SOIL MECHANICS

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SYLLABUS

1. Origin of Soil
2. Soil water Relationship
3. Classification of Soil
4. Clay Mineral & Soil structure.
5. Compaction of soil
6. Permeability
7. Effective Stress
8. Seepage

→ Important for Gate.

9. Compressibility & Consolidation
10. Shear Strength of soil
11. Vertical Stress.
12. Earth Pressure theory
13. Shallow & Deep foundation.
14. Exploration, Expansive soil, Stability of Slope

} (Application)

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ORIGIN OF SOIL

CHAPTER-1

(#) Soil: ⊙ Soil is an unconsolidated material composed of Solid Soil particles produced by the disintegration of rock or decomposition of vegetative matter.

⊙ Formation of soil is due to disintegration and transportation of igneous rock called parent rock or due to the decomposition of vegetative matter.

⊙ Disintegration of rock is due to weathering which are classified as:

(a) Physical weathering
(b) Chemical weathering.

The physical weathering processes can be:

① Erosion of rock due to wind, water of glacier.
② Expansive force due to freezing water.
③ Sudden change in temperature.
④ Organic activity like growth of plant roots in the existing crack, rodents and worms --- etc.
⑤ Unloading and resulting cracking.

⊙ Soil formed by physical weathering has same mineralogical composition as that of the parent rock.

⊙ The particles that are formed are bulky like Sand and Gravel.

⊙ The structural arrangement is Single grain structure

⊙ Single Grain structure has no attraction & repulsion btw the particles

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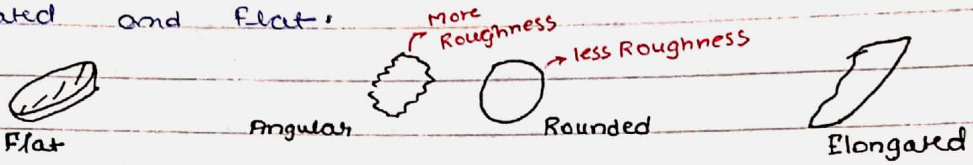
Hydrous Mica → clay Mineral (Illite)

Mica → Not clay Mineral.

⊙ The shape of particle formed are angular, rounded, elongated and flat:



(Based on surface Roughness)

(Not clay Mineral)

⊙ Quartz, a feldspar, and Mica are primary soil Minerals formed due to physical weathering.

⊙ Chemical Weathering: The various chemical processes during chemical weathering are:

(i) Hydration → Reaction of minerals with H^+ and OH^- of water and these ions replace the existing cation.

(ii) Oxidation → Acid produce during this process causes weathering.

(iii) Carbonation → CO_2 in water dissolves minerals and they are carried away.

(iv) Desilication → Leaching of Silica.

Leaching means when a water soluble parts are dissolved and washed out, the process is called leaching. In this process, mineral form of parent rock ~~drop~~ changes.

⊙ Chemical weathering process results in the formation of crystalline particles of colloidal size (small size) known as clay Minerals like Kaolinite, Illite and Montmorillonite.

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clay (<2μ)

Note: ⊙ If (-2μ) size of particle have cohesion b|w them then it is called clay particle but if there is no cohesion, they it is clay sized particle.

⊙ Most clay mineral particles have plate like form, having high specific surface (high surface area per unit volume or mass) with the result that the behaviour is affected significantly due to the pressure of surfaces forces & water can significantly affect the behaviour of clay.

Residual and Transported soil

⊙ If the soil is still located at the place of its origin, it is called residual soil and if it has been transported by wind water of glacier, it is called Transported soil.

⊙ Residual soils have better engg. properties and they have angular soil particles.

Laterite is an Example of Residual soil.

⊙ Transported soil have smaller grain particles and larger amount of pores.

⊙ Depending upon the transporting agency, soils are classified as:

Alluvial soil → deposited due to → Running water (River)

Lacustrine soil → deposited in → Still water (Lake)

Marine soil → deposited in → Sea water

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Bentonite
Black cotton → Montmorrilonite clay

collapse
Loess → silt, air deposited, uniform size

< 2μ → clay
2μ – 75μ → silt
75μ – 4.75mm → sand
> 4.75mm → Gravel

High compressibility
Aealine soil → Transported by winds (also large permeability)
Glacial deposits → Transported by ice or glaciers

Names of different types of soil :

① **Bentonite:** It has high percentage of clay mineral Montmorrilonite. (Expansive soil). It is highly plastic and results from the decomposition of Volcanic Ash, and it is highly water absorbent and greater swelling and shrinkage tendancy.

② **Black cotton soil:** It is a residual soil containing high percentage of Montmorrilonite.

⊙ It has low Bearing capacity, high swelling shrinkage and formed due to chemical weathering of Basalt rock (dark in colour, good for growing cotton).

Ⓐ This soil is stablized using lime.

③ **Loess:** ⊙ It is a fine Grained (Silty size) [< 75μ].

⊙ < 75μ particles are fine Grained particles.

⊙ It is homogenous, friable (Easily crushable), air borne deposits having very uniform Grain size and high void ratio.

⊙ It can stand deep vertical cut because of slight cementation b/w particles due to $CaCO_3$ and Montmorrilonite.

⊙ It has continuous root holes and hence permeability in vertical direcn is much greater than in the

$K_V > K_H$

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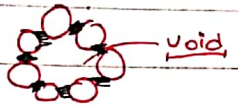
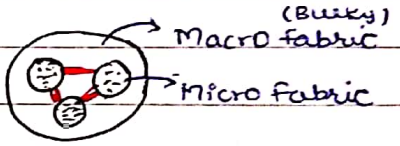
Marl → Greenish
Varved → Glacial Lakes

horizontal direcn. It is found in Arid and Semi-arid region and it is highly porous and subjected to collapse when saturated.



⊙ It has loose meta stable (unstable) fabric. (structure (arrangement), low density and high compressibility.

⊙ It has low wearing capacity.



⊙ Its Macro fabric is Bulky Granular.

④ Till Soil: It is an unstratified deposit formed by melting of Glaciers. The deposit consist of particle of diff sizes ranging from (>80mm) Boulders to clay.
So, it can be easily densifed by compaction and it is a well graded soil.

⊙ It has high shear strength and this soil is also known as Boulder clay.

⑤ Marl Soil: ⊙ Its a stiff, marine, calcareous clay of (Greenish) colour.

⑥ Varved clay: ⊙ These are sedimentary deposits consisting of alternate thin layers of clayey (Adjective) silt (Noun) or silty (Adjective) clay (Noun)

Main component is known.

⊙ These clays are results of depositon in glacial lakes

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Lithification → Indurated clay → doesnot soft under prolonged wetting

Black cotton, Basalt, Laterite

during periods of alternate high and low waters.

⊙ Shear resistance of varved clay is much smaller along horizontal plane and Hydraulic conductivity is much greater in Horizontal plane as compared to that in vertical plane.

⑦ Indurated clay: ⊙ Induration means clay has become more firm, and does not soft under prolonged wetting.
⊙ Induration is the term used to describe degree of Lithification.
⊙ Lithification is the conversion of unconsolidated sediments into sedimentary rock by porosity destruction through compaction and cementation.

⑧ Diatomaceous Earth Soil: ⊙ Diatoms are minute unicellular marine organisms.
⊙ Diatomaceous Earth is a fine, light grey, soft, sedimentary deposit of silicious remains of skeletons of diatoms.

⑨ Laterite Soil: Lateritic soils are formed by decomposition of rock, removal of Silica and accumulation of Iron and Al oxides.
⊙ It's a residual deposit forms from Basalt.
⊙ Presence of Fe oxide gives redish colour and high specific grasity. Generally, it's hardens with passage of time.

⑩ Marine Deposit: It has low shearing strength, high compressibility and contains organic matters.
⊙ It is soft and highly plastic.

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(11) Colluvial Soil: → Gravity deposited soil.

- It is also named as Talus.
- It has irregular and coarse particles.

(12) Peat Soil: (Not used) *** It is an organic soil having fibrous particles formed from vegetative matter under condⁿ of excessive moisture.

- It is highly compressible and not suitable for foundations.

(13) Muck Soil: Mixture of fine soil particles and highly decomposed organic matter and black in colour and extremely soft inconsistency.

(14) Loam Soil: It is a mixture of sand, silt and clay.

(15) Hard Pan Soil: It disin doesnot disintegrate when submerged in water and offers great resistance to penetration.

(16) Tuff Soil: It is a fine grained soil composed of particles ejected from volcanoes and deposited by wind and water.

(17) Dispersive clay: (easily Erodable) : It occurs in soil of low and medium plasticity that contains Montmorrilonite.

- It is treated with lime to make it non-dispersive.

(18) Collapsing Soil: Soil succeptible to large decrease in volume when saturated.

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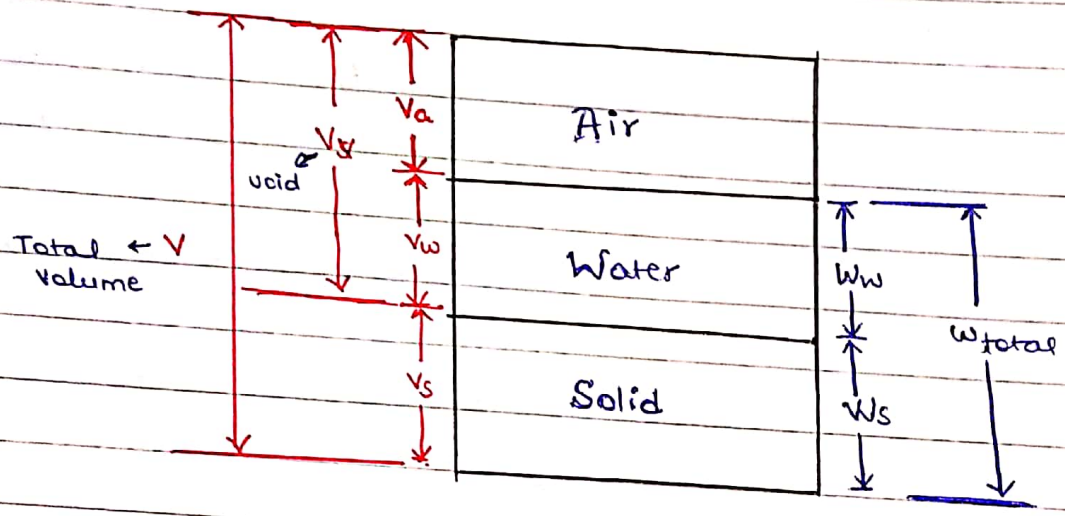
CHAPTER-2

SOIL - WATER RELATIONSHIP

PHASE DIAGRAM of SOIL



⊙ Soil is generally a 3-phase soil but completely saturated soil and completely dry soil are called 2-phase system.

Basic Definitions

① Water Content (w):

$$w = \frac{W_w}{W_s} = \frac{\text{Weight of water}}{\text{Weight of Solid}}$$

✱ In Environmental Engg.. water content is weight of water divided by total weight

⊙ It has no upper limit and for dry soil it is zero.

⊙ Fine grained soil have greater moisture content compared to coarse grained soil.

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② Void Ratio (e):

$$e = \frac{V_v}{V_s}$$

- It has no upper limit
- Void ratio of fine grained soil are generally higher than in coarse grained soil.
  Although the size of voids will be greater in coarse grained soil.

③ Porosity (n):

$$n = \frac{V_v}{V}$$

- Its upper limit is 1.

④ Degree of Saturation: (S)

$$S = \frac{V_w}{V_v}$$

Its limit is (0 → 1)

- For dry soil ⇒ $S = 0$ and for saturated soil ⇒ $S = 1$

⑤ Percentage Air Void: (n_a)

$$n_a = \frac{V_a}{V}$$

⑥ Air content: (a_c)

$$a_c = \frac{V_a}{V_v}$$

or $a_c = 1 - S$

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⑦ Bulk Unit Weight (γ_t) :

$$\gamma_t = \frac{W_{Total}}{V_{Total}}$$

⑧ Unit Weight of Solid (γ_s) :

$$\gamma_s = \frac{W_{solid}}{V_{solid}}$$

⑨ Unit Weight of Water (γ_w) :

$$\gamma_w = \frac{W_{water}}{V_{water}} = 9.81 \frac{KN}{m^3}$$

⑩ Dry Unit Weight (γ_d) :

$$\gamma_d = \frac{W_{solid}}{V_{Total}}$$

⊙ γ_d is used as a measure of denseness of soil.
i.e; higher value of γ_d indicates that more solids are packed in the unit volume of soil.
⊙ Hence, soil is more compacted.

⑪ Saturated Unit Weight (γ_{sat}) :

$$\gamma_{sat.} = \frac{\text{Weight of saturated soil}}{\text{Volume of saturated soil.}}$$

⊙ Saturated unit weight will be a function of water

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content and as water content increases, $\gamma_{sat.}$ decreases.

(12) Submerged Unit Weight:

$$\gamma_{sub.} = \gamma_{sat.} - \gamma_w$$

⊙ When the soil exists below groundwater table. i.e; when it is under submerged condition, buyont force acts on the soil particles and hence the buyont force / weight divides by the volume of soil is called submerged unit weight.

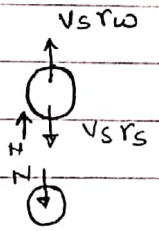
$$\gamma_{sub.} = \frac{W_{Buoyant}}{V_{Total}}$$

$$\gamma_{sub.} \cdot V = (\gamma_{sat} - \gamma_w) V$$

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

$$= (V_s \gamma_s + V_w \gamma_w) - \gamma_w (V_s + V_w)$$

$$= V_s (\gamma_s - \gamma_w)$$



$$V_s\gamma_w + N = V_s\gamma_s$$

$$N = V_s(\gamma_s - \gamma_w)$$

Note: ⊙ A saturated soil may not be the submerged soil like soil saturated with capillary saturation is not called submerged.

Soil below the water table is said to be submerged.

(13) Specific Gravity of Solid (G_s):

$$G_s = \frac{\gamma_s}{\gamma_w}$$

⊙ Generally, for inorganic soils, G_s is b/w (2.6 - 2.7) & for organic soils, it is (2.5 - 2.)

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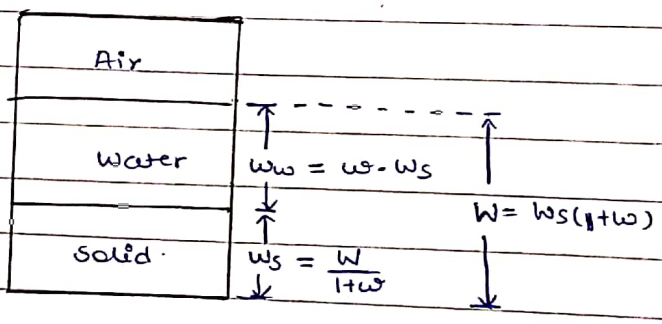
⑥ Specific Gravity of solid is also called absolute Specific Gravity or Grain Specific Gravity.

⑧ ~~Mass~~ Spe

⑭ Mass Specific Gravity of Soil (G_m):

$$G_m = \frac{\gamma_t}{\gamma_w}$$

Some Important Relationships

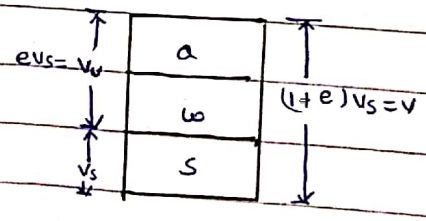


① $W_s = \frac{W}{1+w}$

② $V_s = \frac{V}{1+e}$

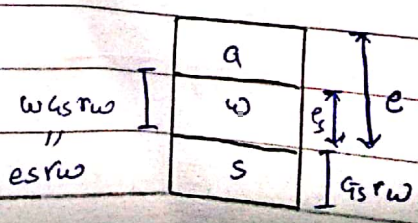
③ $n = \frac{V_v}{V} = \frac{e}{1+e}$

④ $e = \frac{V_v}{V_s} = \frac{n}{1-n}$



⑤ $Se = wG_s$

⑥ $\gamma_t = \frac{G_s\gamma_w(1+w)}{1+e}$



⑦ $\gamma_t = \frac{(G_s + Se)\gamma_w}{1+e}$

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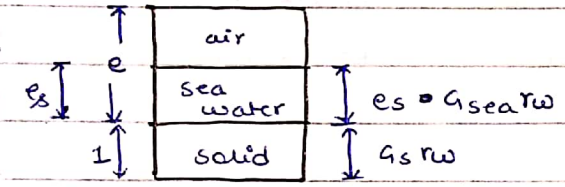
$$\gamma_{sea} = G_{sea} \cdot \gamma_w$$

(8) $$\gamma_t = \frac{(G_s + S e \cdot G_{sea})\gamma_w}{(1+e)}$$



(9) $$\gamma_{sat.} = \left(\frac{G+e}{1+e}\right)\gamma_w$$

(10) $$\gamma_{sub.} = \frac{G+e}{1+e}\gamma_w - \gamma_w = \frac{(G-1)\gamma_w}{1+e}$$

(11) $$\gamma_d = \frac{G\gamma_w}{1+e}$$

(12) $$\gamma_d = \frac{\gamma_t}{(1+w)} \quad \text{or} \quad \gamma_d = \frac{G\gamma_w}{1+\frac{wG}{S}}$$

only for No volume change condn.

S → saturation of wet soil

(13) $$1 - n_a = 1 - \frac{V_a}{V} = \frac{V_w + V_s}{V} = \frac{W_w}{\gamma_w V} + \frac{W_s}{G_s \gamma_w V}$$

$$= \frac{W_s}{V\gamma_w}\left[\frac{1}{G_s} + \frac{W_w}{W_s}\right] = \frac{W_s}{V\gamma_w}\left[\frac{1+wG_s}{G_s}\right] = \frac{\gamma_d}{\gamma_w}\left(\frac{1+wG_s}{G_s}\right)$$

$$\Rightarrow \gamma_d = \frac{(1-n_a) G_s \gamma_w}{1+wG_s}$$

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(14) $\gamma_d = \frac{G\gamma_w}{1+wG}$

(Zero air void dry density)

⊙ Displace the air without changing moisture content by applying roller.

Note: ⊙ Zero air void dry density is the theoritical max^m dry density at a particular moisture content.
(means it can never be achieved).

Ques→ Total unit wt. of a soil is 16 KN/m³. The Specific gravity of solid particle of soil is 2.67. Water content of the soil is 17%, assuming $\gamma_w = 9.81$ KN/m³, calculate:

(a) Dry unit weight
(b) Porosity
(c) Void ratio
(d) Degree of Saturation.

Soln:

$\gamma_t = 16$ KN/m³ $G_s = 2.67$, $w = 17\%$, $\gamma_w = 9.81$ KN/m³

(a) $\gamma_d = \frac{\gamma_t}{1+w} = \frac{16}{1+0.17} = \frac{16}{1.17} = 13.675$ KN/m³

(b) $13.675 = \frac{G\gamma_w}{1+e} \Rightarrow 1+e = \frac{2.67 \times 9.81}{13.675}$

$\Rightarrow e = \frac{1.91-1}{} = 0.915$

(c) $n = \frac{e}{1+e} = \frac{0.915}{1.915} = 0.477$

(d) $es = wG$

$S = \frac{wG}{e} = \frac{0.17 \times 2.67}{0.916} = 0.495$

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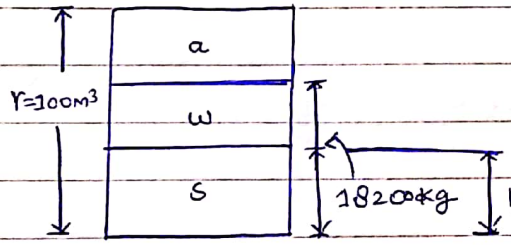
Ques→ For the construction of an embankment, soil is transported from Borrow area using a truck which can carry 6m³ of soil at a time. Following data are given:

Property	Borrow Area	Truck (loose)	Field (compacted)
γ_t (g/cc)	1.66	1.15 g	1.82
w/c (%)	8%	6%	14%

Find the quantity of soil to be excavated from Borrow pit in m³ for the compacted earthfill of 100m³ and also find the no. of trucks required.

Solⁿ:

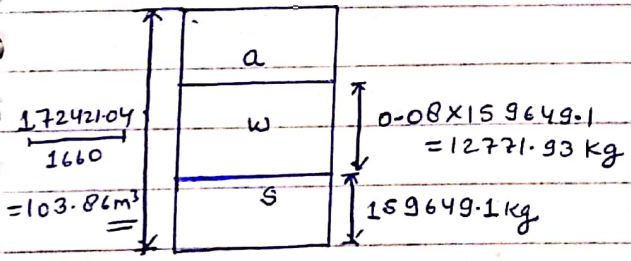


$1.82 \frac{g}{cc} = 1820 kg/m^3$

$1 + \frac{W_w}{W_s} = 1 + w$

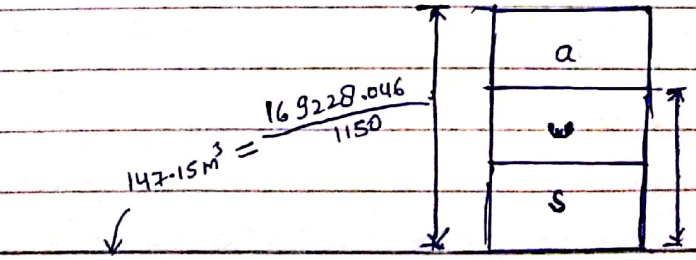
$\frac{W}{W_s} = 1 + w$

$W_s = \frac{W}{1+w}$



⇒ Volume = 103.86m³

No. of trucks = 147.15/6 = 25 truck loads.



No. of trucks = 147.15/6

= 24.5 = 25 Truck loads

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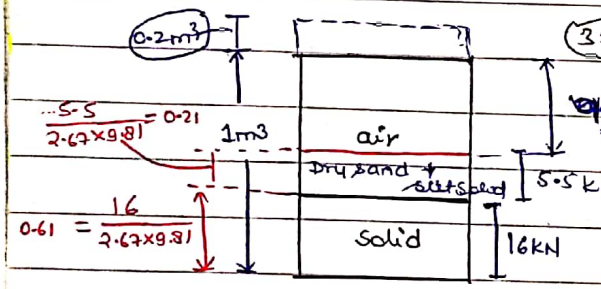
Alternate:

Ques→ Insitu dry unit weight of coarse sand was 16 KN/m³. for mechanical stabilization of soil, 5.5 KN of mixture of dry soil sand and silt was added to 1m³ of coarse sand. and the volume got increased by 20%. find the reduction in the porosity of soil and take $G_s = 2.67$ for all the materials and assume the coarse sand to be dry initially.

If after the addition of mixture of dry sand and silt, 1.5 KN of clay at moisture content of 10% was added such that no further increase in volume of soil resulted. G_s For clay = 2.55. find the reduction of porosity.

Soln: $\gamma_{d_1} = 16$ KN/m³ $\quad V_1 = 1$m³

$\gamma_{d_2} = \frac{16 + 5.5}{1.2} =$



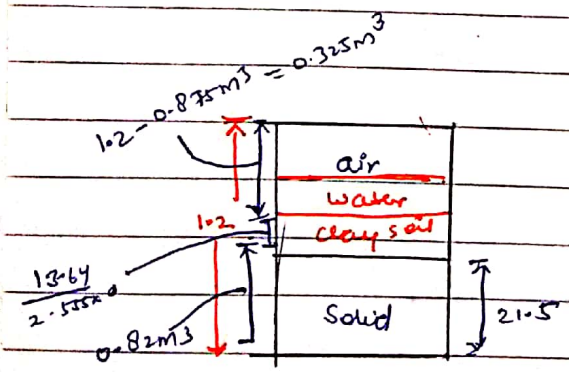
1.2 − (0.21 + 0.61) = 0.38

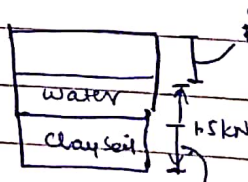
Initial porosity = $\frac{V_{initial}}{V_{initial}}$

$= \frac{0.39}{1m^3} = 0.39$

(Coarse sand)

Porosity after addition of 5.5 kN of mix. of sand & silt = $\frac{V_v}{V} = \frac{0.38}{1.2} = 0.317$





$\frac{1.5}{1.1} = 1.364$ kN

$(\text{Porosity})_{final} = \frac{0.325}{1.2} = 0.271$

⇒ further redn is porosity = 0.317 − 0.271 = 0.046

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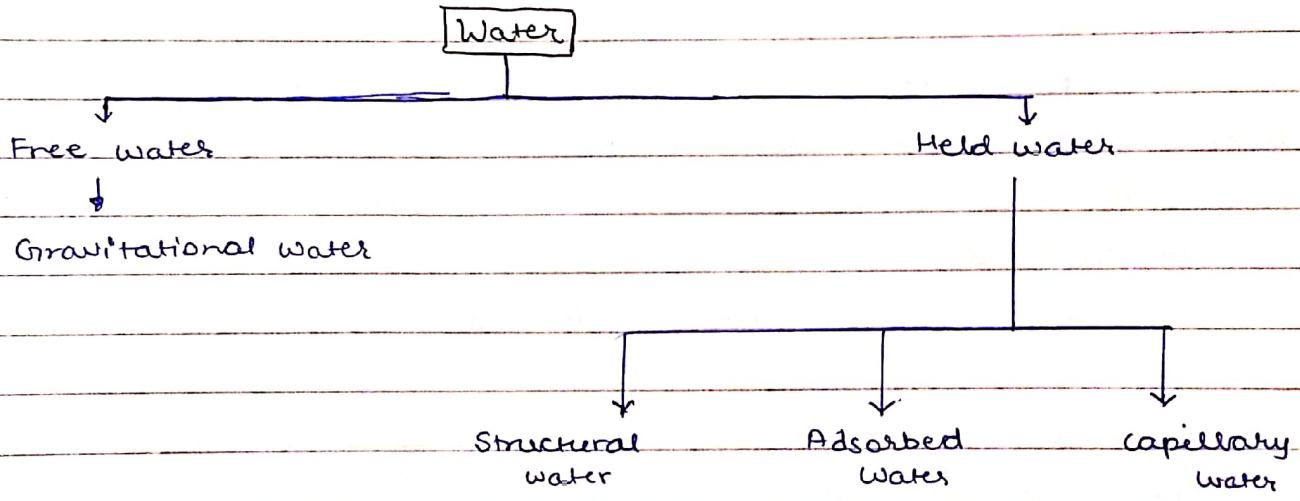
Water Content Determination

⊙ Water content of soil significantly affects the property of fined grained soil, particularly the clayey soil.

Hence, we determine the water content before determining any other property because storage, transportation and handeling can change the water content.

Classification of Soil Water



⊙ Water free to move through soil under the influence of gravity is called free water.

and that held against gravity is called Held water.

⊙ Structural Water: The water chemically combined in the crystal structure of soil.

It can't be removed without breaking the structure of soil particles and this water is not removed by heating upto 105-110°C (used for drying the soil under lab. condn)

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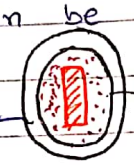
(#) Adsorbed Water: Adsorbed water is the water held by fine grained soil particle due to electrochemical force of adhesion. It can be completely removed only above 200°C and at 105 - 110°c, it cannot be completely removed but the part that can be removed is called Hygroscopic water.



(#) Capillary Water: The water held in soil mass due to capillary action. Capillary water can exist on a macroscopic scale compared to other types of held water which can exist on microscopic scale.

⊙ From the point of view of interparticle forces, the soil water can be classified as:

Ⓐ Adsorbed water
Ⓑ Pore Water (Capillary & free water)

⊙ Water content of the soil sample can be determined by following methods:

1. Oven-drying Method
2. Pycnometer Method
3. Sand Bath Method
4. Alcohol Method
5. Calcium Carbide Method
6. Torsion Balance method
7. Radiation Method.

Oven Drying Method

⊙ It is a standard laboratory method and is very accurate. The soil sample in a container is taken and dried at a temp. of 105-110°c. for 24 Hrs.

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⊙ Above 110°c, absorbed or structural water ~~er~~ may lost.

⊙ If However the soil contains significant quantity of organic matter then heating temp. recommended is 60-80°c because at higher temperature, organic soil tends to decompose and get oxidise.

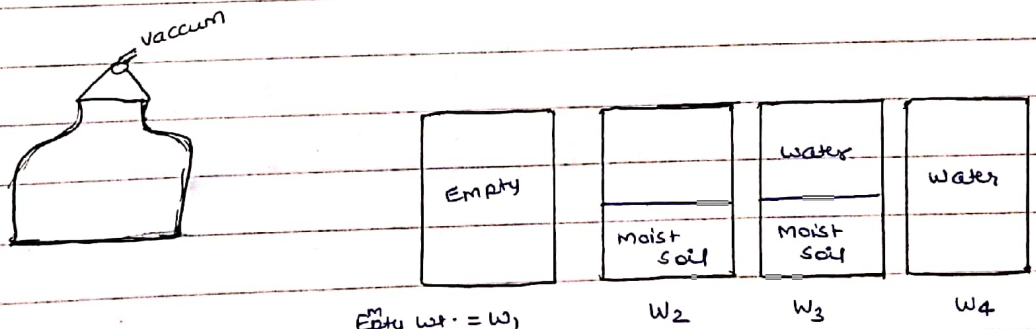
$$W/c = \frac{W_2 - W_1}{W_1} \times 100$$

W_2 = Weight of moist soil

W_1 = Weight of dry soil.

Pycnometer Method

⊙ It is a quick method and it is used to find out water content and Specific Gravity of soil. This method is suitable for cohesionless soil only because it is difficult to remove entrapped air from the cohesive soil.



$$W/c = \frac{W_w}{W_s} = \frac{W - W_s}{W_s} = \frac{W}{W_s} - 1$$

$$w = \frac{W_2 - W_1}{W_s} - 1$$

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$w_4 - w_1 = w_3 - w_1 = w_s + v_s r_w$

$= w_3 - w_1 = w_s + \frac{w_s}{G_s r_w} \cdot r_w$

$\Rightarrow w_3 - w_4 = w_s \left(1 - \frac{1}{G_s}\right)$

$w_s = \frac{w_3 - w_4}{\left(1 - \frac{1}{G_s}\right)}$

$w = \frac{w_2 - w_1}{w_3 - w_4}\left(1 - \frac{1}{G_s}\right) - 1$

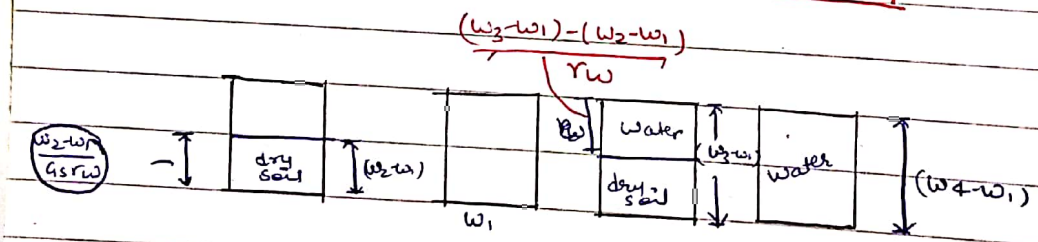
$\Rightarrow w = \left[\frac{(w_2 - w_1)}{(w_3 - w_4)}\left(1 - \frac{1}{G_s}\right)\right] - 1$

⊙ Pycnometer method can be used for w.c determination only if specific gravity is known.

Determination of Specific Gravity using Pycnometer

⊙ In this case, we take the dry soil instead of the moist soil.

$G_s = \frac{w_s}{v_s r_w} = \frac{w_2 - w_1}{(w_4 - w_1) - (w_3 - w_2)}$



$\frac{w_4 - w_1}{r_w} = \frac{w_2 - w_1}{G_s r_w} + \frac{(w_3 - w_1) - (w_2 - w_1)}{r_s}$

Solve to get G_s

⊙ The value of G_s is reported at 27°C. If the temp. during testing is different then we should convert the G_s value at 27°c to report it value

$(G_s r_w)_{t°c} = (G_s r_w)_{27°c}$

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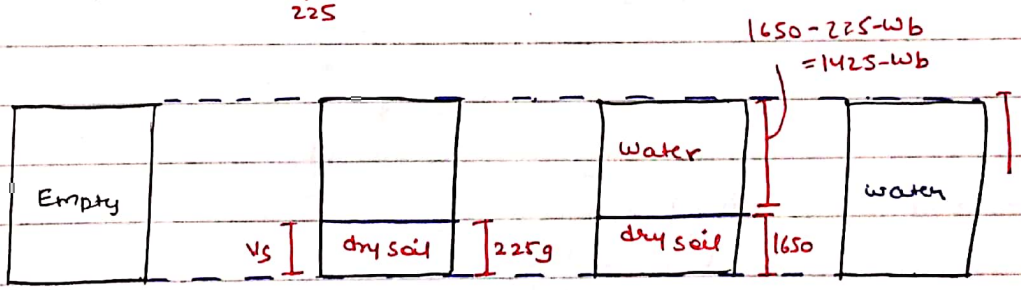
Ques→ 225 gm of soil was oven dried and placed in a Specific Gravity bottle and then filled with water upto a constant volume mark made on the bottle. Mass of bottle with water and soil is 1650 gm. The Specific Gravity ⇒ bottle was filled with water alone upto the constant volume mark and weight. Its mass was found to be 1510 gm. Find Specific gravity of soil.

Soln:

$w_2 - w_1 = 225 gm$ $\quad w_3 = 1650 gm$ $\quad w_4 = 1510 gm$

$$G_s = \frac{w_2 - w_1}{(w_4 - w_4) - (w_3 - w_2)} = \frac{225}{1510 - 1650 + 225} = 2.647$$

1510 — 1650 — 225



w_1

$$G_s = \frac{225}{Vs} = 2.647$$

$$\frac{W_s}{V_s \rho_w} = G_s$$

Ques→ It is required to determine the w/c of a wet silty sand sample of soil weighing 400 gm. This mass of soil was placed in a pycnometer and water is filled upto the top of conical cap and weight. Its mass was found to 2350 gm. The pycnometer was next filled with water and weight, its mass was found to be 2200 gm. Assuming $G_s = 2.67$. Determine the w/c of the soil sample.

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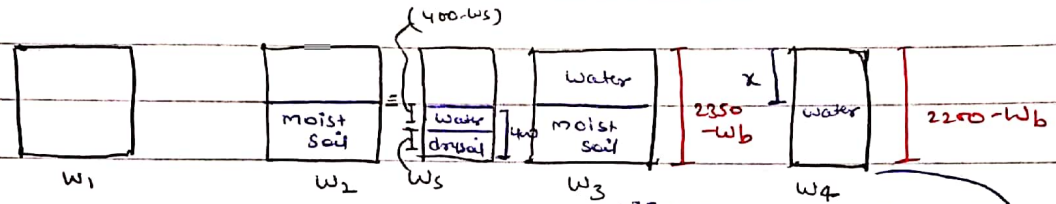
Soln: $w_2 - w_1 = 400\,gm$ $\quad w_3 = 2350\,gm$ $\quad w_4 = 2200\,gm$

$$w = \frac{(w_2 - w_1)}{(w_3 - w_4)}\left(1 - \frac{1}{G_s}\right) - 1 = \frac{400}{(2350-2200)}\left(1 - \frac{1}{2.67}\right) - 1$$

$$\Rightarrow w = \frac{400}{150} \times \left(1 - \frac{1}{2.67}\right) - 1 = 0.6679$$



$(w_2 - w_1 = 400g)$

$(V_s G_s \gamma_w + x) \cdot e$ $\quad$ $(V_s \gamma_w + x)$

$2350 - W_b$ $\quad$ $2200 - W_b$

$$w = \frac{400 - W_s}{W_s}$$

$V_s G_s \gamma_w$

$$(2350 - 2200) = V_s \gamma_w (G_s - 1)$$

$$V_s \gamma_w = \frac{150}{G_s - 1}$$

$$G_s \gamma_w V_s = \frac{150 G_s}{G_s - 1} = \frac{150 \times 2.67}{1.67} = 239.82\ gm$$

$$w = \frac{400 - 239.82}{239.82} = 0.667.$$

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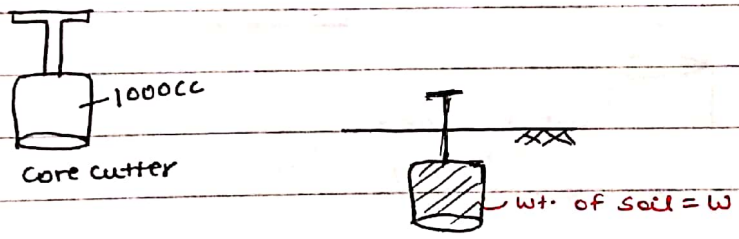
Measurement of Unit Weight

⊙ Bulk unit weight is calculated using various methods like :

(1) Core Cutter method (2) Sand Replacement Method
(3) Water displacement method (4) Water Baloon method
(5) Radiation method.
→ (Non- destructive method)

Core - Cutter Method



$\gamma_t = \frac{W}{1000cc}$

⊙ It is a field method and it is applicable for soil which is soft and fine grained. It cannot be used for stony, Gravelly and dry soil

⊙ This method is practical only when the surface of soil is exposed and core cutter can be easily punched.

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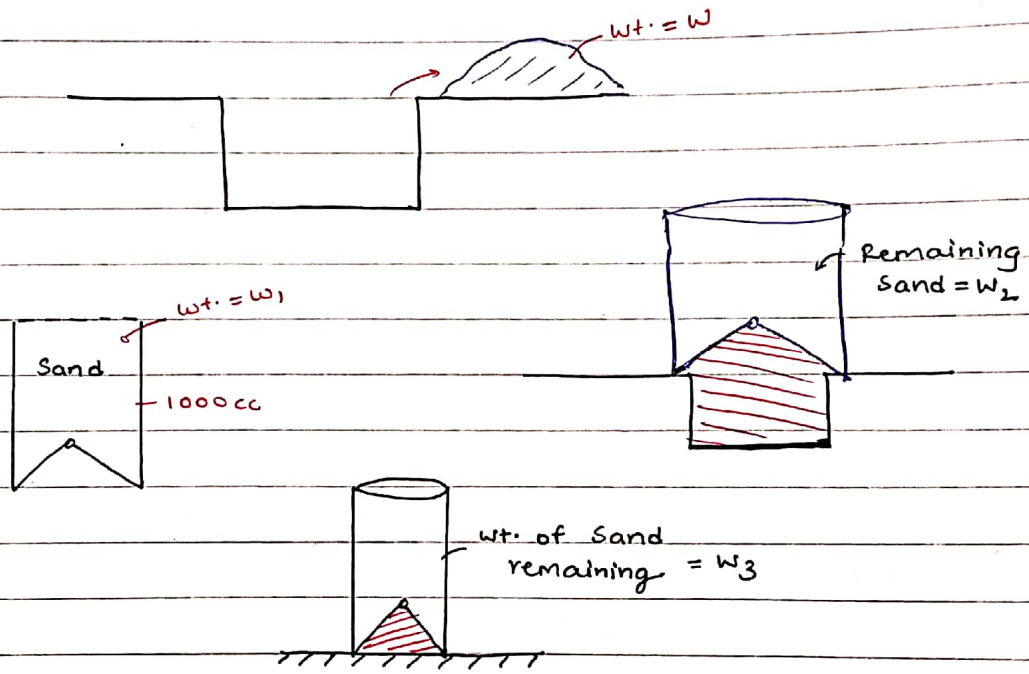
Sand Replacement Method

① It is a field method & is suitable for Gravelly, Sandy and dry soil when core cutter method cannot be used.



$W_1 - W_2$ = Cone weight (Sand) + Pit weight (Sand)

$W_2 - W_3$ = Cone weight (Sand)

$(W_1 - W_2) - (W_2 - W_3)$ = Wt. of Sand in pit

$$V = \text{Vol. of pit} = \frac{(W_1 - W_2) - (W_2 - W_3)}{\gamma_{sand}}$$

$$\gamma_t = \frac{W}{V}$$

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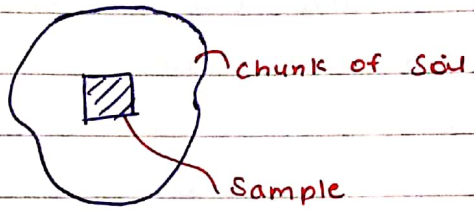
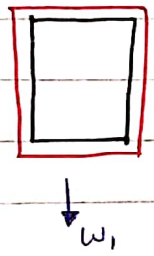
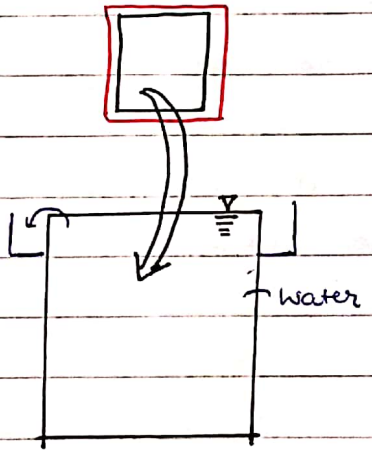
Water displacement Method



Weight of Sample = W_0





Wt. of water overflow = W_2

Vol. of Sample + Vol. of wax = $\frac{W_2}{\gamma_w}$

Vol. of wax = $\frac{W_1 - W}{\gamma_{wax}}$

Volume of Sample = $\frac{W_2}{\gamma_w} - \frac{W_1 - W}{\gamma_{wax}} = V$

$$\gamma_t = \frac{W}{V}$$

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Index Properties of Soils

⊙ Index properties are those properties which help in assessing the Engg. behaviour of soil and in classifying the soil into diff. groups so that a particular group is representative of a particular behaviour.

⊙ For coarse grained soil, properties of particles (Angularity, roundness) and the relative state of compaction are the most significant properties. and for fine grained soil, consistency and plasticity are important.

⊙ The various index properties for coarse and fine Grained soil are as listed below:

(#) Coarse Grain Soil:

PROPERTY	TEST
⊙ Range of particle size and distribution of size	⊙ Particle size distribution Analysis (Grain Size Analysis)
⊙ Shape of Particle	⊙ Visual Examination.
⊙ Presence of fine Grained particles	⊙ Sieve Analysis & Sedimentation Analysis
⊙ Relative state of compaction	⊙ Relative density test

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Fine Grain Soil

PROPERTY	TEST
⊙ Consistency (strength and type of structure in the undisturbed state)	⊙ UCS Testing (Unconfined compressive ~~test~~ strength)
⊙ Change in consistency due to remoulding	⊙ UCS Testing
⊙ Water content	⊙ W/C test
⊙ Plasticity	⊙ Atterberg limits (Liquid & Plastic limit)
⊙ Presence and type of Clay	⊙ Indirect method of determination like Plasticity, consistency and direct method like Clay mineral Analysis is done

⊙ The index properties are related to :

(a) Individual soil grains.
(b) Aggregate soil mass.

⊙ The soil grain property can be determined from remoulded or disturbed sample and they depend on the shape and size of grains and their mineralogical composition.

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⊙ It is independent of mode of soil formation.

⊙ Soil Aggregate properties depend on mode of soil formation, soil history and soil structure. & these properties are determined from undisturbed sample or preferably from insitu test.

Note: ① Grain Shape : Particle shape plays an important role in the mechanical behaviour of coarse grained soil.

⊙ Particle grains are defined in terms of Sphericity and roundness.

⊙ Sphericity depends on elongation and roundness depends on sharpness of angular protudations from the particles.

⊙ Larger is the angularity, greater is the void ratio.

⊙ Larger is the angularity, greater is the shear strength.

② Relative Density : (Dr)

$$Dr = \frac{e_{max.} - e}{e_{max.} - e_{min.}}$$

$e_{max.}$ = The maxm void ratio in the loosest state and

$e_{min.}$ = The minm void ratio in the densest state.

e = Void ratio under nature deposits.

⊙ Normally, $e_{max.}$ is taken as 0.91 (Sphere) & $e_{min.}$ is taken as 0.35 (Tetrahedral)

$e_{max.}$ → Simple cubic arrangement of Sphere

e_{min} → Tetrahedral Arrangement

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Relative Density	Classification
DR < 15 %	Very Loose
15 - 35 %	Loose
35 - 65 %	Medium
65 - 85 %	Dense
> 85 %	Very Dense.

