_2 = i_3$

For flow parallel to the planes of stratification, the loss of head (h) over a length L is the same for all the layers.

Therefore, the hydraulic gradient (i) for each layer is equal to the hydraulic gradient of the entire deposit.

Total discharge $\Rightarrow q = q_1 + q_2 + q_3$, $i = i_1 = i_2 = i_3$

As per Darcy's law, $q = kiA$

$$= K_{avg} \cdot i \cdot (Z_1 + Z_2 + Z_3) 1$$

$$q = K_{avg} \cdot i \cdot \Sigma Z_i$$

$$q_1 = K_1 \cdot i \cdot A_1 = K_1 i (Z_1 \cdot 1)$$

$$q_2 = K_2 i (Z_2 \cdot 1)$$

$$q_3 = K_3 i (Z_3 \cdot 1)$$

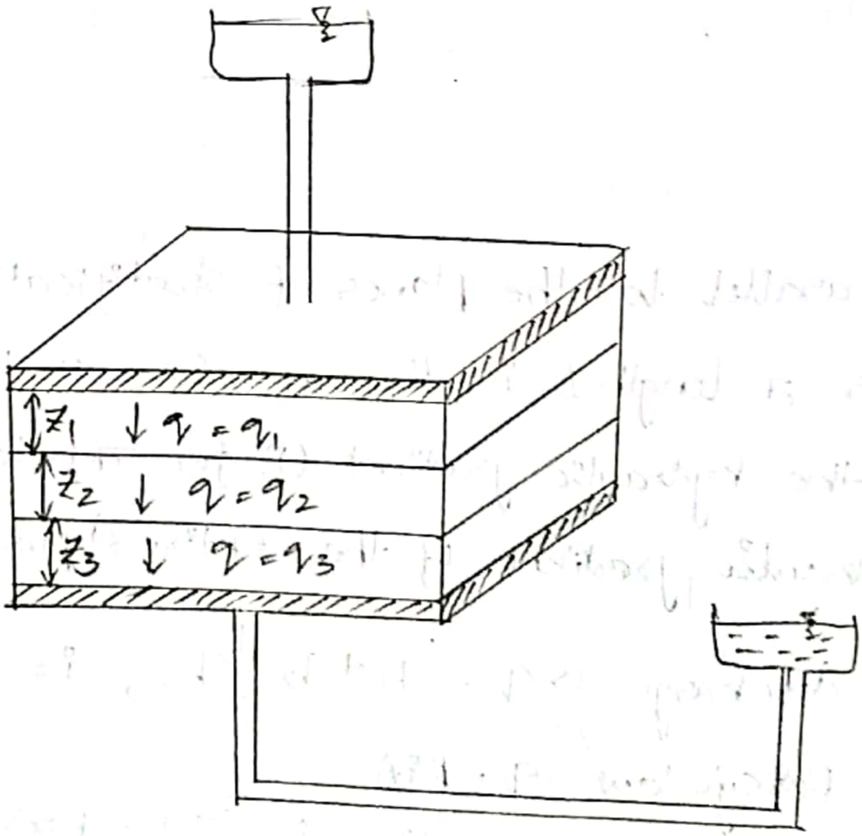
$$q = q_1 + q_2 + q_3$$

$$K_{avg} \cdot \cancel{i} \cdot \Sigma Z_i = K_1 \cancel{i} Z_1 + K_2 \cancel{i} Z_2 + K_3 \cancel{i} Z_3$$

$$K_{avg} = \frac{K_1 Z_1 + K_2 Z_2 + K_3 Z_3}{Z_1 + Z_2 + Z_3}$$

$$\boxed{K_{avg} = \frac{\Sigma K_i Z_i}{\Sigma Z_i}}$$ Horizontal permeability

Case 2) Flow normal/Perpendicular to planes of stratification:



Let us consider a soil deposit consisting of 3 layers of thickness Z_1, Z_2 & Z_3 in which the flow occurs ||to the plane of stratification.

In this case, Discharge per unit width is same for each layer and is equal to the discharge in the entire deposit.

$$q = q_1 = q_2 = q_3 \qquad \text{Total Head Loss} = H_{L_1} + H_{L_2} + H_{L_3}$$

As per Darcy's law,

For entire Deposit :

$$q = K_{avg} \cdot i \cdot A$$

$$= K_{avg} \cdot \frac{H_L}{L} \cdot A$$

$$= K_{avg} \cdot \frac{H_L}{(Z_1+Z_2+Z_3)} \cdot A$$

$$H_L = \frac{q \cdot (Z_1+Z_2+Z_3)}{K_{avg} \cdot A}$$

$$H_L = \frac{q \cdot \Sigma Z_i}{K_{avg} \cdot A} \quad \text{— } 1$$

For individual layers :

$$q = q_1 = q_2 = q_3$$

$$q = K_1 \cdot \frac{H_{L_1}}{Z_1} \cdot A = K_2 \frac{H_{L_2}}{Z_2} \cdot A = K_3 \frac{H_{L_3}}{Z_3} \cdot A$$

$$H_{L_1} = \frac{q \cdot Z_1}{K_1 \cdot A} \qquad H_{L_2} = \frac{q \cdot Z_2}{K_2 \cdot A} \qquad H_{L_3} = \frac{q \cdot Z_3}{K_3 \cdot A}$$

$$H_L = H_{L_1} + H_{L_2} + H_{L_3}$$

$$\frac{q \cdot \Sigma Z}{K_{avg} \cdot A} = \frac{q \cdot Z_1}{K_1 \cdot A} + \frac{q\, Z_2}{K_2\, A} + \frac{q\, Z_3}{K_3\, A}$$

$$\cancel{\frac{q}{A}}\left[\frac{\Sigma Z}{K_{avg}}\right] = \cancel{\frac{q}{A}}\left[\frac{Z_1}{K_1} + \frac{Z_2}{K_2} + \frac{Z_3}{K_3}\right]$$

$$\frac{\Sigma Z}{K_{avg}} = \frac{Z_1}{K_1} + \frac{Z_2}{K_2} + \frac{Z_3}{K_3}$$

$$\frac{K_{avg}}{\Sigma Z} = \frac{1}{\frac{Z_1}{K_1} + \frac{Z_2}{K_2} + \frac{Z_3}{K_3}}$$

$$K_{avg} = \frac{\Sigma Z}{\frac{Z_1}{K_1} + \frac{Z_2}{K_2} + \frac{Z_3}{K_3}}$$

$$K_{avg} = \frac{Z_1 + Z_2 + Z_3}{\left(\frac{Z_1}{K_1}\right) + \left(\frac{Z_2}{K_2}\right) + \left(\frac{Z_3}{K_3}\right)}$$

$$\boxed{K_{avg} = \frac{\Sigma Z}{\Sigma \frac{Z}{K}}}$$ Vertical permeability

Determine the average Coefficient of permeability in the horizontal and vertical directions for a deposit consisting of 3 layers of thickness 5 m, 1m & 2.5 m and having coefficient of permeability of 3×10^{-2} mm/sec, 3×10^{-5} mm/sec and 4×10^{-2} mm/sec respectively.