⊛ Higher the relative density, higher is the shear strength and lower is the compressibility.

$$D_r = \frac{e_{max.} - e}{e_{max.} - e_{min.}}$$

$$\gamma_d = \frac{G\gamma_w}{1+e}$$

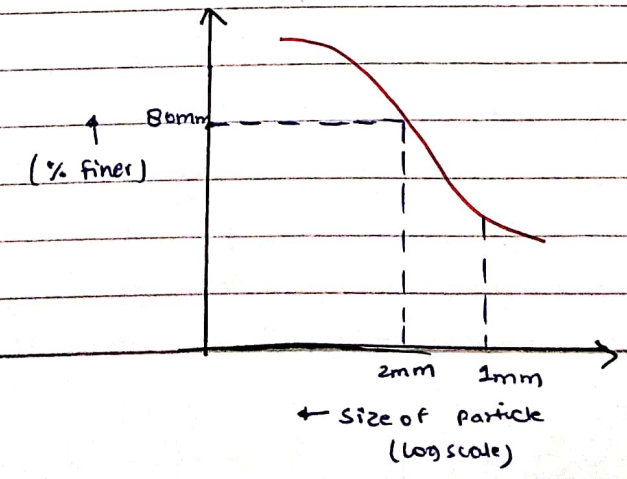
$$e = \frac{G\gamma_w}{\gamma_d} - 1$$

$$= \frac{\left(\frac{G\gamma_w}{\gamma_d} - 1\right)_{max.} - \left(\frac{G\gamma_w}{\gamma_d} - 1\right)}{\left(\frac{G\gamma_w}{\gamma_d} - 1\right)_{max.} - \left(\frac{G\gamma_w}{\gamma_d} - 1\right)_{min.}} = \frac{\left(\frac{1}{\gamma_d}\right)_{min.} - \frac{1}{\gamma_d}}{\left(\frac{1}{\gamma_d}\right)_{min.} - \left(\frac{1}{\gamma_d}\right)_{max.}}$$

Density Index

$$= \frac{\gamma_d - \gamma_{d\,min.}}{\gamma_{d\,max.} - \gamma_{d\,min.}} \times \frac{\gamma_{d\,max.}}{\gamma_d}$$

③ Grain size Analysis: finding out the proportion by weight of different size particles in soil is called Grain size analysis and this analysis yields grain size distribution curve.



⊛ 80% particles are finer than 2 μ

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Coarse Grain: 65% of particles are > 75μ.

- Grain size analysis is important mainly for coarse grained soil because in fine grained soil, inter particle forces and water content are more important.
- This analysis is used along with other physical properties for classification of soil.

Sizes of Soil Particles

80mm - 4.75 mm	Gravel
4.75mm - 75 μ	Sand
75 μ - 2 μ	Silt
< 2 μ	Clay

Note: 300mm - 80mm → ~~Cobble~~ Boulder cobble
>300mm → ~~Cobble~~ Boulder
} Not Grouped as Soil.

- Particles finer than 60μ can't be seen by Naked Eye.
- Grain size analysis of coarse grained soil is carried out by Sieve Analysis.
- Fine grained soil are analysed using sedimentation analysis.
- There are two methods:
  - Pipete Method. → Standard Method.
  - Hydrometer Method.

- Pipete method is the standard method
- Hydrometer Method is secondary Method.

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Sieve

Coarse Sieve

80 mm

20 mm

10 mm

4.75 mm

Adopted for Gravel

Always dry Analysis is done

Fine sieve

2mm

1mm

600μ

425μ

212μ

150μ

75μ

Adopted for Sand.

Dry or wet analysis is done.

Note: For wet analysis, soil passing (4.75 mm) sieve is washed over 75μ sieve and then oven dried. & Fine Sieve analysis is performed with this oven dried Sample.

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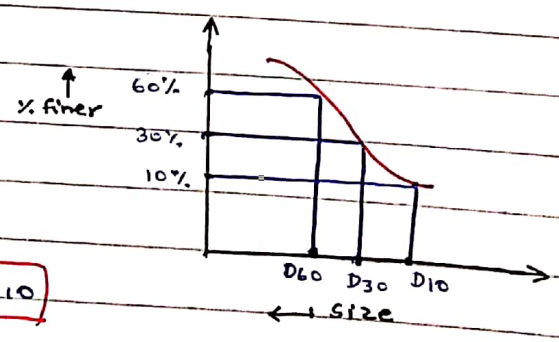
% finer = 100 - % Retained

S.No.	Sieve size	Total weight of soil taken	Wt. Retained (gm)	Cumulative Wt. Retained	% Retained	% finer
11	80 mm					
10	20 mm		10	10	(10/500) x100 = 2	98
9	D_{60} 10 mm		165	175	35	65
8	D_{30} 4.75 mm		100	275	55	45
7	2mm		85	360	72	28
6	1mm	500gm	60	420	84	16
5	D_{10} 600 μ		20	440	88	12
4	425 μ		40	480	96	4
3	212 μ		10	490	98	2
2	150 μ		5	495	99	1
1	75 μ		3	498	99.6	0.4
			1	499	99.8	0.2
					Σ = 828.4	

D_{10} → Effective size of particle



$D_{60} > D_{30} > D_{10}$

D_{10} means 10% particles are finer than D_{10} size.

Fineness Modulous = $\frac{\Sigma \text{ cum \% retained}}{100} = 8.284$

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** Fineness modulous is a sieve no, at which avg. size of particles are said to lie when counted from the lower order sieve size to the higher order sieve size.

In the previous example, avg. size of particle is said to lie b/w sieve no. 8 to 9.
i.e; b/w 4.75mm and 10mm.

Certain parameters are found out from the grain size distribution curve which are used for classification of soil.

$$C_u = \text{Uniformity coeff.} = \frac{D_{60}}{D_{10}}$$

$$C_c = \text{Coeff. of curvature} = \frac{D_{30}^2}{D_{60} \times D_{10}}$$

C_u, coeff. of uniformity, basically represents the slope of the curve b/w D_{60} and D_{10} and hence it is related to the degree of uniformity of the sample.

If $C_u = 1$ then the soil is perfectly uniformely graded. i.e; curve will become vertical.

If $C_u < 2$ or 3, then the soil is considered to be uniform.

If various sizes of particles are present then C_u will have a larger value. & generally when various sizes are present, soil is said to be well graded.

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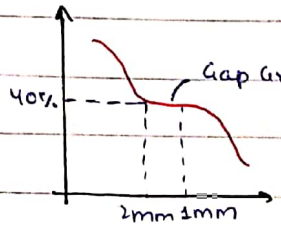
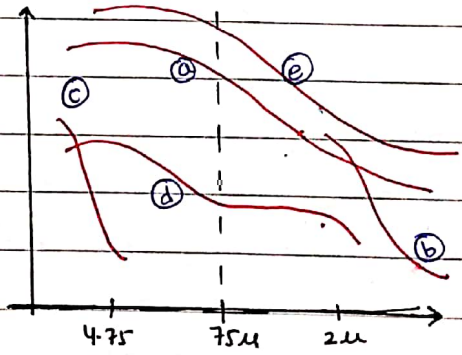
⊙ If $Cu > 4$ and $1 < Cc < 3$ → soil is said to be well graded Gravel.

⊙ If $Cu > 6$ and $1 < Cc < 3$ ⇒ Soil is well graded Sand.



⇒ There is no particle blw 1 mm and 2mm.

⊙ If Cc does not lie blw 1 and 3, the soil is said to be gap graded. & gap graded means certain particle sizes are missing



a → well graded soil

b → Poorly Graded fine Grained Soil

c → Poorly Graded coarse Grained Soil

d → Gap Graded Soil

e → well Graded Soil

a, e → Similarly Graded Soil

⊙ Note: ⊙ Position of curve indicates type of soil & type of curve indicates grade of Soil.

⊙ Smaller slope leads to better gradation.

⊙ Advantage of semi log plot is that the Grain size curve of

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Soil having equal uniformity are identical in shape regardless of the avg. ~~grade~~ grain size.

⊙ With increasing age of soil, avg. grain size decreases due to weathering.

⊙ Gap grading results if materials from two layers are mixed before grain size distribution is performed.

⊙ A conspicuous break in the continuity of grain size curve indicates the simultaneous deposition of soil by two diff. agents.

⊙ While designing the soil filter, it is found that D_{10} (Effective size) is the diameter of an artificial sphere that will produce approx. same effect as an irregularly shaped particles

⊙ The ease with which water flows through a soil is a func^n of D_{10} and particle size < D_{10}, affects the permeability more than the remaining 90% of the soil.

Sedimentation Analysis

$$\% \text{ Finer} = \frac{\text{Conc}^n \text{ of Sample collected at time } t \text{ from height } He}{\text{original conc}^n \text{ of Sample}}$$

$$\frac{He}{t} = \frac{(r_s - r_w) d^2}{18 \mu}$$

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⊙ Sedimentation analysis will be done for particle size less than 75 μ and the analysis is based on Stokes law. i.e;

$$V_s = \frac{(\gamma_s - \gamma_w)}{18\mu} d^2$$

γ_s → Unit wt. of soil particles

γ_w → Unit wt. of water

μ → Dynamic Viscosity

d → Size of particle

⊙ Stoke's law is applicable for particle diameter ranging from 0.2mm to 0.0002mm. (0.2 μ)

⊙ Generally, 50gm of (-75μ) soil is taken and mixed with known volume of distilled water.

⊙ Deflocculating agent or dispersing agent like Sodium Hexa meta phosphate, is added to the solⁿ.

⊙ At t=0, soil particles are assumed to be uniformely distributed throughout the suspension and after sometime 't', 10ml sample is taken out from depth He and weight of solid in it is calculated by oven drying method and concⁿ of the sample taken out from depth He is found out and hence the % finer than size d is calculated as:

$$\% \text{ finer} = \frac{\text{concⁿ of sample collected at time t from height He}}{\text{Original concⁿ of sample.}}$$

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$\frac{\text{Wt. of solute}}{\text{vol. of soln}}$ = concn.

⊙ Size of particle 'd' is calculated from:

$$\frac{He}{t} = \frac{(\rho_s - \rho_w)\, d^2}{18\mu}$$

⊙ Thus, one single point on the Grain size distribution curve is obtained.

⊙ Other points are calculated by taking out the sample from depth He at various time.

Ques→ 50 gm of dry soil of $G_s = 2.5$ was added with sufficient water to make 1L of suspension. From a depth of 10cm after a time of 2min, from the starting of settlement, 10ml of suspension was taken out using pipette and dry to obtain 0.2gm solid.

Find the corresponding point on Grain size curve.

Take $\rho_w = 1\ g/cc$, $\mu = 8.4 \times 10^{-4}\ kg/m\text{-}s$

Solun:

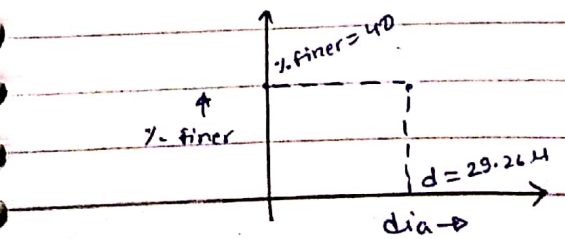
$G_s = 2.5$, $He = 10cm$, $t = 2min.$, concn = $\frac{0.2g}{10ml}$

$$\%\ \text{finer} = \frac{0.2g/10ml}{50g/1000ml} = 0.40 = 40\%$$

$$\frac{(\rho_s - \rho_w)\, d^2}{18\mu} = \frac{(2.5-1) \times \rho_w d^2}{18 \times 8.4 \times 10^{-4}} = \frac{He}{t}$$

$\rho_w = (1000 \times 9.81)\ kg/m^3$, $He \rightarrow 10 \times 10^{-2} m$, $t \rightarrow 2 \times 60$ sec.

$$d = 29.26 \times 10^{-6} m \Rightarrow d = 29.26 \mu m$$



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Consistency Limits

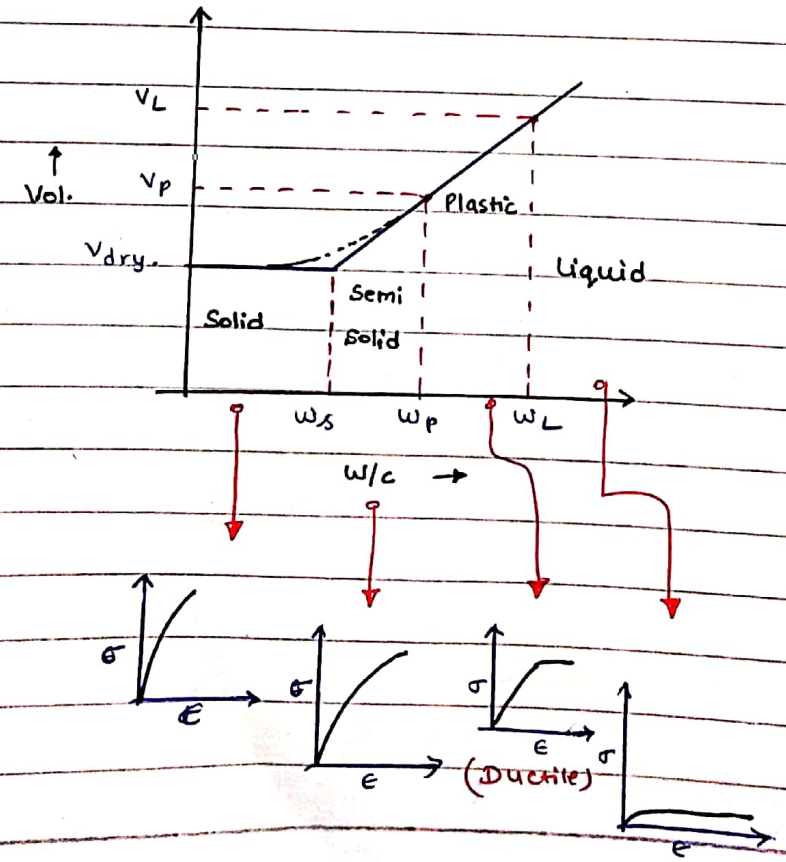
- Consistency represents the relative easy with which the soil can be deformed.

  It is related to the clayey soil and it is related to the water content.

- change of w/c leads to change in consistency of the soil.

- Atterberg classified the consistency in four stages:

(a) Solid stage
(b) Semi-solid stage
(c) Plastic stage
(d) Liquid stage.



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w_L = ⊙ Liquid limit moisture content

⊙ It is the boundary moisture content b/w liquid & plastic state

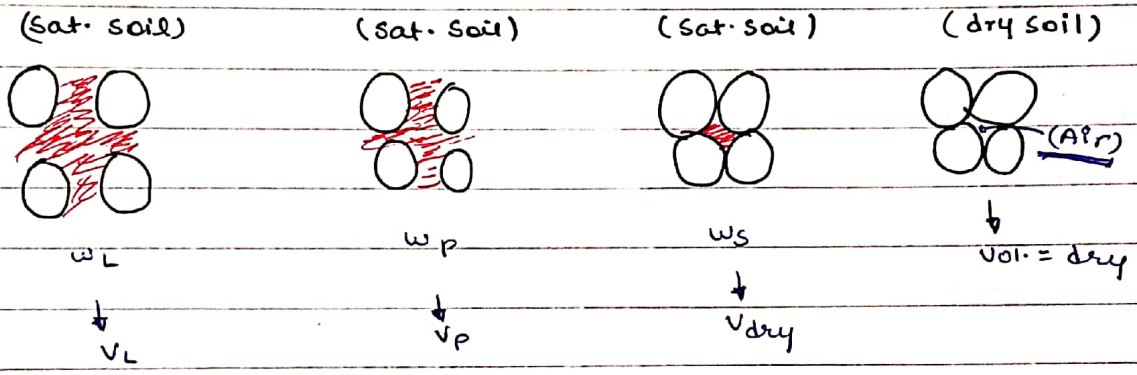
w_p = ⊙ Plastic limit M/c

⊙ It is the boundary m/c b/w plastic & semi solid state

w_s = ⊙ Shrinkage limit w/c

⊙ It is the boundary w/c b/w solid and semi-solid.

⊙



⊙ Shrinkage limit moisture content is the least moisture content at which soil is saturated, at water content < shrinkage limit, there would be no volume reduction on drying because air will start occupying the space in the voids.

Note: ⊙ Smaller is the shrinkage limit, greater is the volume reduction on drying. and hence greater is the shrinkage in the soil.

⊙ Similarly, greater would be the swelling on water addition.

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Amit Barwal
Amit Barwal

Atterberg Limits

⊙ Atterberg limits are used for identification, description and classification of cohesive soil and as a basis for preliminary assesment of their mechanical properties.

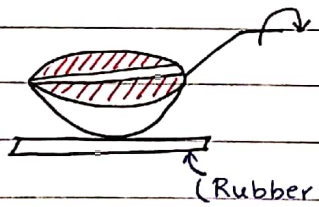
(i) **Liquid limit:** Minm w/c at which the soil has the tendancy to Flow is called Liquid limit of w/c.

⊙ All soils at LL will have same shear strength (approx. 2.7 kN/m²)

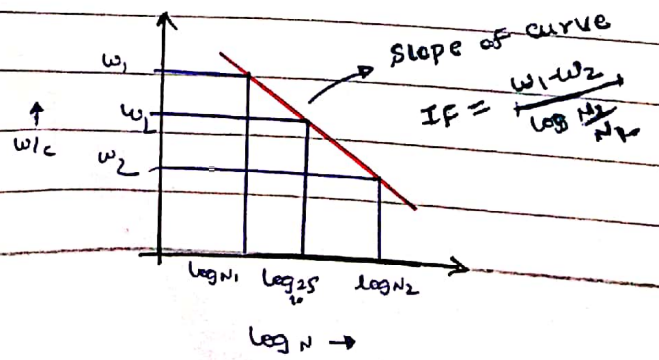
⊙ It is found out using:

(a) Casagrande tool (b) Cone-Penetration Method.

Casagrande Tool



⊙ In the casagrande's apparatus, w_L moisture content is that moisture content at which (25) no. of blows will close the groove.



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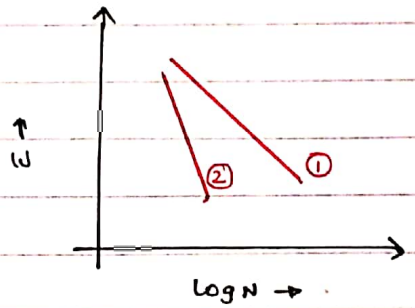
Slope = $-I_f$ = flow Index :

$$I_f = \frac{w_1 - w_2}{\log N_2/N_1}$$

$$I_f \propto \frac{1}{\text{Shear strength}}$$

$$\text{Slope} \propto \frac{1}{\text{Shear strength}}$$



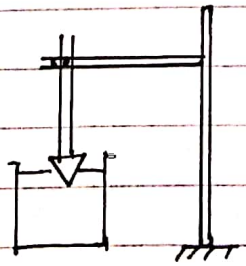
Soil ① has greater shear strength.

Note: ⊙ If the rubber pad used during casagrande testing is harder than the standard rubber pad, the WL reported will be smaller.

⊙

Is-code

CONE PENETRATION METHOD



⊙ w/c at which cone penetrates by 20mm is the Liquid limit moisture content.

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$w_L > w_P > w_s$

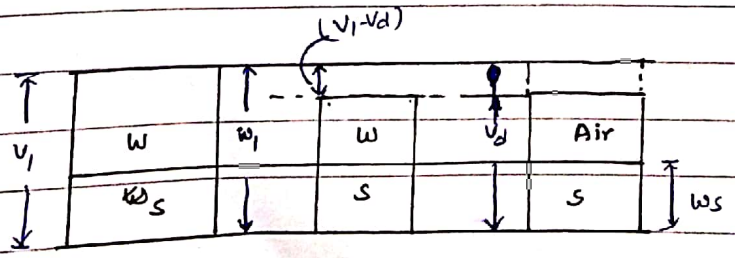
(ii) PLASTIC LIMIT

- Plastic limit is the minm w/c at which soil is in plastic stage, is called plastic limit water content.
- Above the plastic limit, soil can be deformed without volume change or cracking. and will retain its deformed shape.
- At plastic limit water content, a soil when rolled into a thread of (3mm) starts to crumble.
- The shear strength at plastic limit on an avg. is about (100) times that at liquid limit but shear strength at plastic limit is not same for all soils.

(iii) SHRINKAGE LIMIT

- The maxm w/c at which further redn in w/c does not cause any reduction in soil volume is called Shrinkage limit w/c.

  It is the minm w/c at which soil is just saturated.



w/c = Shrinkage limit w/c

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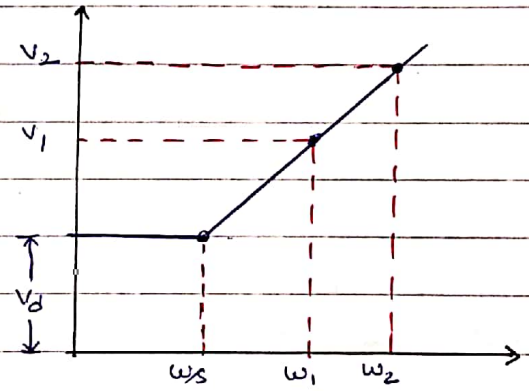
$$w_s = \frac{(w_1 - w_s) - (V_1 - V_d)\gamma_w}{w_s}$$ ⇐ mercury displacement method.

⊙ Shrinkage limit test can be used find out G_s :

$$G_s = \frac{W_s}{V_s \gamma_w} = \frac{W_s}{\left[V_1 - \left[\frac{w_1 - w_s}{\gamma_w}\right]\right]\gamma_w}$$

SHRINKAGE RATIO



Shrinkage ratio : $= \frac{(V_2 - V_1)/V_d}{w_2 - w_1} = \frac{\frac{V_2 - V_1}{V_d}}{\frac{(V_2 - V_1)\gamma_w}{w_s}} = \frac{\gamma_d}{\gamma_w}$

$$SR = \frac{V_2 - V_d/V_d}{w_2 - w_s}$$

$\frac{\gamma_d}{\gamma_w}$ → Shrinkage Ratio

⊙ SR is defined as the volume change in soil about shrinkage limit, expressed as percentage/fraction of dry volume for unit change in w/c about shrinkage limit.

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Volumetric Shrinkage

$$V_s = \frac{V_1 - V_d}{V_{d_1}}$$

V_1 is the Volume above ws.

Ques→ A saturated specimen of clay was immersed in mercury & displaces 21.8 gcc Volume. The weight of sample is 32.2 gm. After the oven drying, the wt. of sample reduced to 20.2 gm & volume came down to 11.6 c.c. find SL = ?

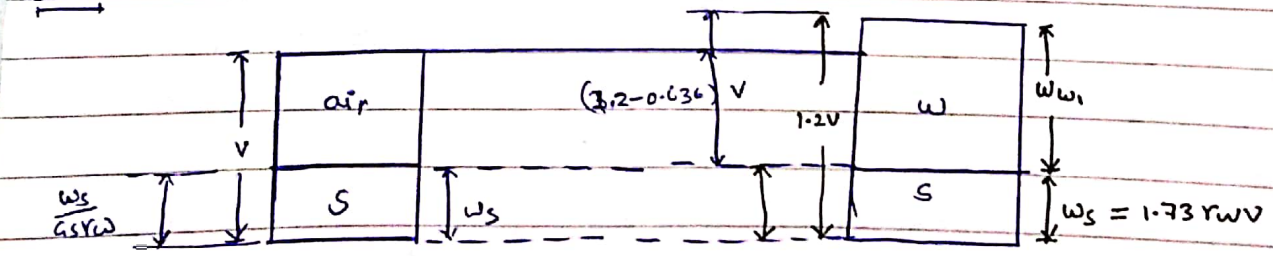
Solun:

$$w_s = \frac{(w_1 - w_s)\rho - (V_1 - V_{d_1})\gamma_w}{w_s} \quad \theta = \frac{1 \cdot (32.2 - 20.2) - (21.8 - 11.6) \times 1}{20.2}$$

$$= 0.089 \text{ i.e. } 8.9\%$$

Ques→ If the unit weight of oven dry sample is $1.73\,\gamma_w$. find the w/c that will increase the volume by 20% of dry soil volume. Take $G_s = 2.72$. Also calculate the shrinkage limit w/c

Solun:



$$\gamma_d = 1.73\,\gamma_w = \frac{w_s}{V}$$

$$G_s = 2.72$$

$$\frac{1.73 \gamma_w V}{G_s \gamma_w} = \frac{w_s}{G_s \gamma_w}$$

$$\frac{1.73}{2.72} V = 0.636 V$$

$$\frac{w_{w_1}}{w_s} = \frac{(1.2 - 0.636) V \gamma_w}{1.73 \gamma_w V}$$

$$= 0.326$$

$$\Rightarrow w/c = 32.6\%$$

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$$\gamma_d = \frac{G\gamma_w}{1+\underbrace{wsG}_{S\to 1}} = \frac{2.72\gamma_w}{1+ws\times 2.72} = 1.73\gamma_w$$

$$\Rightarrow ws = 0.21$$

Ques→ A fully saturated soil has a w/c of 40%. and its mass specific Gravity is 2. on oven drying, the dry density of the soil becomes $1.03\gamma_w$. Which of the following statements are correct?

(i) w/c of saturated soil is at w_s,

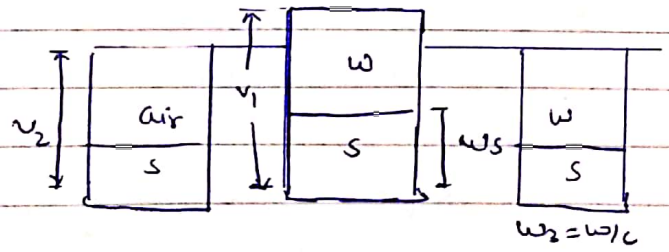
(ii) w/c of saturated soil is above w_s

(iii) Sp. Gravity of solid = 3.33

(iv) w_s w/c = 25.5%

Soln:

⊙ Of the given saturated soil has dry density smaller than the unit wt. of oven dried oven sample/ condn, then the w_s met is > w_s.



$G_m = 2$ ⇒ $\gamma_t = \frac{\gamma_t}{1+w} = \frac{2\gamma_w}{1.4} = 1.428\gamma_w$

$\frac{\gamma_t}{\gamma_w} = 2$ ⇒ γ_d on oven drying $= 1.8\gamma_w$

$\gamma_d < \gamma_{oven\,dried}$

⇒ w/c < > shrinkage limit w/c

$1.428\gamma_w = \gamma_d = \frac{G\gamma_w}{1+\underbrace{wsG}_{S\to 1}}$ (0.4 → 0.1)

⇒ ~~$1.8\gamma_w$~~

⇒ $G = 3.33$

$1.8\gamma_w = \gamma_d = \frac{G_s\gamma_w}{1+\underbrace{w_s G_s}_{S\to 1}}$

$w_s = 25.5\%$ ⇒

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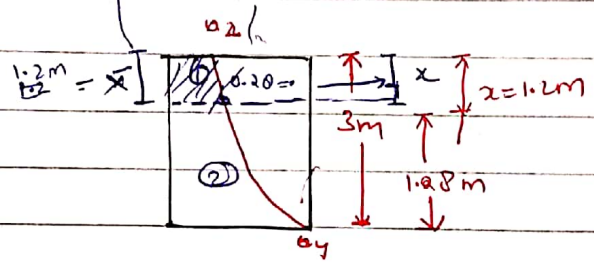
Ques→ A stratum of saturated clay ($G_s = 2.7$) has initial w/c of 40% throughout its depth. After long dry spell, the w/c at ground surface is at 20% and at depth of 3m, the w/c is 40%. w/c varies linearly with depth & SL = 28%.

If $\frac{\Delta V}{V} = 0.75 \frac{\Delta H}{H_0}$ find the amount of settlement of the ground.

Solⁿ.



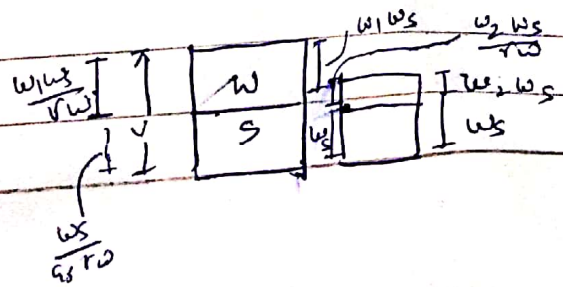
① Volume change in soil take place only upto & at shrinkage limit of the S.L. w/c occur at a depth of 1.2m. Hence in depth of 1.2m, the volume changes will occur due to w/c. change in ~~volume~~ (~~for~~ ~~0.2~~ ~~to~~ from 0.2 to 0.8.

② If b the depth below 1.2m, w/c is greater than shrinkage limit.

Hence, as volume change will take place due to change in w/c from 40% to w/c w ($w > 20\%$)

$$0.28 = 0.2 + \frac{0.2}{3} x \Rightarrow x = 1.2m$$

$$\Delta V = \frac{W_s}{\gamma_w} \times (\Delta w)$$



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$$\frac{\Delta V}{V_0} = \frac{\Delta w w_s}{\gamma_w \left[\frac{w_s}{G_s \gamma_w} + \frac{w_0 w_s}{\gamma_w}\right]} = \frac{\Delta w}{\frac{1}{G_s} + w_0} = \frac{\Delta w}{\frac{1}{2.7} + 0.4} = 1.298(\Delta w)$$

$$\frac{\Delta H}{H_0} = \frac{1}{0.75} \times \frac{\Delta V}{V_0} = \frac{1.298}{0.75} \cdot \Delta w = 1.730\ (\Delta w)$$

$$\frac{\Delta H}{H_0} = 1.73\ \Delta w$$

Zone-I

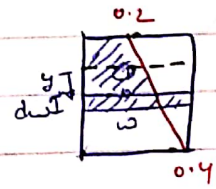
$$w = 0.4 - 0.28 = 0.12$$

$$\frac{\Delta H_1}{H_0} = 1.73 \times 0.12$$

$$\Delta H_1 = 1.73 \times 0.12 \times 1.2m = 0.25m$$

Zone-II

$$d(\Delta H) = 1.73\ (\Delta w)dy \quad = 1.73\ (0.4 - w)\ dy$$



$$\Rightarrow w = 0.28 + \frac{0.12}{1.8} y$$

$$d(\Delta H) = 1.73\left(0.4 - 0.28 - \frac{0.12}{1.8} y\right) dy$$

$$\Delta H = \int_0^{1.8} 1.73\left(0.12 - \frac{0.12\, y}{1.8}\right) dy$$

$$\Delta H = 0.187m$$

$$\text{Total } \Delta H_T = 0.25 + 0.187$$

$$= 0.437m$$

⊙ Since, the moisture variation is linear & change in height is directly positional to change in w/c. Hence, for zone 2, the change in height can be calculated from avg change in w/c

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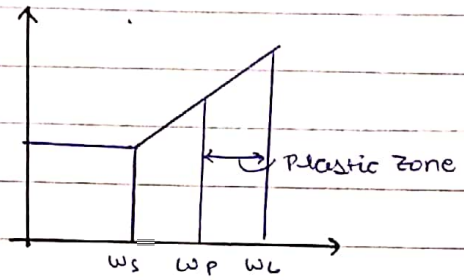
$\Delta w_{av.} = 0.4 - \left[\frac{0.28+0.4}{2}\right] = 0.06$ & $\frac{\Delta H_2}{1.8} = 1.73 \times 0.06 \Rightarrow \Delta H_2 = 0.187m$

PLASTICITY INDEX

- The range of w/c within which the soil behaves as plastic material is called Plasticity index.

$$I_p = w_L - w_p$$



$$I_p = w_L - w_p$$

- This property is due to the presence of clay minerals.
- Clay soils with high specific surface area and charge surface are able to bind water molecule and overall soil will still behave as plastic soil and such soil will have high Plasticity Index.
- Greater is the Quantity of clay in soil, greater is its plasticity index.
- Low plastic soils are used for Embankment construction because they are easy to compact.
- Greater is the plasticity, greater is the shrinkage on drying.

  i.e; lower is the shrinkage limit

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Compressibility → Load se vol. change

Swelling Shrinkage → W/c removal se vol. change

⊙ Soil with larger Ip and liquid limit are called Fat clay (highly plastic).

Soil with lower Ip and liquid limit are called lean clay (No plasticity).

Ip ↑ ⇒ Plasticity ↑ Ws ↓ Swelling Shrinkage ↑
Permeability ↓ clay fraction ↑ creep rate ↑
Organic content ↑ **dry strength ↑
compressibility ↑ cohesion ↑
Smaller compacted density & larger optimum W/c ↑

Ip & Consistency

Ip	Consistency
0	Non-plastic
0<Ip<7	Low plastic
7-17	Medium Plastic
>17	Highly plastic

Shrinkage Index

$$I_s = W_p - W_s$$

Ques→ Ip of a local soil = 15 and Ip of Sand = 0. If the desired Ip is 6, find the % of sand in the mix.

Soln: Ip ∝ clay content/fraction

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$$\frac{IP_1}{IP_2} = \frac{CF_1}{CF_2} \Rightarrow \frac{15}{6} = \frac{(x/x)}{(x/x+y)}$$

x = clay

y = sand

$$\frac{y}{x+y} = ?$$

$$\Rightarrow \frac{15}{6} = \frac{1}{\frac{1+y/x}{}} \Rightarrow 1 + \frac{y}{x} = \frac{15}{6}$$

$$\frac{y}{x} = 2.5 - 1 \Rightarrow \frac{y}{x} = 1.5$$

$$\Rightarrow \frac{y}{x+y} = \frac{y/x}{1+y/x} = \frac{1.5}{2.5} = 0.6$$

CONSISTENCY INDEX (I_c):

(Relative Consistency)

$$I_c = \frac{w_L - w}{w_L - w_P}$$

I_c	Consistency
<1	very stiff (Brittle)
1-0.75	Stiff
0.75-0.5	Medium
0.5-0.25	Soft
0.25-0	very soft
<0	Liquid state

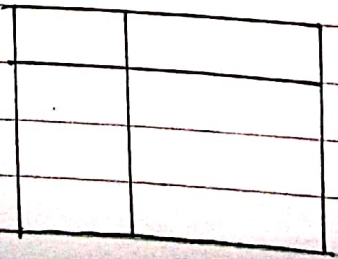
Plastic Range

⊙ Insitu behaviour of saturated fine grained soil at its natural w/c is studied using consistency index.

Larger is the consistency index, greater is the shear strength and better is the material for making foundations.

Liquidity Index

$$I_L = 1 - I_c$$



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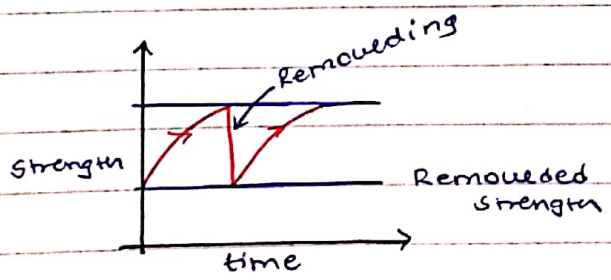
Toughness Index

$$I_T = \frac{I_p}{I_F}$$

- Toughness index indicates shear strength at plastic limit.
- For most of the soil, it is b/w 0 to 3.
- If $I_T < 1$, soil is friable. (easily crushable)
- For most of the clay, I_T is between 1 → 3.

THIXOTROPY

- It is the property of soil due to which loss of strength (shear strength) on remoulding can be regained if left undisturbed for sometime.



- The increase in strength with passage of time is due to tendancy of clay soil to gain their chemical Eqm with the reorientation of water molecule in the adsorbed layer.

- Clay soil have large thixotropy than single grained soil

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Sensitivity

⊙ Degree of disturbance on remoulding is called Sensitivity.

$$\text{Sensitivity } (S_t) = \frac{q_u \text{ undisturbed}}{q_u \text{ remoulded}}$$

q_u = unconfined compressive strength.

Sensitivity

- 1 → Insensitive → Graves, coarse sand
- 2-4 → Normal sensitive →
- 4-8 → Sensitive → Silt / clay
- 8-16 → Extra sensitive
- ≥16 → Unstable / quick clay

⊙ Generally, sensitivity value is >1 but it can be <1 also for stiff clays having fishers and cracks.

⊙ Higher is the sensitivity, higher is the thixotropic hardening.

⊙ Remoulded strength of some clay is so low that UCS test can't be performed. Hence, vane shear test is used to find out the sensitivity.

⊙ The various causes of sensitivity are:

→ Unstable (meta stable) fabric
→ cementation
→ weathering

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Flocculating → More strength
Dispersed → Less strength

→ Leaching : Ion exchange, change in monovalent & diavalent ratio

Activity No.

$$\text{Activity (Ac)} = \frac{I_P}{\%\text{ of clay size particles}}$$

⊙ As per Skempton, volume changes during swelling and shrinkage is a func^n of Ip and % clay fraction. Hence, activity no. is used to study the swelling behaviour.

Activity No.
- < 0.75 → Inactive
- 0.75 – 1.25 → Normal Active
- > 1.25 → Active

⊙ Active means more prone to volume change.

⊙ Kaolinite → Ac < 0.35, (Inactive)
⊙ Illite → 0.38 < Ac < 0.9, (Normal Active)
⊙ Montmorrilonite → 0.9 < Ac < 7.2, (Highly active)

WL ↑ : Compressibility (volume change) ↑
Organic matter ↑
Rate of volume change ↓

Activity No. ↑ : Compressibility ↑
Swelling Shrinkage ↑

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Organic content ↑ : ~~Organic content~~ ↑ (w/c more) ↑

density of soil ↓

Plasticity Index ↑

UCS ↓ (q_u),

Compressibility ↑

Secondary compression ↑

Shrinkage ↑,

Permeability ↓

Charcteristics	w_L same, I_p ↑ (wp ↓) (*)	I_p same, w_L ↑ (wp ↑) (**)
⊙ Dry strength (Ip)	↑	↓
⊙ Toughness near wp (Ip)	↑	↓
⊙ compressibility (wL)	Approx. same	↑
⊙ Permeability (wp)	↓	↑
⊙ Rate of Volume change (wp)	↓	↑

(*) If wp is less, it means that at smaller w/c also, soil is plastic. This means that there must be greater clay fraction

(**) If wp is more means soil is plastic at higher w/c only. This means that there must be lesser clay fraction.

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Ques →

	Soil A	Soil B
W_L	35	65
W_P	15	20
I_f	8	4
W	30	30

- Which soil is more plastic → $(I_P)_A = 20$ $(I_P)_B = 45$

  (I_P)

  $(I_P)_B$ = more plastic

- Which soil is better for foundation → $(I_c)_A = \frac{35-30}{35-15} = \frac{5}{20}$

  (I_c)

  $(I_c)_B = \frac{65-30}{65-20} = \frac{35}{45}$

  $(I_c)_B > (I_c)_A \Rightarrow$ B is better for foundation.

(#) Greater value of (I_f) is a representation of greater loss of shear strength with increase in w/c.

- Which soil has better strength at plastic limit?

  (I_T)

  $(I_T)_A = \left(\frac{I_P}{I_f}\right)_A = \frac{20}{8} = 2.5$

  $(I_T)_B = \frac{45}{4} = 11.25$

Toughness Index $\propto I_P$ (strength)

$(I_T)_B > (I_T)_A$

- Which soil is more compressible? (W_L)

  $(W_L)_B > (W_L)_A$

  $\Rightarrow$ Soil B is more compressible.

- Which soil is more permeable (Ip)

  $I_P \propto \frac{1}{\text{permeability}}$

  $(I_P)_B > (I_P)_A$

  $\Rightarrow$ A is more permeable.

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WS, Phase diagram, Index properties
↳ Numerical
theory
(Borrow Pit)

- Which soil has greater rate of vol. change on loading?

W_L & I_P of B > W_L & I_P of A

⇒ Soil B has lesser rate of Volume change.

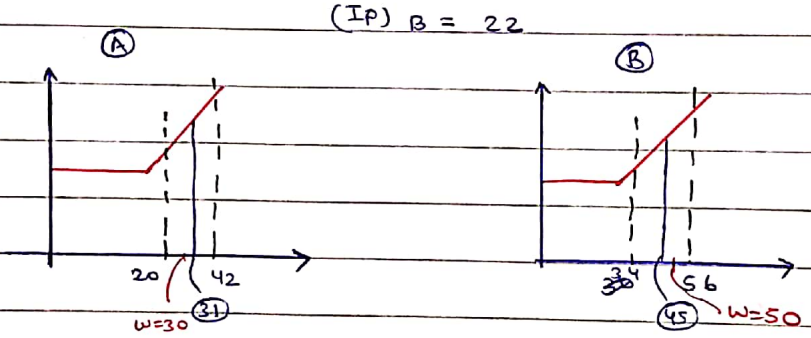
Ques→

	Soil A	Soil B
W_L	40	56
W_P	20	34
W	30	50

- Which is more plastic?

$(I_P)_A = 22$

$(I_P)_B = 22$



B is more plastic than A

(#) Ip same & W_L↑ ⇒ more dry strength [from Table]
less dry strength ⇒ less plastic soil.

(@) Greater is the Gap b/w W_P and W, more plastic is the soil in that state of water content.

(&) In General, when the natural W/c are not given, we will follow the approach of comparing two soils at same Ip on the basis of table given earlier.

(⊙) Hence, Soil A is more plastic.

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CLASSIFICATION OF SOIL

⊙ Soil classification is done on the basis of Indian Standard, IS-1498.

In this, soil is classified as coarse grained, fine grained and peat (organic soil).

⊙ Prefix and Suffix used in soil classification:

Soil Type	Prefix
Gravel	G
Sand	S
Silt	M
Clay	C
Organic	O
Peat	Pt

Sub-Group	Suffix	
Well Graded	W	
Poorly Graded	P	
Silty	M	
Clayey	C	
$W_L < 35$	L	→ low compressibility
$35 < W_L < 50$	I	→ Intermediate compressibility
$W_L > 50$	H	→ High compressibility

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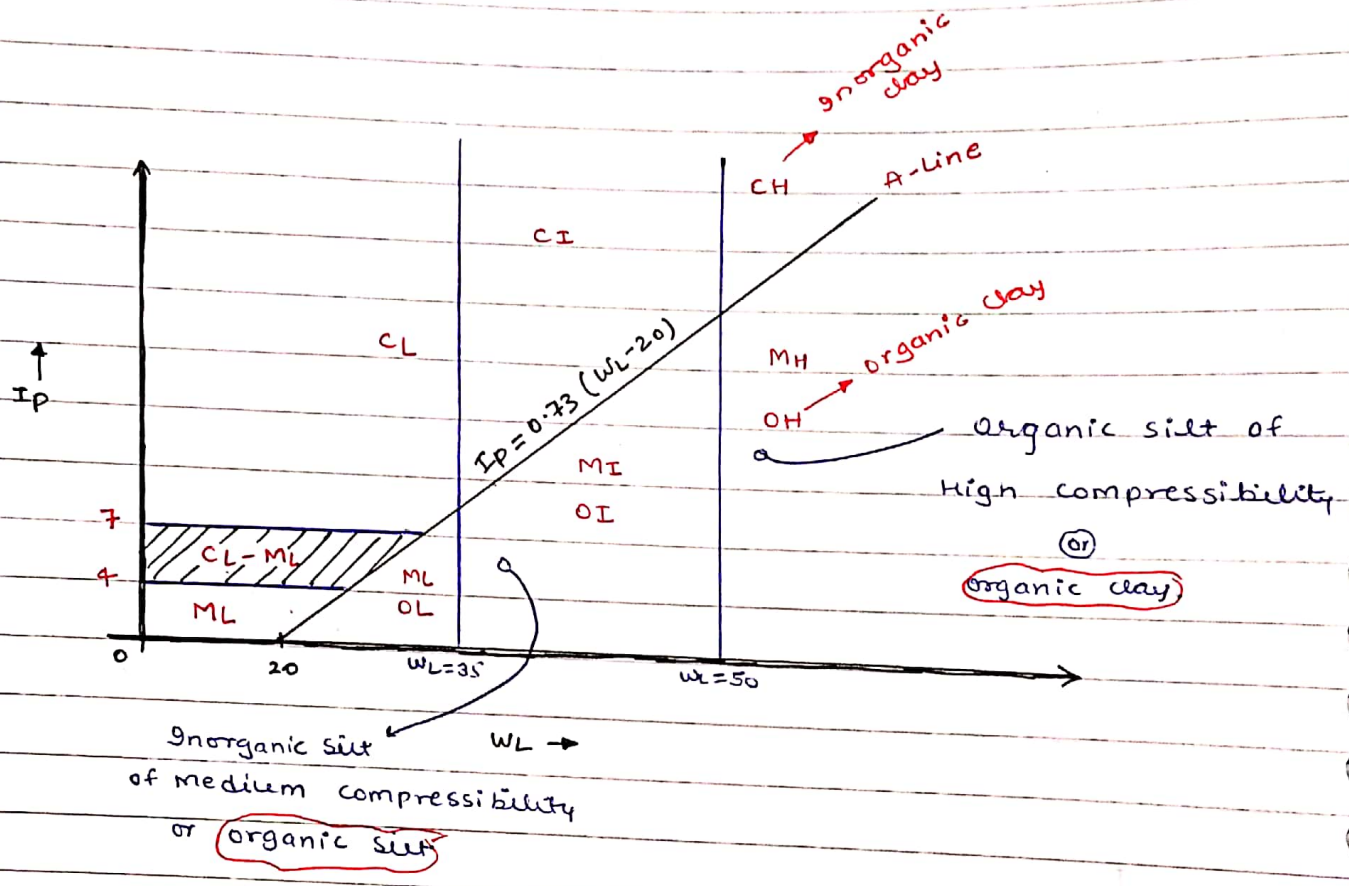
PLASTICITY CHART



① Points which represents diff. samples from same soil stratum, define a line that is roughly parallel to the A-line.

Thus, soil sample from common geological origin will have relationship b/w Ip and WL which will be parallel to the A-line.

CLASSIFICATION of COARSE GRAN SOIL

① It is done on the basis of :

(i) Grain size.

(ii) Gradation characteristics.

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(iii) % of Fines (-75 μ particles) present in soil by weight.

Case-I Fines < 5%

↓

GW	SW
GP	SP

GW

GW	GP
① More than 50% of the soil is retained on 75 μ sieve ⇒ Coarse Grained Soil ② More than 50% of the ~~soil~~ coarse fraction is between 80mm - 4.75mm ⇒ Gravel ③ less than 5% of the soil weight passing 75 μ ④ $C_u > 4$ → Gravel $1 < C_c < 3$ → well graded	
SW	**SP**
①, ② → same as above ③ → More than 50% of coarse fraction is b/w 4.75 mm - 75 μ ④ → $C_u > 6$ → Sand $1 \leq C_c \leq 3$ → well Graded	

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Case-II Fines > 12%

↓

GC	SC
GM	SM
GC-GM	
SC-SM	

⊙ Gravel and Sand are Named as discussed before and for fines, we use the plasticity chart.

If fines gets plotted above A-line, fines are clayey, if gets plotted below A-line, fines are silty. and if fines gets plotted in hatched zone, dual symbol is used. like GM-GC or SM-SC

Case-III Fines b/w ~~12~~ (5-12) %.

⊙ It is a border line case and ~~Border~~ dual symbols are used.

⊙ Various Names can be:

GW — GC / GM GW-GC GW-GM

GP — GC / GM

SW — SC / SM

SP — SC / SM

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⊙ First part of Dual grad symbol indicates gradation, & second part indicates nature of fines.

CLASSIFICATION OF FINE GRAINED SOIL

⊙ Classification is done on basis of plasticity chart.

⊙ If more than 50% of the soil is passing 75 μ then it is fine Grained.

For (-425 μ) sized particles, W_L and W_p are determined and plotted on the plasticity chart and name is given as per zone in the plasticity chart in which the fine gets plotted.

Note:

⊙ Organic soil is distinguished from the inorganic one by colour and odour.

Organic soil is dark in colour and it has distinctive order and order is made more noticible by heating the wet sample.

⊙ If organic content is still doubtful, material can be oven dried and remixed with water and tested for (W_L).

* If reduction in (W_L) after oven drying is ~~by~~ more than ($\frac{1}{4}^{th}$) of the (W_L) before oven drying., then it is an organic soil.

↓

(Confirmatory Test)

Note: ⊙ Boundary classification for coarse Grained soil:

→ Boundary classification can occur within gravel and within sand group., b/w Gravel and Sand group,

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b/w coarse and fine grained soil.

⊙ Within Gravel Group & within Sand Group

(a) Fines > 12% (closer to A-line or in hatched zone)

→ Name can be = GM-GC or SM-SC

(b) Fines b/w (5-12) %

→ Name can be = GW-GM, SW-SM

↑

(#) Fines below A-line or below Hatch zone

(#) Above A-line above hatch zone

↓

→ GW-GC, SW-SC

⊙ When fines b/w (5-12) % and near A-line or in Hatch zone, we prefer non-plastic classification. Hence, the name can be GW-GM (Not GW-GC),

SW-SM (Not SW-SC)

↓

(M) → Silt → less plastic

(#) Fines <5%, Gravel and $C_c \rightarrow [1,3]$ and $C_u = 4$

⇒ GW-GP

⊙ B/w Gravel and Sand Group

(A) Equal Sand & Gravel & Fines <5%.

Name can be GW-SW

→ $C_u > 6$

→ $1 < C_c < 3$

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Field test for identifying Silt
↳ Dilatency test.

GP-SP when $C_u < 4$ or ~~$C_u > 6$~~ $C_c \neq [1,3]$

(B) ⊙ Equal Sand and Gravel + Fines > 12%.

GM-SM → fines plotted below A-line
GC-SC → If fines plotted above A-line
GM-SM → If fines are on A-line or Hatched Boundary ↗ zone.

⊙ Boundary Blw Coarse & Fine Grained Soil:

⊙ 50% ~~fo~~ Coarse Grained & 50% fine Grained.

Hence, fines are (> 12%)

Name can be →

GM-ML	SM-ML
GM-MI	SM-MI
GM-MH	SM-MH
GC-MCL	SC-CL
GC-CI	SC-CI
GC-CH	SC-CH

Boundary classification for Fine Grained Soil

⊙ In this case, the boundary condn can be points getting closure to A-line as closure to W_L of 35 and 50.

In this case, dual symbols are used.

like MI-MH

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Field Identification Procedures

⊙ Is - 1498 recommends the field identification procedure by the following tests:

1. Visual Examination
2. Dilatency test
3. Toughness Test
4. Dry Strength Test
5. Organic content & colour
6. Other identification Test

⊙ We cannot see particle size < 75 μ. Hence, soil containing more than 50% of visual (visible particles are coarse grained.

⊙ If the soil is classified as fine Grained then following manual identification tests are performed?

(i) Dilatency Test (Reaction to Shaking):

Sufficient water is added to small soil sample to just saturate it and soil is placed in open palm and shaked horizontally.

Appearance and disappearance of water quickly indicates silt and non appearance of slow appearance and disappearance indicates clay.

(ii) Toughness Test (consistency near Plastic Limit):

The soil used in Dilatency test is worked upon and 3mm thread are made by rolling and again lump and re rolled to 3mm thread.

Procedure is repeated till 3mm thread starts

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oven dry → hygroscopic w/c removed
air dry → hygroscopic w/c not removed.

crumbling of the lump can still be moulded under high pressure, soil is more tough.

⊙ Highly organic clay exhibits very weak and spongy feel at plastic limit and non-plastic soil cannot be rolled into a 3 thread of 3mm at any moisture content.

(iii) Dry Strength Test (Crushing Resistance):

Clay of high plasticity exhibits high dry strength, fine sands & ~~inorganic~~ silts show low dry strength and ~~fine sands & organic silts~~ inorganic silts exhibits medium dry strength.

(iv) Organic Content & Colour: Fresh wet organic soils have distinctive colour and odour.

Odour can be made more noticible by eating the sample.

(v) Other identification Test:

(a) Acid test: It using dilute HCl acid is basically a test for $CaCO_3$.

Soil with high dry strength indicates that the strength could be due to calcium carbonate as cementing agent rather than due to clay.

(b) Shine Test: It is a quick test for determining the presence of clay.

Test is performed by cutting a lump of dry or slightly moist soil with a knife.

The shiny surface indicates high plastic soil

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and dull surface indicates silt or low plastic clay.

Ques→ Sieve Analysis conducted on a given sample showed that 67% of particles pass through 75 μ sieve and w_L and plastic limit of finer fractions are 45 and 33% respectively. Name the soil as per IS.

Soln:

$w_L = 45$ $\quad I_p' = 12$

I_p for A-line

$= 0.73 (45-20)$

$I_p = 18.25 > I_p'$

$\Rightarrow$ Silt

$\because w_L$ is b/w (35-50).

So, MI

Ques→ A laboratory sieve analysis was carried out on a soil sample out of 500 gm of soil used, in the test, 200 gm was retained on 600 μ sieve, 250 gm was retained on 500 μ sieve and remaining 50 gm was retained on 425 μ sieve. Find the coeff. of Uniformity of soil

Soln:

$$C_u = \frac{D_{60}}{D_{10}} = \frac{0.6}{0.5} = 1.2$$

Total weight = 500 gm

200g → 600μ $\quad$ % passing $= \frac{300}{500} \times 100 = 60\% = D_{60}$

250g → 500μ $\quad$ $10\% = D_{10}$

50g → 425μ $\quad$ $C_u = \frac{600}{500} = 1.2$

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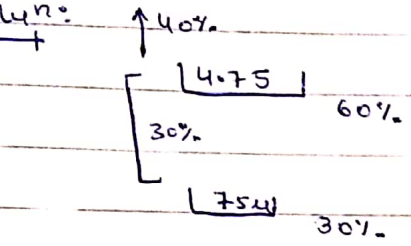
Ques→ Lab. test of a soil sample are given below. The % finer than 4.75 mm is 60. % finer than 75 μ is 30. $W_L = 35$, $W_P = 27$. Name the soil.

Soln:



70% particles are larger than 75μ
⇒ Coarse grained.
Fines are 30%. >12%.

GC
GM
SC
SM

} Soil is Gravel b/c of the coarse fraction (70%), 40% are of size > 4.75mm

$W_L = 35$

$I_p = 8$ $\qquad (I_p)_{A\text{-line}} = 0.73(35-20) = 10.8 > 8$

⇒ points gets plotted below A-line.
⇒ fines are silty.
⇒ Soil is GM.

Ques→ Based on Grain size distribution analysis, D10, D30 and D60 are 0.23 mm, 0.3mm, and 0.41 mm respectively. Name the soil.

Soln:

$$C_u = \frac{D_{60}}{D_{10}} = \frac{0.41}{0.23} = 1.78 < 2 \text{ or } 3 \quad \text{uniform}$$

$$C_c = \frac{D_{30}^2}{D_{60} D_{10}} = \frac{0.3 \times 0.3}{0.41 \times 0.23} = 0.95 \quad \text{poorly graded.}$$

Poorly Graded Sand.

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CLAY MINERAL & SOIL STRUCTURE

- Soil can be organic and inorganic in nature.
- Inorganic soil is obtained from the weathering of rock, and organic soils are obtained from decomposition of vegetative matter.
- Weathering of rock produces clay minerals and non-clay minerals.

  clay minerals are: Kaolinite, Halloysite, Illite, & Montmorrilonite (Smectite)
- The Non-clay minerals are: Quartz, feldspar Mica, Calcite, Gypsum, Dolomite, Pyrite.
- Grains of clay are composed of clay minerals and they are responsible for plasticity and cohesion.

STRUCTURE OF CLAY MINERALS

- Platlets of plates of clay soil are made up of many crystal sheets which are made up of repeating atomic structure.

  the atomic structure of clay minerals are made up of two fundamental crystal sheets.

(i) Tetrahedral or Silica Sheet (made up of Tetrahedral unit)
(ii) octahedral sheet or Alumina sheet. (made up of octa-hedral units)

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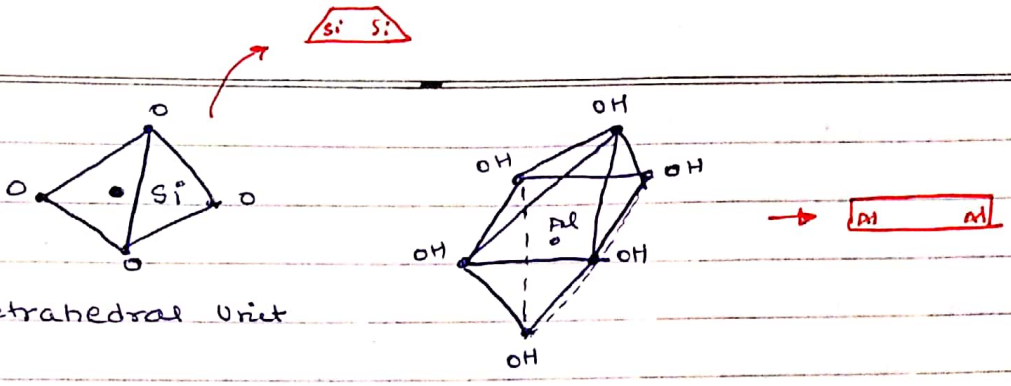
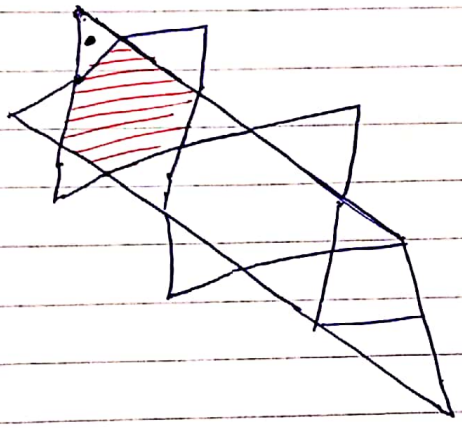
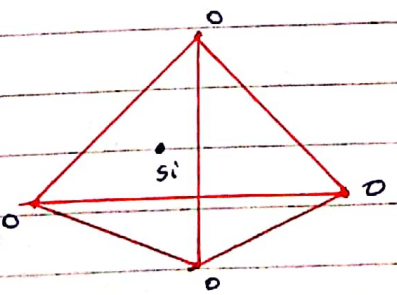
Generally, shape of clay particle is → (FLAKY)





① The mode of stacking of these sheets and nature of bonding forces b/w them and the diff. metallic ions in the crystal lattice. go on to make diff. clay minerals.

Tetrahedral Unit & Silica Sheet



① (4) oxygen atom placed at the tip of tetrahydron enclosing the silicon atom

① (3) Oxygen atom at the base are common to 2 units, hence net charge on one unit is:

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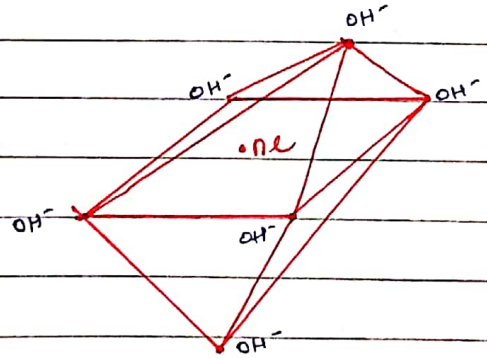
$= \left(-\frac{2}{2}\right) + \left(-\frac{2}{2}\right) + \left(-\frac{2}{2}\right) + (-2) + 4 = -1$

- There is a hexagonal opening at the base of silica sheet and this shape is shown symbolically as:

Octahedral Unit & Alumina Sheet

- It has (6) hydroxyl ion (OH^-) at the tip of an octahedron enclosing an aluminium or magnesium or other metallic ion.

The rows of OH^- ion in the sheet are in two planes with each OH^- ion in 1 unit is common to (3) octahedral units.



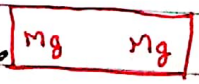
$\Rightarrow \frac{6(-1)}{3} + 3 = +1$

↓ If Al is in centre.

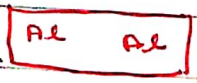
$6\left(-\frac{1}{3}\right) + 2 = 0$

↳ If Mg is in centre.

- Octahedral sheet is represented as:



Brucite Sheet.



Gibbsite Sheet

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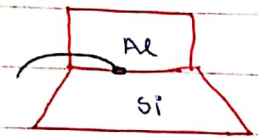
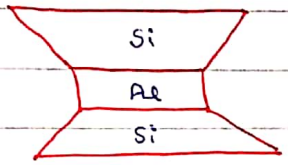
⊙ Clay Minerals have unit cells consisting of (2) or (3) Sheets



⇒ Two Sheet Mineral

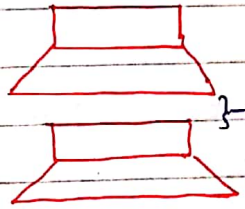
(1:1) Clay mineral



⇒ 3 sheet Mineral

(2:1) Clay mineral

In ⊙ The unit cells, tetrahedral and octahedral sheets are held very strongly by primary valence bond, (Ionic and covalent)



} → Secondary bond.

⊙ The bond holding various unit cells is called secondary bond. like wanderwall force, Hydrogen bond. & also through cations that enter into the layer. to maintain electrical neutrality

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Isomorphous Substitution

- Ideally, Silicon is present at the centre of tetrahedral and Al is present at the centre of octahedral Gibbsite unit.

In clay minerals, however, some of the tetrahedral units may have Al in place of Silica and in Gibsite unit, Mg in place of Al. This presence in an octahedral and tetrahedral units of cations other than ideally found without change in the crystal structure, is called **Isomorphous Substitution.**

- Isomorphous Substitution occurs during the initial formation or subsequent alteration of minerals.

- Isomorphous Substitution leads to net -ve charge on clay particles.

Various Types of clay Minerals

- Clay minerals continue to change from one form to another.

Early weathering produces non-clay minerals and during intermediate weathering, Illite and montmorillonite are formed & during advance weathering, Kaolinite is formed.

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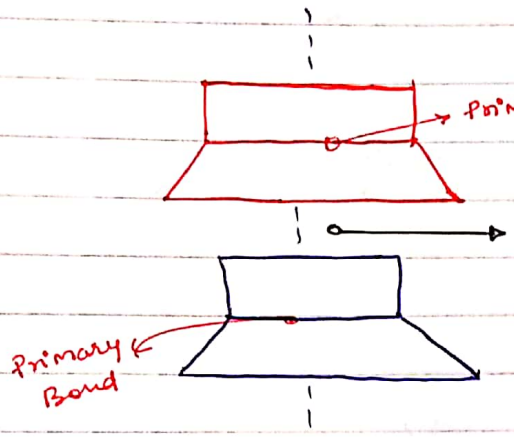
KAOLINITE



⊙ Unit cells are held strongly and water can't enter blw structural units. Hence, swelling shrinkage is less. i.e; Activity is least.
It has little or low isomorphous substitution.

⊙ Kaolinite is 1:1 Mineral (1-silica, 1-Alumina)

⊙ Kaolinite formation is favoured when Alumina is in excess, when cations have low electrolyte content (cation content), low pH.
It is found in old deposits in highly weathered soil with good drainage.

⊙ Kaolinite particles posses net -ve charge on account of substitution of Al^{3+} in Silica sheet or divalent ions for Al in octahedral sheet.
But replacement of 1 silica ion in 400 units will be adequate.

⊙ The hydrogen of OH^- (hydroxyl) may be replaced with

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Ip & Activity

Kaolinite
- low pH → +ve charged
- High pH → -ve charged

Exchangable cations,

⊙ Broken bonds around the particle edge may give unsatisfied charge.

Note: ⊙ In High pH environment, Hydrogen ion from OH^- goes into solution giving the -ve charge to the surface. Hence, in low pH environment, Kaolinite surface will be +vely charged.
(means -ve charge ka kam hona)

** ⊙ Hence, in low pH environment, cation absorption capacity will be less.

Halloysite

⊙ It is a clay mineral of Kaolinite Group and the structure units are seperated by water molecules & when dried they might get converted into Kaolinite.

⊙ Kaolinite and Halloysite are used in making china mare and Anti diarrheal medicine.

Halloysite is formed in high rainfall volcanic area.

Halloysite $\xrightarrow{\text{dried}}$ Kaolinite

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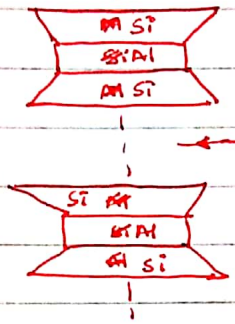
Smectite

Montmorrilonite



- It is also called ~~cementite~~. smectite.
- It is a 2:1 clay mineral. (2 silica, 1 Alumina)
- Thus, it is found in the abundance of silica.
- It is has largest specific surface among all clay minerals. & successive unit cells are held by vanderwall forces.

  It has significant isomorphous substitution.
- Normally, every 6th Al is replaced by Mg.
- Since, it has larger -ve charges on the surface, it has high cation exchange capacity.

  (cation को Apne surface pe retain करने की property)

- Since, interlayer forces are weak, & huge quantity of water can enter b/w interlayers, causing the layers to be seperated. & it has significant volume change.

- It is found in Bentonite and Black cotton soil.
- It is used as drilling mudy (ड्रिलिंग)
- It is used as Grout material for seepage barriers.
- It commonly found in arid areas and formed due to weathering of volcanic ~~rock~~ under poor drainage condn.

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ILLITE → K-Bond (Potassium)

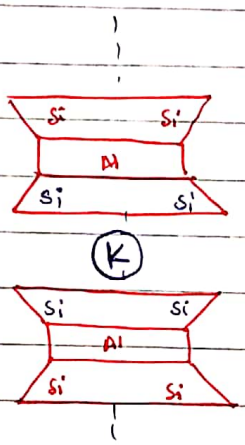
① Condⁿ favouring its formation are:

High pH, high electrolyte concⁿ, poor drainage, poor leaching, arid conditions

ILLITE

↳ Mica like clay Mineral → Hydrous Mica.

②



③ It is a (2:1) clay mineral having substential isomorphous substitution and (K^+) ion bonds, the two -ve surfaces of silica sheet tightly and K^+ ion occupies a hexagonal hole in the silica sheet.

④ Bcoz of strong bonding b/w interlayers, it is less prone to cleavage or splitting and hence doesnot swell as much as the Montmorrilonite but more swelling than kaolinite.

Clay Mineral	Grain Size	Cation Exchange Capacity	Ip	Dry Strength	Activity
Montmarillonite	Smallest	Largest	Largest	Largest	Largest
Kaolinite	Largest Intermediate	least	Least	least	least
Illite	~~Largest~~ Intermediate	Intermediate	Inter mediate	Inter - mediate	Inter - mediate

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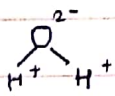
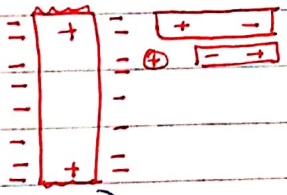
CLAY WATER RELATION SHIP

(a) Property of clayey soil is affected significantly by the presence of water but granular soil is generally not sensitive to the presence of water.

(b) Water is a dipolar molecule  with ca of +ve and -ve charge not coinciding.

Thus, it can be treated as a dipole.

(c) The clay particles always carry -ve charges except at the edges.



Reason for charge accumulation are:

(i) Isomarphous Substitution by lower Valency cation.

(ii) Dissociation of OH^- ions.

(iii) Breakage of Edges.

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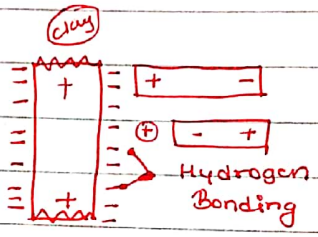
[pore → Non-oriented water molecule.]

General Mechanism of clay - water Interaction

* (i) Electrostatic Attraction b/w -ve charge of face of prate and +ve face of water dipole.

* (ii) Attraction b/w cation in water and -ve charge end of water dipole

* (iii) Hydrogen Bonding.



(iv) Attraction by Osmosis.

(v) Capillarity → Minor

(vi) Vanderwaals forces

Adsorbed water layer & Diffused Double layer

⊙ Attraction of water to the particle surface is very strong near the surface and tapers off with distance.

The layer of water surrounding the clay particle and strongely attracted through powerful force of electrostatic attraction is called adsorbed water layer.

⊙ Thickness of adsorbed layer is around 10 A° thick (approx.)

⊙ Beyond this, there is an outer layer which is attracted to a lesser degree and is more mobile.

This layer extends upto the limit of attraction

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Double layer thickness thin → more repulsion

and is known as Diffused Double layer.

⊙ Water beyond the diffused double layer is Non-oriented water.

© Thickness of double layer affects the behaviour of soil. The conc^n of ions (+ve charge, cation) is also max^m near the clay surface. and tapers off gradually and ultimately becomes equal to the conc^n in normal or non-oriented water.

Thickness of Double layer: x

$$x = \sqrt{\frac{\epsilon_0 D K T}{2 n_0 e^2 v^2}}$$

$x \propto \frac{1}{\sqrt{n_0}}$

$x \propto \frac{1}{v}$

n_0 = Electrolyte conc^n (cation conc^n)

D = Dielectric const. of medium.

T = Temperature.

v = valency of cations.

e = charge

Note: $(D \times T)$ is approx. constant bc if temp. increases, dielectric constant decreases. Hence, temp. doesnot have much effect on double layer thickness.

However, with increase in temperature, Double layer thickness decreases.

⊙ Decrease in double layer thickness leads to ↓ red^n in repulsion b/w interacting clay particles. and decreased repulsion leads to clay structure becoming flocculated and red^n in swelling pressure

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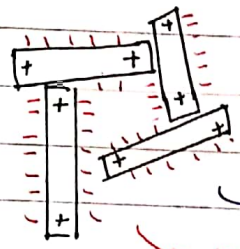
Compaction of clay ⇒ Pneumatic Roller.

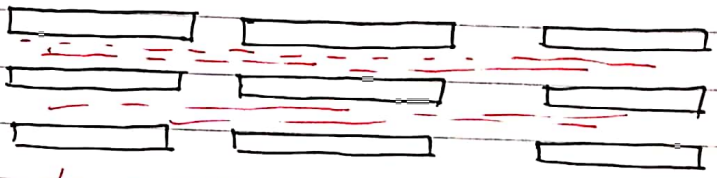
in expansive soil.



(Flocculated structure) → If net force is Attractive.

↳ Edge to Edge configuration

↳ More voids



less voids

(Dispersed structure) → If Net force is Repulsive

↓

Face to face configuration

Other factors affecting Double layer thickness

① In Kaolinite :

pH ↑ ⇒ ϰ ↑ → {Double layer thickness}

with increase in ion size ⇒ Double layer thickness increases.

② In Montmorillonite :

pH doesnot affect the double layer thickness

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Cation Exchange Capacity

- Ability of clay particles to adsorb ions on the surface or edges is called cation Exchange capacity or base exchange capacity.
- Cations are termed as exchangable because one cation can easily be exchange with other cation.
- Diff. clays have diff. charge efficiencies and have varying ability to adsorb exchangable cation.
- Depositional Environment, weathering action, leaching --etc. will govern the kind of ions present in the clay deposits.
- Non-Marine soils have Ca > Mg > Na and K present in decreasing order of abundance.
- In marine Soils, Na is the dominent cation.

Cation Replacability

- Cations of one type can replace the cations of another type and the ease of replacement depends on: valency, concn, and ion sizes.
- Smaller cations tends to replace larger cations. Higher concn cations can replace lower concn cations.
- Under Normal condn, the replacement capacity is as shown below:

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$Na^+ < H^+ < Mg^{2+} < Ca^{2+} < Al^{3+} < Fe^{3+}$

- The above replacement capacity trend can be rep reversed by providing greater conc^n of lower replacing capacity cations.
- The above replacement capacities made use of stabilization of sodium clay, using lime $Ca(OH)_2$ → slacked / Hydrated lime.

CLAY PARTICLE INTERACTION

- Soil structure of clay depends upon forces of attraction and repulsion b/w the soil particles.
  There is an adsorbed layer of water around a particles and hence two particles interact through adsorbed layer.
- The repulsive force b/w the clay particles is due to the double layer and attractive force is due to Vanderwaal force.
- If the conc^n of cation or the valency of cation increases, it results in the reducn of double layer thickness and hence reduces the repulsive force b/w the particles.
- As the two particles come closer, the net force may become attractive and flocculated structure is formed & if net force is repulsive, dispersed structure is formed.

Flocculated structure mostly have edge to face contig arrangt

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Ca → decrease double layer thickness.

But can have edge to edge configuration also. and dispersed structure have face to face configuration.

*** (H) Tendancy towards flocculation is usually caused by :

Increasing ↑	Decreasing ↓
(cation) Electrolyte conc^n	Dielectric constant
Ion Valency	pH
Temperature	Anion Adsorption
	size of Hydrated ions.

** ⊙ Dispersed structure have lower void ratio as compared to flocculated structure

⊙ Marine clay has flocculated structure and lacustrine clay have dispersed structure.

⊙ Sodium clay can adsorb more water. Hence, swelling is more and if sodium (Na) is replaced by calcium, swelling will decrease

⊙ Flocculated structure is stronger and can resist greater force due to stronger bond b/w particles

⊙ Flocculated structure is insensitive to vibration. Flocculated structure has lower compressibility at lower stress level., high permeability & high shear strength.

⊙ Dispersed structure develops in clay that has been remoulded and reworked.

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It has low shear strength, high compressibility but low permeability.

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SOIL COMPACTION

CHAPTER-5

⊙ Soil Compaction is the process of packing the soil particles closer by reducing air voids through application of Mechanical means.

⊙ It is achieved by application of static and Dynamic loading.

Various Advantages of compaction are:

(i) Settlement can be reduced or prevented.
(ii) Soil strength increases and stability can be improved.
(iii) Load carrying capacity of pavement subgrade can be improved.
(iv) Swelling and shrinkage can be controlled.
(v) Permeability can be reduced.
(vi) Density can be increased.
(vii) Liquifaction tendancy can be reduced.

Compaction is measured quantitatively in terms of dry unit weight.

Difference B/w Compaction and Consolidation

Compaction	Consolidation
① Instantaneous Phenomena	① Time dependent phenomena
② Soil is always partial saturated	② Soil is always complete saturated
③ Densification is due to reduction in voids at a given water content.	③ Volume Reduction is due to explusion of pore water from voids
④ Rollers, tempars are used	④ Static loading is Applied.

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COMPACTION OF COHESIVE SOIL

⊙ Clay cannot be compacted by Vibration. For compacting clay, ~~post~~ positions of particles are changed by forcing the contact point along the adjacent surface to a more parallel position with reduce voids. Application of static pressure causes Compaction of soils.

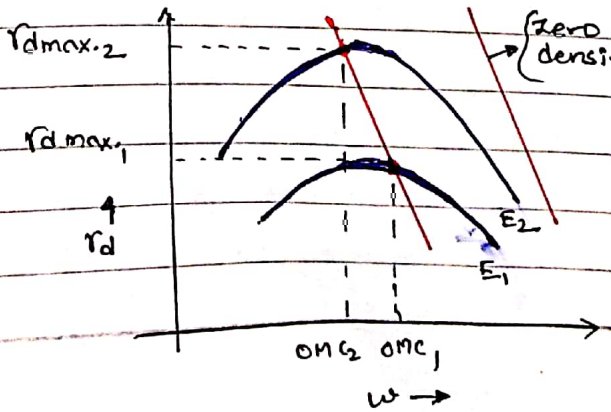
#Compaction Test:

⊙ The objective in compaction is to achieve desired engineering property at least cost. For this, we need to know Relationship b/w soil behaviour and placement details of a particular soil. This information is obtained from laboratory compaction Test.

⊙ Generally, dynamic compaction test is done in lab and compactive effort and moisture content to be adopted in the field economically, is judged from correlation b/w field compaction method and laboratory compaction method.

PROCTOR TEST

⊙ As per Proctor, a definite relationship exist b/w a soil moisture content and degree of dry density to which soil can be compacted.



(Zero air void density) → can never be achieved.

$E_2 > E_1$

E → Compactive Effort.

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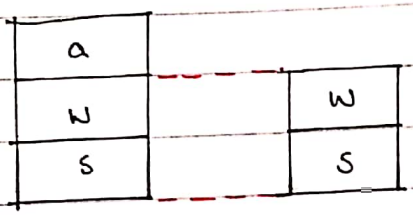
⊙ For a specific amount of compaction energy, there is one w, termed as optimum moisture content (OMC) at which a particular soil attains max^m dry density. Maxm dry Density achieved is a function of type of soil, compactive effort and method of compaction.



$$\text{Dry density} = \frac{G\gamma_w}{1+wG}$$

Zero air void density.

Note: ⊙ Typical value of $\gamma_{d\,max.}$ = 16-20 kN/m^3
OMC is around (10-20%)

Standard Proctor Test	Modified Proctor Test
Weight of Rammer = 2.495 kg	4.54 kg
Height of fall = 304.8 mm	457.2 mm
Volume of mould = 944 cc	944 cc
Layers = 3	5
Blows = 25/layer	25/layer
Compacted Energy applied per unit volume (E_1) = 592.7 kJ/m^3	= 2696.3 kJ/m3

$\frac{E_2}{E_1} = 4.55$ → In modified Proctor Test, Energy imparted per unit volume is 4.55 times that of standard Proctor Test.

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IS Light Compaction Test	IS Heavy Compaction Test.
Weight of Rammer = 2.6 kg	4.9 kg
Height of fall = 310 mm	450mm
Volume of Mould = 1000 cc	1000 cc
Layers = 3	5
Blows = 25/layer	25/layer
E_1 = 593.01 KJ/m3	E_2 = 2703.88 KJ/m^3

Factors Affecting Compaction

(A) Moisture content (B) Soil type

(C) Compactive Effort

Type of Effort

Amount of Effort.

Moisture Content:

(A) Lubrication Theory: At low w, soil is stiff and offers more resistance to movement. As w↑, a film of water surrounds the soil particle and tends to lubricate the particle and it make easier for particles to go into denser packing. this phenomena continues upto OMC and beyond OMC, water starts occupying the space which could have been occupied by the soil particles.

(B) Lambe's Explaination: At low w, Double layer is not fully developed and hence repulsive force is small and net force is attractive due to Vanderwaals

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forces. This makes it difficult for particles to move closer. Hence, γ_d is smaller. As w↑, Double layer thickness ↑ and hence repulsive force ↑ and particle can go easily into denser packing thus γ_d ↑, double layer development is complete at OMC and further addition of water will start occupying the space which could have been occupied by the soil particle and hence γ_d ↓.

Note: ⊙ 10% Air void density ≠ 90% Saturation density
⊙ 0% Air void density = 100% Saturation density
⊙ 10% air content density = 90% Saturation Density.

(#) Compactive Effort: For a given type of compaction higher is the compactive effort, higher is the γ_{dmax}. achiever and lower is the OMC.

⊙ on wet side of optimum increase of compactive Effort will have little influence on the γ_d.

(#) Line ~~of~~ of Optimum is Generally Parallel to Zero air void line.

Note: ⊙ Degree of saturation at OMC is almost same in all cases

⊙ If large compactive effort is applied on wet side of optimum, density will not increase significantly instead pore pressure may built up creating slope stability problem and induces time dependent settlement

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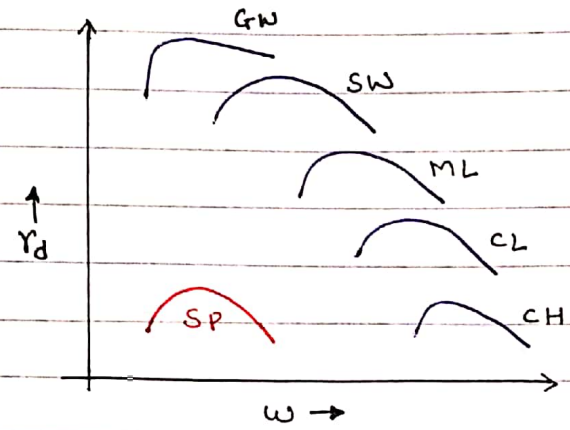
Type of Soil:

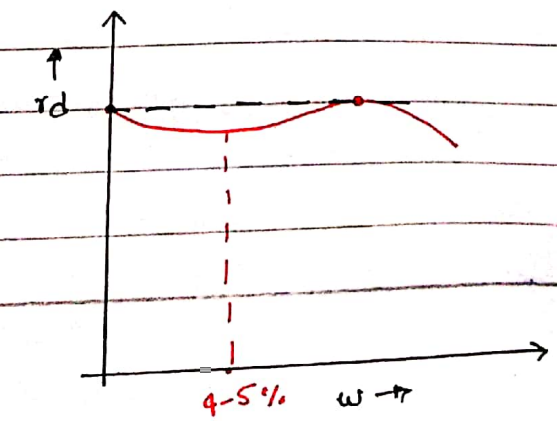


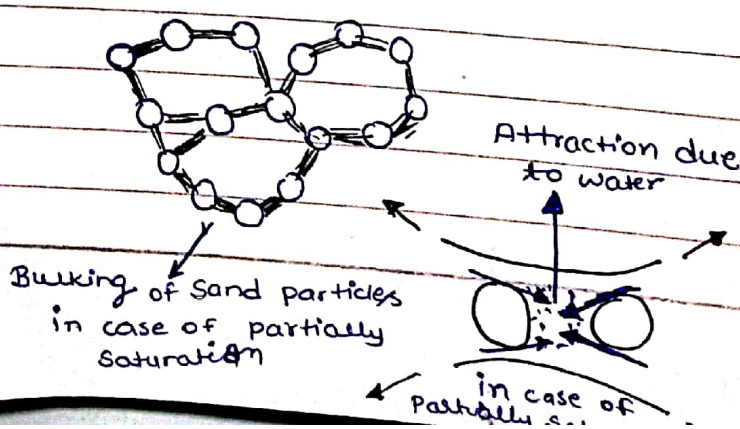
- Well Graded soil can be compacted to high γ_d specially if it contains some amount of fines.
  As fines ↑, $\gamma_{d(max.)}$ ↑.
- Poorly Graded, Uniform sand leads to lowest $\gamma_{d(max.)}$.
- In case of clays, $\gamma_{dmax.}$ ↓ as Plasticity ↑

→ Heavy clays with high plasticity has lower $\gamma_{dmax.}$ and high OMC

→ Increase in organic content increases OMC but decreases $\gamma_{dmax.}$

Compaction of Sand





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- Typical inverted V-shape of compaction curve exists only for soil possessing some amount of plasticity.
- In case of pure sand compaction, curve is not having inverted V-shape
- In case of pure sand, γ_d initially ↓ with w ↑. This is due to capillary tension in the pore water which prevents soil particles to move closer on load application and this is called Bulking of Sand.
- Maxm bulking occurs at w of about 4-5%.
- With increase in w ↑, γ_d starts increasing ↑ due to breakage of capillary menisci and particles are able to move in closer packing.

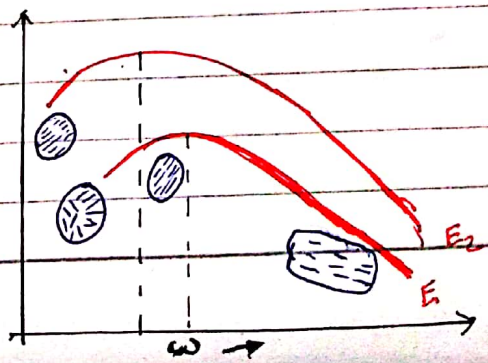
- γ_{dmax}. results when soil is fully saturated and with further increase in w, γ_d starts ↓

- γ_{dmax}. for completely dry and completely saturated condition is nearly the same hence sand should be compacted either in completely dry condition or in completely saturated condition.

Effect of Compaction on Properties of Soil

(I) Soil structure:

Soil compacted at w < OMC has floculated structure and at w > OMC have dispersed structure.



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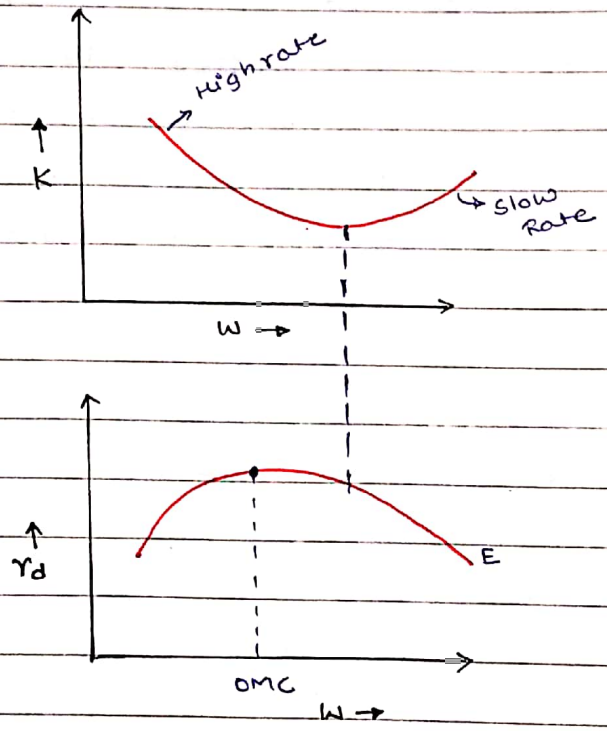
⊙ As w↑, repulsive force ↑ and particles gets oriented in disperse structure and similarly with E↑, the particles becomes more oriented.

② Permeability :



Due to w↑ for given E, there is an improvement in orientation of particles and corresponding reduction in size of voids which causes decrease in permeability K in min. slightly beyond omc and thereafter increases but at very slow rate, generally remains smaller than that of dry side.

⊙ If the compactive Effort E↑, K↓ due to increase in γ_d ↑ and better orientation of Particle.

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⊙ Kneading compaction gives better oriented structure due to higher shear strain and hence leads to smaller permeability.

③ Swelling: Soil compacted dry of optimum has higher deficiency implies more water than the soil compacted wet of optimum and therefore swells more.

④ Construction Pore Water Pressure: construction pore water pressure is more on wet side of optimum and after compaction load has been removed, -ve pore water pressure may develop.

⑤ Shrinkage: Soil compacted on dry side of optimum shrinks less on drying compared to that compacted wet of optimum. Drying shrinkage results from particle movement due to pore water tension develop by capillary meniscus.

⊙ Two samples of clay at same initial w/c but have different fabric, the one which is more disperse shrinks more and this is because the pore size is smaller in dispersed structure thus allowing more capillary stress and easier movement of particle.

⊙ kneading shrinkage will be greater than static shrinkage.

⑥ Compressibility: Flocculated structure on dry side of optimum offers greater resistance to compression due to the bonding b/w the particles.

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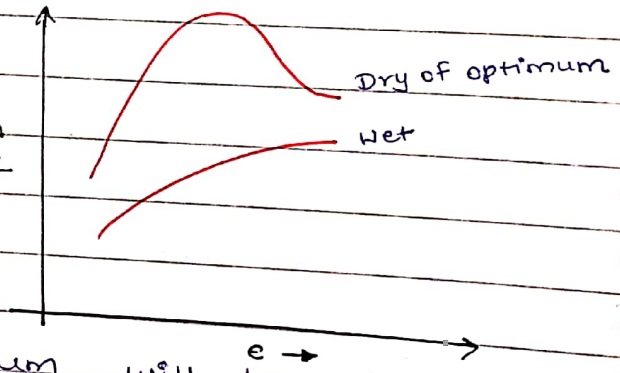
Hence, on dry side of optimum, the compressibility will be less at lower stress level.

- Kneading or impact type of compaction which creates more disperse structure, this will lead to greater compressibility at dry side of optimum.

⑦ Sensitivity: On dry side of optimum, remoulding will break the flocculated structure and greater loss of strength will occur hence sensitivity on dry side of optimum is more.

⑧ σ-ε Relationship: Soil compacted on dry side of optimum has steeper σ-ε curve than wet side of optimum.



- Modulous of Elasticity for soil compacted on dry side of optimum will be higher.
- Soil will have brittle failure on dry side of optimum like dense sand and overconsolidated clay.
- At higher strain level, both soil will have similar strength.

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Property	Dry of Optimum	Wet of Optimum
Structure after compaction	Flocculated (Random)	Dispersed (oriented)
Water Deficiency	More	Less
Swellability	More	less
Construction Pore Water Pressure	Less	More
Shrinkage	Less	More
Permeability	More (isotropic)	Less ($K_h \uparrow$ $K_v \downarrow$) (Anisotropic)
Compressibility		
(A) At low stress	less	More
(B) At High stress	More	less
σ-ϵ Behaviour	Brittle	Ductile
	High Modulous of Elasticity (High Peak)	Low Modulous of Elasticity (No Peak)
Sensitivity	More	Less
Shear Strain	More	Less

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Suitability of Compaction Equipment

Type of Equipment	Suitability of Soil type	Nature of Project
① Rammer, Tamper, or frog Hammer	All types of soil	In confined areas such as fills behind retaining walls, Basement fills, trench fills.
② Smooth Wheeled Roller	Crust rock, Sand, Gravel and Balast. Not suitable for embankment Not for soft subgrade Not for uniform Sand	Road construction (Subgrade or Base course)
③ Pneumatic Tyred Roller	Sands, silts, gravel and clay of all compressibility and not suitable for uniformely graded soil	Base, Subbase, embankment, Earthern dams, clay of all compressibility Air fields.
④ Sheep foot roller	Clayey soil, silty and clayey sand and silty clay of all compressibility	core of Earthern Dam
⑤ Vibratory Roller	Sand, Granular Base, course, Asphalt (pure sand)	Embankment for oil storage tank
⑥ Grid Roller	Well Graded coarse soil, Weathered Rock, Crushing oversize material Not suitable for clayey soil, silty clay or uniform soil.	SubBase and SubGrades.