$Z_1 = 5m$, $Z_2 = 1m$, $Z_3 = 2.5m$

Given Data,

$K_1 = 3\times10^{-2}$ mm/sec, $K_2 = 3\times10^{-5}$ mm/sec, $K_3 = 4\times10^{-2}$ mm/sec

$$K_h = \frac{K_1 Z_1 + K_2 Z_2 + K_3 Z_3}{Z_1 + Z_2 + Z_3}$$

$$= \frac{(3\times10^{-2} \times 5000) + (3\times10^{-5} \times 1000) + (4\times10^{-2} \text{ mm/sec} \times 2500)}{5000 + 1000 + 2500}$$

$K_h = 0.0294$ mm/sec

$$K_v = \frac{Z_1 + Z_2 + Z_3}{\frac{Z_1}{K_1} + \frac{Z_2}{K_2} + \frac{Z_3}{K_3}}$$

$$= \frac{5000 + 1000 + 2500}{\frac{5000}{3\times10^{-2}} + \frac{1000}{3\times10^{-5}} \times \frac{2500}{4\times10^{-2}}}$$

$= 2.5\times10^{-4}$ mm/sec

$= 0.00025$ mm/sec

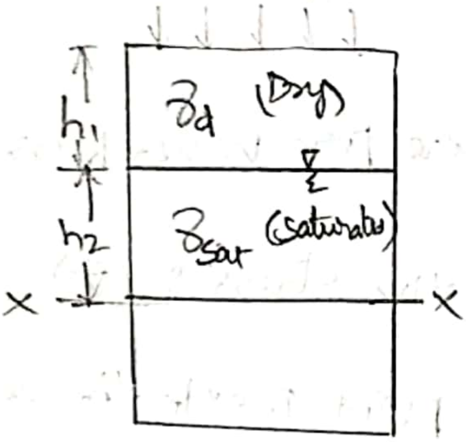
Effective Stress & Seepage through Soils

Total, Neutral and Effective Stress:

Total Stress: [σ]

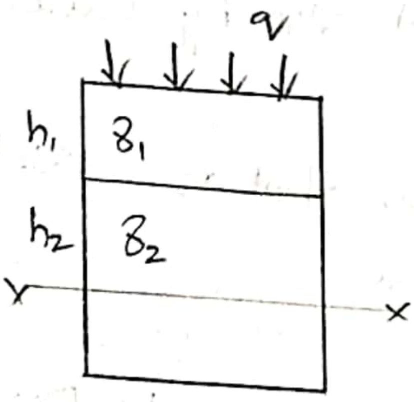
Total Stress at a layer present at a certain depth below ground level is the total weight of the soil present above that layer per unit surface area of the soil mass.

★ Total Stress also includes the stresses due to externally applied loads, if any



Total stress at a layer present at any depth from the ground surface is given by

$$\sigma_{x-x} = \gamma_d \cdot h_1 + \gamma_{sat} h_2 \quad \text{—} ①$$

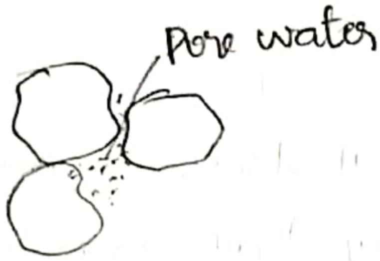


$$\sigma_{x-x} = q + \gamma_d h_1 + \gamma_{sat} h_2 .$$

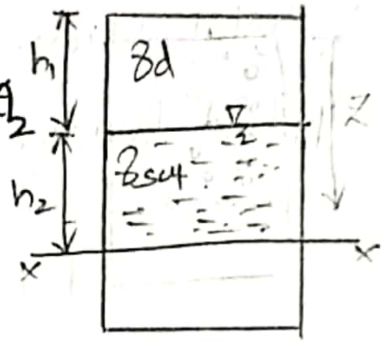
Neutral stress : (u)

As we all know, the soil is a three-phase system i.e., it contains solid, water & air voids. In the case of saturated soils, the pressure exerted by the pore water to its surrounding solid mass is called pore water pressure and is also called "Neutral stress".

Neutral stress can be defined as the stress exerted by the pore water



$$u = \gamma_w \cdot h_2$$
$\downarrow$
(2)



* Piezometer is used to measure pore water pressure.

Effective stress / Principle of Effective stress : ($\bar{\sigma}$)

The total stress at any point inside the soil is resisted by the soil grains and also by water present in the pores or void spaces.

This portion of total stress that is resisted by soil grain is called effective stress.

(or)

Total stress (σ) = Pressure exerted by water present in pores (u) + Pressure exerted by soil ($\bar{\sigma}$)

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

$$\bar{\sigma} = \sigma - u$$

Substitute ① & ② in $\bar{\sigma} = \sigma - u$

$$\bar{\sigma} = (\gamma_d h_1 + \gamma_{sat} h_2) - \gamma_w h_2$$

$$= \gamma_d h_1 + (\gamma_{sat} - \gamma_w) h_2$$

$$\bar{\sigma} = \gamma_d h_1 + \gamma' h_2$$

Where,

γ_d = Dry unit weight of soil

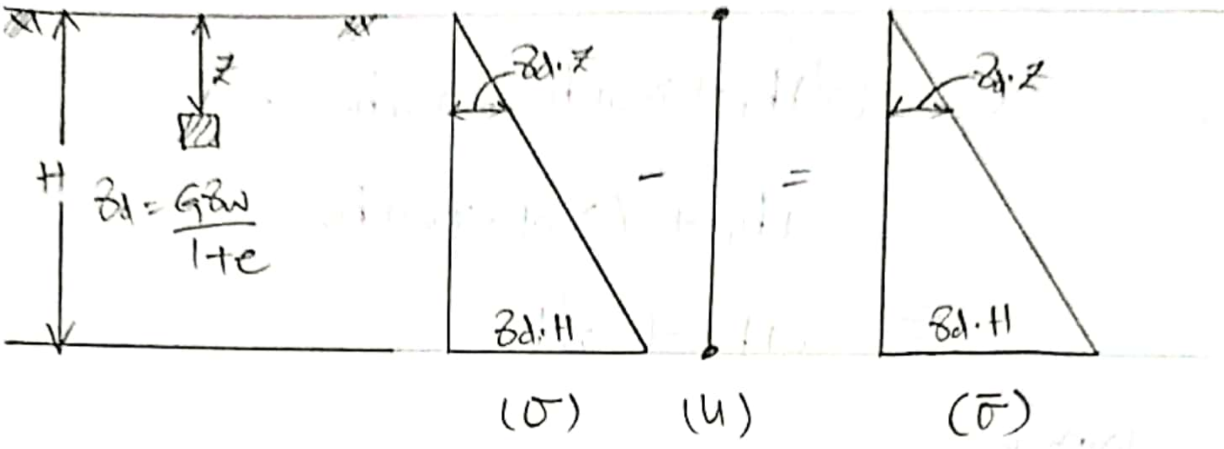
γ_{sat} = Saturated unit weight of soil

γ' = Submerged unit weight of soil.

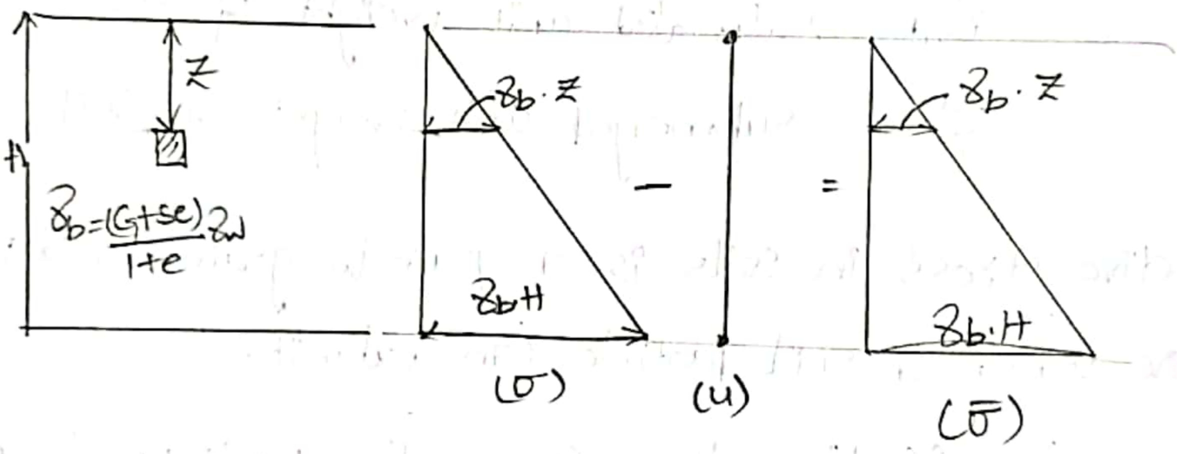
* Effective stress in soils is a grain to grain contact pressure which a soil particle can exhibit.
* Increase in effective stress causes the particles to pace more closely, decreases the void ratio, leads to a decrease in compressibility and increases in the shearing resistance of the soil.
* When there is an equal increase in the total stress and the pore pressure, then effective stress remain unchanged.