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Field Control of Compaction

- In field control testing, we need to find out γ_d and w.

$$\gamma_d = \frac{\gamma_t}{(1+w)}$$

γ_t can be calculated by methods calculated earlier and proctor needle can be used to find out w.

- A calibration chart is made for the proctor Needle in the laboratory relating the penetration resistance and w and in the field, penetration resistance is found out and w is determined from caliberation chart.

Relative Compaction : $R.C. = \frac{\gamma_{d\ field}}{(\gamma_{d\ max.})\ lab.}$

Compaction for deeper layers of soil

① **Vibrofloatation:** It is used for compacting granular soil only and can compact sand upto 30m depth

② **Dropping of Heavy weight:** 500 kN dropped from 40-50m height can be used for compacting cohesive as well as cohesionless soil.
Compaction can be done upto 10-12m.
Also known as deep dynamic compaction.

③ **Balasting:** It is used for compacting loose granular soil.

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Other Soil Improvement Techniques

① **Preloading:** It is used to densify soft to very soft cohesive soil. In order to remove water and to reduce the period of consolidation, horizontal or vertical sand drains are provided.

② **Sand Compaction Piles / Stone Columns:** They are used for compacting the soil by driving hollow steel pipes with closed bottom and while withdrawing pile, bottom plate opens up and sand fills the hole.

- Stone columns are generally preferred in soft cohesive soil

③ **Chemical Stabilization:**

(A) **Cement Stabilization:** It involves mixing of soil, cement and water and compacting this mixture to high density.

- All types of soil except highly organic clay can be improved using cement.
- If sulphates and organic matter are present in soil, we can't adopt cement stablization.

Advantages → most of the soil can be stablized. Strength increases with age, soil will have considerable flexural strength.

Disadvantage → cement is costly material and hydration of cement leads to volume changes which gives rise

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to Shrinkage Crack.

Mechanism ⇒ Cement combines with soil particles to form strong bridge blw aggregates. thus producing hard durable material.

⊙ In Granular Soil, cementation produces like Calcium Silicate hydrate and Calcium Aluminate hydrate are formed providing the bond.

⊙ However, in fine grained soil apart from bridge formation, reaction blw free lime present in the cement paste and clay minerals occurs causing clay particles to aglomarate in a larger size particle imparticle friability and increases strength.

⊙ Vibratory rollers can be used for friable soils.

(B) Lime Stablization: → Used for Heavy clay (Ip↑)

⊙ When lime is added to clayey soil, it lowers WL reduces Ip, WL↑.

⊙ It makes clayey soil friable and swelling decreases↓, OMC decreases ↓

⊙ It has little effect on highly organic soil and on soil with little or low clay content.

Generally, 5-10% of lime is used for plastic soil

Lime Stabilization ⇒ Strength ↑, Stiffness ↑, swelling potential and swelling pressure ↓.

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- Not recommended for sandy soil except when lime is added in combination with clay, fly ash, and other pozzolanic material.

Various Mechanism during lime Treatment are:

① **Cation Exchange:** → (Ca^{2+} replacing Na^{+})

- Result in binding of clay, reduction in clay size particle and reduction in plasticity.

② **Pozzolanic Reaction:** When silica, Alumina reacts with available calcium (ca) in lime, they produce very stable calcium silicates and Aluminates that acts as cementing agents.

Note: - **Carbonation:** Reacn of lime with CO_2 from atmosphere resulting in poor cementing agent like $CaCO_3$, $MgCO_3$. These carbonates leads to soil becoming more plastic and carbonation generally occurs when excess lime is added or when soil does not contain sufficient clay.

(c) **Bituminaus Stabilization:** Clayey soil generally have satisfactory bearing capacity at low w. Hence, by encorporating Bitumin in such soils, water proofing results and lower w is maintained.

Thus, will have higher Bearing capacity.

In case of non-cohesive Granular soil → It serves as a banding or cementing agent.

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(d) Stabilization Using Electroosmosis Method:

⊕ Electroosmosis method is used to densify the insitu cohesive soil. Electric current induces of flow of water from anode to cathode, the cathode is generally made of perforated metal pipe which is used as a well point for removing water.

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PERMEABILITY

⊙ Permeability is an ease with which water can flow through any medium and knowledge of permeability is essential for the competition of settlement of building, yield of well, seepage through or below the earthern dam and in the control of hydraulic stability of the soil mass.

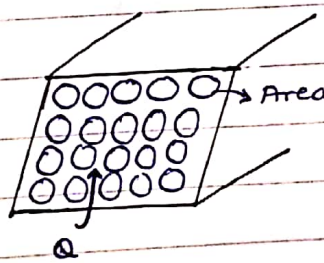
Darcy's law

⊙ For 1D flow, through fully saturated soil, discharge is given by:

$$Q = KiA$$

$$\frac{Q}{A} = V = Ki$$ → Discharge Velocity

A → Total area of flow.
K → Permeability.
i → Hydraulic Gradient.



$$\frac{Q}{A_{void}} = V_{seepage}$$ = Seepage Velocity (Actual Velocity)

$$\frac{Q}{A_{void}} = V_{seepage} = \frac{Q}{\frac{A_{void}}{A} \times A} = \frac{V}{n}$$ → Porosity

$$V_{seepage} = \frac{V}{n}$$

$$i = -\frac{dh}{dx}$$

h = Total Head = $\frac{p}{\gamma} + z + \frac{v^2}{2g}$

Note: ⊙ Seepage Velocity is generally small and hence in seepage calculation, the total Head is taken

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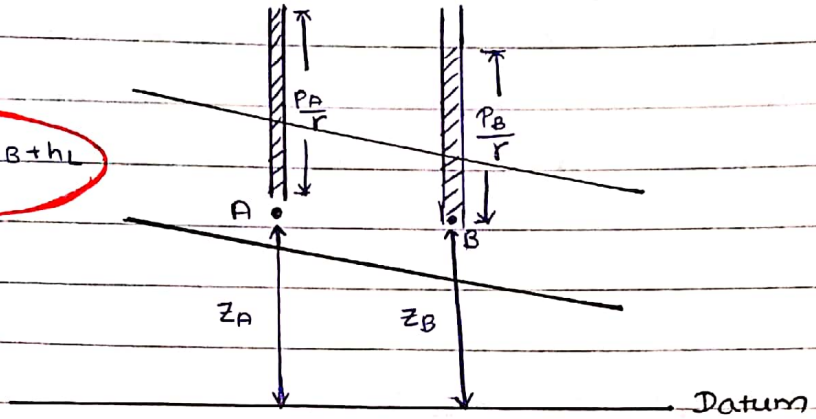
corresponding to the Piezometric Head only.

$$\frac{P_A}{\gamma} + z_A = \frac{P_B}{\gamma} + z_B + h_L$$



- Difference b/w 2 points in a flow is the head loss b/w those two points.
- Difference in the level of liquids in two piezometers inserted at 2 points in flow is the head loss b/w those two points
- The actual rise in the piezometer is equal to pressure head.

Darcy's law is valid when -

- The flow through soil is laminar, soil is saturated, flow is steady and flow is irrotational.
- Flow through soil finer than coarse soil is generally laminar in field. However, flow through gravel and rockfills generally becomes turbulent and Darcy's law is not applicable to it.
- If the volume and shape of water passage varies with time due to blockage of pores and change

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Of fabric elements etc., Darcy's law will become Invalid.

Coefficient of Permeability / Hydraulic Conductivity

⊙ It is defined as the superficial velocity of flow which will occur under unit hydraulic gradient.

⊙ Unit of coefficient of Permeability is that of velocity.

Soil Type	K (m/s)	Property
Gravel	≥ 1	Pervious
Sand	$1 \rightarrow 10^{-3}$	Pervious
Silt	$10^{-3} \rightarrow 10^{-6}$	Slightly Pervious
Clay	$< 10^{-6}$	Impervious.

$\left(\frac{K}{n}\right)$ → Coefficient of Percolation

n → Porosity.

Factors Affecting Permeability

$$K = C_s \frac{\gamma_w}{\mu} \cdot \frac{1}{S_o^2} \cdot \frac{e^3}{1+e} \cdot S^3 \qquad C_s = \frac{1}{K_0 T^2}$$

$$K = C_d e^2 \frac{e^3}{1+e} S^3 \frac{\gamma_w}{\mu}$$

(C_d) → Shape factor

(μ) → Dynamic Viscosity

(γ_w) → Unit weight of fluid

(S_o) → wetted surface Area/ Unit volume of particles and it depends on particle size and soil fabric.

S_t is a measure of pore size.

(T) → Tortousity factor

(K_0) → Pore shape factor ,

(S) → Degree of saturation

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Note: ⊙ Smaller is the particle, more tortous is pore and Greater is the tourtousity factor.

de → Effective Grain size.

$K \propto s^3$ | $K \propto \frac{e^3}{1+e}$ | $K \propto \frac{\gamma_w}{\mu}$ | $K \propto de^2$ | $K \propto \frac{1}{S_o^2}$

$$K = k_i \left(\frac{\gamma_w}{\mu}\right)$$

k_i → Intrinsic Permeability

unit → m^2 or Darcy

1 Darcy = $9.87 \times 10^{-13}\ m^2$

① Particle size:

Allen - Hazen formula for Sand

$$K\left(\frac{cm}{s}\right) = 100\, D_{10}^2\ (cm)$$

⊙ Permeability of Granular Soil depends mainly on c/s Area of pore channels and Smaller particle size actually controls the size of pore channel.

② Void - Ratio:

$$K \propto C \cdot \frac{e^3}{1+e}$$

C → shape factor

⊙ In case of sand, shape factor changes only slightly with void ratio. Hence,

$$\frac{K_1}{K_2} = \frac{e_1^3/1+e_1}{e_2^3/1+e_2}$$

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Note: $\frac{K_1}{K_2} = \frac{e_1^2}{e_2^2}$ for sand

$\frac{\log K_1}{\log K_2} = \frac{e_1}{e_2}$ for silt and clay

⊙ Although clays have Greater void ratio but still have smaller permeability and this is because non-uniform pore size and fabric majorly affecting the permeability.

③ Permeant:

$$\frac{K_1}{K_2} = \frac{(\gamma_w/\mu)_1}{(\gamma_w/\mu)_2}$$

With change in temperature, the unit weight of a fluid and μ both will change but change in γ is insignificant. Hence,

$$\frac{K_1}{K_2} = \frac{\mu_2}{\mu_1}$$

⊙ As per IS-2720, permeability of soil is expressed at 27°C as:

$$K_T \mu_T = K_{27°C}\, \mu_{27°C}$$

Note: ⊙ $\left(\frac{\gamma_w}{\mu}\right)$ is inadequate to account for the pore fluid differences in case of clayey soil because conch of cations, type of cation and dielectric constant of flowing fluid will affect the soil fabric and hence the permeability.

④ Shape of Particle:

$K \propto \frac{1}{S_A^2}$ $S_A \rightarrow$ Specific Surface Area.

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⊙ for soil with same void ratio, soil with angular particles are less permeable than those for rounded particles because angular particles having greater specific surface area.

⊙ In natural deposits, however, angular particles will lead to greater void ratio and hence will have greater permeability.

⑤ Hydraulic Conductivity of Unsaturated Soil :

$K \propto s^3$

Hydraulic Conductivity ↓ with matrix Suction ↑

Hydraulic Conductivity ↑ , w ↑

Note: Matrix Suction, w are related to each other. Due to suction, tension applied by soil particle on pore water is greater if the suction is more. Suction is characterised as Matrix Suction and Osmotic Suction.

⇒ Matrix Suction is due to Capillary Effect. Hence, depends on water content,

⇒ Osmotic Suction is due to difference in conc^n of cations near the particle surface and in the pore fluid. Hence, not affected by w.

⑥ Effect of Entrapped Air (Occluded Air Bubble) :

i.e; Non-continuous pore channel.

⊙ Occluded Air Bubble in o/w Saturated Soil blocks

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the flow channel and decreases the permeability.

- Bubble size ↓ with increase in water pressure and hence permeability ↑ with water pressure.

⑦ Effect of Impurities in Water :

- K ↓ if flow passes is plugged due to foreign matter.
- Organic matter in the soil are washed into the voids and hence blocks the voids and consequently K ↓
- In well compacted soil, slacking (घुल जाना) and dispersion of clay particles will cause blocking of pores and K ↓ with time and in poorly compacted soil, these particles can be carried away by percolating water and hence K ↑ with time.

⑧ Effect of adsorbed cation on clay Mineral surface:

Adsorbed cations on clay mineral surface can change the soil fabric, two clays at same void ratio, one having disperse structure and other having floculated structure will have different permeability.

- Flocculated structure leads to greater K and disperse structure leads to reduction in K due to reduction in size of voids.

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⊙ At same void ratio $K_{Kaolinite} > K_{Illite} > K_{Montmorillonite}$

Montmorillonite :

$$K^+ < Na^+ < H^+ < Ca^{2+}$$

Kaolinite :

$$Na^+ < K^+ < Ca^{2+} < H^+$$

Note: ⊙ Na-clays have smaller K. Hence, used in clay of Earthern Dam.

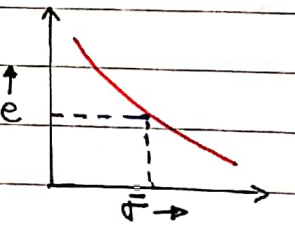
⑨ Effect of Effective stress:

Increase in Effective stress leads to decrease in void ratio. Hence (K↓)

⑩ Effect of Activity of clay:

⊙ for the same void ratio,

as Ac↓, ⇒ K↑



⑪ Effect of Anisotropy:

for Varved clay $(K_h > K_v)$

⊙ Preferred orientation of clay particles also effects K and $(K_H > K_v)$ if orientation is in horizontal plane.

⊙ Marine clay has random flocculated fabric.

Hence, $\left(\frac{K_H}{K_v} = 1\right)$

⑫ Effect of Compaction w for fine Grained Soil:

w < OMC ⇒ (K↑)

w > OMC ⇒ (K↓)

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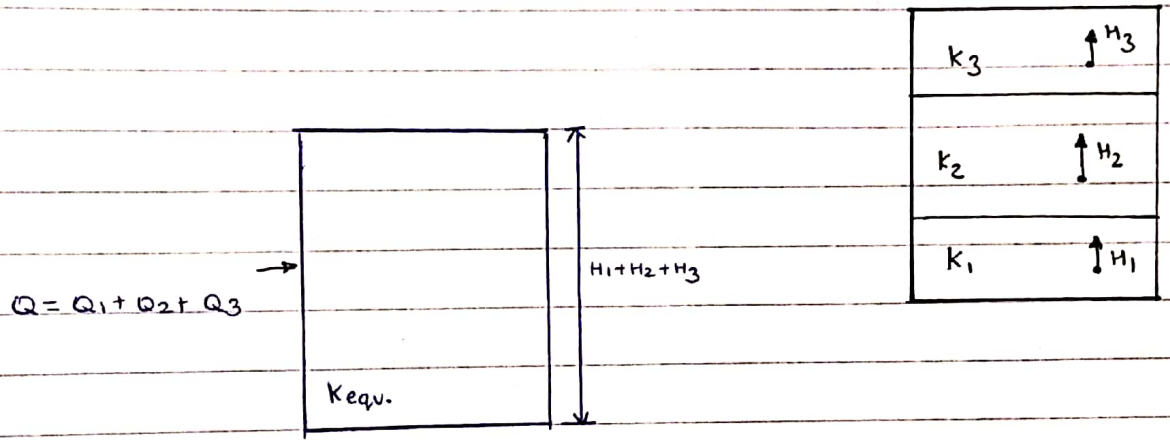
(13) **Effect of Remoulding:** Breakdown of flocculated open fabric and destruction of large pores on remoulding leads to reduction in K.

Permeability of Stratified Soil:

(1) Flow Parallel to Bedding Plane:

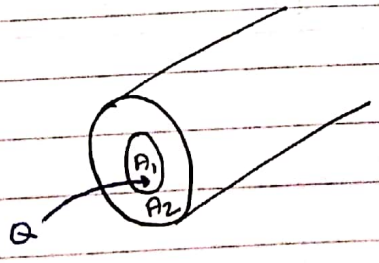


$h_{L_1} = h_{L_2} = h_{L_3} = h_L$

$$Q = K_{equ.} \frac{h_L}{L} (H_1 + H_2 + H_3) \cdot 1 = K_1 \frac{h_L}{L} \cdot H_1 + K_2 \frac{h_L}{L} \cdot H_2 + K_3 \frac{h_L}{L} \cdot H_3$$

$$K_{equ.} = \frac{K_1 H_1 + K_2 H_2 + K_3 H_3}{H_1 + H_2 + H_3}$$

Note:



$$K_{equ.} = \frac{K_1 A_1 + K_2 A_2 + \cdots}{A_1 + A_2 + \cdots}$$

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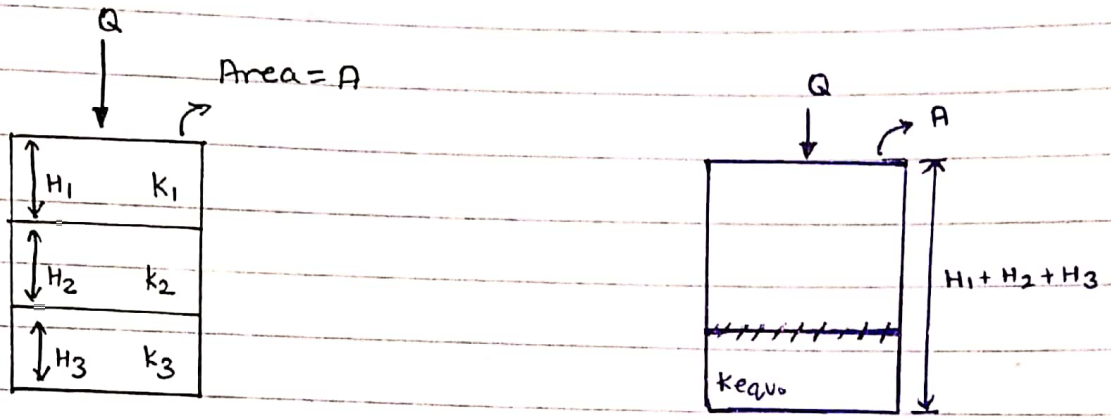
② Flow Perpendicular to the Bedding Plane:



$$Q_1 = Q_2 = Q_3 = Q$$

$$h_L = h_{L1} + h_{L2} + h_{L3}$$

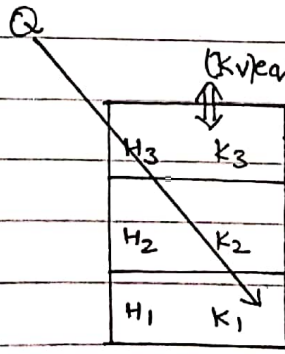
$$Q = K_1 \frac{h_{L1}}{H_1} \cdot A = K_2 \frac{h_{L2}}{H_2} \cdot A = \frac{K_3 h_{L3}}{H_3} \cdot A = \frac{h_L K_{eq.} A}{H_1 + H_2 + H_3}$$

$$h_L = \frac{Q(H_1 + H_2 + H_3)}{K_{eqv.} A} = \frac{Q H_1}{K_1 A} + \frac{Q H_2}{k_2 A} + \frac{Q H_3}{k_3 A}$$

***

$$K_{eqv.} = \frac{H_1 + H_2 + H_3}{\frac{H_1}{K_1} + \frac{H_2}{K_2} + \frac{H_3}{K_3}}$$

Note:



$$K_{eqv.} = \sqrt{(K_H)_{eqv.} \times (K_V)_{eqv.}}$$

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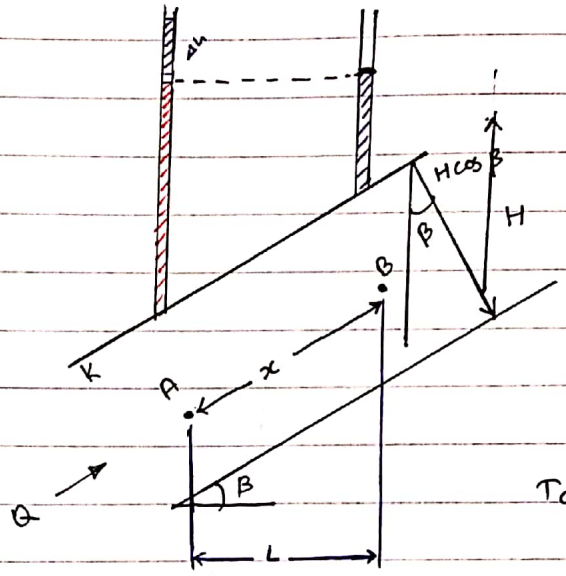
Ques →



Q = ?

Take Inside width = 1

$Q = KiA$

$i = \frac{\Delta h}{L/\cos\beta} = \frac{\Delta h \cos\beta}{L} = \frac{\Delta h}{x}$

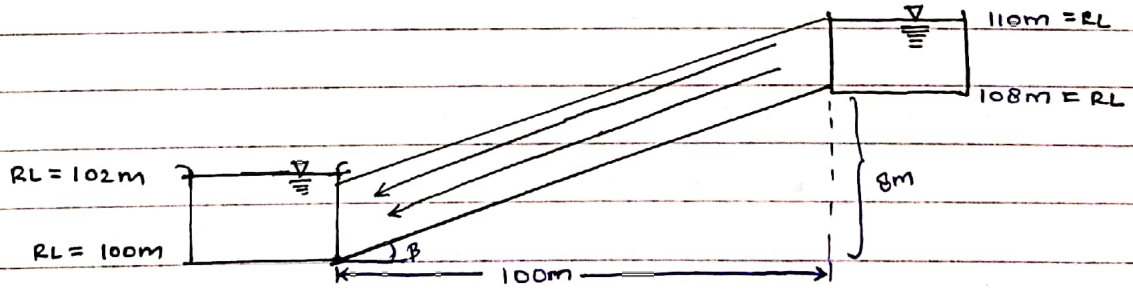
$Q = K \frac{\Delta h}{L} \cos\beta \, H\cos\beta \cdot 1$

$$\boxed{Q = K \frac{\Delta h}{L} H\cos^2\beta}$$

Ques →

$K_H = 10^{-5}$ cm/s

$K_V = 2\times10^{-9}$ cm/s



Find flow rate into ditch for 1m length of ditch.

$\cos\beta = \frac{100}{\sqrt{100^2 + 8^2}} = 0.996$

$K_{equ.} = \sqrt{2\times10^{-9}} = 4.48\times10^{-5}$ cm/s

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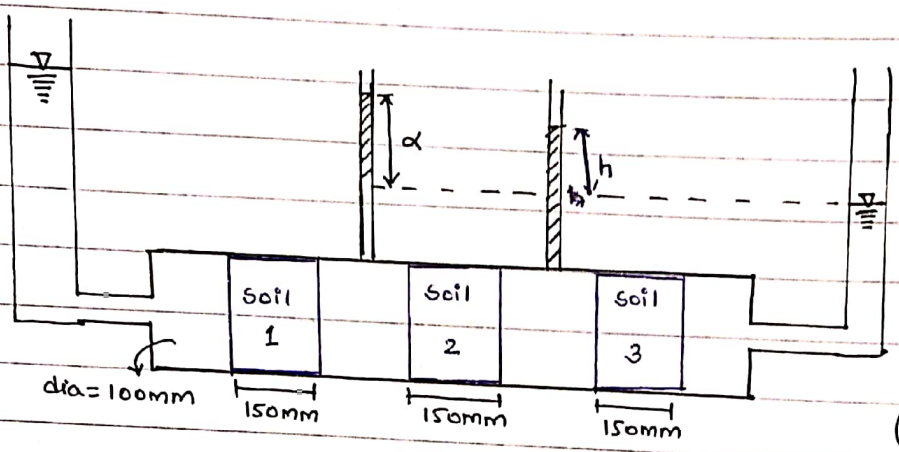
$Q = kiH \times l$

$= 4.48 \times 10^{-5} \times 8 \times \cos\beta \times 2\cos\beta = 0.0712\ cm^3/s$

$= 7.12\ m^3/s$

Ques →



$k_1 = 0.01\ cm/s \quad k_2 = 0.003\ cm/s \quad k_3 = 0.03\ cm/s$

Solun: $Q = \frac{k_1 h_{L1}}{L} \times A + \frac{k_2 h_{L2}}{L} \cdot A + \frac{k_3 h_{L3}}{L} \cdot A$

$L = 0.15m$

$A = \frac{\pi}{4}(0.1)^2$

$k_1(0.56-\alpha) = k_2(\alpha-h) = k_3 h$

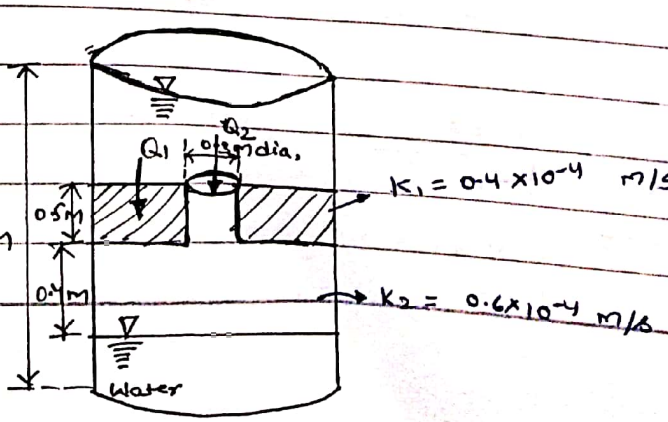
$1 \times (0.56-\alpha) = 0.3(\alpha-h) = 3h$

$0.56-\alpha = 3h$

$0.3\alpha - 0.3h = 3h$. $\alpha = 11h$

$h = 40mm$

Ques →



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Solun: Head causing flow = 2m

2m Head is lost while flow through the soil :

$$Q = Q_1 + Q_2$$

$$Q_2 = k_2 \times \frac{2}{0.9} \times \pi \frac{(0.3)^2}{4}$$

$$Q_1 = K_1 \cdot \frac{h_{L1}}{0.5} \left[(0.8)^2 - (0.3)^2\right] = \frac{K_2 (2-h_{L1})}{0.4} \times \frac{\pi}{4} (0.8^2 - 0.3^2)$$

$$\frac{K_1 h_{L1}}{0.5} = \frac{K_2 (2-h_{L1})}{0.4}$$

$$\frac{0.4 \times h_{L1}}{0.5} = \frac{0.6 (2-h_{L1})}{0.4}$$

$$0.92\, h_{L1} = 1.2 \qquad h_{L1} = \frac{1.2}{0.92}$$

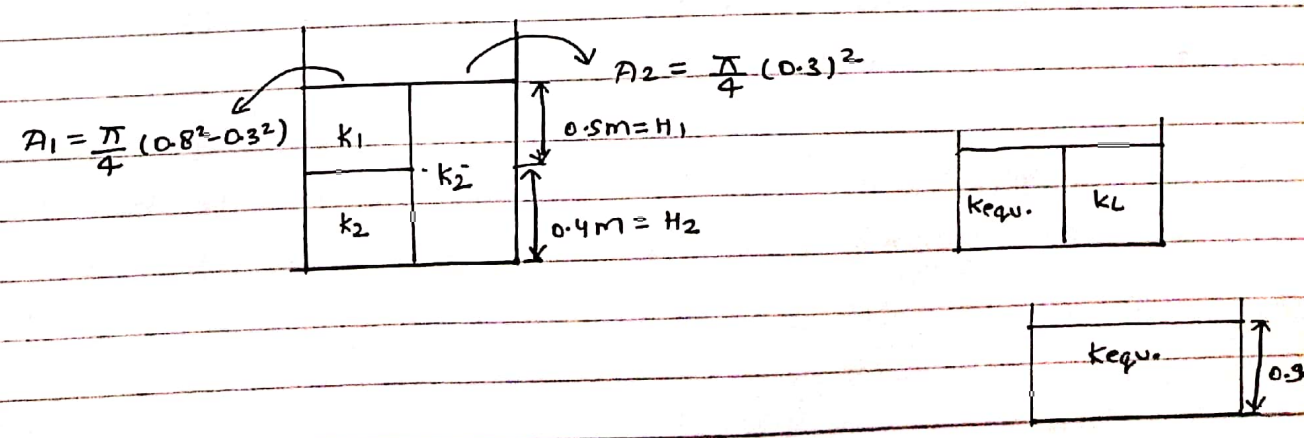
$$Q_1 = \frac{K_1 h_{L1}}{0.5} \cdot \frac{\pi}{4} \left[(0.8)^2 - (0.3)^2\right]$$

$$Q_1 = 0.4 \times 10^{-4} \times \frac{1.2}{0.92 \times 0.5} \times \frac{\pi}{4} \left((0.8)^2 - (0.3)^2\right) = 4.507 \times 10^{-5} \ m^3/sec.$$

$$Q_2 = 0.6 \times 10^{-4} \times \frac{2}{0.9} \times \frac{\pi}{4} \times (0.3)^2 = 9.424 \times 10^{-6} \ m^3/sec.$$

$$Q = Q_1 + Q_2 = 5.449 \ m^3/sec.$$

Alternate:



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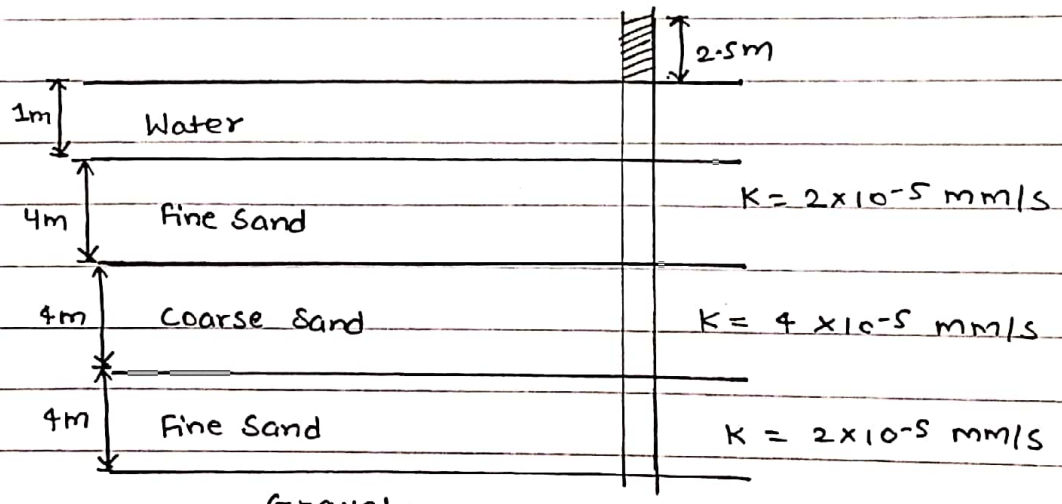
$$K_{equ.} = \frac{H_1 + H_2}{\frac{H_1}{K_1} + \frac{H_2}{K_2}} = \frac{0.9}{\frac{0.5}{0.4\times10^{-4}} + \frac{0.4}{0.6\times10^{-4}}} = 0.469\times10^{-4} \text{ m/s.}$$

$$K_{equ.} = \frac{K_{eq.} A_1 + K_2 A_2}{A_1 + A_2} = \frac{0.469\times10^{-4} \times \frac{\pi}{4}(0.8^2 - 0.3^2) + 0.6\times10^{-4} \times \pi/4 (0.3)^2}{\frac{\pi}{4}(0.8)^2}$$

$$Q = K_{eq.} \times \frac{2}{0.9} = 5.45\times10^{-5} \text{ m}^3/\text{sec.}$$

Ques →



Under Artesian Pressure.

Determine the quantity of flow per unit area and excess hydraulic head at the junction of each soil layer.

Head causing flow in this case is 2.5 m (Diff. B/w u/s & d/s levels)

Total Head of 2.5m will be lost during flow from A → D Because losses do not occur in water, it occurs only in the resisting medium.

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Solun: Excess Hydraulic Head at B = 2.5 - h_{LAB}

Excess Hydraulic Head at C = 2.5 - h_{LAB} - h_{LBC}

Excess Hydraulic Head at D = 0

$$Q = K_{equ.} \times \frac{2.5}{12} \times A$$

$$K_{equ.} = \frac{H_{AB} + H_{BC} + H_{CD}}{\frac{H_{AB}}{K_{AB}} + \frac{H_{BC}}{K_{BC}} + \frac{H_{CD}}{K_{CD}}}$$

$$K_{eq.} = \frac{12}{\frac{4}{2\times10^{-5}} + \frac{4}{2\times10^{-5}} + \frac{4}{2\times10^{-5}}} = \frac{12\times10^{-5}}{5} = 2.4\times10^{-5} \text{ mm/sec.}$$

$$Q = 2.4 \times 10^{-8} \times \frac{2.5}{12} \ m^3/s/m^2 = 5\times10^{-4} \ m^3/s/m$$

$$Q = K_{AB} \cdot \frac{h_{LAB}}{4} \cdot A = 5\times10^{-9} \times A$$

$$h_{LAB} = \frac{5\times10^{-9}\times4}{2\times10^{-8}} \ m = 1m$$

$$h_{LBC} = \frac{5\times10^{-9}\times4}{4\times10^{-8}} = 0.5m$$

$$h_{LCD} = 1m$$

Excess Hydraulic Head:

at B = 2.5-1 = 1.5 m

at C = 1.5 - 0.5 = 1m

at D = 0m

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Determination of Coefficient of Permeability

Permeability can be determined using:

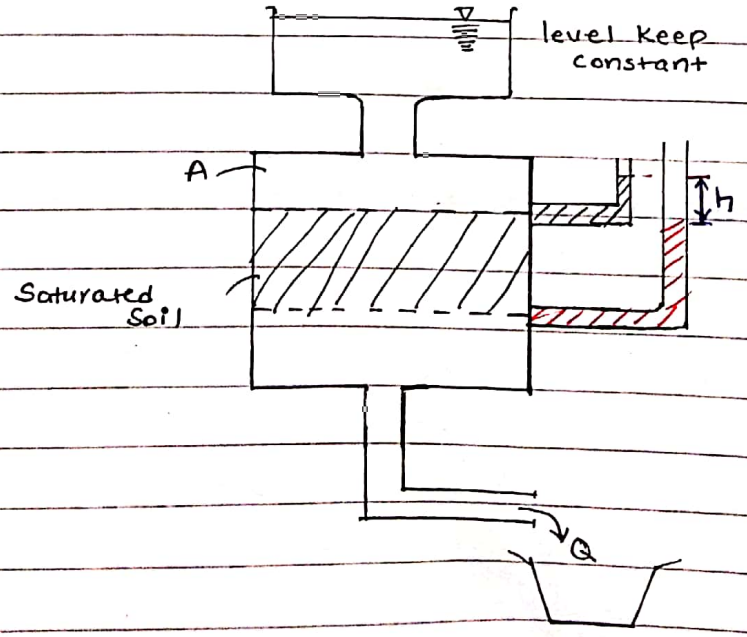
① Laboratory Method
- (A) Constant Head Permeability Test
- (B) Variable Head Permeability Test

② Field Method
- (A) Pumping out Test
- (B) Pumping in Test

③ Indirect Method
- Using various Soil Parameters.

① Constant Head Permeability Test

(Used for coarse Grained soil)



$$Q = K \cdot \frac{h}{L} \cdot A$$

find K ←

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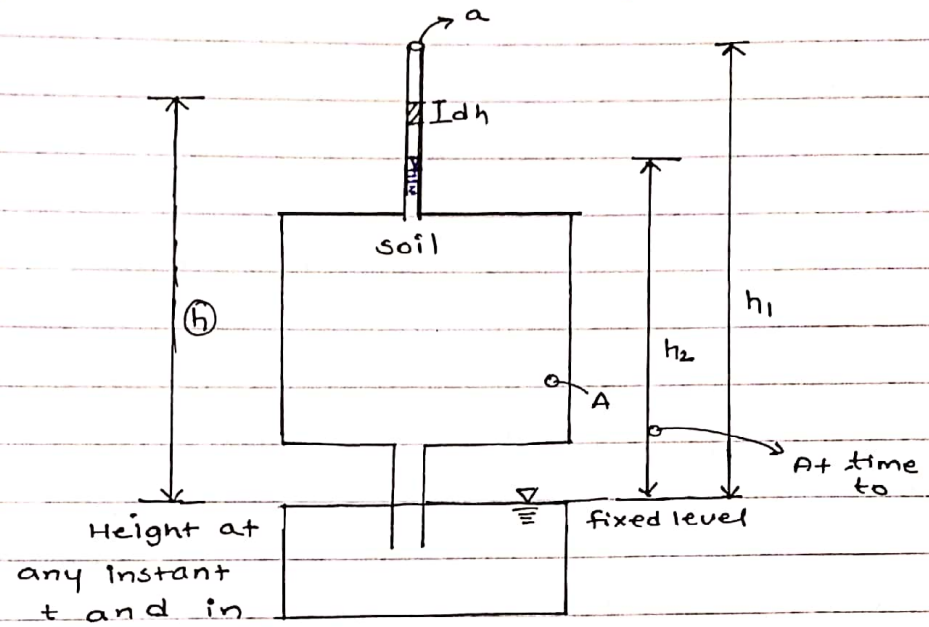
② Variable Head Permeability Test:

(Used for fine Grained soil)



$$a(-dh) = Q \cdot dt = K \cdot \frac{h}{L} \cdot A dt$$

$$\int_{h_1}^{h_2} \frac{-dh}{h} = \frac{KA}{La} \int_0^{t_0} dt$$

$$K = \frac{L \cdot a \ln(h_1/h_2)}{A t_0}$$

Field Methods:

Since, sampling of soil leads to disturbance. Hence, more reliance is placed on the in-situ testing pumping out test influences the large area around the pumping well and gives an overall value of the coefficient of permeability of soil deposits.

However, pumping in test influences a small area around the hole and therefore gives value of coefficient of permeability of soil surrounding

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the hole.

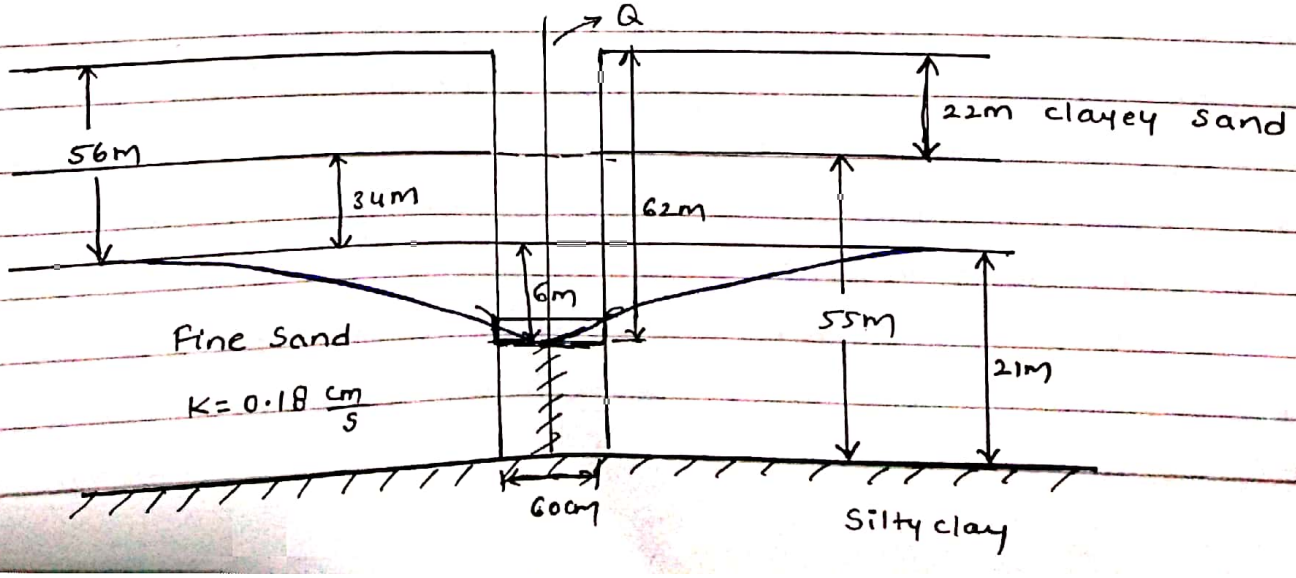
Note: ⊙ Capillary permeability test is used for determining the coefficient of permeability of unsaturated soil.

⊙ We have already studied in Hydrology in ground water.

⊙ Value of k can be determined from pumping out test of unconfined and confined aquifer as discussed in Hydrology.

Note: ⊙ During the testing done in Lab, water used must be de-aired tap water because the Air Bubbles may obstruct the flow.

Ques→ A municiple water supply well is to be installed in the Aquifer as shown in the figure Below- The well has 20cm dia. casing and 60cm dia. boring with a submergible pump that draws the water level down to a depth of 62M below the ground level. Compute the maxm flow rate that can be produced by the well.



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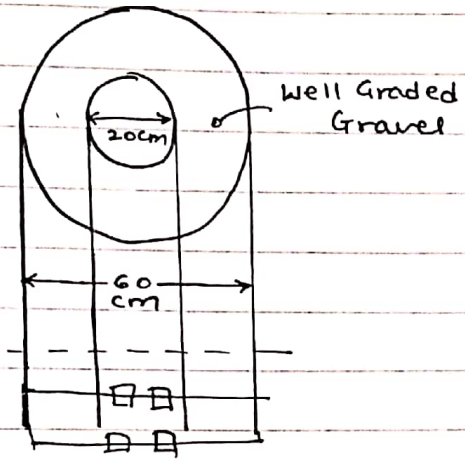

Soln: Unconfined Aquifer:

$$Q = \frac{K\pi(h_2^2 - h_1^2)}{\ln(r_2/r_1)} = \frac{K\pi(H^2 - h^2)}{\ln R/r_w}$$

$r_w \rightarrow 0.3m$

$R = 3000\, s_w \sqrt{K} \rightarrow (m/s)$

{Drawdown Intensity}

$$R = 3000 \times 6 \times \sqrt{0.18 \times 10^{-2}}$$

$$\Rightarrow R = 763.67\,m$$

$$Q = \frac{0.18 \times 10^{-2} \times 3.14 \times (21^2 - 15^2)}{\ln\left(\frac{763.67}{0.3}\right)} = 0.1556\ m^3/sec.$$

Ques → Pumping is done from a confined aquifer using a fully penetrating well of dia. 0.4m at a constant discharge of 500 m^3/hr. The confined aquifer has a thickness of 30m and porosity of 0.4, Calculate the time required for a contaminent to reach the well if introduced to the aquifer 200m from the well.

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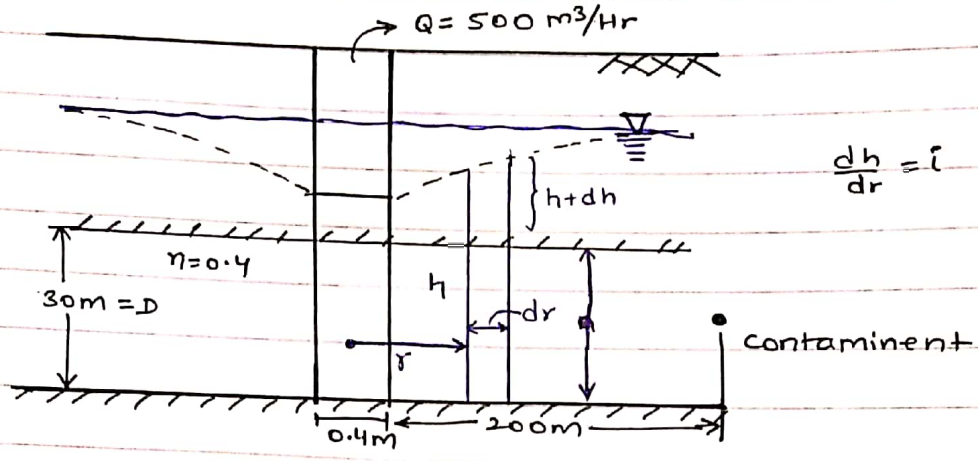
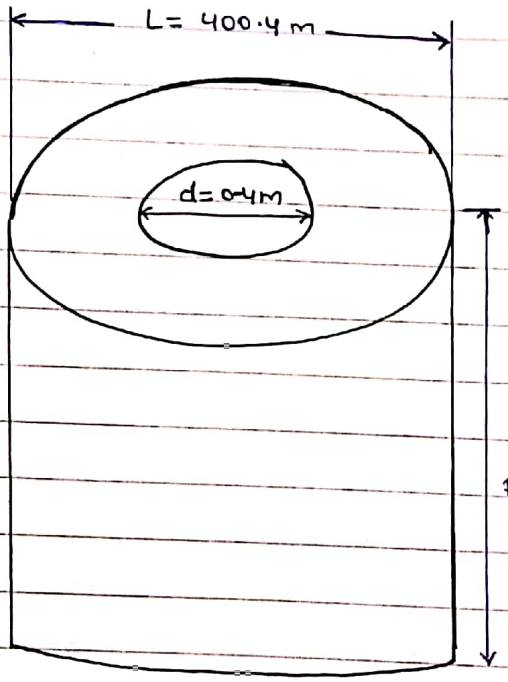


$\frac{dh}{dr} = i$



$$\left[\frac{\pi(L^2-d^2)}{4} \times D\right] \times n = Q \times t$$

$t \rightarrow$ Time Required for impurities to come into the well.

$$-\frac{dx}{dt} = \frac{Q}{\frac{2\pi x D}{n}}$$

$$-\int_{200.2}^{0.2} x\,dx = \int_{0}^{t} \frac{Q}{2\pi n D}\,dt$$

$$\frac{(200.2)^2-(0.2)^2}{2} = \frac{Q}{2\pi n D} \cdot t$$

$$t = \frac{\pi n D}{Q}\left[(200.2)^2-(0.2)^2\right] = 125.91 \text{ days.}$$

(0.4, 30 m, 500 m³/hr)

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EFFECTIVE STRESS & CAPILLARITY

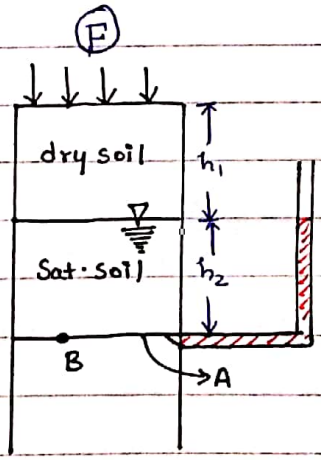
⊙ Soil particles interact physically and chemically like in case of clays, the interaction b/w the particles through double layer is called chemical interaction, but in coarse grained soil, there would not be any chemical interaction b/w particles.

⊙ Due to physical interaction b/w particles, contact force b/w particles change and effective stress concept is used to study the physical interaction b/w soil particles.

⊙ Effective stress concept was proposed by Terzaghi and applies to a fully saturated soil and it relates total stress and pore water pressure.

(Eff. stress) $\bar{\sigma} = \sigma - u$ (Pore water Pressure)

(Total stress)



$$\sigma_B = \frac{F + h_1 A \gamma_d + h_2 A \gamma_{sat.}}{\text{Area}}$$

$$\sigma_B = \frac{F}{A} + h_1 \gamma_d + h_2 \gamma_{sat.}$$

$u = h_2 \gamma_w$

⊙ Total stress on a plane within a soil mass is the force/unit area transmitted in normal direcn on a plane across the soil mass.

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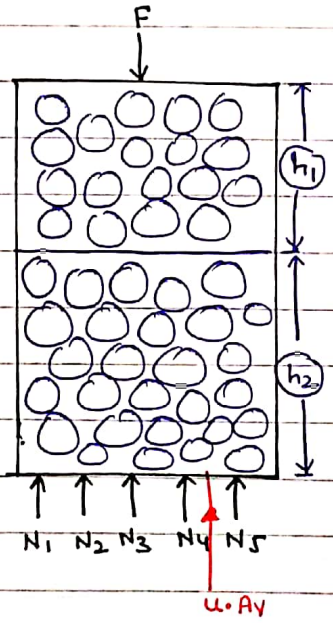
In field, If loaded area >>> thickness of soil
↳ (laterally confinement)

⊙ Pore water Pressure is the pressure of fluid in the pores of soil mass.

⊙ Pore water Pressure can be measured using any pressure management device like piezometers, U-tube Manometers ---- etc.

⊙ Pore water Pressure in stationary fluid is also called Neutral stress because it acts on all sides of particle and doesnot cause particles to press against each other.



$A_V = 0.97A \simeq A$

$$F + \gamma_d h_1 A + \gamma_{sat} \cdot h_2 A = \Sigma N + u \cdot A$$

$$F + \gamma_d h_1 A + \gamma_{sat} \cdot h_2 A = \Sigma N + h_2 \gamma_w \cdot A$$

$$\frac{\Sigma N}{A} = \left[\frac{F}{A} + \gamma_d h_1 + \gamma_{sat} \cdot h_2\right] - (\gamma_w h_2)$$

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

⊙ Effective stress doesnot represent the contact stress because we are not dividing ΣN with the area of contact blw grains rather we are dividing it by total area. Hence, Eff stress doesnot have a physical meaning, it is just a term defined as:

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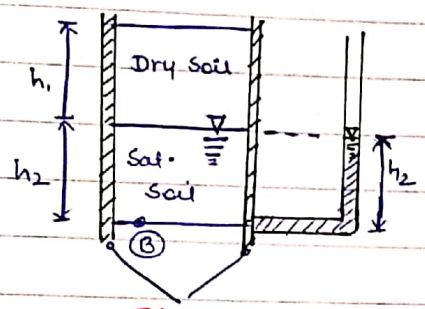
$\sigma - u = \bar{\sigma}$

22/10/19

$$\bar{\sigma} = \frac{F}{A} + \gamma_d h_1 + \gamma_{sub} \cdot h_2$$

Effect of External load on Pore water Pressure

Before Application of Load:



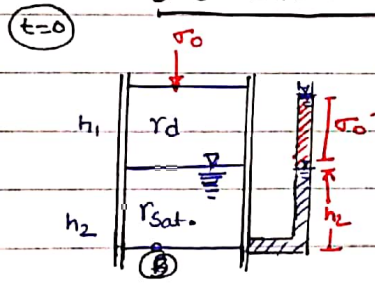
Rigid wall
Laterally Restrained

$P_2 = \gamma_w h_2 = u_B$

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

$$= \gamma_d h_1 + \gamma_{sub} \cdot h_2$$

Just after application of load:



Excess Pore water pressure created at $t=0$

$$\sigma_B = \sigma_0 + \gamma_d h_1 + \gamma_{sat} \cdot h_2$$

If:

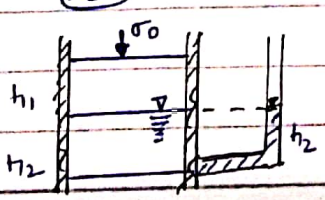
① Soil particle & water are incompressible.
② Soil is laterally restrained.
③ Soil is Saturated.

Now;

$$\bar{\sigma}_B = \sigma_B - u_B = (\sigma_0 + \gamma_d h_1 + \gamma_{sat} \cdot h_2) - (\sigma_0 + \gamma_w h_2)$$

$$= \gamma_d h_1 + \gamma_{sub} \cdot h_2$$

Long time after load Application:



$$\bar{\sigma}_B = \sigma_B - u_B$$

$$= [\sigma_0 + \gamma_d h_1 + \gamma_{sat} \cdot h_2] - h_2 \gamma_w$$

$$\bar{\sigma}_B = \sigma_0 + \gamma_d h_1 + \gamma_{sub} \cdot h_2$$

Load area is large & depth of soil is less.

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⊙ Immediately after load application (i.e; at $t=0$), the soil particles try to occupy a position closer together but as the soil particles and water are incompressible, soil is laterally confined and soil is saturated, rearrangement of soil particles is not possible without expulsion of pore water.

Thus, as the particle rearrangement is resisted by pore water, excess pore water pressure will be created (i.e; over and above the hydrostatic pore water pressure)

Hence, at $t=0$, $u = \sigma_0 + r_w h_2$

$$\sigma_B = \sigma_0 + r_d h_1 + r_{sat} \cdot h_2$$

$$\boxed{\bar{\sigma}_B = r_d h_1 + r_{sub} \cdot h_2}$$

($\bar{\sigma}$ is same as before load application)

Thus, at $t=0$ effective stress does not change and thus, grain to grain contact force doesnot change. i.e; load is not transferred to the soil grains.

Long time after load Application

⊙ As time passes, pore water pressure dissipates and gradually grain contact force increases and after long time when excess pore water pressure has dissipated out, pore pressure reduces to Hydrostatic pore pressures and effective stress becomes: $\bar{\sigma} = \sigma - u$

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

$$= (\sigma_0 + r_d h_1 + r_{sat} h_2) - (h_2 r_w)$$

Hence; $\boxed{\bar{\sigma} = (\sigma_0 + r_d h_1 + r_{sub} \cdot h_2)}$

Thus, in long term, $\bar{\sigma}$ increases by amount of externally applied stress.

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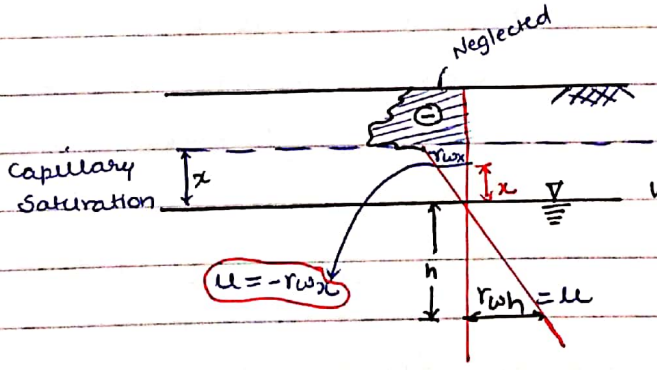
i.e; externally applied stress gets transferred to the soil grains.

Note: ⊙ In the field, lateral confinement is assumed if the loaded area is large as compared to the thickness of the soil. (as in case of uniform fill condition).

⊙ In this case, at every depth, the stress increment will be taken to be constant.

⊙ When the soil is subjected to reduction in total stress, the excess pore pressure develop will be (-ve) and in long term, water will be sucked in & the volume will increase and gradually excess (-ve) pore pressure will dissipate and swelling will occur.

Effect of Capillarity in the Soil



⊙ Due to capillary effect, pore water pressure becomes (-ve) and hence effective stress becomes increases.

⊙ certain depth above W.T., is completely saturated with capillary water and it is called capillary

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saturation zone and in this zone, the pore pressure is taken to vary linearly.

⊙ Depth of capillary saturation zone (α) for sand and silt is given by:

$$\alpha = \frac{0.03}{0.2 D_{10}}$$ (m) → for Sand and Silt

(D_{10} in mm)

Other Emperical formula:

for capillary rise:

$$\alpha = \frac{C}{e D_{10}}$$ (cm)

(D_{10} in cm)

Emp constant having value (0.1 → 0.5 cm²)

⊙ Two soils having same D_{10} value can have diff capillary rise depending upon soil structure and Geological History.

⊙ Due to increase in Effective stress on account of capillary action, Shear strength of the soil increases.

Bulking of sand is also due to capillary Effect.

Effect of Seepage on Effective Stress

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- Flow lines are drawn in such a way that b/w two successive flow lines, discharge (Q) is constant.

- Equipotential lines are locus of points having same total head,
  i.e; piezometers inserted at any two points on the same equipotential line will rise to the same level.

- Equipotential lines are drawn in such a way that b/w two successive equipotential lines, losses are same,

- No. of Equipotential drop (Nd) = No. of Equipotential lines - 1.
- Losses b/w two successive Equipotential lines = $\frac{H}{Nd}$

H → Head causing flow
↙
Diff. b/w u/s and D/s water level.

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⊙ Difference in the level of liquid in the two piezometers inserted at two points = Head loss b/w those two points.

⊙ Actual rise in piezometer = Pressure Head. Each flow field is an elimentary square.

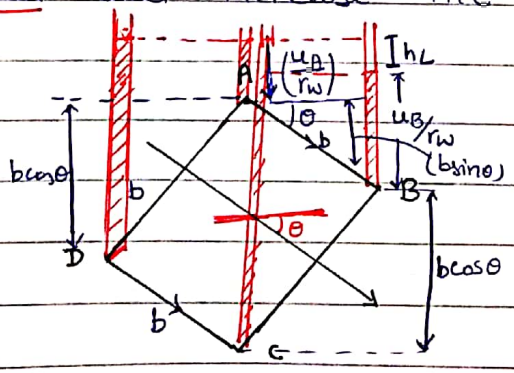
↓

Avg. dist b/w two flow lines and Avg. dist. b/w two equipotential lines are same.

⊙ When water is seeping through the soil, total head is dissipated as viscous friction producing a frictional drag in the direction of flow on the soil particles.

This drag force results in seepage force. on the soil mass which acts in the direc" of flow.

⊙ Component of this seepage force acting vertically upward will reduce the effective vertical stress and component of this seepage force acting vertically downward will increase the effective vertical stress.



AD, BC → Equipotential lines

AB, CD → Flow/stream lines.

$$\frac{u_B}{\gamma_w} = \frac{u_A}{\gamma_w} + b\sin\theta - h_L$$

$$\frac{u_C}{\gamma_w} = \frac{u_A}{\gamma_w} + b\sin\theta + b\cos\theta - h_L$$

$$\frac{u_D}{\gamma_w} = \frac{u_A}{\gamma_w} + b\cos\theta$$

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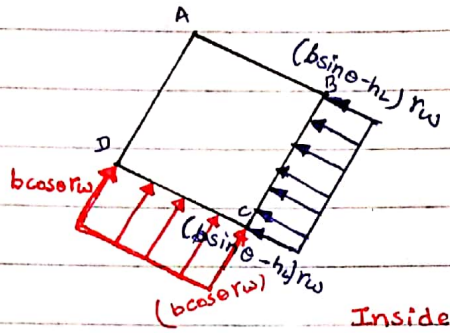

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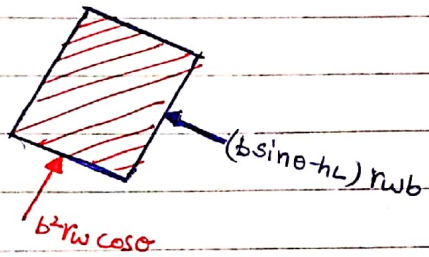
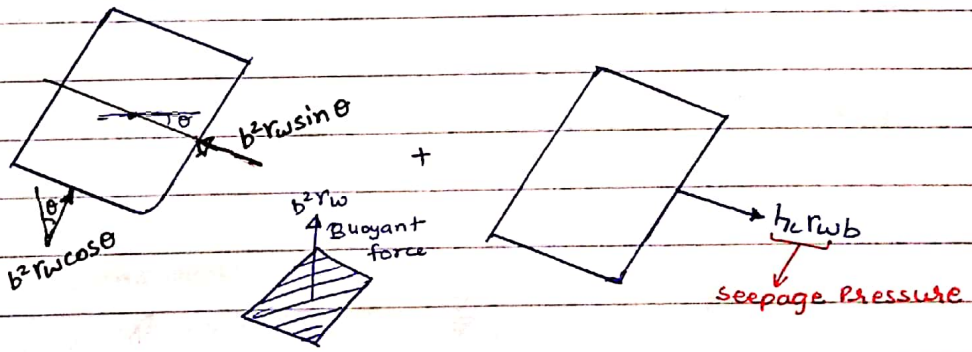


Surface force on the Element.



$h_L\gamma_w$ → Seepage Pressure.

$b \times 1$ → Area.

$h_L\gamma_w b$ → Seepage force.

⊙ Under no flow situation, the surface force acting is only equal to Buoyant force acting in vertically upward direction and under flowing condⁿ, apart from the buoyant force, there will also be

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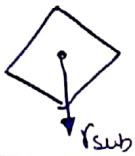


$r_{sub.} \times b^2 \times 1$ = Buoyant weight.

a seepage force acting in the flow direction and

Seepage force = Seepage Pressure × Area

$= (h_L r_w) \times (b \times 1)$

$= \boxed{h_L r_w b}$

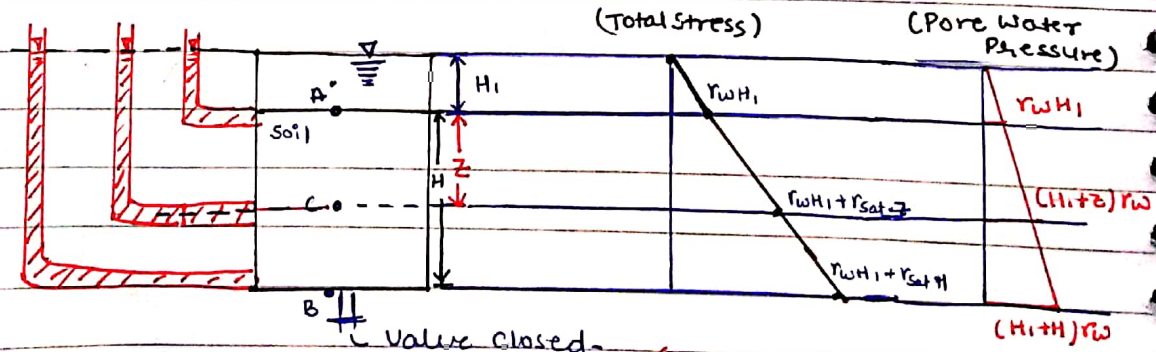
$\frac{\text{Seepage force}}{\text{Volume}} = \frac{h_L r_w \times b \times 1}{b \times b \times 1} = \frac{h_L}{a_b} \times r_w$

$= \boxed{i r_w}$

Note: ⊙ Resultant force on the soil mass is the vector summation of Buoyant weight and Seepage force.

⊙ The above resultant force will be balanced by contact forces and vertical component of the contact force will be equal to buoyant weight ± vertical component of seepage force.

No Flow condition:



(Not affected by datum location)

	Datum Head	Pressure Head	Total Head
A	H	H_1	$H+H_1$
B	0	$H+H_1$	$H+H_1$
C	$H-Z$	H_1+Z	$H+H_1$

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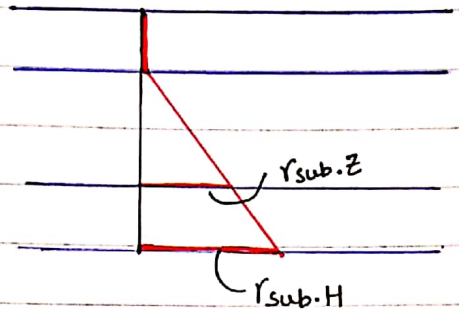
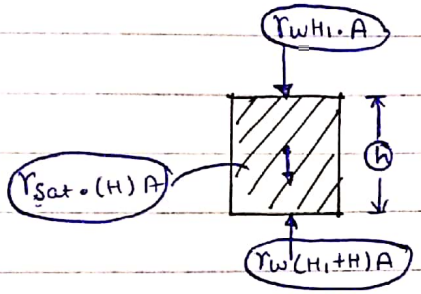
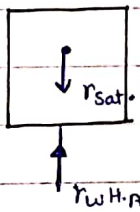
Buoyant weight ⇒ No Flow condⁿ mein
Effective stress ∄ Equal hoga.

(Effective stress)
$= (\sigma - u)$

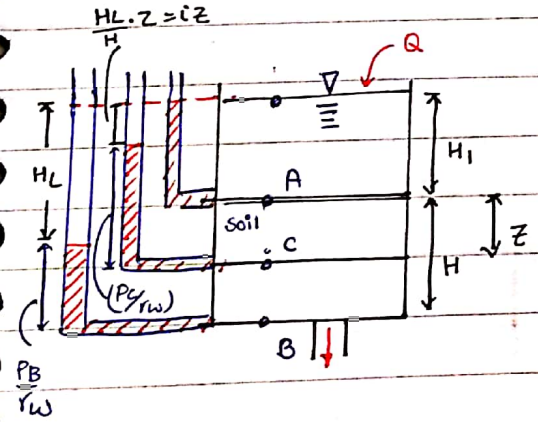




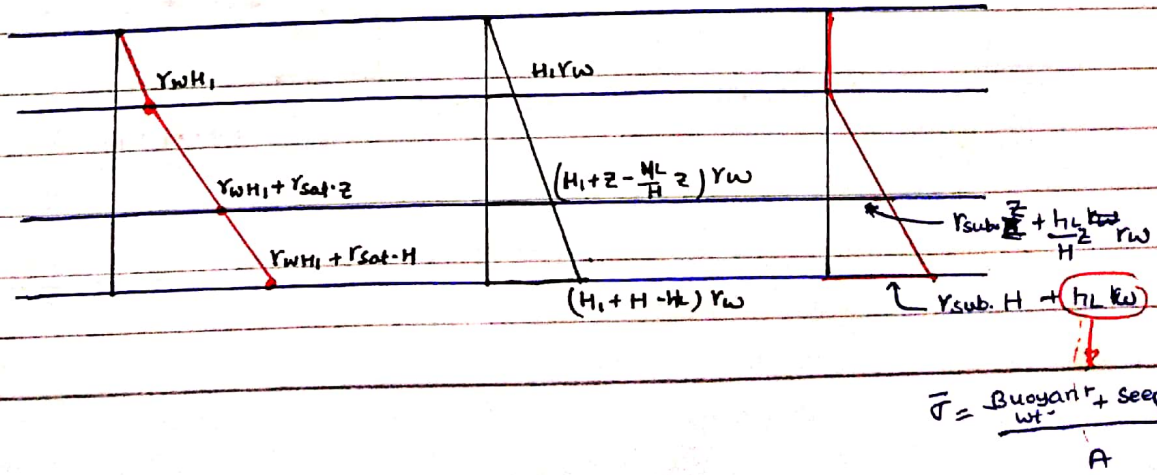


⇒ Buoyant ~~force~~ weight = $\gamma_{sub} \cdot H \cdot A$

Downward Flow Condition



	Datum Head	Pressure Head.	Total Head
A	H	H_1	H_1+H
B	0	H_1+H-H_L	H_1+H-H_L
C	$H-Z$	$H_1+H-\frac{H_L}{H}\cdot Z$	$H_1+H-\frac{H_L}{H}\cdot Z$



$\bar{\sigma} = \frac{\text{Buoyant wt.} + \text{Seepage force}}{A}$

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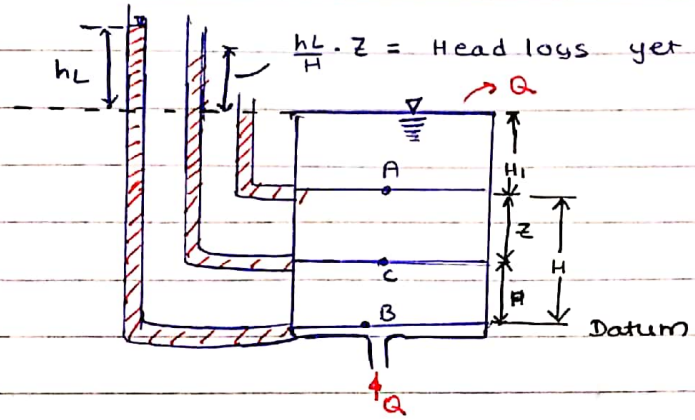
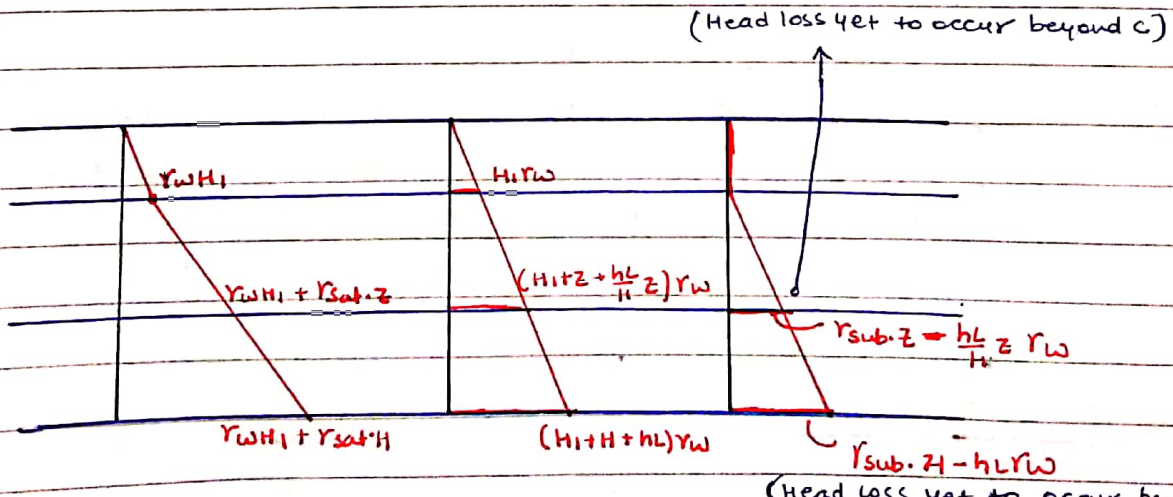
Note: ⊙ Buoyant weight = Effective stress under No flow situation.

Upward Flow Condition



	Datum Head	Pressure Head	Total Head
A	H	H_1	$H_1 + H$
B	0	$H_1 + H + h_L$	$H_1 + H + h_L$
C	H-z	$z + H_1 + \frac{h_L}{H} z$	$H_1 + H + \frac{h_L}{H} z$



effective stress = $\frac{\text{Buoyant Wt.} - \text{seepage force}}{A}$

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Note: ⊙ If we have a downward load acting at top then effective stress is written as:

↓ Flow

Eff. Stress = $\frac{P + \text{Buoyant wt} \pm \text{Seepage Force}}{A}$

↑ Flow

$\bar{\sigma}$ = { eff. stress under no flow condn } ± Seepage Pressure

↓

$h_L r_w$

h_L = loss already occured upto the point under consideration in downward flow.

h_L = loss yet to occur beyond the point under consideration in upward flow.

⊙ Effective vertical stress at any point for any direcn of flow can be simply calculated as total vertical stress - pore water pressure.

Quick Sand Condition

⊙ In case of upward seepage flow, if the upward seepage force becomes equal to Buoyant force/weight of soil, effective stress at every point in the soil will become =0

At B; $\bar{\sigma}_B = r_{sub} \cdot z - \frac{h_L}{H} \times z \times r_w = 0$

$\Rightarrow r_{sub} \cdot z = \frac{h_L}{H} \times z \times r_w \Rightarrow r_{sub} = i r_w$

$\Rightarrow \boxed{i = \frac{r_{sub}}{r_w}}$

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$i_{cr} = \frac{G-1}{1+e}$

$\bar{\sigma} = 0$

At B: $\gamma_{sub} z - i z \gamma_w = 0$

$i = \frac{\gamma_{sub}}{\gamma_w} = i_c$ → critical.

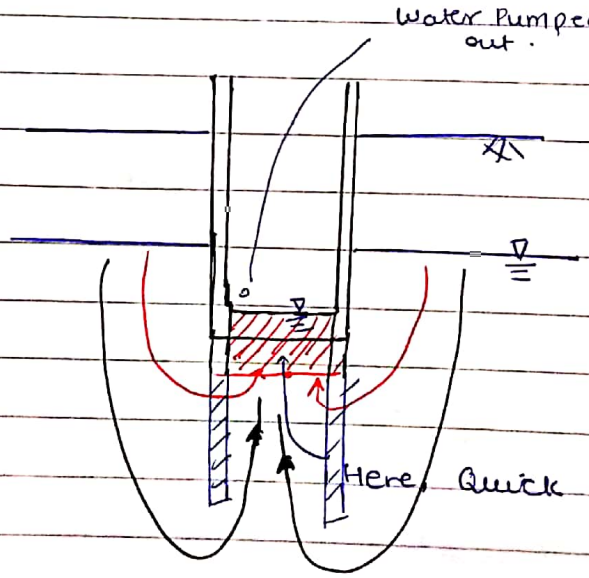
① Critical hydraulic cond' gradient at which $\bar{\sigma} = 0$ at every point in the soil mass $= i_{cr} \Rightarrow \frac{\gamma_{sub}}{\gamma_w} = \left(\frac{G-1}{1+e}\right)$

Critical Hydraulic Gradient.

② In case of sand: $\tau = c + \sigma' \tan\phi$

where $c = 0$.

Hence if effective stress is zero, sand will loose its shear strength completely, this condⁿ is called Quick Sand condⁿ.



③ To safeguard against quick sand condⁿ during excavation, the options are:

① Lowering the water table by pumping.
② Increasing the depth of embedment of sheet pile.
③ By keeping some water depth in the pit

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⊙ Sometimes, we also add Graded filter on the d/s side of Masonary wall. because at that point, there could be chance of the quick sand condn taking place.

⊙ FOS against quick sand condn = Buyont weight / Seepage force.

= Eff. stress under no flow condn / Seepage pressure)

⊙ In case of clay, if the pore water pressure exceeds the total stress, we will have a heaving problem & that may lead to cracking of edge.

⊙ To safeguard against heave, we have to ensure that on the $\sigma = ?$

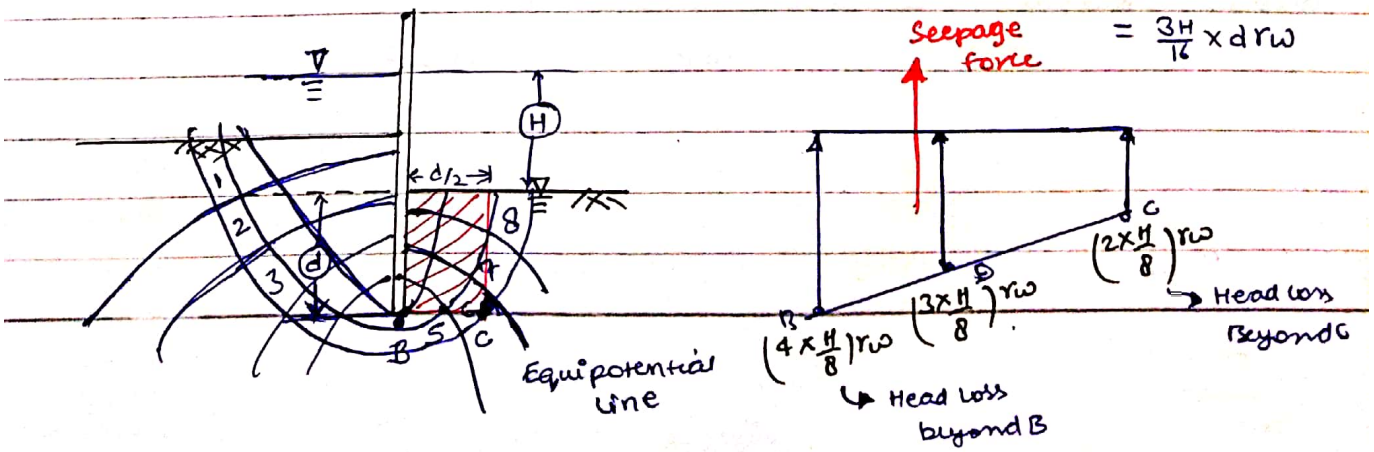
Pore water pressure & FOS against heave =

= Total Stress / Pore water Pressure

Condn Adjacent to Sheet Piles

Seepage force $= \frac{1}{2}\left[4\times\frac{H}{8} + 2\times\frac{H}{8}\right]\gamma_w \times \frac{d}{2}$

Seepage force $= \frac{3H}{16} \times d\gamma_w$



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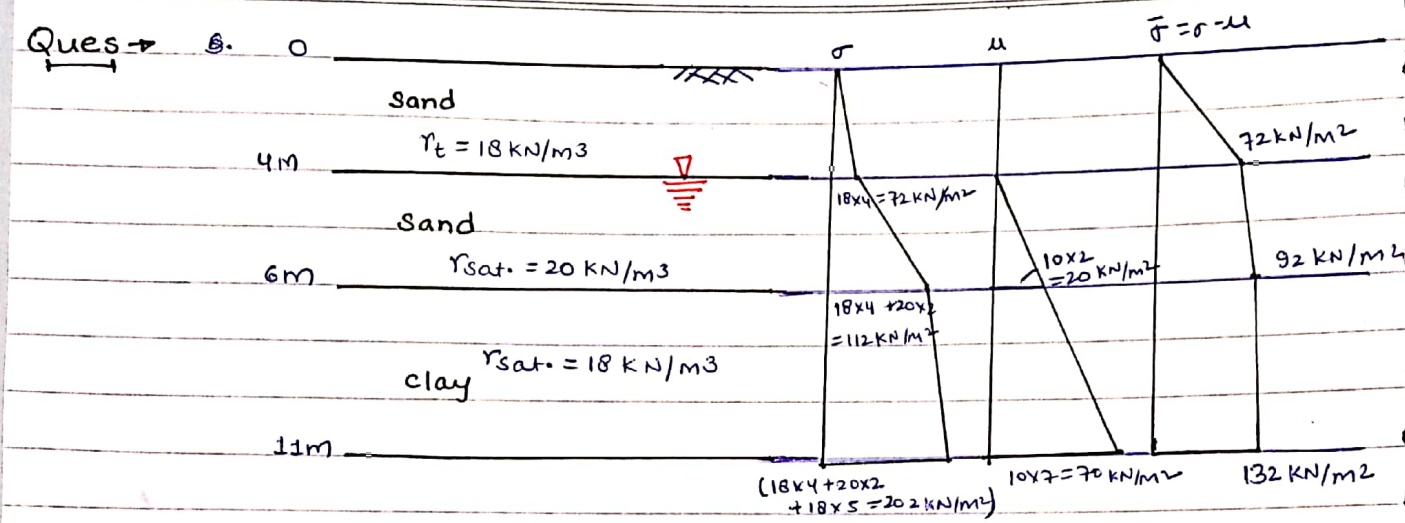
Ques → 8. 0



→ Plot variation of total stress, pore water pressure & effective stress against depth assuming no capillary rise above WT.

→ Find the total pore water pressure & σ with depth when sand upto 1m above w.r. saturated with capillary water.

→ when there is no capillary rise, a 5m deep gravel layer with γ_t = 20kN/m3 is placed over large area at top, find the effective vertical stress at the centre of clay layer under the following condn:

(a) Immediately after placement of fill

(b) long time after placement of fill.

→ find total stress, pore water pressure, when water table rises upto the ground level.

find σ, u, $\bar{\sigma}$ variation when the w.t. rises above ground surface by 2m.

Take γ_w = 10kN/m3

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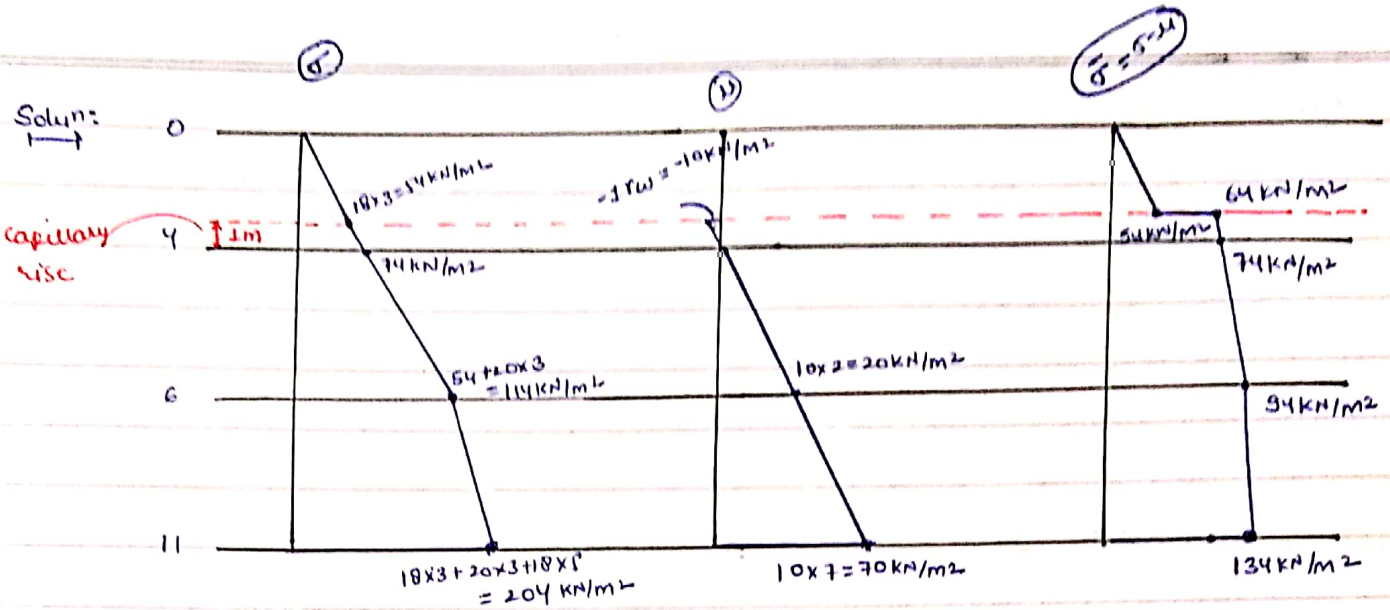
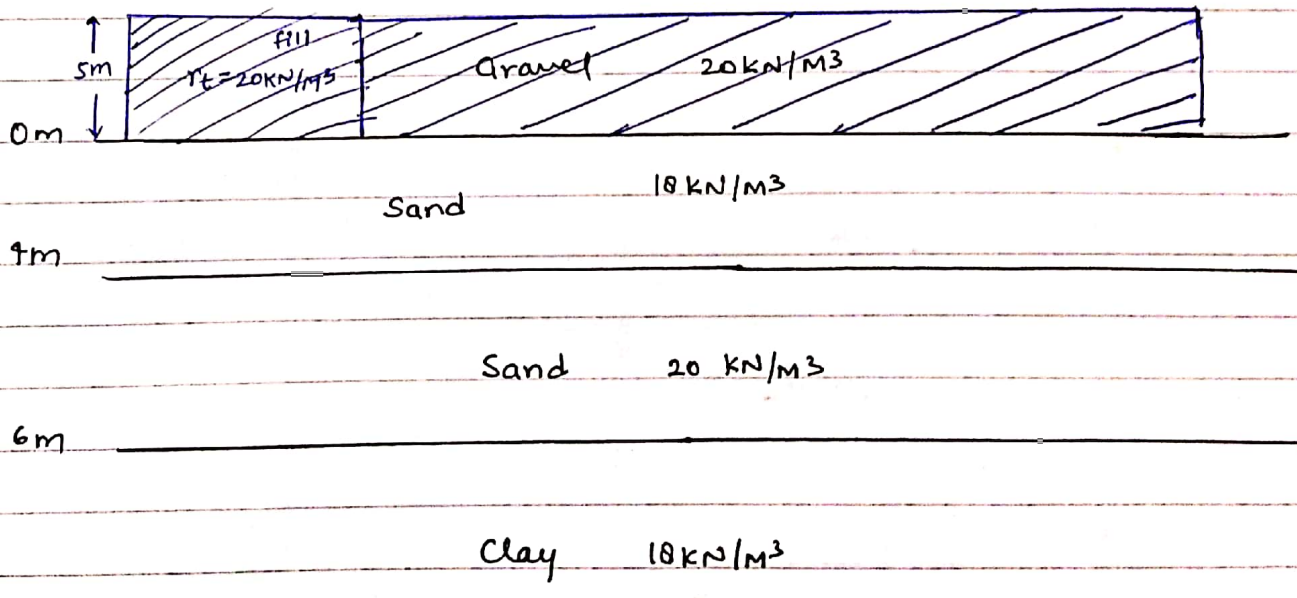


(i) In case of capillary rise, the pore water pressure below the W.T. will not change.

Hence, the effective stress below the water table will change only corresponding to increase in the total stress due to capillary saturation.



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① Effective stress at centre of clay before fill placed

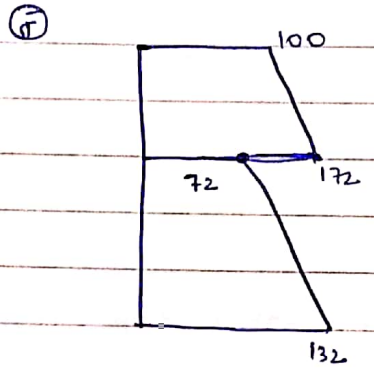
$= 18 \times 4 + 20 \times 2 + 18 \times 2.5 - (2+2.5) \times 10 = 112\ KN/M^2$

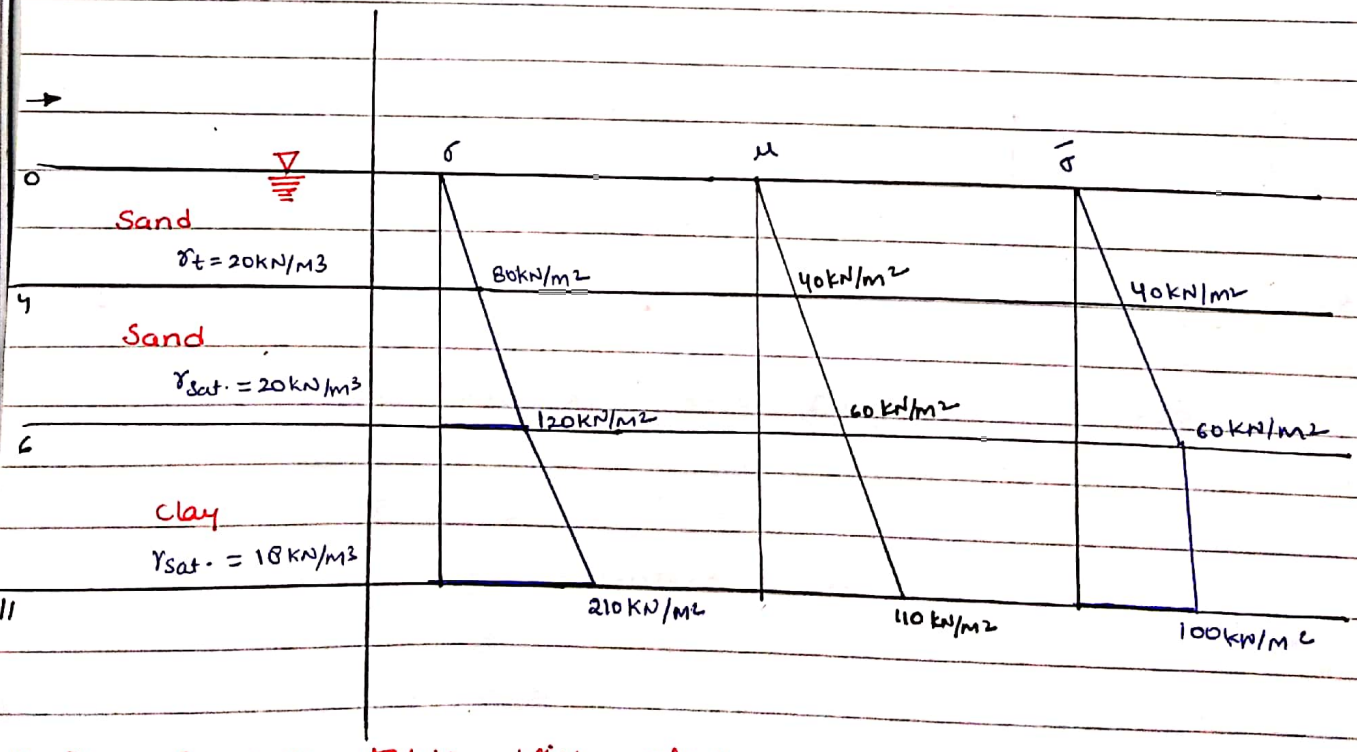
Ⓐ Immediately after placement of fill ⇒ $\bar{\sigma_B} = 112\ KN/m^2$

Ⓑ Long time after placement of fill ⇒ $\bar{\sigma_B} = 112 + 5 \times 20$

$= 212\ KN/M^2$

Immediately after load Application:





⊙ Rise of water table will always cause decrease in Effective stress. But this is true upto rise to ground level.