$\sigma, u, \bar{\sigma}$ in different soil Conditions:

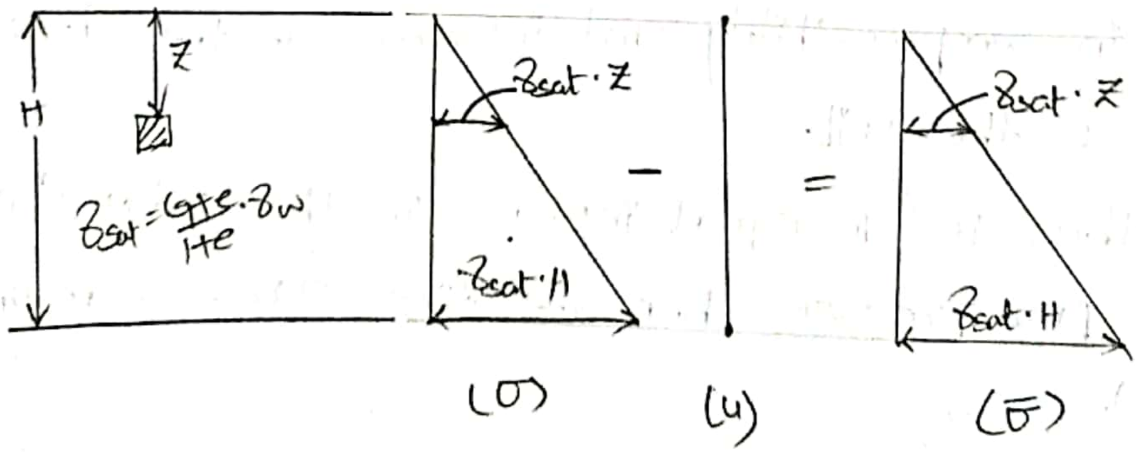
1. When soil is dry:



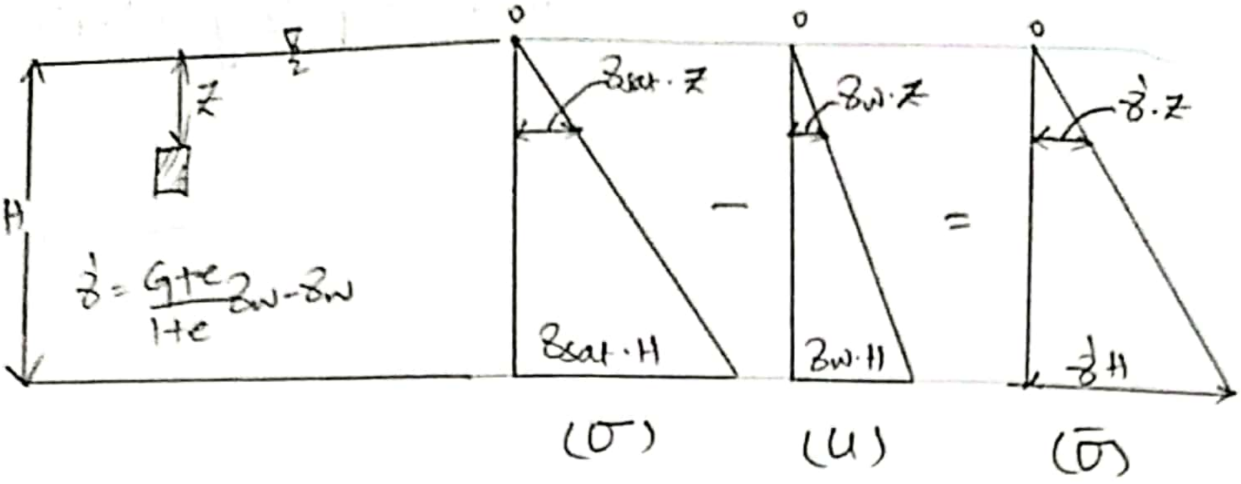
2. When soil is moist:



3. When soil is saturated:



4. When soil is submerged:



$\sigma_{(z)} = \gamma_{sat} \cdot z$

$\sigma_{bottom} = \gamma_{sat} \cdot H$

$u_{(z)} = \gamma_w \cdot z$

$u_{(bottom)} = \gamma_w \cdot H$

$\bar{\sigma}_{(z)} = \sigma - u$

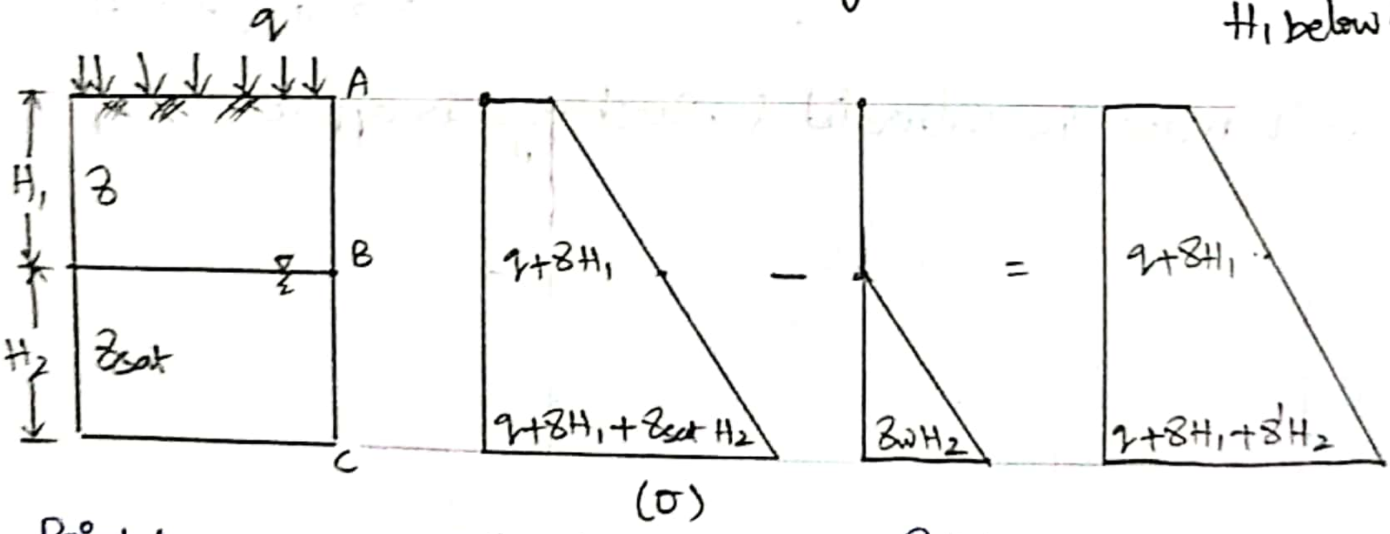
$= \gamma_{sat} \cdot z - \gamma_w \cdot z$