Rise of water level or fluctuation of water level

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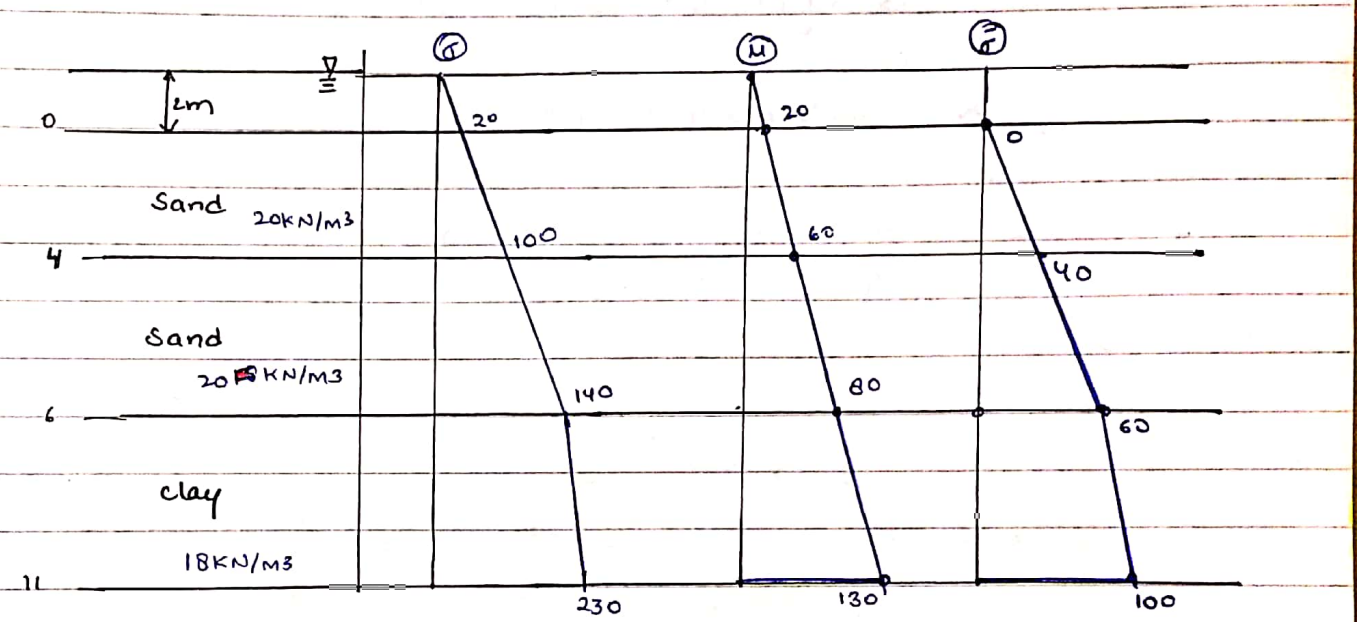
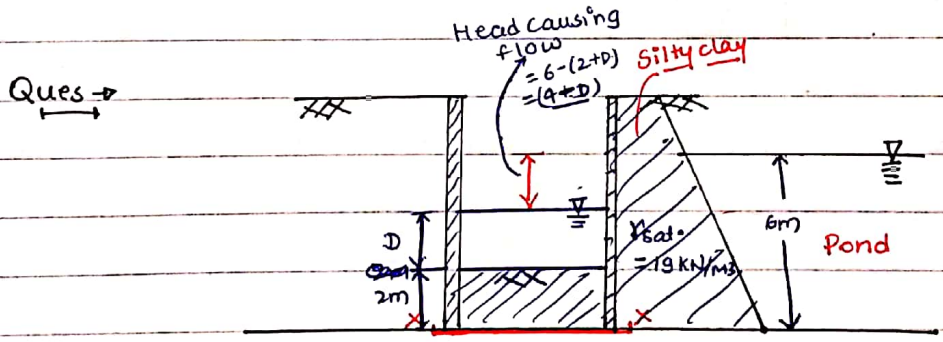
above the Groundlevel will not cause any change in Effective stress. Because due to rise above Ground level, the total stress and pore water pressure at every point increases by the same amount. Hence, effective stress does not change.





$\gamma_w = 10 KN/M^3$.

Upto Section:- x-x, (15%) of Head causing flow is lost. Find the depth of water to be kept in the pit if Boiling condn is to be avoided.

If (FOS = 2) is ensured and no water is kept in the pit, how much of lowering of Water level in the pond will be required if under this condn, (10%) of Head causing flow is to be lost upto x-x.

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Soln: Loss in 2M of soil = $0.85(4-D)$

For no Boiling;

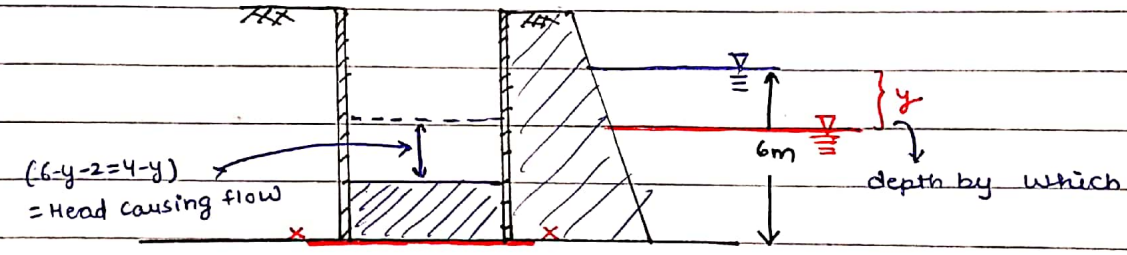
Buoyant weight at (B) > Seepage force

$\Rightarrow A\left[(\gamma_w D + \gamma_{sat} \times 2) - (2+D)\gamma_w\right] > 0.85(4-D)\gamma_w \cdot A$

$\Rightarrow \gamma_{sub} \cdot 2 > 0.85(4-D)\gamma_w$

$(19-10)\times 2 > 0.85 \times (4-D) \times 10$

$D > 1.88\,m$



coarse sand $\gamma_w = 10\,kN/m^3$

(Impermeable)

$6-y-2 = 4-y$ ⇒ Head causing flow.

Head loss beyond $x-x = 0.4(4-y)$

$FOS = 2 = \dfrac{\text{Boyant wt.}}{\text{Seepage force}}$

$\Rightarrow 2 = \dfrac{\gamma_{sub} \times 2 \times A}{0.9(4-y)\gamma_w A}$ $\Rightarrow y = 3\,m$

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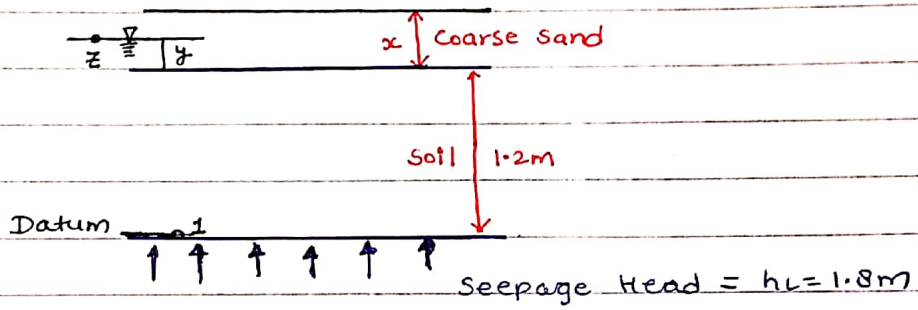
Piping, Boiling → Quick sand condⁿ

Ques→ 1.2 m layer of a soil is subjected to an upward seepage head of 1.8m. A layer of coarse sand is laid above the soil layer to attain a FOS of 2 against piping. Both the soil and the coarse sand has the same value of $G_s = 2.67$ and $e = 0.67$. There is Negligible head loss in the coarse sand layer. Find the required depth of coarse sand layer.

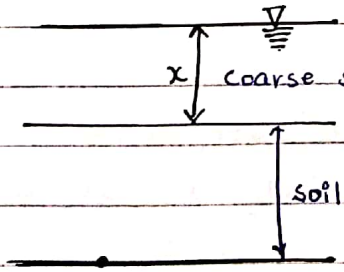
Solⁿ:



$$FOS = \frac{\text{Buoyant wt.} + F}{\text{Seepage Force}}$$

(Eff. stress at bottom of soil layer) A

On this case, it is not possible to determine to what height, the water will rise in the coarse sand layer. Hence, as a worst case, we can assume the coarse sand also to be completely submerged.



$$\gamma_{sub.} = \frac{G-1}{1+e}\gamma_w = \gamma_w$$

(G = 2.7, e = 0.67)

$$\gamma_{sat.(cs)} \cdot x + \gamma_{sat\,soil} \times 1.2 - (1.2+x)\gamma_w$$

$$\text{Buoyant wt.} = \left[\gamma_{sub.(cs)}\, x + \gamma_{sub.soil} \times 1.2\right] \times A$$

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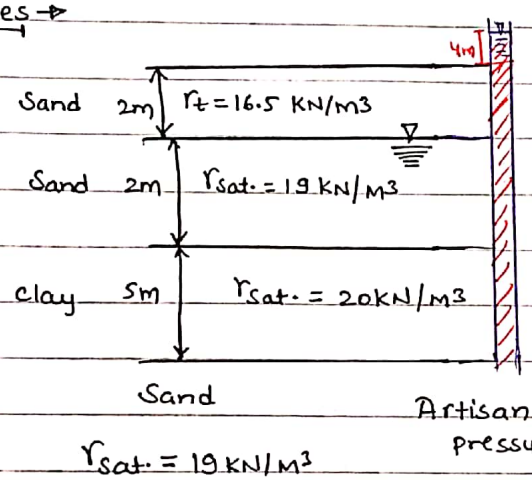
$$2 = \frac{\left[\gamma_{sub.(cs)} \cdot x + \gamma_{sub.soil} \times 1.2\right] A}{h_L \gamma_w A}$$

$$2 = \frac{\gamma_w (x + 1.2)}{1.8\, \gamma_w} \Rightarrow x = 2.4m$$

Ques →



Check if there is stability problem in case of clay or layer or not.

Soln: $\sigma_B = (2 \times 16.5) + (2 \times 19) + (5 \times 20) = 171$ KN/m²

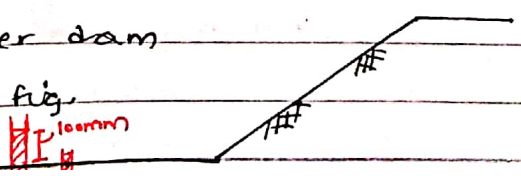
$u_B = (5 + 2 + 2 + 4)\gamma_w = 13\gamma_w = 13 \times 9.81 = 127.53$ KN/m²

$\sigma_B > u_B$

⇒ NO Stability problem in clay.

FOS against heave $= \frac{171}{127.53} = 1.34$

Ques → Steady seepage is taking place through a soil element at Q, 2m below the ground surface immediately d/s of the toe of an earther dam as shown in the fig.



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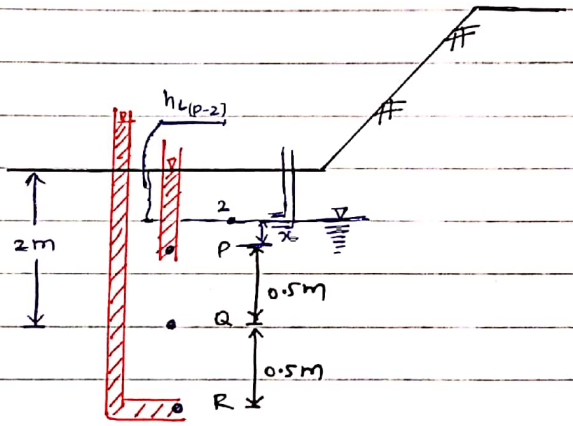
Water level in the piezometer inserted at point P is at the ground surface and water level in the piezometer inserted at R is 100 mm above the ground surface. Saturated unit wt. of soil is 18 kN/m³ and γ_w = 9.81 kN/m³. Find the vertical effective stress at Q.

Soln:

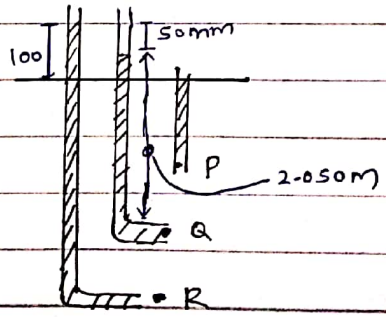


$$\frac{1.50 - x}{x} = \frac{0.1}{1} = i$$

$$x = 1.364\,m$$

If we assume the flow to be taking place vertically then the water level will rise above point P by 1.364 metre. Hence, to calculate the total stress at Q, we will take γ_t above the water table and $\gamma_{sat.}$ below the water table.

But γ_t is not given. Hence we will assume the water level to go upto the top or we can take $\gamma_t = \gamma_{sat.}$



$$\sigma_Q = \gamma_{sat.} \times 2$$

$$u_Q = 2.050\,\gamma_w$$

$$\bar{\sigma}_Q = 2 \times 18 - 2.05 \times 9.81$$

$$= 15.899\ kN/m^2$$

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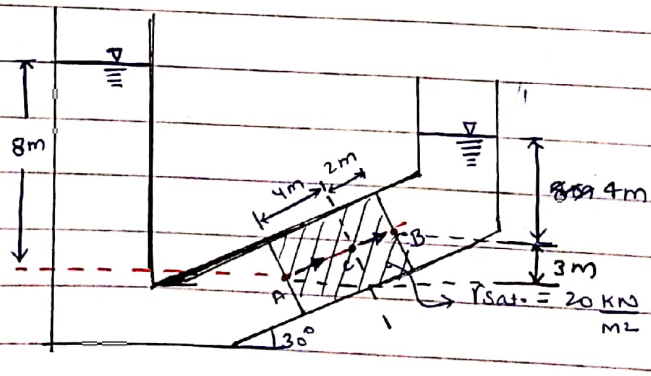
Note: $\sigma_a = \gamma_{sub} \times z - i \times z\gamma_w$

$= (18 - 9.81) \times 2 - \frac{0.1m}{1m} \times 2 \times 9.81$

$= 14.41 \text{ KN/m}^2$

Alternate way:

Ques →



(a) Check if there is a chance of Quick Sand condⁿ in the soil shown

(b) If permeability of the soil is 4×10^{-6} m/s, find the pore water pressure at point C & discharge through the sand.

Take $\gamma_w = 10 \text{ kN/m}^3$.

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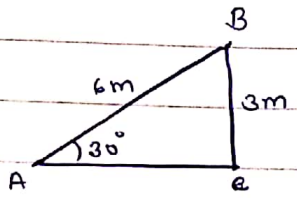


$\sigma_A = 4\gamma_w + 3\gamma_{sat.} = 40 + 3\times 20 = 100\ kN/m^2$

$u_A = 8\gamma_w = 8\times 10 = 80\ kN/m^2$

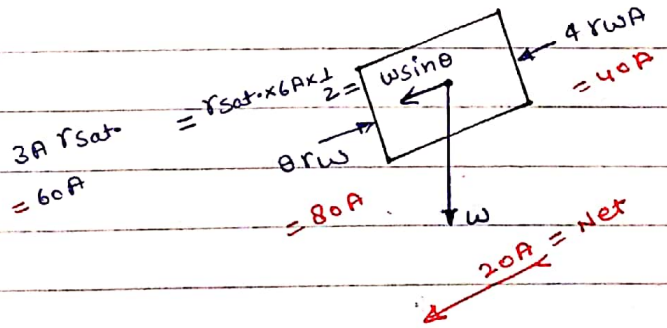
$\sigma_A > u_A \Rightarrow$ No quick condition

$\frac{\text{Buoyant wt.}}{A} = (4\gamma_w + 3\gamma_{sat.}) - 7\gamma_w = 3\gamma_{sub.} = 30\ kN/m^2$

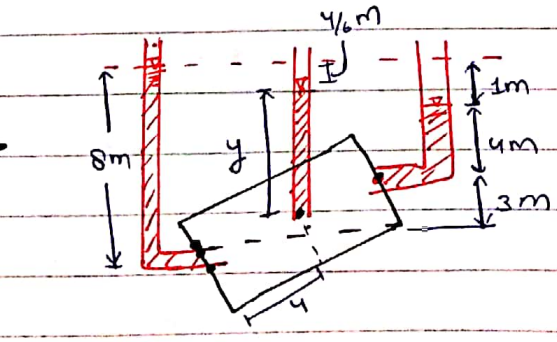
Seepage Pressure $= 1\times\gamma_w = 10\ kN/m^2$

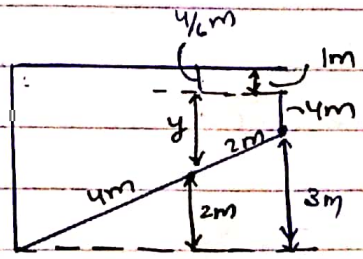
Buoyant wt. > Seepage force

$\Rightarrow$ No quick condition.



So; (No quick Sand Condition)





$Q = kiA$

$Q = \frac{1}{6}\times K\times A$

$2 + y + \frac{4}{6} = 8$

$y = 6 - \frac{2}{3}$

$y = \frac{16}{3}\ m$

$P_c = y\gamma_w = \frac{16}{3}\times 10$

$= 53.3\ kN/m^2$

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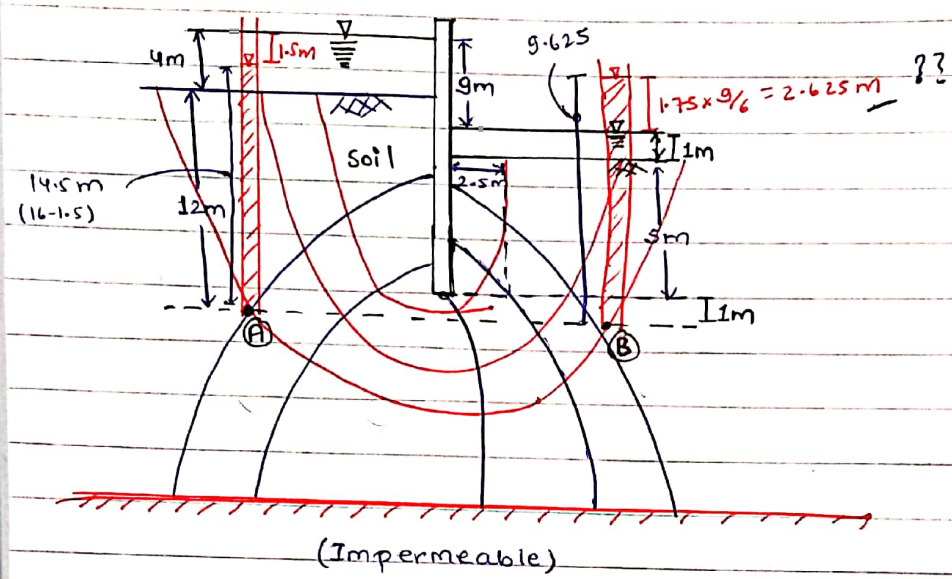
Ques →



$\gamma_{sat.} = 20\ kN/m^3$

No. of Equipotential drops upto (B) = 4.25 $\quad \gamma_w = 10\ kN/m^3$

Find the eff. stress at (A) x (B) and also check: the stability of sheet pile

Soln:

$\sigma_A = 4 \times 10 + 20 \times 12 = 280\ kN/m^2$

$u_A = 14.5\,\gamma_w = 145\ kN/m^2$

$\bar{\sigma}_A = 280 - 145$

$= 135\ kN/m^2$

$\sigma_B = 1 \times 10 + 6 \times 20 = 130\ kN/m^2$

$u_B = 9.625 \times \gamma_w$

$\bar{\sigma}_B = 130 - 96.25 = 33.75\ kN/m^2$

Head causing flow = 9m

No. of Equipotential drops = 6

Upto (A), no. of equipotential drops = 1

⇒ Head loss upto (A) = 1.5 m

⇒ $u_A = 14.5\,\gamma_w = 145\ kN/m^2$

$\bar{\sigma}_A = \sigma_A - u_A = 280 - 145$

$= 135\ kN/m^2$

$\sigma_B = 1\gamma_w + 6\gamma_{sat.} = 1 \times 10 + 6 \times 20$

$= 130\ kN/m^2$

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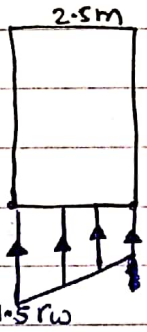
Check for stability of the sheet Pile:



Seepage force $= \left(\frac{4.5\gamma_w + 3\gamma_w}{2}\right) \times 2.5 \times 1$

$= 9.375\gamma_w = 93.75\,kN$

$3\gamma_w = \left(\frac{2 \text{ Equipotential}}{\text{drop}}\right) \times \frac{g}{6}\gamma_w$

$= \left(\frac{3 \text{ equipotential}}{\text{drop}}\right) \times \frac{g}{6}\gamma_w$

Buoyant wt. $= (5 \times 2.5 \times 1)\,\gamma_{sub.}$

$= 125\,kN$

Buoyant wt. > Seepage force

$\Rightarrow$ No stability Problem

$FOS = \frac{125}{93.75} = 1.33$

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SEEPAGE THROUGH SOIL

⊙ Seepage is the process in which liquid leaks through a porous medium from high head towards low head.

⊙ Seepage Problem could be loss of water from the Reservious, uplift force below the dam & piping problems Hence, we need to calculate the quantity of seepage, uplift pressure and exit Gradient. → Quick Sand Condⁿ & Piping.
All of these things can be calculated using flownet.

⊙ Flownet is a graphical representation of path taken by water particles and the head variation along the path.

⊙ The concept of flownet is based on laplace Eqⁿ

Laplace Eqⁿ For 2D Flow

⊙ following assumptions are made in 2D flow:

(i) Soil is fully saturated and Darcy's law is valid.
(ii) Soil mass is Homogeneous and isotropic (Kx=Ky)
(iii) Both the soil grains and pore fluid are incompressible.
(iv) Steady state condⁿ exist.

$$V_x = K_x\left(-\frac{\partial h}{\partial x}\right) \quad , h = (\text{Total Head})$$

$$V_y = K_y\left(-\frac{\partial h}{\partial y}\right)$$

$$\frac{\partial V_x}{\partial x} + \frac{\partial V_y}{\partial y} = 0 \quad (\text{continuity Eqn})$$

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$$-K_x \frac{\partial^2 h}{\partial x^2} - K_y \frac{\partial^2 h}{\partial x^2} = 0$$

For isotropic condⁿ:

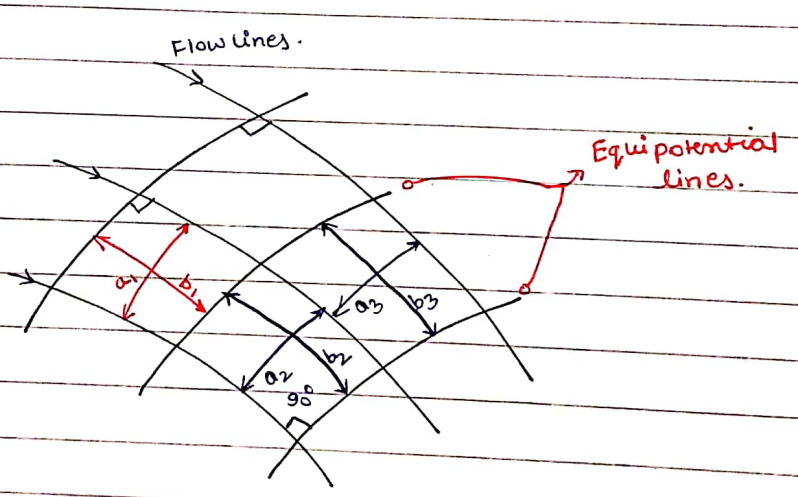
$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} = 0$$ → Laplace Eqn.

⊙ Solⁿ of laplace eqⁿ graphically yields two sets of curves which are orthogonal to each other.

One set is known as flow lines and other is known as equipotential lines.

Flow lines are nothing but stream lines.

Properties and use of Flow Net



(i) Flow lines and Equipotential lines are orthogonal to each other for isotropic soil.

(ii) Space b/w two adjacent flow line is called flow channel and the area b/w two adjacent flow lines and two equipotential lines is called a flow field.

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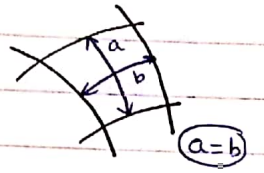
Head causing Flow = (U/s level − D/s level.)

$\left(\frac{N_f}{N_d}\right)$ → depends only on Boundary conditions and Extent of flow.

Flow fields are Elimentary square.



i.e; $\frac{q_1}{q_2}$ $\boxed{\frac{a_1}{b_1} = \frac{a_2}{b_2} = \frac{a_3}{b_3} = 1}$

(iii) Head loss b/w two Successive Equipotential lines are same and discharge through each flow channel is same.

N_f = No. of flow channels = No. of Flowlines − 1

N_d = No. of Equipotential drop = No. of Equipotential lines − 1

$$\boxed{q = K\frac{h_L}{b_1} \cdot a_1 \times 1} = K \cdot \frac{h_L}{b_2} \cdot a_2 \times 1 = K\frac{h_L}{b_3} \times a_3 \times 1$$

$$\Rightarrow q = K \cdot h_L \times 1 \left[\frac{a_1}{b_1} = \frac{a_2}{b_2} = \frac{a_3}{b_3} = 1\right]$$

Total discharge = qN_f = $K \cdot N_f \cdot h_L \times 1$

$h_L = \frac{H}{N_d}$, H = Head causing flow.

Total discharge per unit width inside = $K \cdot N_f \cdot \frac{H}{N_D} \times 1$

$$\boxed{Q = KH\frac{N_f}{N_D}}$$

$\frac{N_f}{N_D}$ = Shape factor.

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⊙ The shape factor $\left(\frac{N_f}{N_D}\right)$ is a funcn of Boundary conditions.

⊙ Shape funcn $\left(\frac{N_f}{N_D}\right)$, will not change if permeability of soil changes.

⊙ It will not change if the head causing flow changes.

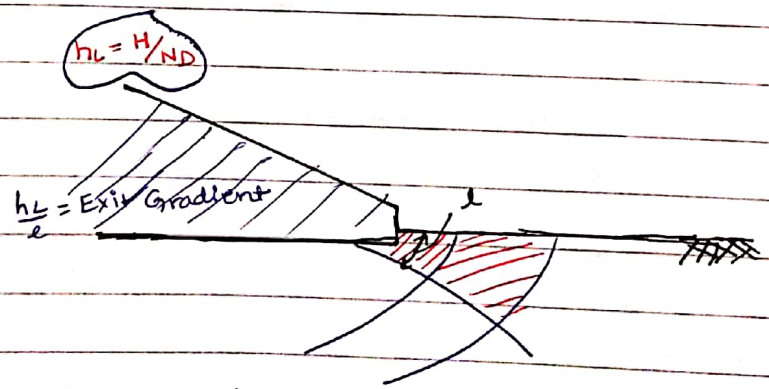
⊙ It will not change if the direcn of flow changes.

⊙ It will not change if N_f and N_D change.

⊙ It will only change only when boundary condn or extent of flow changes.

USES OF FLOW NET

1. Seepage calculation
2. Uplift pressure calculation
3. Exit Gradient calculation.



⊙ Exit Gradient is maxm near the toe. because the flow field is smallest here.

If the Exit gradient is more, piping begins near the d/s toe and lengthens progressively towards the u/s side

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as the seeping water gradually washes away more and more of soil particles leaving a void or pipe in the soil.

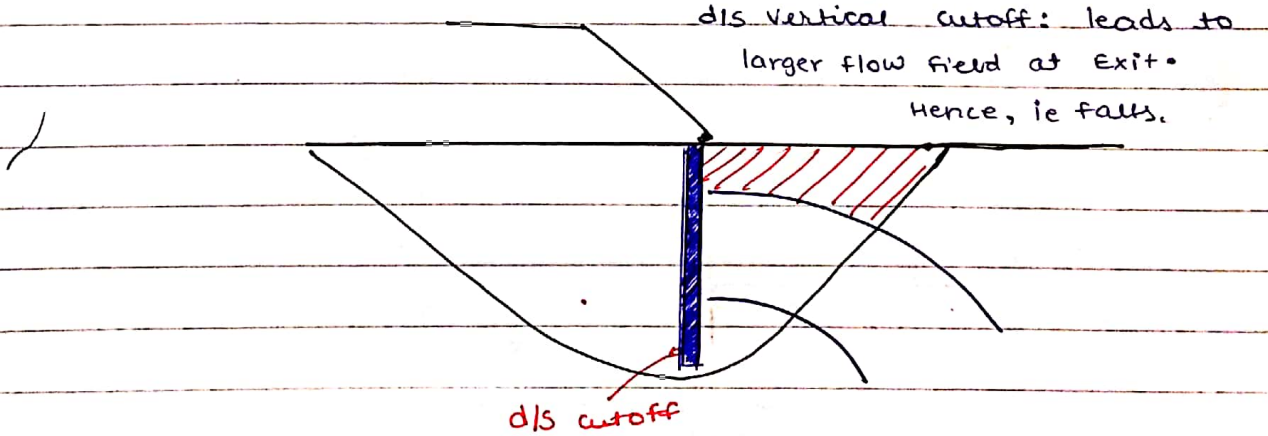
⊙ Piping can procede backwards along the base of the dam or along the bedding plane in the soil strata. where the resistance is minm.

The dam may collapse into the pipes created and it would be a catastrophic failure.

⊙ Factor of Safety (FOS) against piping = $\frac{i_c}{i_e}$ where

i_c = critical Hydraulic gradient : $i_c = \left(\frac{G-1}{1+e}\right)$ and $i_e = \frac{h_L}{L_e}$

i_e is the exit gradient. This FOS is taken conservatively minm at 6.



⊙ U/s cutoff helps in reducing the uplift force below the dam because more no. of equipotential drops would have already occured by the time water reaches the base of the dam.

Thus, pressure at the base of the dam reduces.

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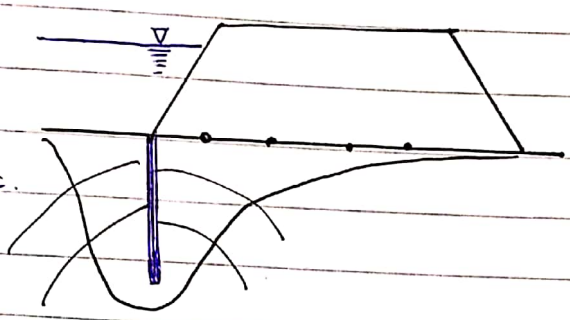
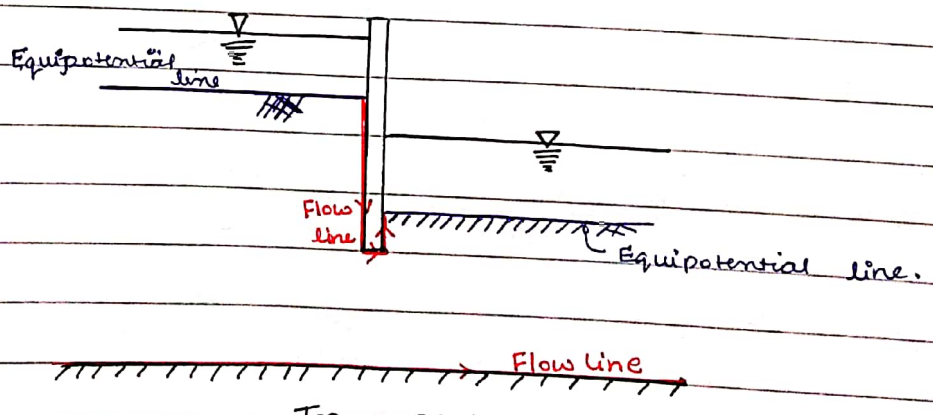


Note: ◉ Provision of d/s cutoff will reduce exit gradient but will increase the uplift pressure.

◉ Provision of u/s cutoff will reduce the uplift pressure but may increase the exit gradient.

◉ The provision of cutoff u/s or D/s will change the $\frac{N_f}{N_D}$ value and hence the discharge.

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



① For solⁿ of Laplace eqⁿ in 2D, we require (4) boundary condns: of the 4 B.C. are known

(21)

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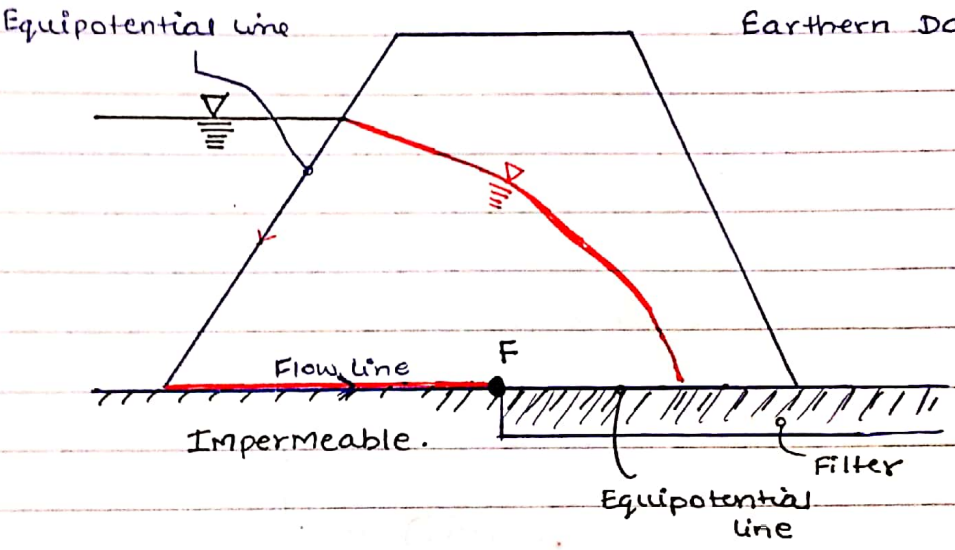
then the flownet drawn is called the flownet for the confined flow.

Examples: Flow under sheet pile, concrete or Masonary Dam.

FLOW NET FOR UNCONFINED FLOW



⊙ In case of Earthern dam, the top flow line is not known in advance and thus we have only 3 Boundary conds known, the flow net drawn for this is called the flownet for the unconfined flow.

Locating the Top flow line for Seepage through Earthern Dam

⊙ A/c to casagrande, the flow lines and Equipotential lines are two sets of parabola with point f as a common focus and the top parabola is called Base parabola.

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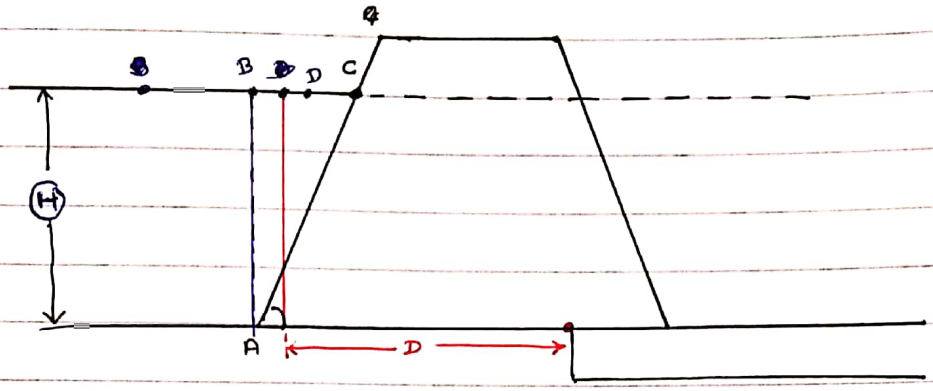
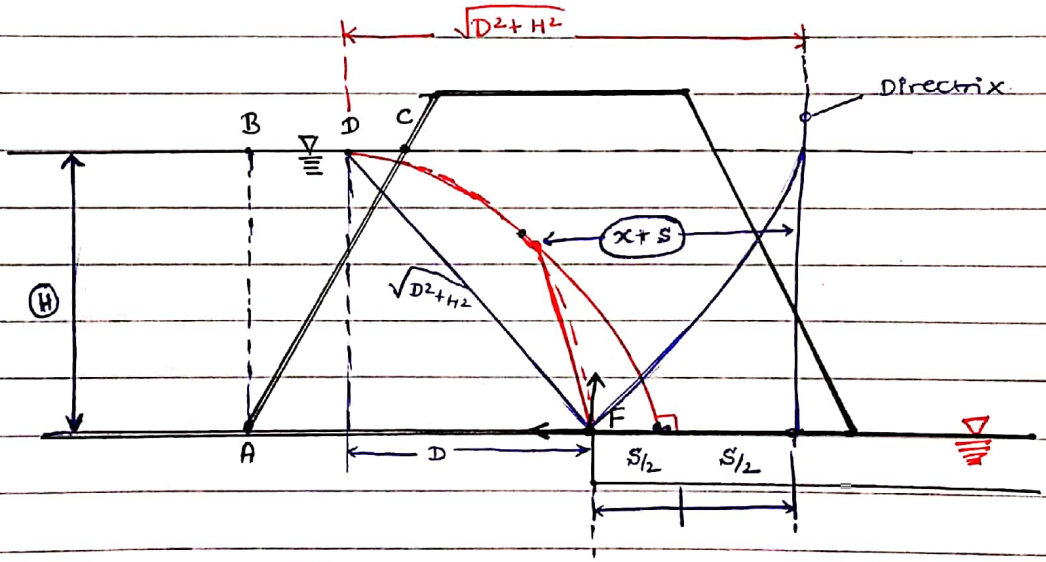




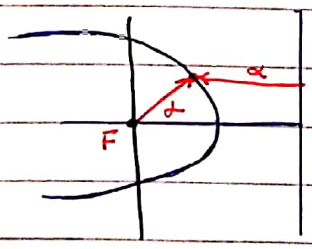
DC = 0.3BC

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

$y^2 = (x+s)^2 - x^2$

$y^2 = s^2 + 2sx$

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



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⑥ Once the top flow line can be located, flownet can be determined and seepage calculation like: $q = K_H \frac{N_f}{N_d}$ can be obtained.

At $x=0$, $y=s$

$$\Rightarrow 2y\frac{dy}{dx} = 2s$$

$$\frac{dy}{dx} = \frac{s}{y} \Rightarrow \left.\frac{dy}{dx}\right|_{x=0} = 1$$

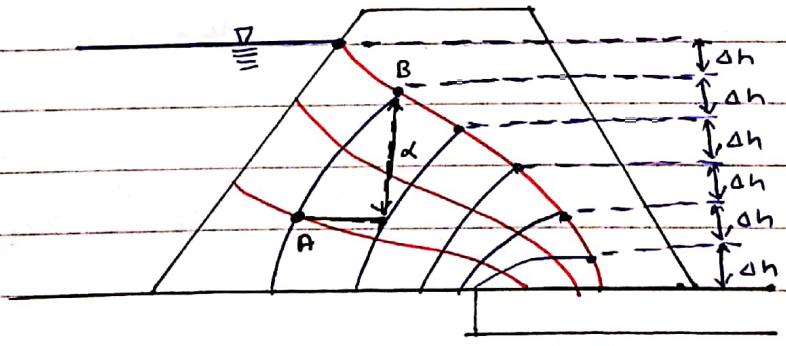
⑦ For top flow line, total Head = Elevation Head (b/c Pressure Head = 0) and hence $\frac{dh}{dx} = \frac{dy}{dx} = i$.

The Hydraulic gradient throughout the depth of flow at $x=0$ can be approximately taken as the hydraulic gradient of phreatic line (top line) at $x=0$

$$Q = K \times i \times A$$

$$q = K \times 1 \times S \times 1$$

$$\boxed{q = KS} \rightarrow \text{Seepage Discharge}$$



$$\frac{P_A}{\gamma} = \alpha$$

$$\frac{P_A}{\gamma} + Z_A = \frac{P_B}{\gamma}^{0} + Z_B$$

$$\frac{P_A}{\gamma} = Z_B - Z_A = \alpha$$

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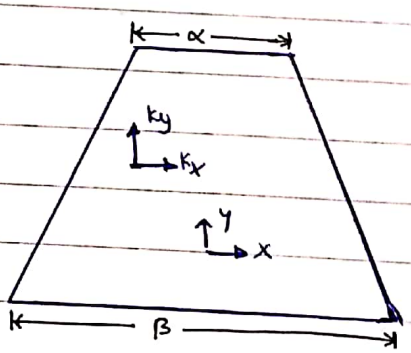
Seepage through Anisotropic soil

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

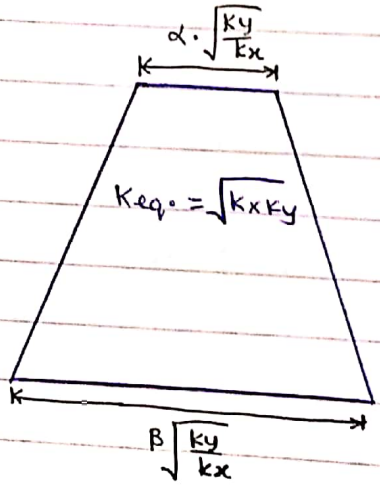
$$\frac{K_x}{K_y} \frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} = 0$$

$$x_t = \sqrt{\frac{K_y}{K_x}} \cdot x$$

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



(Anisotropic)



(Equivalent Isotropic condⁿ)

⊙ In Anisotropic condⁿ, we transform the section into equivalent isotropic system by reducing the horizontal dimension by a factor of $\sqrt{\frac{K_y}{K_x}}$ or $\sqrt{\frac{K_V}{K_H}}$ where y is the vertical direcⁿ and x is the horizontal direcⁿ keeping dimension intact.

⊙ On the transformed section, Flownet can be drawned and the flowlines and equipotential lines will be perpendicular to each other. in the transformed section.

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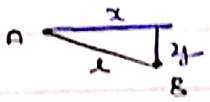
The equivalent permeability of the transformed section is taken as: $\sqrt{K_x K_y}$

$$Q = \sqrt{K_x K_y} \cdot H \times \frac{N_f}{N_d}$$

H = Head causing flow

⊙ Transformed section can be used to determine pore water pressure head at any point but to determine the hydraulic gradient, we should use the length in the original section.

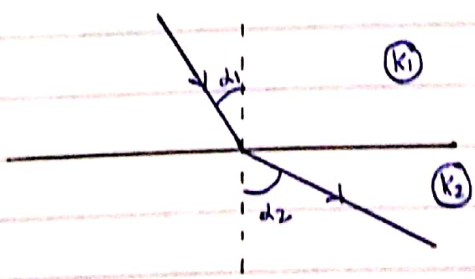
$$i_{actual} = \frac{h_L \text{ (b/w) two points}}{\text{original dist. b/w those two points}}$$



$$l_{AB\ (actual)} = \sqrt{y^2 + \left(x\sqrt{\frac{K_x}{K_y}}\right)^2}$$

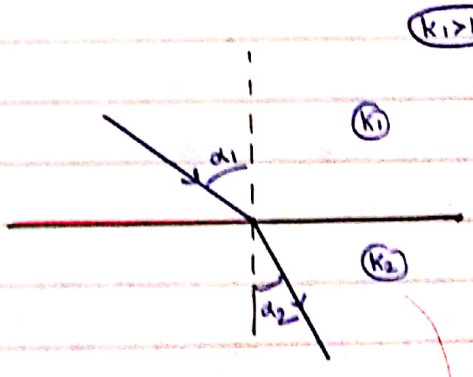
Flow through Non-Homogenous Section

$K_1 < K_2$



$$\frac{K_1}{K_2} = \frac{\tan\alpha_1}{\tan\alpha_2}$$

$K_1 > K_2$



$$\frac{K_1}{K_2} = \frac{\tan\alpha_1}{\tan\alpha_2}$$

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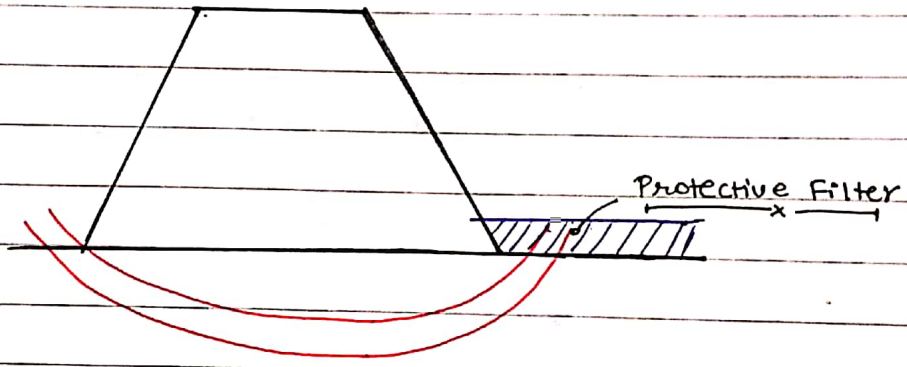
Prevention of Erosion

⊙ To prevent the possibility of erosion and piping, two approaches are used:

(i) Control of Seepage and Seepage force.
(ii) Use of Protective filter.

⊙ Seepage force and Erosion can be prevented by provision of cutoff walls or increasing the length of flow path.



⊙ Use of protective filter prevents erosion and helps in safeguarding against piping.

⊙ Filter will prevent migration of finer particles without significant loss of head in filter.

If the voids in filter are large, particles will be washed out & if the voids are small, seepage force may develop in the filter.

To achieve this, Filter must have grain sizes that satisfy certain requirements:

(i) $\frac{D_{15}\text{ (filter)}}{D_{85}\text{ (Protected material)}} < 5$

⊙ This will ensure no significant invasion of particles from protected material to the filter occurs &

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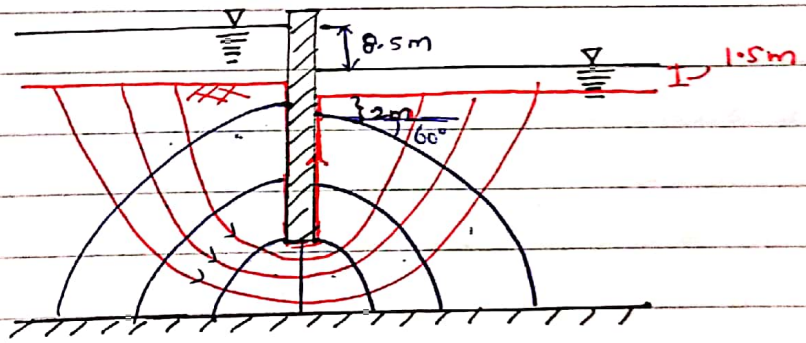
thus governs the upper limit of Grain size for the filter material.

(ii) $4 < \frac{D_{15}\text{ (filter)}}{D_{15}\text{ (Protected material)}} < 20$

⊙ This will ensure that seepage pressure doesnot build up in the filter and governs the lower limit of material size.

(iii) Additional criteria: $\frac{D_{50}\text{ (filter)}}{D_{50}\text{ (protected material)}} < 25$

Ques→



$K = 0.09$ m/d (isotropic), $G_s = 2.7$, $e = 0.8$

Find the seepage loss, FOS against piping.

Soln: $q = K H \frac{N_f}{N_d} = 0.09 \times 8.5 \times \frac{4}{8} = 0.3825$ m³/s/m.

$$FOS = \frac{i_c}{i_e} = \frac{\left(\frac{G-1}{1+e}\right)}{\left(\frac{8.5/8}{2}\right)} = \frac{\left(\frac{2.7-1}{1.8}\right)}{\frac{8.5}{16}} = 1.778$$

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Ques→ The flownet shown in previous question if corresponds to equivalent isotropic condⁿ for an anisotropic soil with horizontal permeability = 2m/day and vertical permeability of 0.8 m/day, find the seepage discharge and the exit gradient.

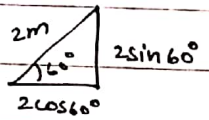
Soln:

$K_x = 2\ m/day \qquad K_y = 0.8\ m/day$

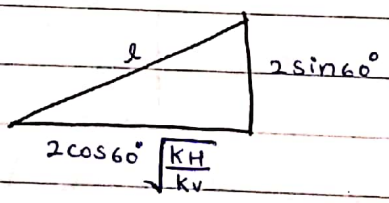
$K_{eq.} = \sqrt{K_x K_y}$ →

$$q = \sqrt{K_x K_y} \cdot H \cdot \frac{N_f}{N_d} = \sqrt{1.6} \times 8.5 \times \frac{4}{8} = 5.376\ m^3/d/m$$

$$i_e = \frac{h_L}{L} = \frac{(8.5/8)}{\ell}$$



For transformed section:



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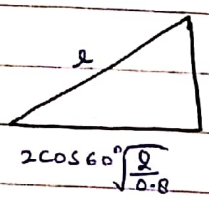

$\Rightarrow \ell = 2.345\ m$

$$i_e = \frac{8.5/8}{2.345} = 0.453$$

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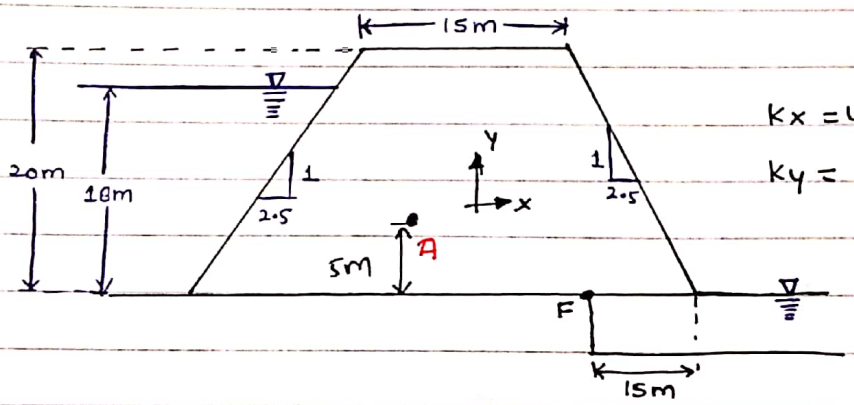
Note: ① Hydraulic Gradient in reality should be calculated corresponding to the least distance b/w the equipotential lines.

However, if the data is not known, we adopt the above approach.

Ques→



$K_x = 4.5 \times 10^{-8}$ m/s

$K_y = 1.6 \times 10^{-8}$ m/s

No. of Equipotential drops (N_d) = 18, upto point A = 3 in the transformed section.

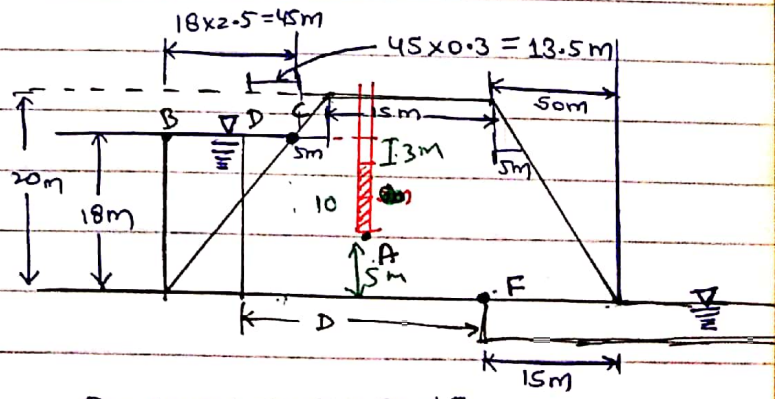
Find out the seepage discharge and pore water pressure at point A.

Soln: $q = K_{eq} \cdot S$

$q = \sqrt{K_x K_y} \cdot S$

$\sqrt{d^2 + H^2} - d = S$

$D\sqrt{\frac{K_y}{K_x}} = d$



$D = 13.5 + 5 + 15 + 50 - 15$

$D = 68.5$ m

$D\sqrt{\frac{K_y}{K_x}} = d = 68.5\sqrt{\frac{1.6}{4.5}} = 40.845$ m

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$\Rightarrow S = \sqrt{(40.845)^2 + (18)^2} - 40.845 = 3.79\,m$

$q = \sqrt{4.5 \times 1.6} \times 10^{-8} \times 3.79\ m^3/s/m$

$q = 10.169 \times 10^{-8}\ m^3/s/m.$

Total $Nd = 18$, $H = 18\,m$

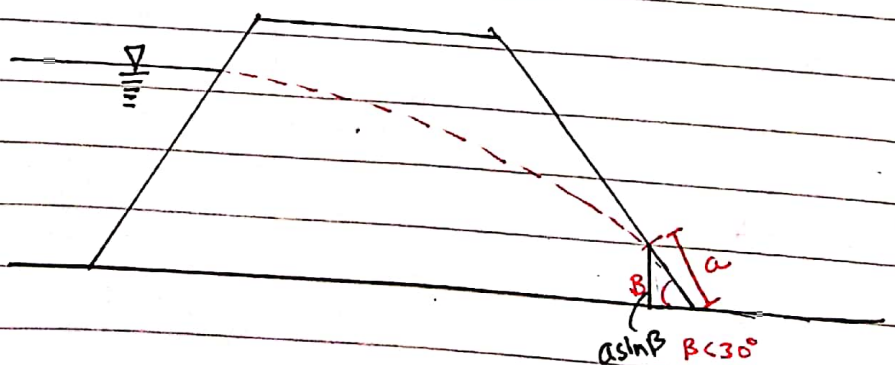
loss per ~~unit~~ Equipotential drop $= 1m$.

Hence, upto point A, total head loss $= 3m$.

∴ Pore water Pressure: ~~Head~~ $h = 18 - 3 - 5 = 10\,m$

$u_A = 10\gamma_w = 98.1\,kN/m^2$

Ques→ A Homogenous earth dam with no Horizontal drainage filter is shown in the figure. The slope of the d/s side is β ($<30°$)



In order to determine the value of A, discharge per unit width through the section of height asinβ will be ____?

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Soln: At the vertical where water surface touches the d/s face, the hydraulic gradient can be assumed to be constant throughout the depth and equal to the slope of phreatic line if d/s slope is $< 30°$.

Slope = Hydraulic Gradient = $i = \tan\beta = \frac{dy}{dx}$ at phreatic line at the intersection with the d/s slope.

$$q = K\left(\frac{dy}{dx}\right)(a\sin\beta \times 1)$$

$$q = K\,\frac{\sin\beta}{\cos\beta}\cdot a\sin\beta$$

$$\Rightarrow q = Ka\tan\beta\cdot\sin\beta$$

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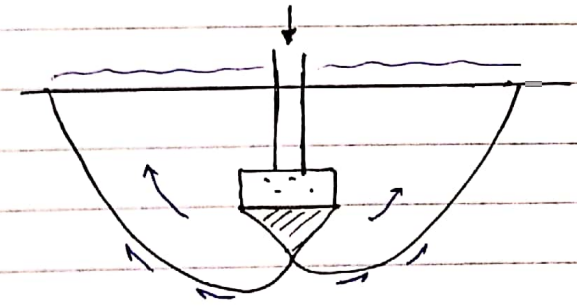
COMPRESSIBILITY & CONSOLIDATION

⊙ Properties of soil due to which decrease in volume occurs under compressive force is known as compressibility of soil.

⊙ Reduction in volume of soil mass may result in change in the lateral and vertical dimensions of soil mass.

⊙ The soil mass is infinitely large in lateral direcn, change in dimension in lateral direcn is small except when loading is applied at the edge of the embankment or when the loaded area is small such as structural foundations.



⊙ The change in the vertical dimension of soil is called settlement of soil.

$$S_{total} = S_{immediate} + S_{primary} + S_{secondary}$$

(#) Immediate Settlement:

(deformation without volume change)

⊙ It occurs due to volume distortion in the foundation soil caused by shear strain. (Volume distortion occuring without volume change). & by soil compression resulting from the decrease in void ratio (due to expulsion or

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Primary & secondary consolidation can occur simultaneously or together.

But for calculation purpose, we consider secondary consolidation after primary consolidation.

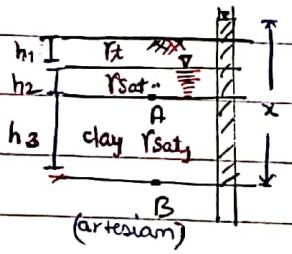
compression of air), and also due to elastic compression of the soil particles.

⊙ If soil particles and water are incompressible and there is lateral confinement in the soil and soil is saturated then immediate settlement will not occur.

⊕ **Primary Consolidation:** It occurs due to increase in effective stress.

⊙ Application of external loading, lowering of water table, decrease of artesian pressure below clay layer

Note:



$\sigma_A = h_1\gamma_t + h_2\gamma_{sat.}$

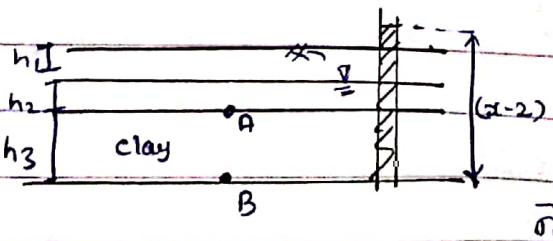
$u_A = \gamma_w h_2$

$\bar{\sigma}_A = h_1\gamma_t + h_2\gamma_{sub.}$

$\sigma_B = h_1\gamma_t + h_2\gamma_{sat.} + h_3\gamma_{sat.1}$

$u_B = x\gamma_w$

$\bar{\sigma}_A =$



$\sigma_B = h_1\gamma_t + h_2\gamma_{sat.} + h_3\gamma_{sat.1}$

$u_B = (x-2)\gamma_w$

$\bar{\sigma}_A = h_1\gamma_t + h_2\gamma_{sub.}$

$\Delta\bar{\sigma}_A = 0$

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$$\Delta\bar{\sigma}_B = \Delta\bar{\sigma}_{Bf} - \Delta\bar{\sigma}_{Bi}$$

$$= -[\sigma_B - x\gamma_w] + [\sigma_B - (x-2)\gamma_w]$$

$$\Delta\bar{\sigma}_B = 2\gamma_w$$

Avg. $\bar{\sigma}$ increase $= \frac{\Delta\bar{\sigma}}{2}$

⊙ Due to increase in loading, particle will respond by rearrangement themselves in denser packing thereby reduction in void ratio will occur but to decrease the void in saturated soil, pore water has to be squeezed out of the soil.

The soil remains completely saturated during the process of consolidation.

⊙ Primary consolidation occurs at a rate at which the stresses are transfered on to the soil particles.

⊙ Rate of primary consolidation = Rate of Expulsion of water

= Rate of decrease in pore water Pressure

= Rate of increase in Effective Stress.

⊙ The time taken in the primary consolidation is called hydrodynamic time lag.

⊛ Secondary Consolidation: The reason for secondary consolidation is that the soil structure is prone to viscous and pre deformations under the action of sustained stress as the fabric elements slowly adjust to more stable configuration.

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2° consolidation → Peat (Highly organic soil) and Soft clays

- The mechanism of secondary consolidation involves sliding at inter plastic contact., expulsion of water from the microfabric element, rearrangement of absorbed water molecules and cations into different orientations.

- Secondary consolidation also occurs due to decomposition of organic matter.

- Secondary consolidation occurs at gradually decreasing rate, because of viscosity of adsorbed water film increases as the molecules move closer resulting in decrease in rate of compression.

- Rate of Secondary consolidation is less than that of the primary consolidation.

- For calculation of secondary consolidation settlement, we assume that 2° consolidation starts after completion of primary consolidation at constant effective stress.

- However, in reality, both primary and Secondary consolidation continues concurrently.

- Secondary consolidation is important in case of Peat (highly organic soil) and soft clay.

- In case of overconsolidated clays and stiff clays, it is not important.

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Creep → time dependent.

2° settlement → Creep after 1° settlement.

- In Granular soil, all of the settlement occurs immediately. In Stiff clay & overconsolidated clay, total settlement will be primary consolidation and immediate settlement.

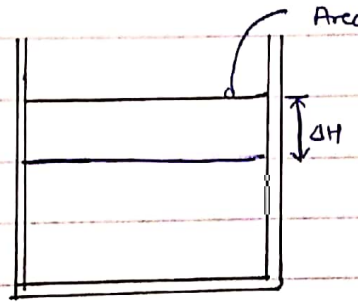
- In Soft clay and organic soil, total consolidation = $S_{immediate} + S_{primary} + S_{secondary}$.

1-D consolidation

Granular → Si

Stiff clay, OC clay → Si + Sp

Soft clay, organic soil → Si + Sp + Ss



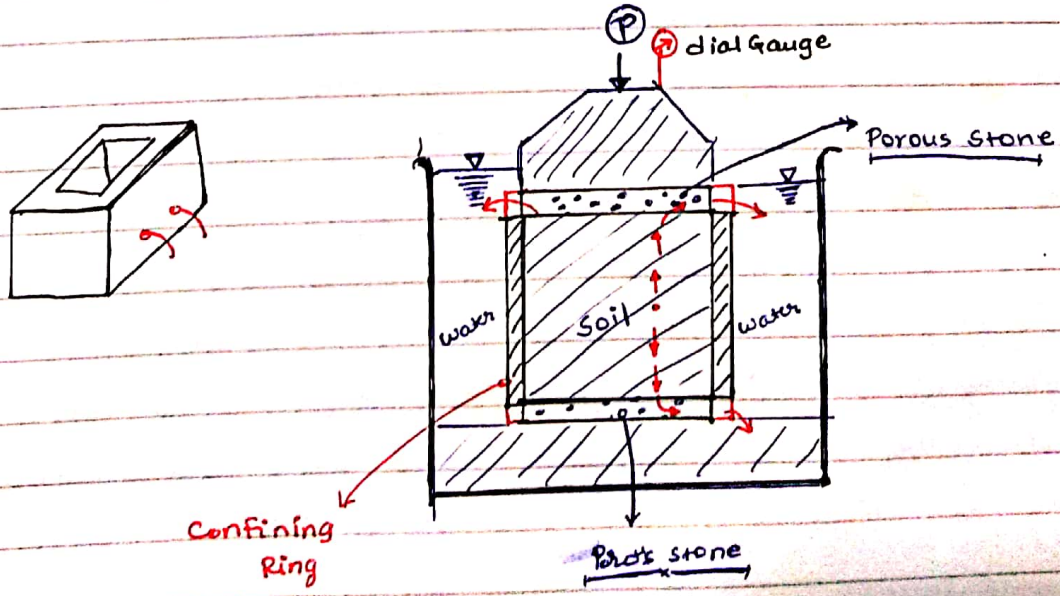
$$\frac{\Delta V}{V} = \frac{\Delta H \cdot A}{H \cdot A}$$

$$\Rightarrow \frac{\Delta V}{V} = \frac{\Delta H}{H}$$

$$\frac{\Delta V}{V} = \frac{\Delta e \cdot V_s}{(1+e)V_s} = \frac{\Delta e}{1+e}$$

$$\Rightarrow \frac{\Delta V}{V} = \frac{\Delta H}{H} = \frac{\Delta e}{1+e}$$

- Oedometer Test:



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⊙ Oedometer testing is done to study the characteristic of soil during 1-D consolidation. In this case, sample in the form of disc of thickness 19-20mm and diameter 75-100 mm is prepared and placed inside a metallic ring.

Metallic ring can be floating or fixed and accordingly, the oedometer test is called floating ring test. or fixed ring test oedometer test.

⊙ Floating ring is free to move vertically whereas fixed ring is clamped to the body of the equipment.

⊙ In floating ring test, compression occurs from both top and bottom while in fixed ring, sample moves only in vertically downward direction relative to the ring.

⊙ In floating ring test, friction blw the ring and the soil ϕ is somewhat less than in the fixed ring test.

⊙ In the fixed ring test, measurement of drainage from the porous stone is Easy and hence it is used to measure the permeability of soil.

⊙ Sample of soil is permanently in contact with water and porous stones are placed at top and bottom of sample signifying double drainage condition.

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- The sample is loaded and dial gauge reading is noted.
- Each load increment is kept for 24 Hr. during which the increment is assumed to have ~~of~~ become effective.
- Load increment ratio (LIR) i.e; $\frac{\Delta\sigma}{\sigma}$ is taken as 1 because it results in evenly spaced data points and provides sufficient no. of data points for adequate definition of the shape of curve.
- Smaller load increment ratio prolongs the ~~of~~ oedometer test.
- After the soil has been sufficiently loaded to cover the stress range in which the soil in the field will be subjected to, unloading will be done in 1 or 2 steps.
- After complete ~~of~~ removal of loading, sample is taken out and tested to find out the water content.
- Oedometer test results are plotted as:

(i) e v/s $\bar{\sigma}$ curve
(ii) e v/s $\log \bar{\sigma}$ curve
(iii) Dial Gauge reading v/s time curve, for a particular stress increment.

- The void ratio at different effective stress are calculated using 2 approaches:

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(i) Change in Void ratio method :

$$\frac{\Delta H}{H_f} = \frac{\Delta e}{1+e_f}$$

$e_f \times 1 = w_f \times G$

ΔH calculated from → dial Gauge reading.

(ii) Height of Solid Method :

H_s = Height of Solid = $\frac{W_s}{G_s \gamma_w A}$ (Volume / area)

$$e = \frac{H - H_s}{H_s}$$

Ques→

Applied Stress (kN/m²)	Sample thickness (mm)
0	19.7
50	19.3
100	19.15
200	18.85
400	18.56
800	18.36
0	18.9

e = ?

Find the void ratio (e) at effective stress of 800 kN/m² and 400 kN/m² if water content of Sample at the end of the test is 28% and G_s is 2.72.

Solu:

$w_f = 0.28$ $G_s = 2.72$

$e_f \times 1 = w_f \times G_s$

$e_f = 0.28 \times 2.72 = 0.7616$

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Void ratio at 800 KN/m² = e_1

$$\frac{\Delta e}{1+e_f} = \frac{\Delta H}{H}$$

(e_f = 0.7616)

$$\frac{e_f - e_i}{1+e_f} = \frac{18.9 - 18.36}{18.9} \Rightarrow e_1 = 0.7112$$

(e_f = 0.7616)

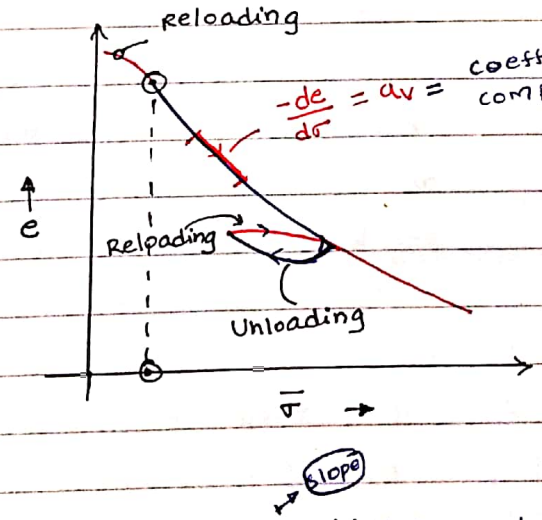
Void ratio at 400 KN/m² = e_2

(e_1 = 0.7112)

$$\frac{e_1 - e_2}{1+e_1} = \frac{18.36 - 18.56}{18.36}$$

$$\Rightarrow e_2 = 0.7298$$

Compressibility characteristics



$-\frac{de}{d\bar{\sigma}} = a_v$ = coeff. of compressibility.

$$a_v = \frac{\Delta e}{\Delta \bar{\sigma}}$$

① Coeff. of compressibility decreases with the increase in effective stress because with each increment of loading, soil becomes more densified. Hence, resistance to further compaction increases.

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Coefficient of Volume Compressibility

$$m_v = \frac{\Delta V/V}{\Delta \bar{\sigma}} = \frac{\Delta H/H}{\Delta \bar{\sigma}} = \left(\frac{\Delta e}{1+e}\right) \times \frac{1}{(\Delta \bar{\sigma})} = \frac{\Delta e}{\Delta \bar{\sigma}} \times \frac{1}{1+e} = \frac{a_v}{1+e}$$

$$\Rightarrow \boxed{m_v = \frac{a_v}{1+e}}$$

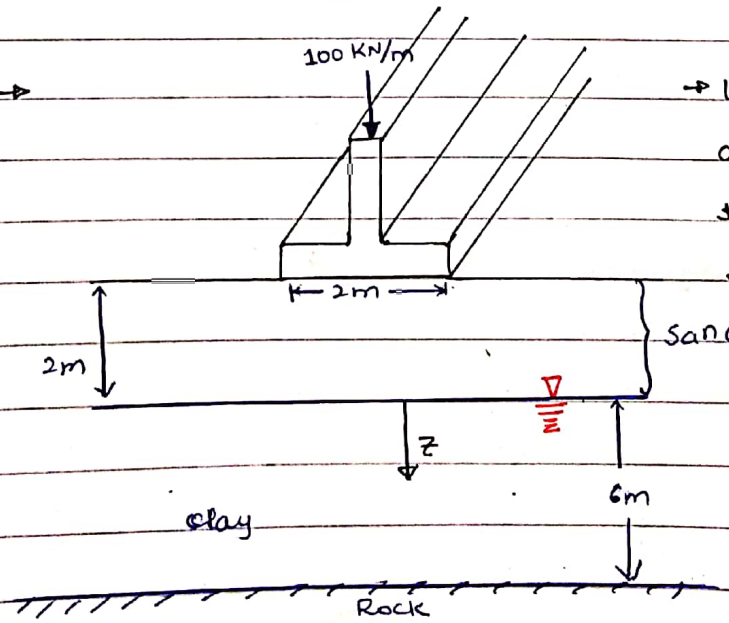
$$\Rightarrow \boxed{\Delta H = m_v \cdot \Delta \bar{\sigma} \cdot H} \Rightarrow \text{we use this formula.}$$

$$\frac{\Delta e}{1+e} = \frac{a_v \cdot \Delta \bar{\sigma}}{1+e} = \frac{\Delta H}{H}$$

$$\boxed{\Delta H = \frac{a_v}{1+e} \Delta \bar{\sigma} \cdot H}$$

⊙ The value of a_v and m_v of the soil is not a constant. It depends on the stress range.

Ques→



→ Load from the strip foundation as show in fig. is assumed to be distributed over the soil stratum as 2V : 1H

The value of m_v varies over the depth of clay stratum as $(22 + 1.2z^2) \times 10^{-5}$ m^2/KN.

z is the distance from top of clay layer. Calculate the consolidation of clay stratum assuming 1-D consolidation.

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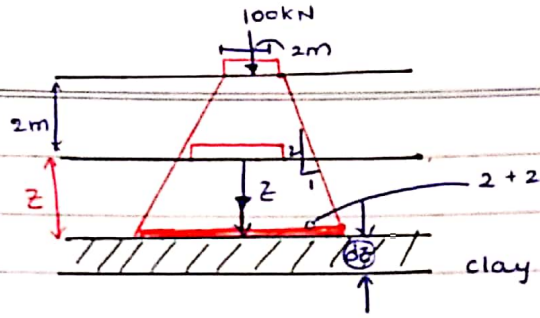
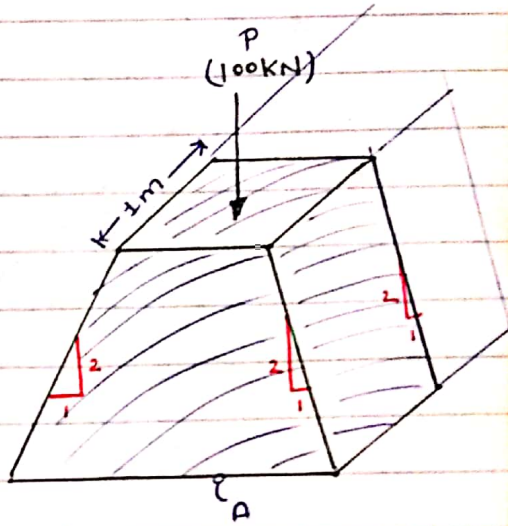
Soln:



$$\Delta H = \int d(\Delta H) = \int_0^{6m} m_v \cdot \Delta\bar{\sigma}\, dz$$

$$= \int_0^6 (22 + 1.2z^2) \times 10^{-5} \times \frac{100}{(4+z)\times 1}$$

$$\Delta H = 0.0305\,m = 30.5\,mm$$



$$\Delta\bar{\sigma} = \frac{P}{A}$$

Note: ⊙ Settlement can be approx. calculated by taking m_v and $\Delta\bar{\sigma}$ at the centre of clay layer and assuming the value to be constant throughout the clay layer.

m_v at centre of clay layer

$$= [22 + 1.2(3)^2] \times 10^{-5} \ \frac{m^2}{kN}$$

$$= 32.8 \times 10^{-5}\ m^2/kN$$

$\Delta\bar{\sigma}$ at centre of clay layer $= \frac{100}{(4+3)} = 14.28\ \frac{kN}{m^2}$

$$\Delta H = m_v \Delta\bar{\sigma} H = [32.8 \times 10^{-5} \times 14.28 \times 6]\,m$$

$$= 28.11 \times 10^{-3}\,m$$

$$= 28.11\,mm$$

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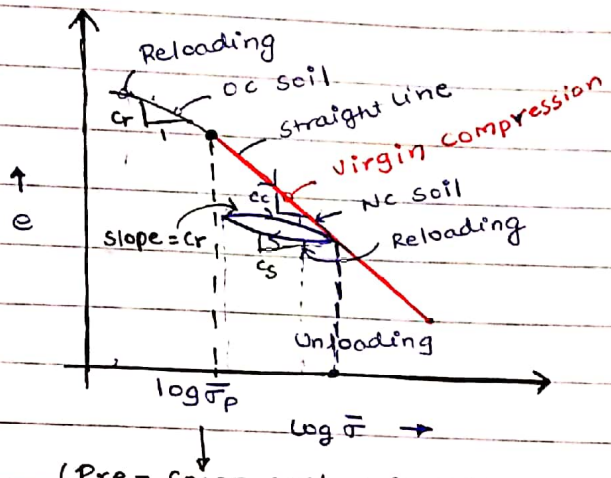
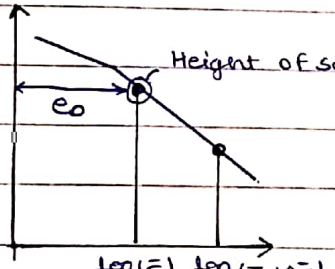
e v/s $\log \bar{\sigma}$ curve



(Pre-compression stress)

$$C_c = \frac{\Delta e}{\log(\bar{\sigma} + \Delta\bar{\sigma}) - \log\bar{\sigma}} = \text{Compression Index}$$



$$C_c = \frac{\Delta e}{\log_{10}\left(\frac{\bar{\sigma} + \Delta\bar{\sigma}}{\bar{\sigma}}\right)}$$

$$\frac{\Delta H}{H} = \frac{\Delta e}{1+e}$$

$$\Delta H = \frac{C_c H}{1+e} \log_{10}\left(\frac{\bar{\sigma} + \Delta\bar{\sigma}}{\bar{\sigma}}\right)$$

(Δe)

Valid only when $(\bar{\sigma})$ & $(\bar{\sigma} + \Delta\bar{\sigma})$ is in N.C range.

If soil is in OC range:

i.e; $\bar{\sigma}$, $\bar{\sigma} + \Delta\bar{\sigma}$ is in O.C range.

$$\Delta H = \frac{C_r H}{1+e} \log\left(\frac{\bar{\sigma} + \Delta\bar{\sigma}}{\bar{\sigma}}\right)$$

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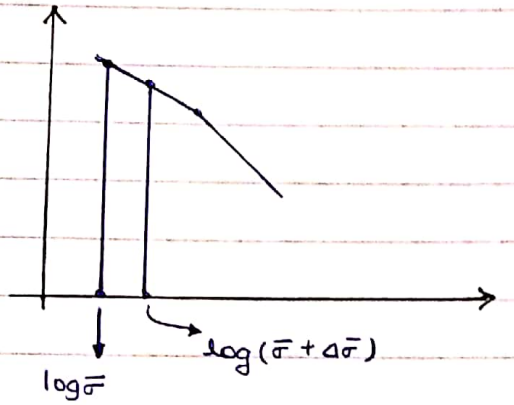

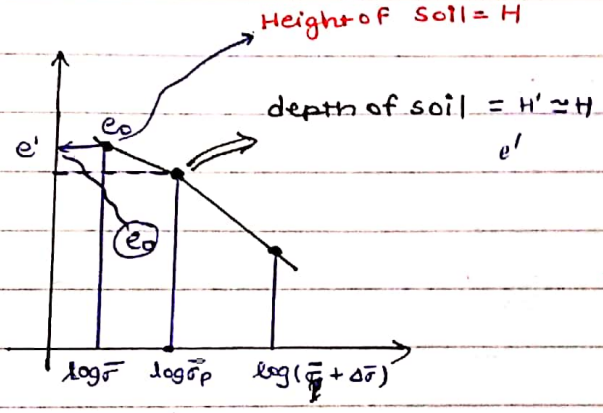

$$\Delta H = \frac{C_r \cdot H}{1+e_0} \log\left(\frac{\bar{\sigma}_p}{\bar{\sigma}}\right) + \frac{C_c H'}{1+e'} \log\left(\frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_p}\right)$$

Swelling:

$$\text{Swelling} = \frac{C_s H}{1+e_0} \log\left(\frac{\bar{\sigma}_1}{\bar{\sigma}_2}\right)$$

Note: ⊙ $\bar{\sigma}$ is calculated due to the weight of soil existing before the external load application at the centre of the clay layer. & $\Delta\bar{\sigma}$ is taken as stress increment due to external loading at the centre of clay layer.

⊙ For $\Delta\bar{\sigma}$ calculation, load dispersion is assumed at 2V : 1H from the base of the foundation upto the centre of clay layer and dispersion will be taken

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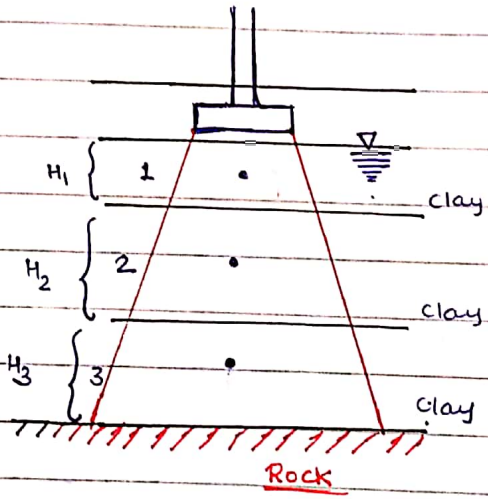
Strip footing → Dispersion → one direcn

Column footing → Dispersion → two direcn.

In one direcn only for strip footing, and in two direcn for column / isolated footing.

⊙ For uniform fill placed over large area, $\bar{\sigma}$ is taken as γh where r is the unit wt. of fill & h is the thickness of the fill for all depths of soil. i.e; there is no dispersion is assumed in case of uniform fill.

i.e; the uniform fill will be assumed as additional layer of soil.



⊙ Sometimes, if the thickness of clay layer is more then we can divide the clay layer into layers of smaller thickness and can calculate $\bar{\sigma}$ and $\Delta\bar{\sigma}$ at the centre of each of the sublayers and calculate the settlement of each of the sub layers and added up to obtain the total settlement.

⊙ If the clay layer continues upto the greater depth then we consider the clay only upto a depth of (2-3) times of b below the foundation.

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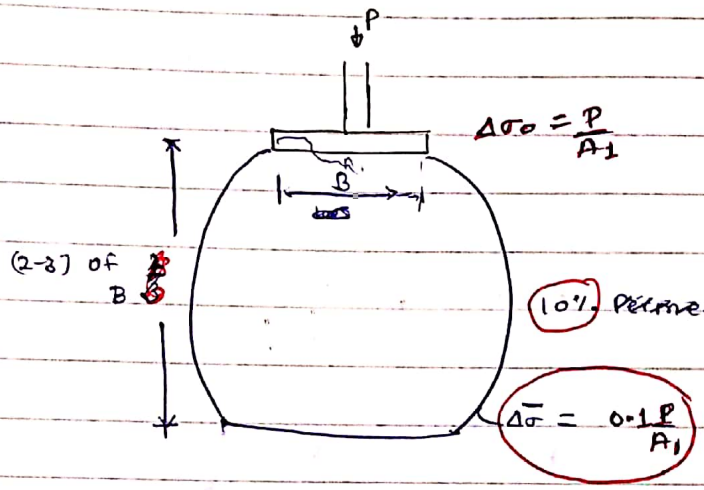
Field wala curve → more steeper

calculate the settlement of only that portion of clay.



Under Consolidated, Normal Consolidated & Over Consolidated Soil

⊙ Three relationship exist b/w the effective stress ($\bar{\sigma}$) and the maxm past effective stress ($\bar{\sigma_p}$):

(i) **Underconsolidated Soil** ($\bar{\sigma_p} < \bar{\sigma}$): ⊙ If the soil has not yet reached the eqm under present overburden due to the time required for consolidation.

Soil will be underconsolidation if the deposition rate of material is faster than rate of consolidation, rapid drop in the GWT

(ii) **Normally consolidated Soil** ($\bar{\sigma_p} = \bar{\sigma}$): ⊙ If the soil is in Effective stress eqm with the present overburden effective stress, the soil is said to be normally consolidated.

⊙ UC soil behaves as NC soil until the end of primary consolidation.

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(iii) Over consolidated soil ($\bar{\sigma}_p > \bar{\sigma}$) : If the soil has been consolidated before at a greater effective stress as compare to the present effect stress, the soil is said to be over consolidated or Pre consolidated soil.

Various Reasons for pre-consolidation are:

① Removal of overburden / Glaciers :

② Lowering of water table in past and seepage condition

③ Drying due to evaporation, vegetation (dessication)

④ Long term 2° consolidation / compression, creep or ageing.

⑤ Bonding due to ion exchange, thixotropy and weathering.

Degree of Over Consolidation:

It is represented by over consolidation ratio.

$$OCR = \frac{\bar{\sigma}_p}{\bar{\sigma}}$$

For UC soil ; $OCR < 1$

For NC soil ; $OCR = 1$

For OC soil ; $OCR > 1$

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Note: ⊙ If OCR > 2, the soil is heavily over consolidated.

⊙ HOCB soil is having volume increase tendancy on shear load application and hence the excess pore pressure developed is (-ve).

Imp. point:

⊙ e v/s $\log \bar{\sigma}$ curve for (NC) soil is straight line and for (OC) soil, it has convex upward curvature.

⊙ NC soil is more compressible than OC soil.

⊙ There are recoverable and non-recoverable components to the volume changes:

(i) Rearrangement of soil particles (Non-recoverable or plastic)

(ii) Elastic strain in particles (recoverable)

(iii) Compression of bound water layer (recoverable)

⊙ The swelling of soil on removal of load will be due to recoverable deformations and compression of soil in the OC range (recompression range) is due to the compression of recoverable portion of deformation.

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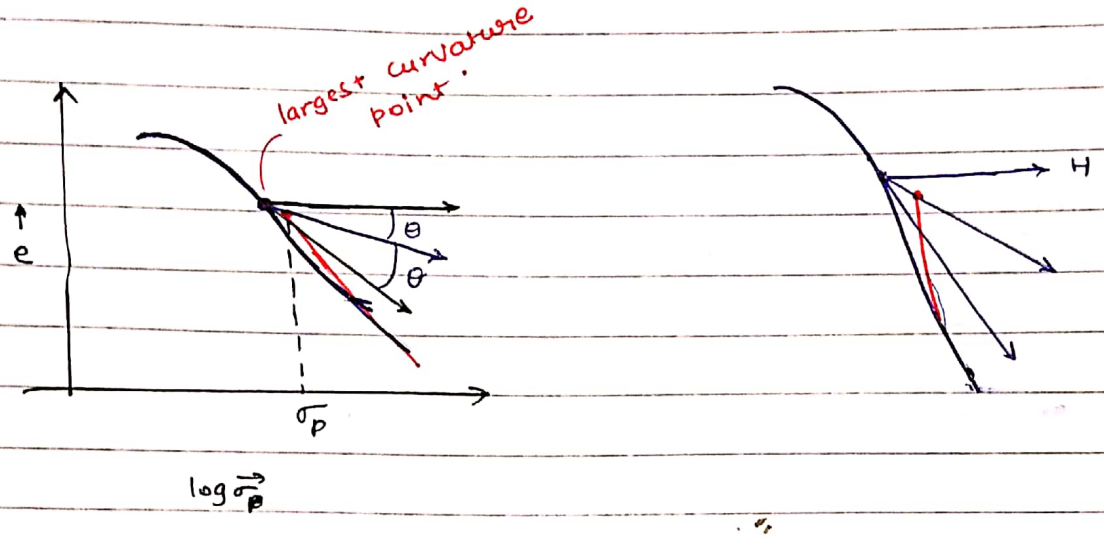
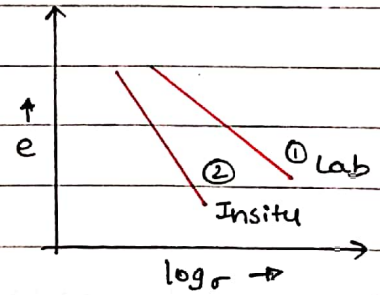
Determination of $\bar{\sigma}_p$



⊙ In situ e v/s $\log \bar{\sigma}$ curve is steeper than the laboratory e v/s $\log \bar{\sigma}$ curve.

This is because the laboratory curve is obtained for sample collected from field. in which unloading has taken place during sampling.



Hence, the laboratory curve will be a recompression curve leading to flatter curve.

⊙ Whenever possible, we shouldnot exceed the preconsolidation stress in the field. This will reduce the settlement of the structure.

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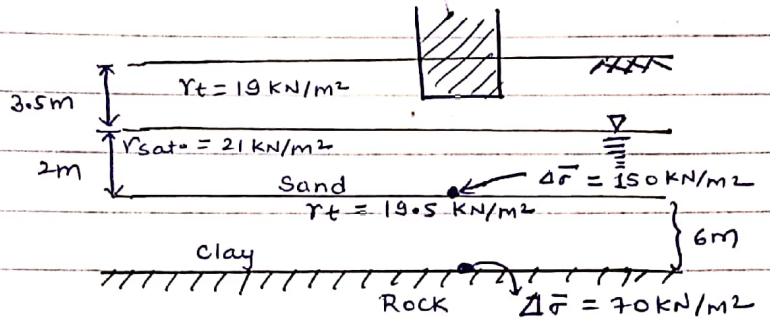
Ques→ It is estimated that additional stress on the clay layer due to the weight of building is 150 KN/M² at the top of the clay and 70 KN/M² at the Bottom of the clay. Compute the settlement of clay layer due to the construction of the Building.



Data from consolidation do test gets the following results:

Applied stress (KN/m²)	e
25	0.89
50	0.882
100	0.865
→ 117.95	→ 0.858
200	0.830
→ 227.95	→ 0.825
400	0.798
800	0.765

$$\frac{\Delta H}{H} = \frac{\Delta e}{1+e}$$

$\bar{\sigma}$ at the centre of clay layer before construction of building = $(3.5 \times 19) + 2(21 - 9.81) + 3(19.5 - 9.81)$

$= 117.95$ KN/M²

$\Delta\bar{\sigma}$ at the centre of clay layer due to construcn of Building $= \frac{150+70}{2} = 110$ KN/M²

$\Rightarrow e_1 = 0.865 - \frac{0.035}{100} \times 117.95 = 0.858$

⇒ Eff. Stress at the centre of clay layer after construcn of Building $= 117.95 + 110 = 227.95$ KN/M²

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$\Rightarrow e_2 = 0.83 - \left[\frac{0.032}{200} \times 27.95\right] = 0.825$

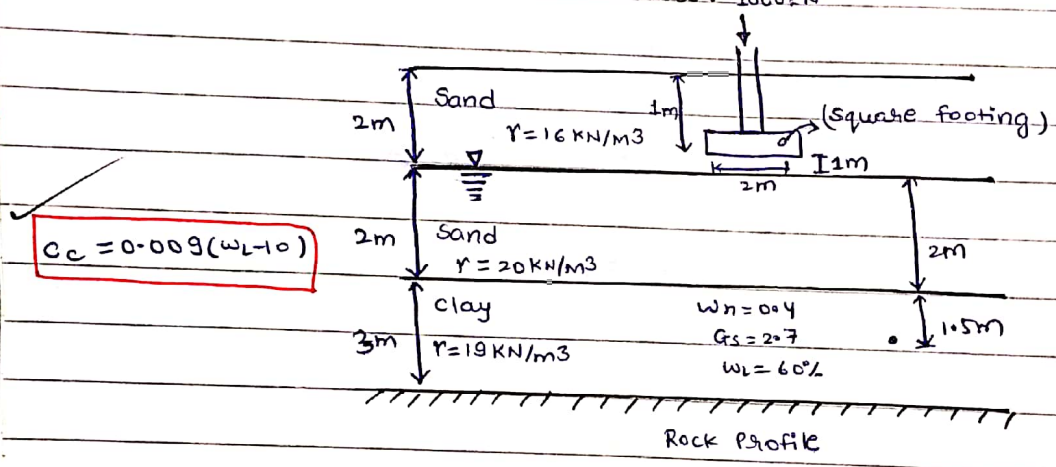
$\frac{\Delta e}{1+e_1} = \frac{0.858 - 0.825}{1.858}$

$= 0.017 = \frac{\Delta H}{6m}$

$\Delta H = 106.5\,mm$

~~$\Rightarrow e_2 = 0.865 - \frac{0.035}{100} \times 17.95 = 0.858$~~

Ques→ The subsoil profile of a proposed site of construction is shown below.