$= (\gamma_{sat} - \gamma_w) z$

$\bar{\sigma}_{(z)} = \gamma' z$

$\bar{\sigma}_{(bottom)} = \gamma' H$

5. Soil mass with uniform surcharge & water table at depth H_1 below GL



Point A

$\sigma = q$

$u = 0$

$\bar{\sigma} = \sigma - u = q$

Point B

$\sigma = q + \gamma H_1$

$u = 0$

$\bar{\sigma} = \sigma - u$

$= q + \gamma H_1$

Point C

$\sigma = q + \gamma H_1 + \gamma_{sat} \cdot H_2$

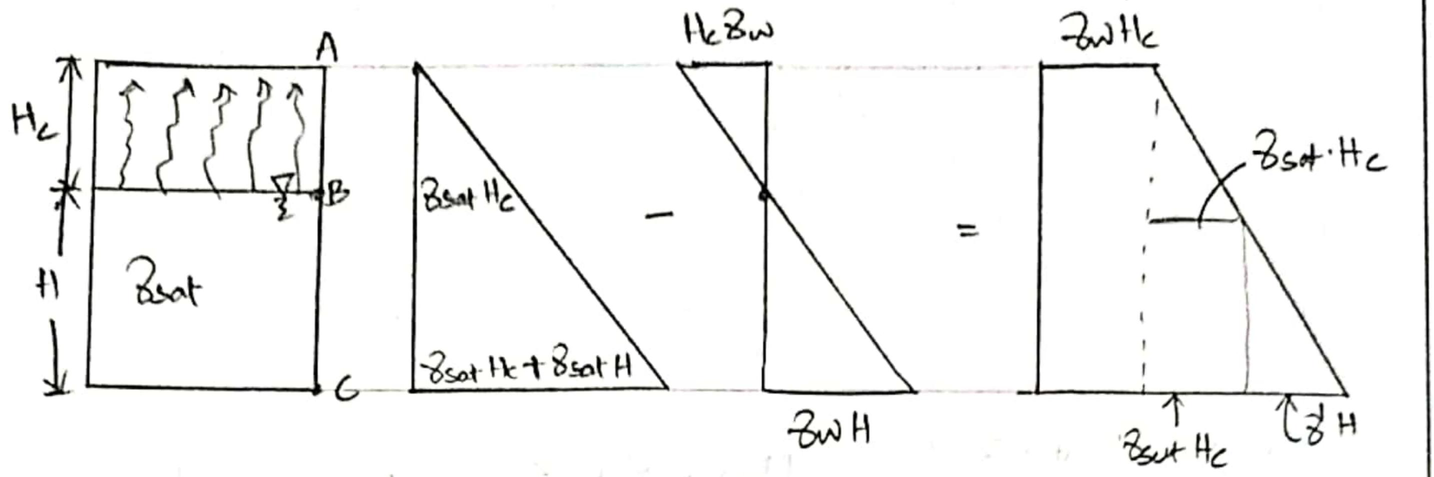
$u = \gamma_w H_2$

$\bar{\sigma} = \sigma - u$

$= q + \gamma_1 H_1 + \gamma_{sat} \cdot H_2 - \gamma_w H_2$

$= q + \gamma_1 H_1 + \gamma' H_2$

6. Soil mass with Capillary (soil is saturated above GWT under Capillary action)



<u>Point A</u>

$\sigma = 0$

$u = -H_c \cdot \gamma_w$

$\bar{\sigma} = \sigma - u$

$= 0 - (-H_c \cdot \gamma_w)$

$\bar{\sigma} = \gamma_w \cdot H_c$

<u>Point B</u>

$\sigma = \gamma_{sat} \cdot H_c$

$u = 0$

$\bar{\sigma} = \sigma - u$

$= \gamma_{sat} \cdot H_c$

<u>Point C</u>

$\sigma = \gamma_{sat} H_c + \gamma_{sat} H$

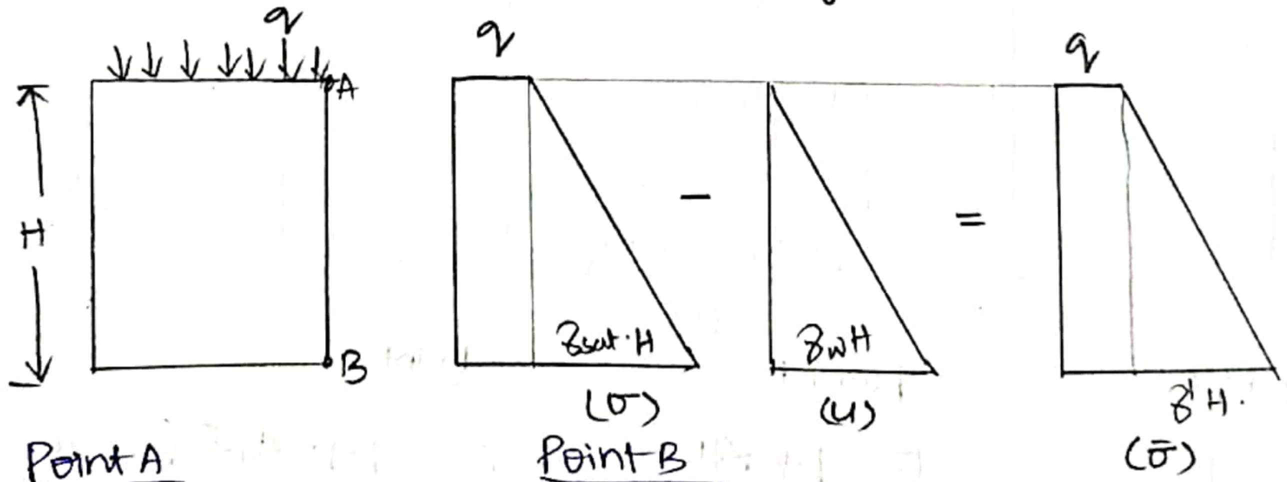
$u = \gamma_w H$

$\bar{\sigma} = \sigma - u$

$= \gamma_{sat} H_c + \gamma_{sat} H - \gamma_w H$

$= \gamma_{sat} H_c + \gamma' H$

7. Soil mass is saturated & surcharge is applied on top



<u>Point A</u>

$\sigma = \cancel{\gamma_{sat} \cdot H}\ q$

$u = 0$

$\bar{\sigma} = q$

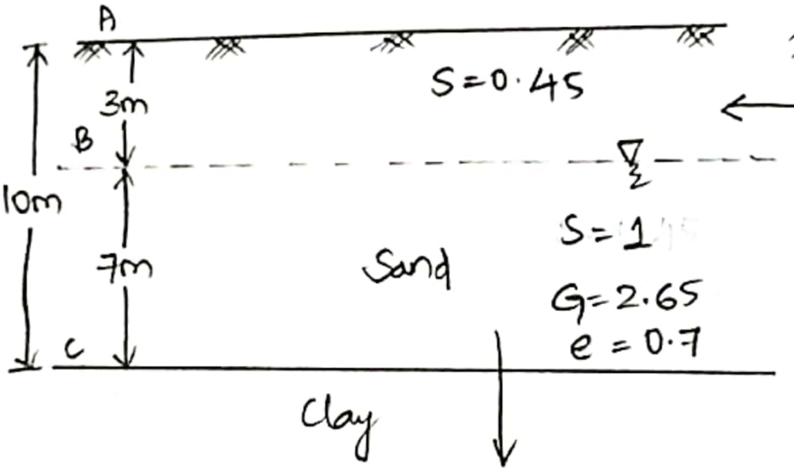
<u>Point B</u>

$\sigma = q + \gamma_{sat} \cdot H$

$u = \gamma_w H$

$\bar{\sigma} = q + \gamma_{sat} \cdot H - \gamma_w H \Rightarrow q + \gamma' H.$

1) A Sand deposit is 10 m thick and overlies a bed of soft clay. The ground water table is 3m below the ground surface. If the sand above the ground water table has a degree of saturation of 45%, plot the diagram showing the variation of the total stress, pore water pressure and the effective stress. The void ratio of the sand is 0.70. Take G = 2.65.



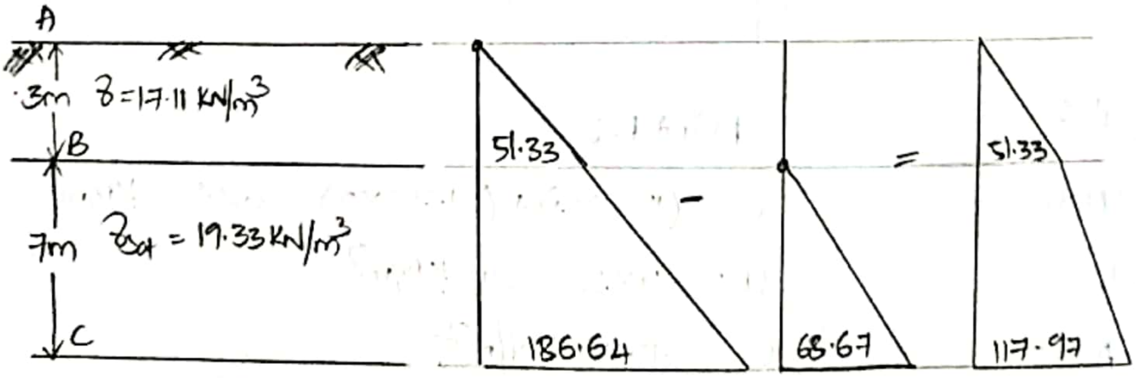
$$\gamma = \frac{G+se}{1+e}\cdot\gamma_w$$

$$= \frac{2.65 + (0.45\times 0.7)}{1+0.7}\times 9.81$$

$$= 17.11 \ KN/m^3$$

$$\gamma_{sat} = \frac{G+e}{1+e}\cdot\gamma_w = \frac{2.65+0.7}{1+0.7}\times 9.81$$

$$= 19.33 \ KN/m^3$$



Point A:

$\sigma = 0$

$u = 0$

$\bar{\sigma} = 0$

Point B

$\sigma = 17.11\times 3 = 51.33 \ KN/m^2$

$u = 0$

$\bar{\sigma} = 51.33 \ KN/m^2$

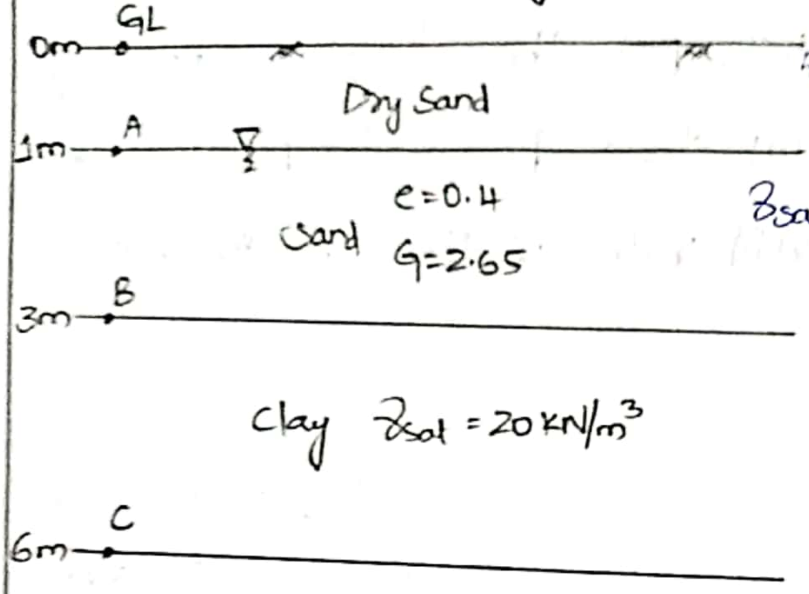
Point C

$\sigma = (17.11\times 3) + (19.33\times 7) = 186.64 \ KN/m^2$

$u = 7\times 9.81 = 68.67$

$\bar{\sigma} = 186.64 - 68.67 = 117.97 \ KN/m^2$

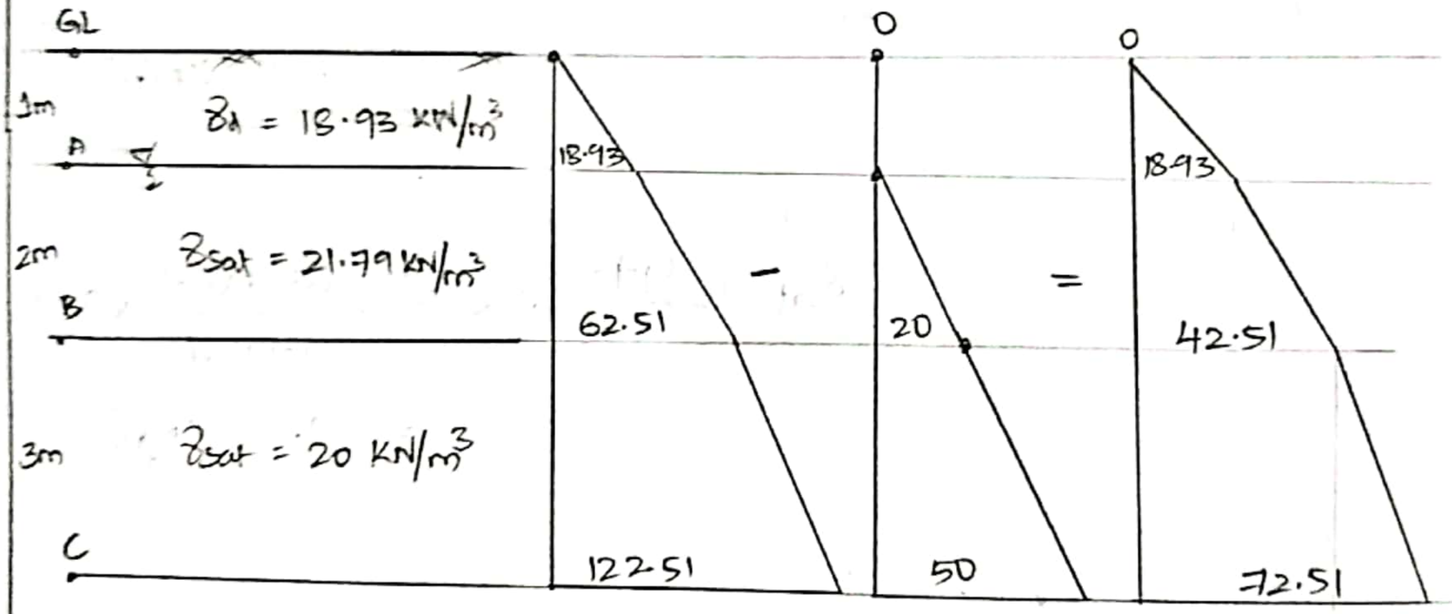
2) For the subsoil condition shown in figure below, Calculate the total, neutral and effective stress at 1m, 3m and 6m below the ground level. Assume $\gamma_w = 10\ KN/m^3$



$$\gamma_d = \frac{G\gamma_w}{1+e} = \frac{2.65\times10}{1+0.4} = 18.93\ KN/m^3$$

$$\gamma_{sat} = \frac{(G+e)\gamma_w}{1+e} = \frac{(2.65+0.4)\times10}{1+0.4}$$

$$= 21.79\ KN/m^3$$



Point GL:

$\sigma = 0,\ u = 0$

$\bar{\sigma} = 0$

Point A:

$\sigma = 18.93\times1 = 18.93\ KN/m^2$

$u = 0$

$\bar{\sigma} = 18.93\ KN/m^2$

Point B:

$\sigma = (18.93\times1) + (21.79\times2) = 62.51\ KN/m^2$

$u = 10\times2 = 20\ KN/m^2$

$\bar{\sigma} = 42.51\ KN/m^2$

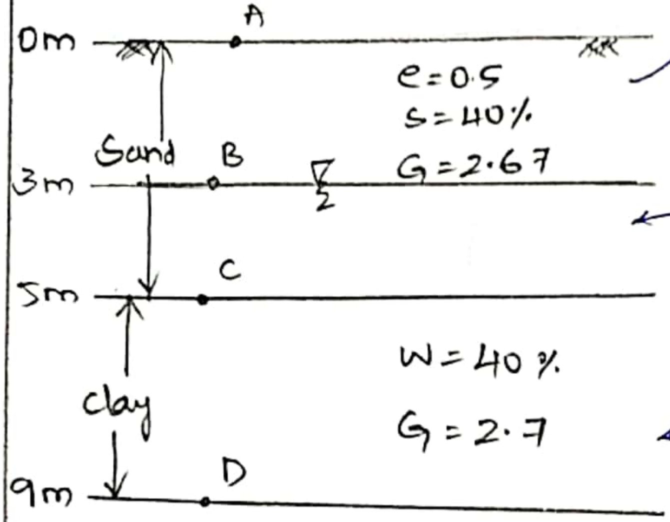
Point C:

$\sigma = (18.93\times1) + (21.79\times2) + (20\times3) = 122.51\ KN/m^2$

$u = 5\times10 = 50\ KN/m^2$

$\bar{\sigma} = 122.51 - 50 = 72.51\ KN/m^2$

3) For the subsoil condition shown in figure. Draw the total, neutral and effective stress diagram upto the depth of 9m, 1



$$\gamma_{Bulk} = \frac{G+Se}{1+e}\cdot\gamma_w = \frac{2.67+(0.4\times0.5)}{1+0.5}\times9.81$$

$$= 18.77\ KN/m^3$$

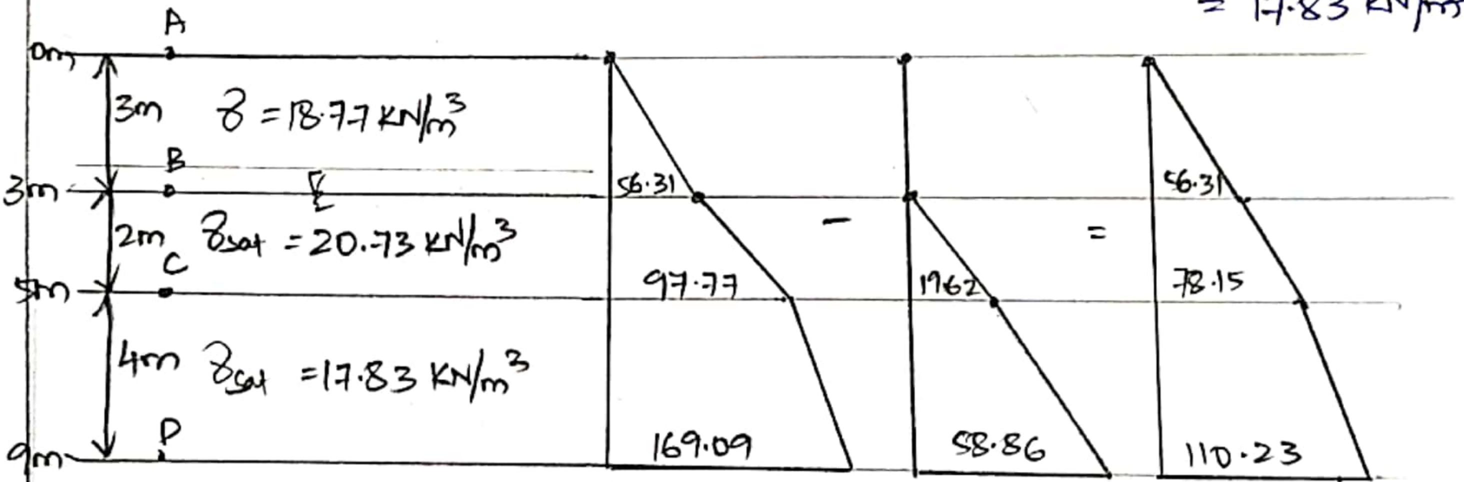
$$\gamma_{sat} = \frac{G+e}{1+e}\cdot\gamma_w = \frac{(2.67+0.5)\,9.81}{1+0.5}$$

$$= 20.73\ KN/m^3$$

$$e = \frac{WG}{S} = \frac{0.4\times2.7}{1} = 1.08$$

$$\gamma_{sat} = \frac{G+e}{1+e}\cdot\gamma_w = \frac{2.7+1.08}{1+1.08}\times9.81$$

$$= 17.83\ KN/m^3$$



Point A:

$\sigma = 0, u = 0$

$\bar{\sigma} = 0$

Point B:

$\sigma = 18.77\times3 = 56.31\ KN/m^2$

$u = 0$

$\bar{\sigma} = 56.31\ KN/m^2$

Point C:

$\sigma = (18.77\times3) + [(20.73)\times(2)] = 97.77\ KN/m^2$

$u = 2\times9.81 = 19.62\ KN/m^2$

$\bar{\sigma} = 97.77 - 19.62 = 78.15\ KN/m^2$

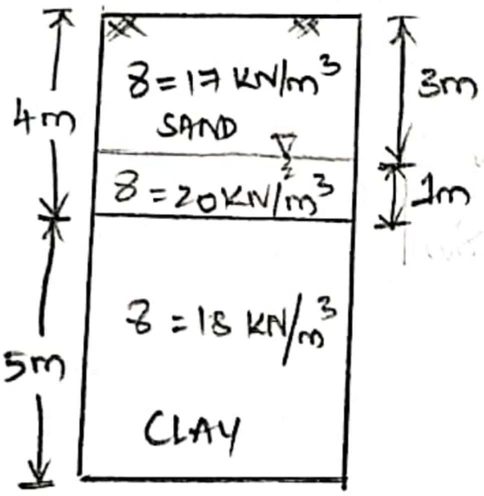
Point D:

$\sigma = (18.77\times3) + (20.73\times2) + (17.83\times4) = 169.09$

$u = 5\times9.81 = 58.86\ KN/m^2$

$\bar{\sigma} = 169.09 - 58.86 = 110.23\ KN/m^2$

A layer of saturated clay 5 m thick is overlain by sand 4 m deep. The water table is 3 m below the top surface. The saturated weights of clay and sand are 18 KN/m³ and 20 KN/m³ respectively. Above the water table, the unit weight of sand is 17 KN/m³. Use $\gamma_w = 9.81$ KN/m³. If the soil gets saturated by capillary, upto the height of 1 m above the water table, then the increase in effective stress at 9 m is



Before capillary

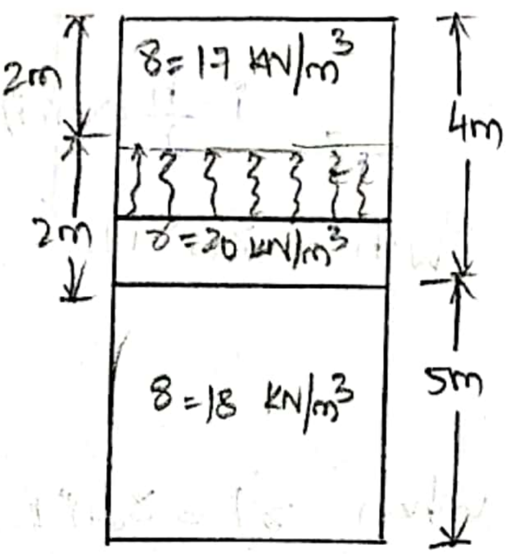
$$\bar{\sigma} = \sigma - u$$

$$\sigma = (3 \times 17) + (20 \times 1) + (5 \times 18)$$

$$= 161 \text{ KN/m}^2$$

$$u = 6 \times 9.81 = 58.86 \text{ KN/m}^2$$

$$\bar{\sigma} = 161 - 58.86 = 102.14 \text{ KN/m}^2$$



After Capillary Rise

$$\bar{\sigma} = (2 \times 17) + (2 \times 20) + (5 \times 18)$$

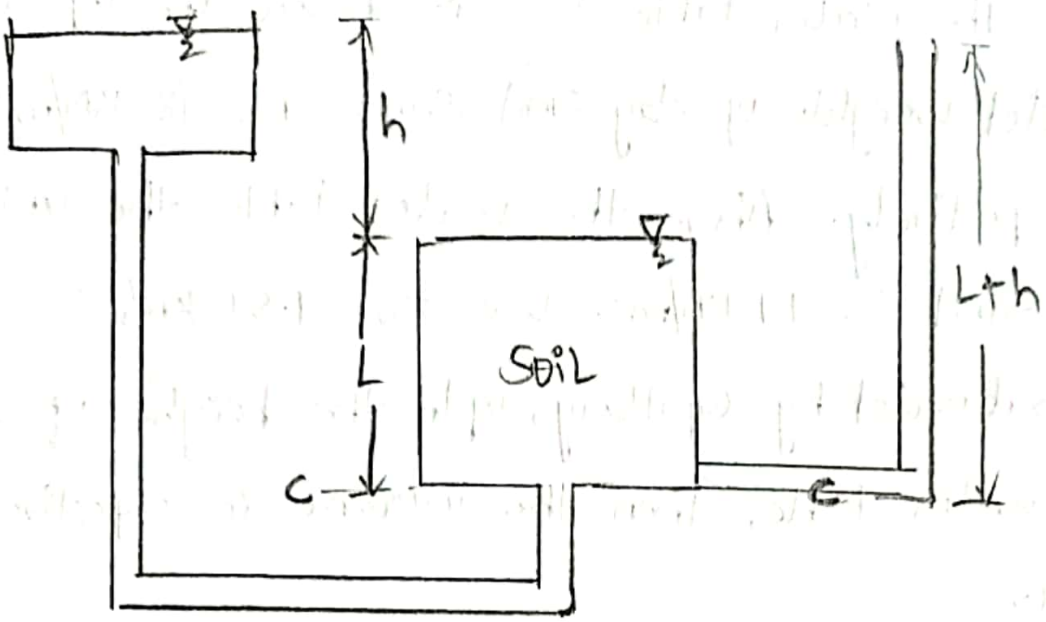
$$= 164 \text{ KN/m}^2$$

$$u = 7\gamma_w - 1\gamma_w = 6\gamma_w = 58.86$$

$$\bar{\sigma} = 164 - 58.86 = 105.14 \text{ KN/m}^2$$

$$\Delta\bar{\sigma} = 105.14 - 102.14 = 3 \text{ KN/m}^2$$

Quick Sand Condition:



Let us Consider the stresses developed at section C-C

$$\sigma = \gamma_{sat} \cdot L \qquad \left[\begin{array}{l} \gamma' = \gamma_{sat} - \gamma_w \\ \gamma_{sat} = \gamma' + \gamma_w \end{array}\right]$$

$$\sigma = (\gamma' + \gamma_w) L$$

$$u = \gamma_w (L+h)$$

$$\bar{\sigma} = (\gamma' + \gamma_w) L - \gamma_w (L+h)$$

$$= \gamma' L + \cancel{\gamma_w L} - \cancel{\gamma_w L} - \gamma_w h \qquad i = \frac{h}{L} \Rightarrow h = iL$$

$$\bar{\sigma} = \gamma' L - \gamma_w h \qquad \gamma_w \cdot h = \gamma_w \times iL$$

$$\bar{\sigma} = \gamma' L - \gamma_w iL$$

Effective stress ($\bar{\sigma}$) becomes zero when $\gamma' L = \gamma_w i L$

$$\gamma' \cancel{L} = \gamma_w i \cancel{L}$$

$$i = \frac{\gamma'}{\gamma_w}$$

$$i = \frac{\gamma'}{\gamma_w}$$

The hydraulic gradient at which the effective stress becomes zero is known as Critical gradient (i_c)

$$i_c = \frac{\gamma'}{\gamma_w}$$

$$= \frac{\left(\frac{G-1}{1+e}\right)\gamma_w}{\gamma_w}$$

$$i_c = \frac{G-1}{1+e}$$

For fine sands & silt for which specific gravity $= 2.65$

void ratio $= 0.65$

$$i_c = \frac{2.65-1}{1+0.65} = 1$$

When flow takes place in upward direction, seepage pressure also acts in upward direction and effective stress is reduced.

If seepage pressure equals to submerged weight of soil mass then effective stress reduces to zero, in such case cohesionless soils mass looses its shear strength & have the tendency to flow along with the water. This phenomenon in which soil particles leaves the soil mass and flow along with the water is termed as Quick sand/

Piping/ Sand Boiling or floating condition

At quick sand condition, cohesionless particles of fine sand may start flowing with the water which may result in piping failure below the hydraulic structure.

- In order to prevent quick sand or piping failure, the hydraulic gradient should be less than critical hydraulic gradient. Hence factor of safety against quick sand failure or piping failure is

$$FOS = \frac{i_c}{i} \qquad i = \frac{h}{L}$$

★ Quick sand is not a type of sand. It is a flow condition which exists in cohesionless soil mass where effective stresses are reduced to zero in upward flow conditions.

★ Quick sand conditions is found only in fine sand and coarse silts and it is not being observed in the case of gravels, coarse sands and clays.

A soil sample (sand) obtained from trench has water content of 38%. The specific gravity of solid is 2.65. Determine the critical hydraulic gradient for the soil. Also determine the maximum permissible upward gradient, if a factor of safety of 3 is considered.

Given Data,

$$w = 38\% = 0.38$$

$$G = 2.65, \quad FOS = 3 \qquad i_c = \frac{G-1}{1+e}$$

For saturated soil,

$$Se = wG$$

$$1 \times e = 0.38 \times 2.65$$

$$e = 1.007$$

$$i_c = \frac{2.65-1}{1+1.007} = 0.822$$

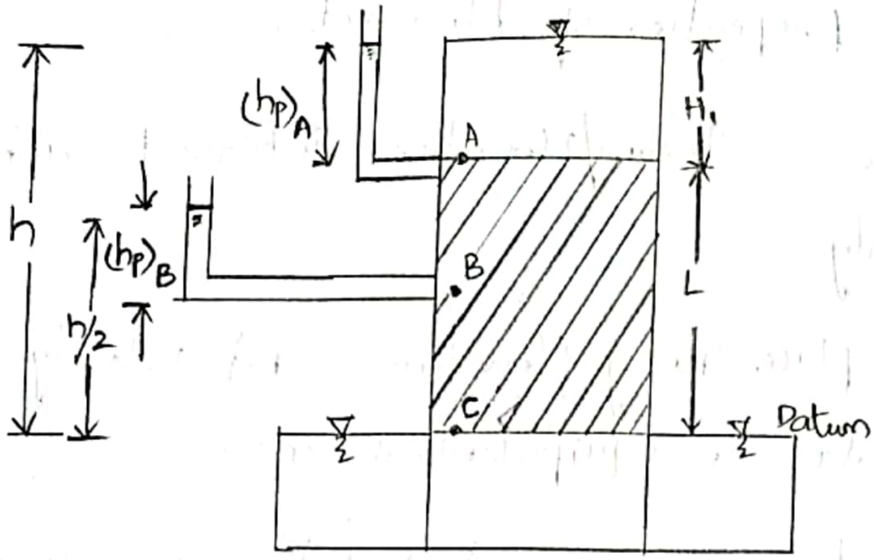
$$FOS = \frac{i_c}{i} \Rightarrow i = \frac{i_c}{FOS} = \frac{0.822}{3} = 0.274$$

Seepage through Soils : Flow net

Seepage is the flow of water through permeable medium under gravitational forces.

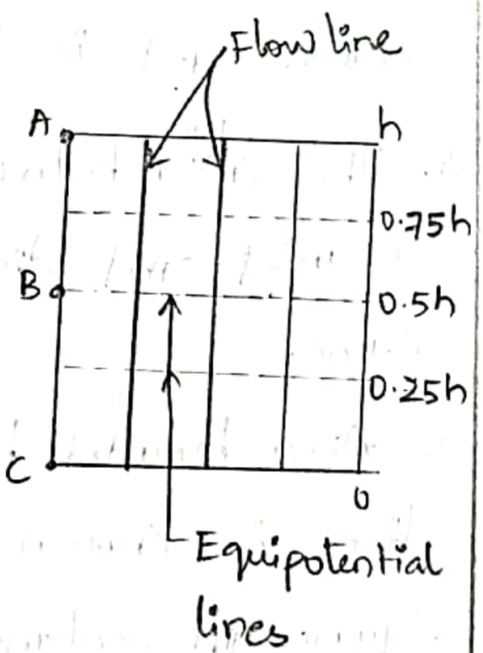
☆ Flow of water takes place from a point of high head to a point of low head. The flow is generally laminar.

- The path taken by a water particle is represented by a flow line. At certain points on different flow lines, the total head will be the same. The lines connecting points of equal total head can be drawn. These lines are known as equipotential lines.
- As flow takes place along the steepest hydraulic gradient the equipotential lines cross flow lines at right angles.
- The flow lines and equipotential lines together form a "FLOW NET".
- The flow net gives a pictorial representation of the path taken by water particles and head variation along that path.

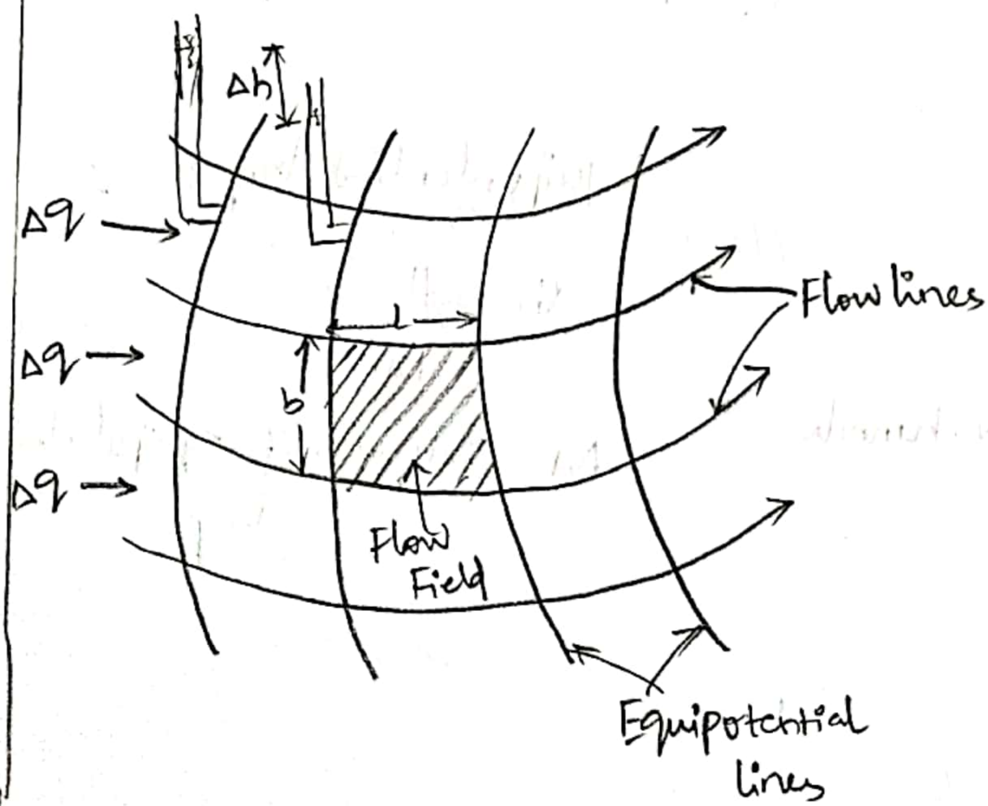


$L + H_1 = h$

Point	Elevation or (h_e) Datum head	Pressure head (h_p)	Total head $H = h_e + h_p$
A	L	H_1	$L+H_1 = h$
B	0.5 L	$0.5L + H_1 - 0.5h$	$0.5L+0.5L+H_1-0.5h = L+H_1-0.5h = 0.5h$
C	0	0	0



FLOW NET



Characteristics or Properties of Flow Net :

1. Equipotential lines & stream/Flow lines intersect each other perpendicularly.

2. Flow will take place along flow lines and velocity of flow is perpendicular to equipotential lines.

3. Loss of head between 2 equipotential lines is always same and is termed as equipotential drop.

4. The ~~area~~ space between 2 flow lines is known as flow channel and discharge through each flow channel is same.

5. Area bounded between 2 equipotential lines and flow lines is known as flow field which are approximately square and rectangular in isotropic & non-isotropic medium respectively.

Total Discharge:

$$q = \Delta q \times N_f$$

N_f = Number of flow channels

$$\Delta q = \frac{q}{N_f}$$

Equipontential drop:

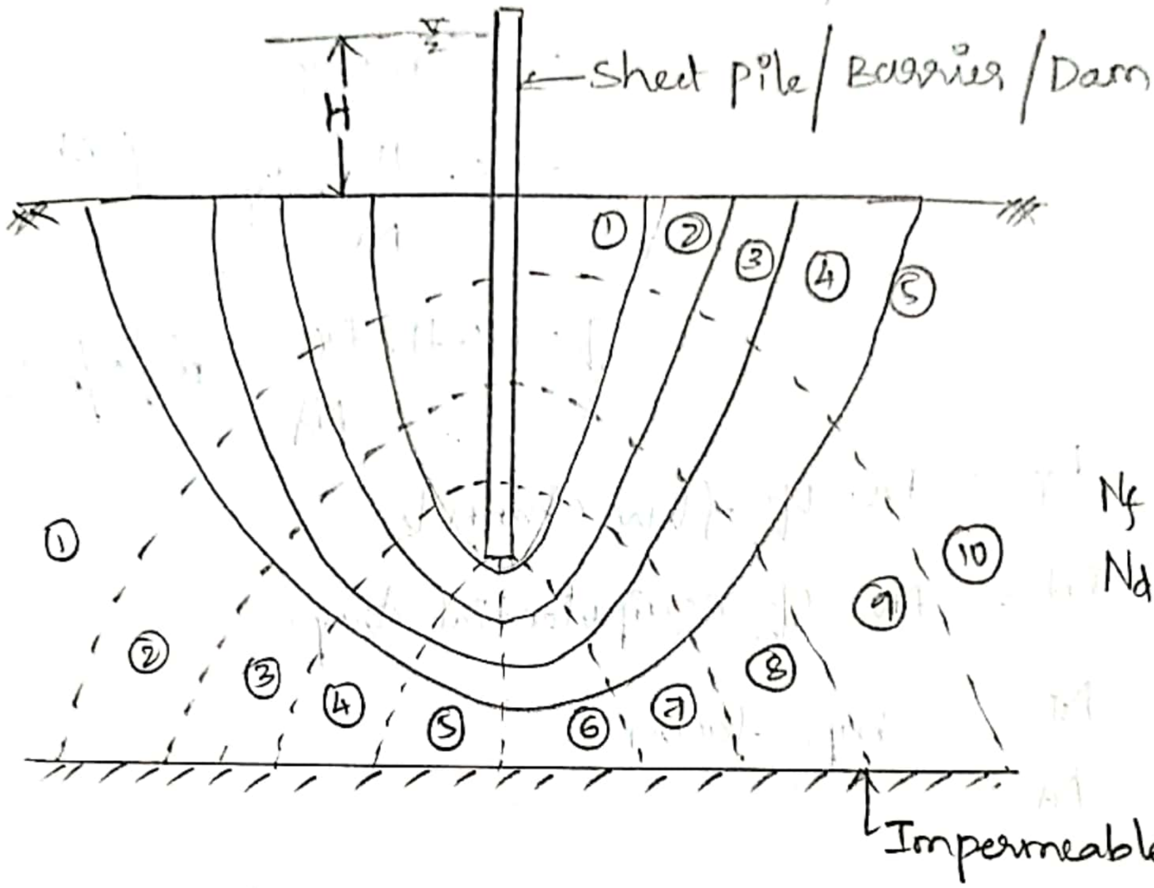
$$\Delta h = \frac{H}{N_d}$$

N_d = Number of equipotential drops.

Applications or Uses of Flow Net :

The flow net can be used for a number of Purposes

1. Discharge :



$N_f = 5$
$N_d = 10$.

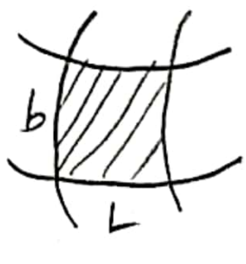
Let us Consider Δq is the discharge through each flow field

Consider 1m length of dam or any barrier

As Per Darcy's law,

$$q = K i A$$

$$i = \frac{\Delta h}{L} \qquad A = (b \times 1m)$$



$$q = K i A$$

$$\Delta q = K . \frac{\Delta h}{L} . b$$

$$\Delta q = K . \Delta h . \left(\frac{b}{L}\right)$$

If $b = L$ [Isotropic medium)

$$\Delta q = K \cdot \Delta h$$

Total discharge $\Rightarrow q = \Delta q \times N_f$

$$= K \Delta h \times N_f$$

$$= K \cdot \frac{H}{N_d} \cdot N_f \quad \left(\Delta h = \frac{H}{N_d}\right)$$

$$q = K \cdot H \cdot \frac{N_f}{N_d} \quad m^3/s/m \text{ length of dam}$$

N_f = No. of flow channels

N_d = No. of equipotential drops

$\frac{N_f}{N_d}$ = Shape factor

2. <u>Total head</u>:

Total head at any point

$$T.H = H - n \Delta h$$

$$T.H = H - n \cdot \frac{H}{N_d}$$

n = No. of equipotential drops at the point which is considered

3. <u>Pressure head</u>:

Total head = Pressure head + Datum head

Pressure head = Total head − Datum head

$$P.H = H - n_1\left(\frac{H}{N_d}\right) - (D.H)$$

4. <u>Hydraulic gradient</u>:

The average value of hydraulic gradient for any flow is given by

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

Δh = Head loss

l = Length of the flow field.