$C_c = 0.009(w_L - 10)$

Assume that post construction settlement of soil is negligible. Determine the consolidation settlement of clay layer on account of construction. The clayey soil is NC and assume 2V:1H load dispersion to estimate the stress increase in the clay layer.

Solun:

$$\Delta H = \frac{C_c}{1+e_0} \cdot H \cdot \log\left(\frac{\bar{\sigma} + \Delta\bar{\sigma}}{\bar{\sigma}}\right)$$

$C_c = 0.009(60-10) = 0.45$

$e_0 \times 1 = w_n \cdot G_s$

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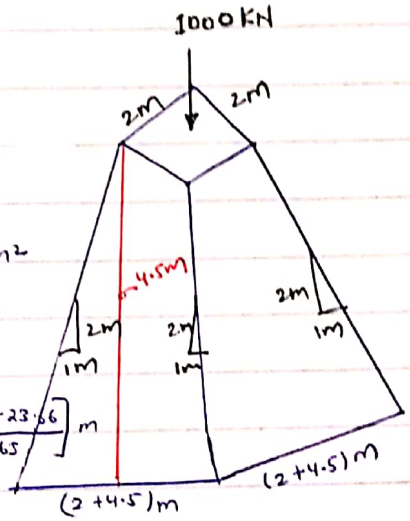
$e_0 = 0.4 \times 2.7 = 1.08$

$\bar{\sigma}\Big|_{\text{at centre of clay layer}} = 16\times 2 + (20-9.81)\times 2 + (19-9.81)\times 4.5 = 66.165 \frac{KN}{m^2}$

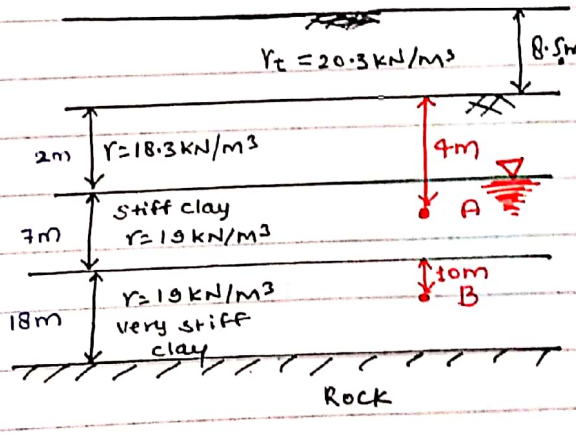


$\Delta\bar{\sigma}$ at the centre of clay layer $= \frac{1000}{(6.5)^2} = 23.66\ KN/m^2$

$\Rightarrow \Delta H = \left[\frac{0.45}{1+1.08} \times 3m \log \frac{66.165+23.66}{66.165}\right] m$

$\Delta H = 86.2mm$

Ques→



Sample-A	Sample-B
$a_c = 0.25$	0.2
$c_r = 0.08$	0.06
$e_0 = 0.66$	0.45
$\bar{\sigma}_c = 101 kPa$	510kPa

⊙A 8.5 m deep compacted fill is placed over a soil profile as shown in figure. Consolidation test on samples collected from point A and B produced the result as given in the table. Pre-consolidation

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Stress at other points can be calculated using the formula:

$$\bar{\sigma}_m = \bar{\sigma}_c - \bar{\sigma}$$

for each layers where $\bar{\sigma}_m$ is pre-consolidation margin which is constant for each layers.

$\bar{\sigma}$ = Eff. stress $\bar{\sigma}_c$ = Pre-consolidation stress at point under consideration.

→ Compute the ultimate consolidation due to the weight of the proposed fill.

Soln: Since, we have a proposed fill over a large area, there would be no dispersion of loading and effective stress increment $\Delta\bar{\sigma}$ will be calculated simply as : (8.5×20.3) KN/m²

$\Delta\bar{\sigma}$ for all depths = $8.5 \times 20.3 = 172.55$ KN/m².

Without proposed fill:

$\bar{\sigma}$ at the centre of 7m thick clay

$= 18.3 \times 2 + (19 - 9.81) \times 3.5$

$\bar{\sigma}$ at the centre of 18 m thick clay

$= 18.3 \times 2 + (19 - 9.81) \times 7 + (19 - 9.81) \times 9$

$= 183.64$ KN/m².

$(\bar{\sigma}_m)_A = (\bar{\sigma}_c)_A - (\bar{\sigma})_A$

$= 101 - [18.3 \times 2 + (19 - 9.81) \times 2] = 46.02$ KN/m²

$(\bar{\sigma}_m)_B = (\bar{\sigma}_c)_B - (\bar{\sigma}_B) = 510 - [18.3 \times 2 + (19 - 9.81) \times 7 + (19 - 9.81) \times 10]$

$= 317.17$ KN/m²

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$\bar{\sigma}_c$ at centre of 7m clay = 46.02 + 68.765 = 114.785 KN/m²

$\bar{\sigma}_c$ at centre of 18m clay = 317.17 + 183.64 = 500.81 KN/m²

At centre of 7m clay:

$\bar{\sigma}$ = 68.765 KN/m²

$\bar{\sigma} + \Delta\bar{\sigma}$ = 68.765 + 172.55 = ~~240.57~~ 241.31 KN/m²

$\bar{\sigma}_c$ = 114.785 KN/m²

$68.765 < \bar{\sigma}_c < 241.31$ ⇒ So, one is in NC and other is in OC.

At centre of 18m clay:

$\bar{\sigma}$ = 183.64 KN/m²

$\bar{\sigma} + \Delta\bar{\sigma}$ = 183.64 + 172.55 = 356.19 KN/m²

$\bar{\sigma}_c$ = 500.81 KN/m²

$\bar{\sigma}_c > 183.64$ $\bar{\sigma}_c > 356.19$ ⇒ So, both are in OC.

Settlement of 7m clay:

$$\Delta H_1 = \left(\frac{C_r H_1}{1+e_1} \log \frac{114.785}{68.765}\right) + \frac{C_c H_1'}{1+e_1'} \log \frac{241.31}{114.785}$$

0.08, 7, 0.075m, 7, 0.66

Settlement of 18m clay:

$$\Delta H_2 = \frac{C_r H_2}{1+e_2} \log \frac{356.19}{183.64} = 0.214m$$

18m, 0.06, 0.45

In 1st layer:

$H_1' = 7m - 0.075m = 6.925m$

$$\frac{\Delta e}{1+0.66} = \frac{0.075}{7}$$

$\Delta e = e_1 - e_1' = 0.0178$

$e_1' = 0.6422$

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7m

$$\frac{C_c H_1}{1+e_1} \log \frac{241.31}{114.785} = 0.3401\ m$$

$C_c = 0.66$, $e_1 = 0.925$

$$\frac{C_c H_1'}{1+e_1'} \log \frac{241.31}{114.785} = 0.3401\ m$$

$e_1' = 0.6422$

$$\Rightarrow \Delta H = \Delta H_1 + \Delta H_2$$
$$= 0.075 + 0.3401 + 0.214$$
$$= 0.6291\ m$$

Ques→ A layer of clay 8m thick lies b/w two layers of sand, the upper sand layer extends from the ground level to a depth of 4m. water table being at a depth of 2m. The lower sand layer is under artesian pressure and the piezometric layer being 6m above G.L. $\gamma_{sat.}$ of clay and sand are both 20 kN/m³. m_v for clay = $9.4 \times 10^{-4}\ m^2/kN$. Due to pumping from the artesian layer, the piezometric level falls by 3m. find the Consolidation settlement.

Soluⁿ:

$$\Delta H = m_v \cdot \Delta\bar{\sigma} \cdot H$$

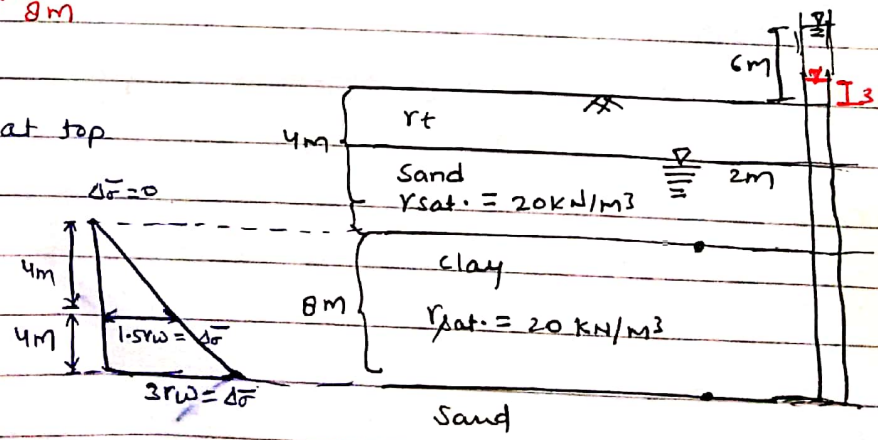
$m_v = 9.4 \times 10^{-4}$, $\Delta\bar{\sigma} = (1.5 \times 9.81)$, $H = 8m$

Change in eff. stress at top of clay layer = 0

$\Delta\bar{\sigma} = 0$

Change in eff. stress at bottom of clay layer

$(\sigma - 15\gamma_w) - (\sigma - 18\gamma_w) = 3\gamma_w$



Artesian Prone.

$$\Rightarrow \Delta H = 0.1106\ m$$

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SECONDARY CONSOLIDATION SETTLEMENT

$$\frac{\Delta H}{H} = \frac{\Delta e}{1+e_o}$$

$$\Delta H = \frac{C_\alpha H_o}{1+e_o} \log \frac{t_2}{t_1}$$

$$C_\alpha = \frac{\Delta e}{\log\left(\frac{t_2}{t_1}\right)}$$

C_α → Secondary compression Index.

H_o → Thickness of clay layer at the end of 1° settlement

e_o → void ratio at the end of 1° consolidation

Note: ⊙ For preliminary investigation, certain results are used as follows:

$$C_c = 0.009 (w_L - 10)$$

w_L → liquid limit in %age.

↓

For medium sensitivity of clay (S<4)

⊙ For remoulded clay, $C_c = 0.007 (w_L - 7)$

⊙ $C_c = 1.15 (e_o - 0.35)$

e_o → void ratio before consolidation

⊙ $C_c = 1.15 \times 10^{-2} \times w$ → for organic soils.

(water content)

⊙ For Most of the NC Soil for medium sensitivity, C_c is b/w (0.2-0.5)

For organic clay, C_c → (>4)

For Peat (Highly organic) C_c → (10-15)

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C_v → Not constant.

Time Rate of Consolidation

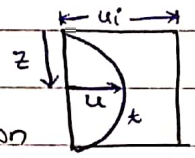
⊙ The rate of settlement is directly related to the rate of dissipation of pore water pressure.

Terzaghi predicted the time rate of consolidation for 1-D consolidation using a mathematical theory.

$$C_v \cdot \frac{\partial^2 u}{\partial z^2} = \frac{\partial u}{\partial t} - \frac{\partial \sigma_v}{\partial t}$$ ← Total stress

If total stress is constant with time:

$$\frac{\partial u}{\partial t} = C_v \cdot \frac{\partial^2 u}{\partial z^2}$$



C_v → coeff. of consolidation

$$C_v = \frac{a_v}{m_v r_w}$$

u → Excess pore water pressure

Basic Assumptions in Terzaghi 1-D consolidation theory:

1. Soil is homogeneous, fully saturated and soil particles and water are incompressible.
2. Compression and pore water flows are 1D (vertical) } He neglected immediate settlement
3. Strains are small.
4. coefficient of Permeability (K_v), coefficient of volume compressibility (m_v) and coeff. of compressibility (a_v) are constant throughout the soil at every stage of consolidation.
5. Darcy's law is valid at all hydraulic gradients.

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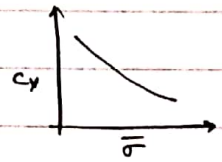
⑥ There is a unique relationship independent of time b/w the void ratio and the effective stress.

⑦ Secondary consolidation is Neglected.

Note: ⊙ As per Terzaghi's theory, m_v and k_v are constant. So, c_v is also constant.

But in reality, c_v depends on stress range.



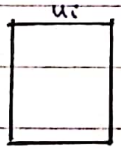
⊙ Solun of Terzaghi's one-dimensional consolidation eqn can be done using Fourier series analysis and the progress of consolidation can be shown by plotting a series of curves b/w u (excess pore water pressure), z and t.

These curves are called Isochrones.

⊙ The form of the curve depends on:

(i) Initial distribution of excess pore water pressure.
(ii) Drainage condⁿ at the boundary.

Initial distribution of Excess Pore Water Pressure

1. 

→ when loaded area is large relative to the thickness of consolidating layer.

2.
→ When the consolidating thickness layer is more.

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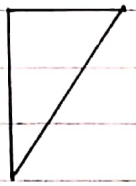
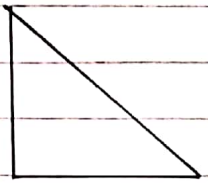
3. 

→ When the consolidating thickness is very large or pumping of water from aquifer lying beneath the compressible layer.

4. 

→ Consolidation of Hydraulically placed layer acted on by no force other than their own weight.

⊙ In our course, we will discuss only constant initial excess pressure throughout the depth.

⊙ Solun of Terzaghi's 1-D consolidation eqn using dimensionless parameters ~~are~~ is done.

These dimensionless parameters are defined as under:

⊛ $Z = \frac{z}{H}$ = Drainage Path ratio

⊛ $T_v = \frac{c_v t}{H^2}$ = Time factor

⊛ $U_z = \frac{u_i - u_z}{u_i}$ = Degree of Consolidation at any depth z.

z → ~~depth~~ dist. of any point from the top of clay layer.

H → Maxm dist. that water has to travel to reach the drainage face.

i.e; the length of longest drainage path.

For double drainage condn.

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⊙ for double drainage condn:

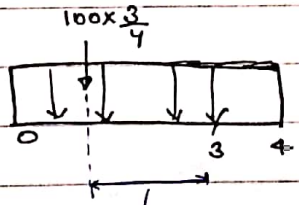
$$H = \frac{\text{depth of clay layer}}{2}$$

⊙ for single drainage condition :

$$H = \text{depth of clay layer.}$$

t → time since the start of consolidation.

⊙ This time could be measured from the midpoint of construction duration. i.e; full instantaneous loading will be assumed at the midpoint of construction duration.



100kN = Total load during construction period.

$\frac{t}{2} = 1.5$ years → Hence for this duration of load = 75 kN we will calculate the settlement & it will be the settlement at the end of 3 years.

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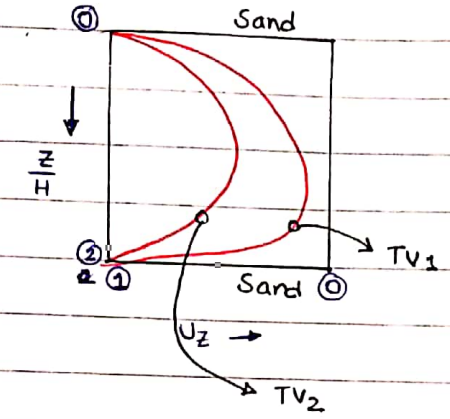
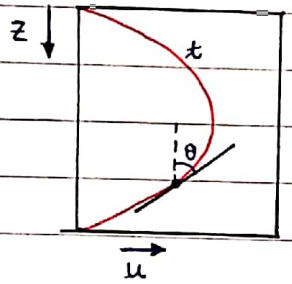
Isochrones in terms of Dimensionless Parameters





$$\frac{\partial v}{\partial z} = \tan\theta$$

consider $h \rightarrow$ ~~total~~ Head

$$r\frac{\partial h}{\partial z} = \tan\theta$$

$$i = \frac{\partial h}{\partial z} = \frac{1}{r}\frac{\partial v}{\partial z} = \text{Hydraulic Gradient}$$

flow → in the decreasing total head direction.

$$U_z = \frac{u_i - u}{u_i}$$

$$Z = \frac{z}{H}$$

$$\partial Z = \frac{1}{H}\cdot \partial z$$

$$\partial U_z = \frac{1}{u_p}\cdot(-\partial u)$$

$$\frac{\partial U_z}{\partial z} = \frac{-\partial u}{u_i \cdot \frac{\partial z}{H}} = \frac{-H}{u_i} \times r\frac{\partial h}{\partial z}$$

$$\Rightarrow \quad \boxed{i = \frac{-u_i}{Hr}\cdot\frac{\partial U_z}{\partial z}}$$

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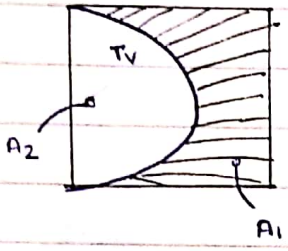
Average Degree of consolidation



$U = \frac{A_1}{A_1 + A_2}$ = Avg. degree of consolidation of the complete clay layer.

$U = f(T_v)$

$U \leq 60\%$.

$T_v = \frac{\pi}{4} U^2$; U in fraction

$U > 60\%$.

$T_v = 1.781 - 0.933 \log_{10}(100 - U\%)$

U%	Tv
0	0
50	0.197
60	0.2827
90	0.848

$T_v = \frac{C_v t}{H^2} \Rightarrow t = \frac{T_v}{C_v} H^2$

Note: ⊙ for a particular degree of consolidation in a soil, Tv and Cv are fixed. Hence, $t \propto H^2$

⊙ Thus, in a particular soil, for a particular loading, time required for a particular degree of consolidation in

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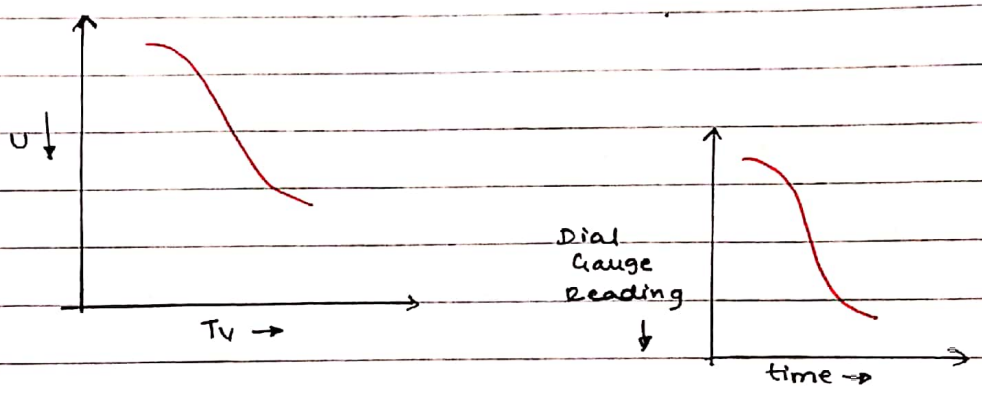
Single drainage is 4 times that is in case of double drainage.

Determination of Cv

① The curve of deformation (dial gauge reading) v/s time obtained from oedometer test is similar to U v/s Tv curve.



② This property is used to determine Cv from oedometer test using curve fitting technique.

Two methods are used:

1. Casagrande's logarithm of time fitting method.
2. Taylor's square root of time fitting method.

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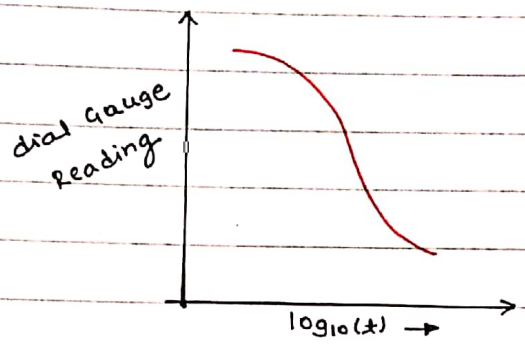
Casagrande's Logarithm of time fitting Method



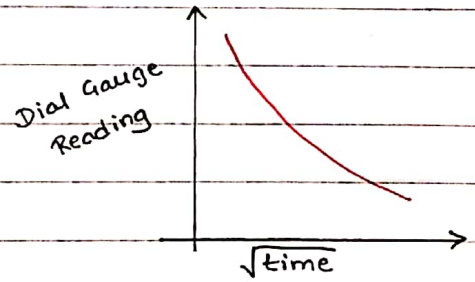
find t_{50}

$$\frac{C_v \cdot t_{50}}{H^2} = T_{v50} = 0.197$$ ★★

⇓

find C_v.

Taylor's Square root of time fitting Method



find t_{90}

$$\frac{C_v \cdot t_{90}}{H^2} = T_{v90} = 0.848$$ ★★

⇓

find C_v.

Note: ⊙ $C_v \propto \frac{1}{w_L}$ $(w_L \uparrow \rightarrow C_v \downarrow)$

⊙ $C_v \propto \frac{1}{PI}$

⊙ $C_v \downarrow$ with increase in ↑ Compressibility of soil

$$C_v \propto \frac{1}{\text{compressibility}}$$

⊙ In over-consolidated soil, faster dissipation occurs.

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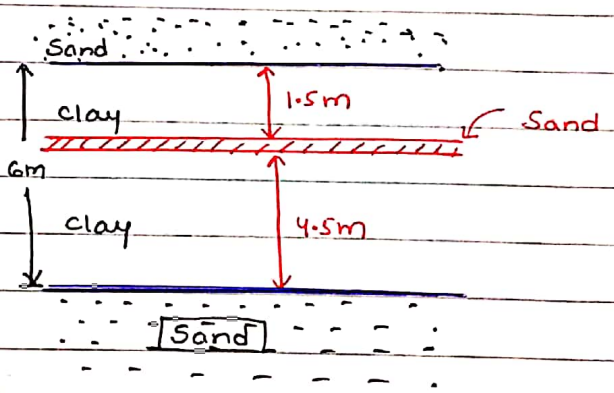
- ⊙ In Normal consolidated soil, → slow dissipation occurs
- ⊙ Organic soil → slow dissipation
- ⊙ Greater Kv → Faster dissipation.

Ques→ A clay layer 6m thick having double drainage settles by 30 mm in 3 years after it has been subjected to some load. The final consolidation settlement is calculated as 120 mm. If a layer of sand of negligible thickness is to be present at a depth of 1.5m from the top of clay layer, what will be the final consolidation settlement of the clay layer and what will be the settlement after 3 years.



Solun: Addition of additional drainage face will only increase the rate of settlement because the maxm distance, that the water has to travel to reach the drainage face gets reduced.

It cannot change the total settlement because total settlement depends only on stress increment, compressibility of soil, thickness of soil layer.

It is independent of drainage condⁿ.

⇒ final settlement will remain = 120mm

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⊙ Total settlement ∝ thickness of clay layer

Hence, top 1.5 m layer will have final settlement

of 1.5m layer $= \frac{120}{6} \times 1.5 = 30\text{mm}$

and final settlement of 4.5 m layer

$= \frac{120}{6} \times 4.5 = 90\text{mm}$

$T_v = \frac{C_v t}{H^2}$

$T_v \propto \frac{1}{H^2} \Rightarrow \frac{T_{v_1}}{T_{v_2}} = \frac{H_2^2}{H_1^2}$

originally ; $H = 3\text{m}$

for 1st layer ; $H = 0.75\text{m}$

$$\frac{(T_v)_{1st\ layer}}{(T_v)_{original}} = \frac{(3)^2}{(0.75)^2}$$

$(T_v)_{1st\ layer} = 16 \times (T_v)_{original}$.

$(T_v)_{original} = \frac{\pi}{4}(0.25)^2$

$(T_v)_{1st\ stage} = 16 \times \frac{\pi}{4}(0.25)^2 = 0.785 > 0.263$

$\Rightarrow U > 60\%$

$\Rightarrow 0.785 = 1.781 - 0.933 \log_{10}(100 - U_1)$

$\Rightarrow$ **$U_1 = 88.31\%$**

$$\frac{(T_v)_{2nd\ layer}}{(T_v)_{original}} = \frac{(3)^2}{(2.25)^2}$$

$(T_v)_{2nd\ layer} = \left(\frac{3}{2.25}\right)^2 \times \frac{\pi}{4}(0.25)^2 = 0.0873 < 0.283$

$U < 60\%$

$\frac{\pi}{4} U_2^2 = 0.0873 \Rightarrow U_2 = 0.33$ i.e; (33%)

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Total settlement in 3 yrs.

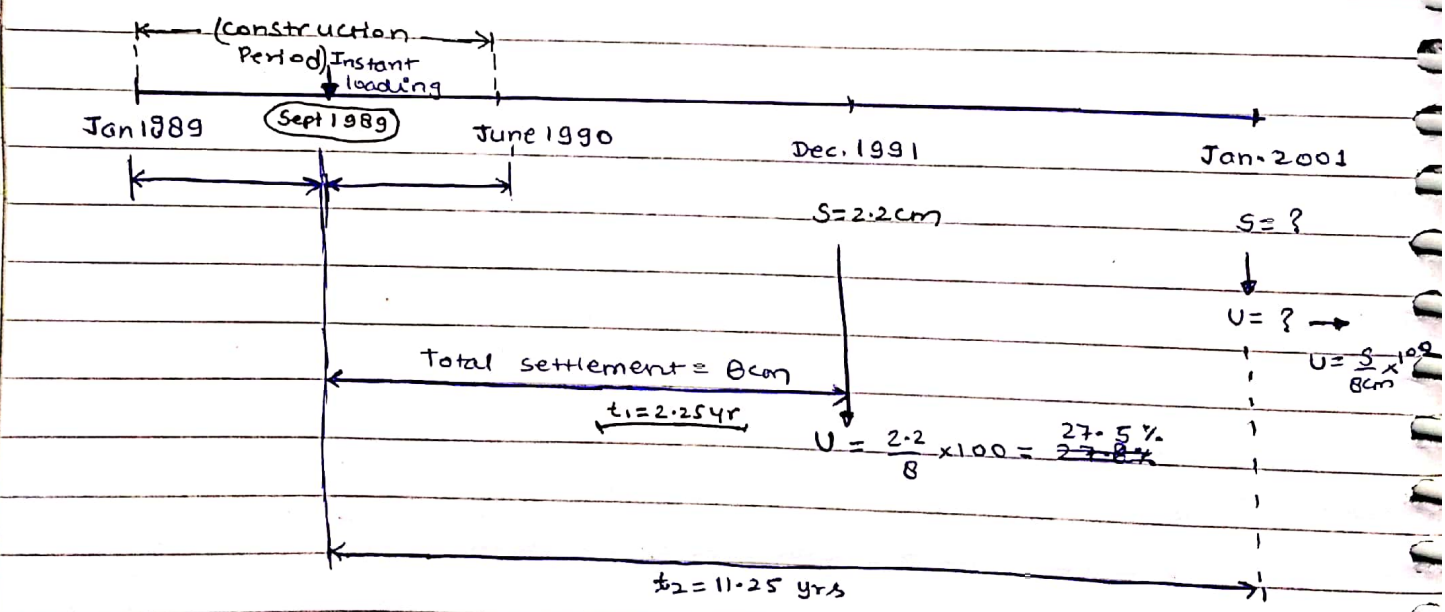
$= U_1 \times 30mm + U_2 \times 90mm$

$= 0.8631 \times 30 + 0.33 \times 90$

$= 56.49mm.$

Ques→ Construction of a multistorey building is started in Jan. 1989. and was completed in June 1990. The total consolidation settlement of the building was estimated to be 8cm. The avg. settlement of the building was measured in Dec 1991 & it was found to be 2.2cm. Compute the probable settlement of the building in Jan. 2001.

Soln:



$$\frac{C_v t_1}{H^2} = T_{V_1} = \frac{\pi}{4} U_1^2$$

0.275

$$\frac{C_v t_2}{H^2} = T_{V_2} = \frac{T_{V_1}}{t_1} \cdot t_2 = \frac{\pi}{4}(0.275)^2 \times \frac{11.25}{2.25} = 0.296 > 0.283$$

$\Rightarrow U_2 > 60\%$

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Compressibility ↑ ⇒ consolidation time ↑

Permeability ↑ = consolidation time ↑

$\Rightarrow 0.296 = 1.781 - 0.933 \log_{10}(100 - U_2)$

$U_2 = 61.09\%$

$\frac{S}{8} \times 100 = 61.09$

$\Rightarrow S = 4.88\ cm$

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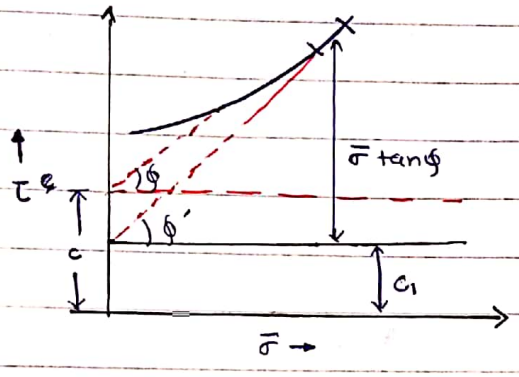
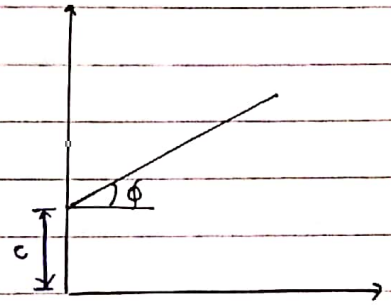
SHEAR STRENGTH OF SOIL

Undrain → total stress formula used.





$\tau = c + \bar{\sigma}\tan\phi$

$\tau = c_1 + \bar{\sigma}\tan\phi'$

- Shear strength is the ~~stress~~ capacity to resist shear stress.
- Generally, soil loaded in compression fails due to shear not due to the crushing of particles.
- Various strength properties of soil like bearing capacity, earth pressure, slope stability etc. depends on shear strength of soil

Mechanism of Shear Resistance
(Shear strength)

- Shear strength is catagorized as:

(a) Frictional strength (b) Cohesive strength.

- Frictional strength ~~effe~~ accounts for grain to grain contact friction and interlocking b/w particles.

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(c, ϕ) → depend on soil type and field conditions

⊙ cohesive strength accounts for:

(i) True cohesion b/w particles
(ii) Apparent cohesion b/w particles.

⊙ True cohesion is due to:

(A) Cementation
(B) Electrostatic and Electromagnetic attraction
(C) Primary Valence Bonding and adhesion

⊙ Apparent cohesion is generated due to capillary stresses.

⊙ Apparent cohesion leads to attraction b/w particles in a partially saturated soil.
It is in reality, a frictional stress. This friction stress is not due to external loading.

⊙ For saturated soil, apparent cohesion would be zero.

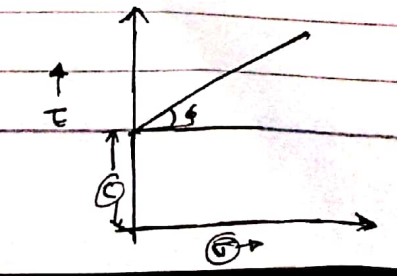
Mohr's Hypothesis

$\tau_{ff} = f(\sigma_{ff})$

⊙ Shear strength on the failure plane at failure is a unique func^n of normal stress on that plane.

Columb's Hypothesis

⊙ As per columb: $\tau_{ff} = c + \sigma_{ff} \tan\phi$



c, ϕ → shear strength parameter of soil called total stress shear strength parameter.

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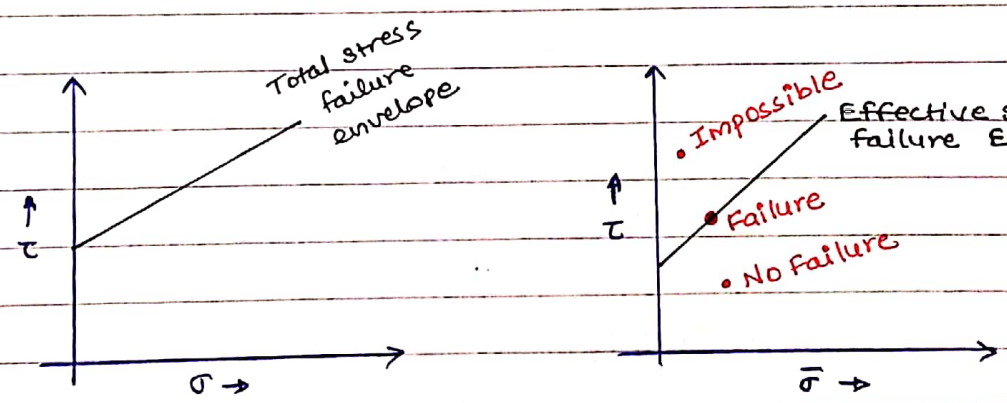
◉ After knowledge of Effective stress, Shear Strength is represented as:

$$\tau_{ff} = c' + \bar{\sigma}_{ff} \tan\phi'$$

c', ϕ' → Effective stress shear strength parameters.

Mohr - Coulumb Criteria

◉ It can be shown that if a series of Mohr circles are drawn corresponding to different tests carried out on different samples of a soil upto failure, then a the common tangent drawn ~~to~~ on these circles represents the failure Envelope.



◉ If the normal and shear stresses on soil mass on any plane gets plotted in the failure envelope, the soil will fail in ~~the~~ shear on that plane

Sign convention for drawing Mohr Circle

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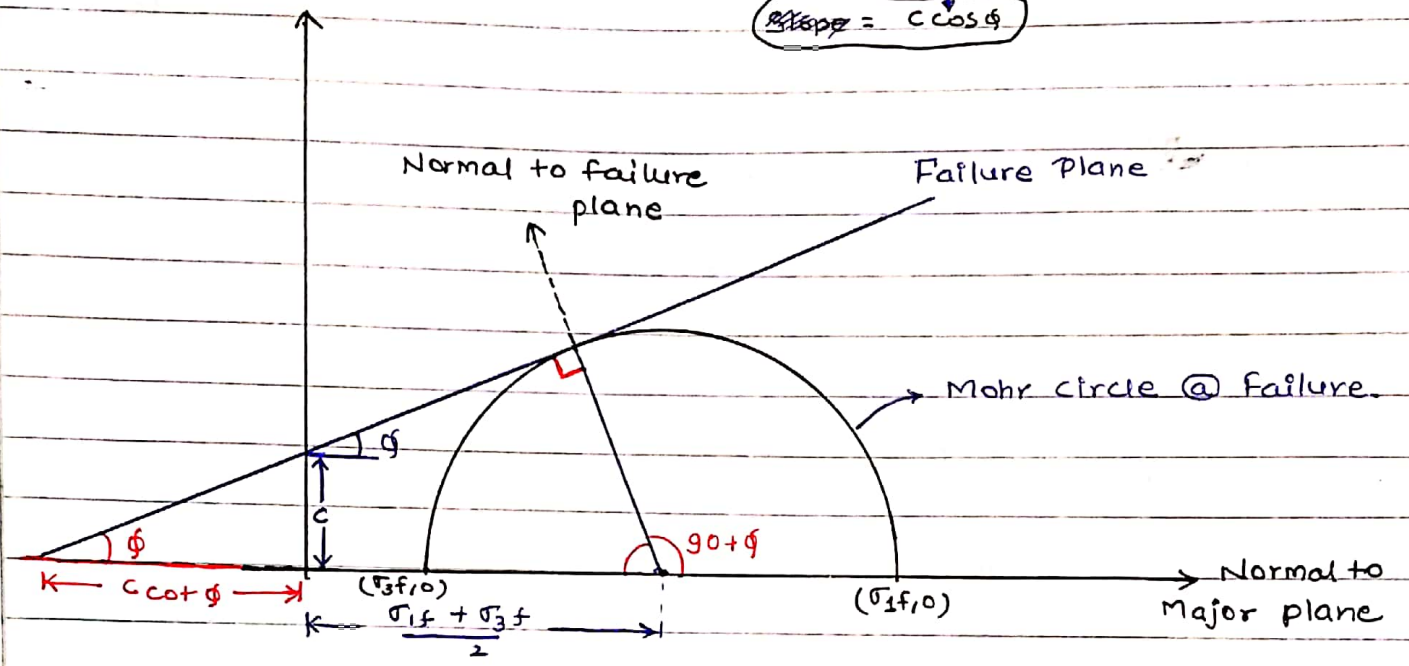
In (p-q) plot

Intercept

Slope = $c\cos\phi$



Slope = $\sin\phi$

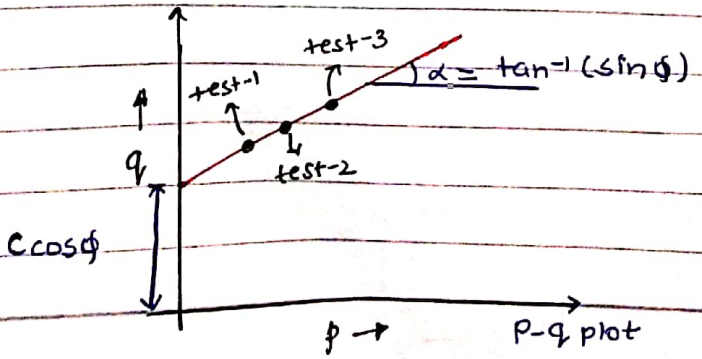
$$\sin\phi = \frac{R}{c\cot\phi + \frac{\sigma_1 f + \sigma_3 f}{2}}$$

$$\sin\phi = \frac{\frac{\sigma_1 f - \sigma_3 f}{2}}{c\cot\phi + \frac{\sigma_1 f + \sigma_3 f}{2}}$$

$$\frac{\sigma_1 f - \sigma_3 f}{2} = c\cos\phi + \frac{\sigma_1 f + \sigma_3 f}{2}\sin\phi$$

$$\frac{\sigma_1 f - \sigma_3 f}{2} = q \qquad \frac{\sigma_1 f + \sigma_3 f}{2} = p$$

$$\boxed{q = c\cos\phi + p\sin\phi}$$



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P-q plot: can also be used to find out c and ϕ value of soil.

$$\sigma_{1f} = \sigma_{3f}\left(\frac{1+\sin\phi}{1-\sin\phi}\right) + 2c\sqrt{\left(\frac{1+\sin\phi}{1-\sin\phi}\right)} \quad \star\star\star$$

$$\sigma_{3f} = \sigma_{1f}\left(\frac{1-\sin\phi}{1+\sin\phi}\right) - 2c\sqrt{\frac{1-\sin\phi}{1+\sin\phi}}$$

$\frac{1+\sin\phi}{1-\sin\phi} = N\phi$ = flow factor

$$\sigma_{1f} = \sigma_{3f} N\phi + 2c\sqrt{N\phi} \quad \star\star\star \quad \text{(A)}$$

$$N\phi = \tan^2(45 + \phi/2)$$

Note: ⊙ from the test results given we are have 3 types/ways to find out c and ϕ values:

(i) Using eq-(A) (in the exam)

↳ When the results for two test are given.

(ii) When the results for minimum 3 tests are given we can use:

(a) Common Tangent method on Mohr Circle

(b) P-q plot

In p-q plot, slope of line is taken as $\tan^{-1}(\sin\phi)$ and intercept is taken as $c\cos\phi$.

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Ques→ Two identical specimen of a backfill soil when tested in the triaxial compression test gives the following stresses at failure.

Sample No.	$\sigma_3 f$ (kg/cm²)	$\sigma_1 f$ (kg/cm²)	Pore water Pressure
1	1.0	3.2	0.6
2	2.0	5.6	1.1

Find out the total stress and effective stress shear strength parameter of soil and also calculate major principle stress at which soil will fail when the minor principle stress = 2.8 kg/cm² (total stress)

Soln: $\bar{\sigma}_{3f} = \sigma_{3f} - u_f$ $\quad\quad$ $\bar{\sigma}_{1f} = \sigma_{1f} - u_f$

0.4 $\quad\quad$ 2.6

0.9 $\quad\quad$ 4.5

① Total Stress shear Strength parameter.

$$3.2 = 1 N\phi + 2c\sqrt{N\phi}$$

$$5.6 = 2 N\phi + 2c\sqrt{N\phi}$$

$$N\phi = 2.4 = \tan^2\left(45 + \frac{\phi}{2}\right)$$

$$\Rightarrow \phi = 24.31^\circ$$

$$3.2 = 1 \times 2.4 + 2c\sqrt{2.4}$$

$$\Rightarrow c = 0.258 \text{ kg/cm}^2$$

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② Effective Stress Shear Strength parameter

$$2.6 = 0.4\, N\phi' + 2c'\sqrt{N\phi'}$$

$$4.5 = 0.9\, N\phi' + 2c'\sqrt{N\phi'}$$

$$0.5\, N\phi = 1.9$$

$$N\phi = 3.8$$

$$\tan^2\left(45 + \frac{\phi}{2}\right) = 3.8$$

$$\boxed{\phi' = 35.68^\circ}$$

$$2.6 = 0.4 \times 3.8 + 2c'\sqrt{3.8}$$

$$\boxed{c' = 0.277}\ kg/cm^2$$

$$\sigma_{1f} = 2.8 \times 2.4 + 2 \times 0.258\sqrt{2.4}$$

$$\sigma_{1f} = 7.519\ kg/cm^2$$

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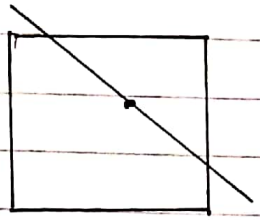
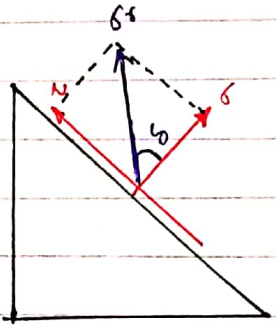
Maximum Obliquity Relationship

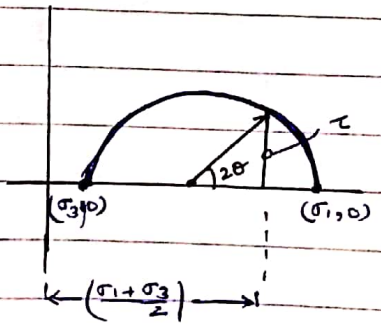




δ = Angle of obliquity.

- Angle of obliquity is the angle blw the resultant stress on a plane and the normal stress on the plane.

Failure plane is the one on which angle of obliquity is maximum (for cohesionless soil)



$\left(\frac{\sigma_1-\sigma_3}{2}\right)$ → Radius

$$\tau = \frac{\sigma_1-\sigma_3}{2}\sin 2\theta$$

$$\sigma = \frac{\sigma_1+\sigma_3}{2} + \frac{\sigma_1-\sigma_3}{2}\cos 2\theta$$

$$\tan\delta = \frac{\tau}{\sigma}$$

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⊙ Determination of c and ϕ:

⊙ c and ϕ are not the inherent properties of soil. They are related to the type of test and the condⁿ under which the test are performed.

⊙ The test performed to determine c and ϕ simulates the field condition.

⊙ The field conditions could be:

(i) Rapid or slow construction.
(ii) Drained or Undrained condition.
(iii) Type of soil.

⊙ The drained and undrained condⁿ depends on the type of the loading and the soil type and these are relative terms.

If the rate of loading is greater than the rate of dissipation of Excess pore water pressure, the condⁿ is undrained and excess pore pressure will exist.

However, if the rate of loading is lesser then the maxm possible rate of excess pore pressure dissipation., we will have drained condition and excess pore pressure will not exist. Only Hydrostatic pressure will exist.

⊙ If we can measure the effective stress in the field then to calculate the shear strength parameter, we use the effective stress approach and c' and ϕ' are

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NC → Compression Tendancy.
DC → Dilation Tendancy

+ve u ⇒ undrained test

-ve u → drained test.

---

calculated in the lab. and shear strength in the field is calculated as:

$$\tau = \bar{c}' + \bar{\sigma} \tan\phi'$$

$\bar{\sigma}$ = Eff. Stress in the field.

⊙ If we cannot measure the effective stress in the field then we use total stress approach and find c and ϕ in the laboratory and shear strength in the field is taken as:

$$\tau = c + \sigma \tan\phi$$

⊙ $\bar{\sigma}$ in the field is taken as:

$\bar{\sigma} = \sigma - u_h - u_e$ → Undrained (u_e = Excess Pore water Pressure)

$\bar{\sigma} = \sigma - u_h$ → Drained (u_h = Hydrostatic Pore Water Pressure)

u_h → hydrostatic Pore Water Pressure.

u_e → Excess Pore Water Pressure

⊙ If the application of loading leads to +ve pore water pressure, depend development then critical condⁿ is the immediately after construction condⁿ and undrained analysis is performed.

However, if -ve pore pressure develops due to load application, critical condⁿ is the long term condⁿ and drained analysis is performed corresponding to long term loading.

Note: ⊙ NC clays and loose sands compresses on shearing and hence on shear load application under undrained condⁿ, +ve excess pore pressure develops.

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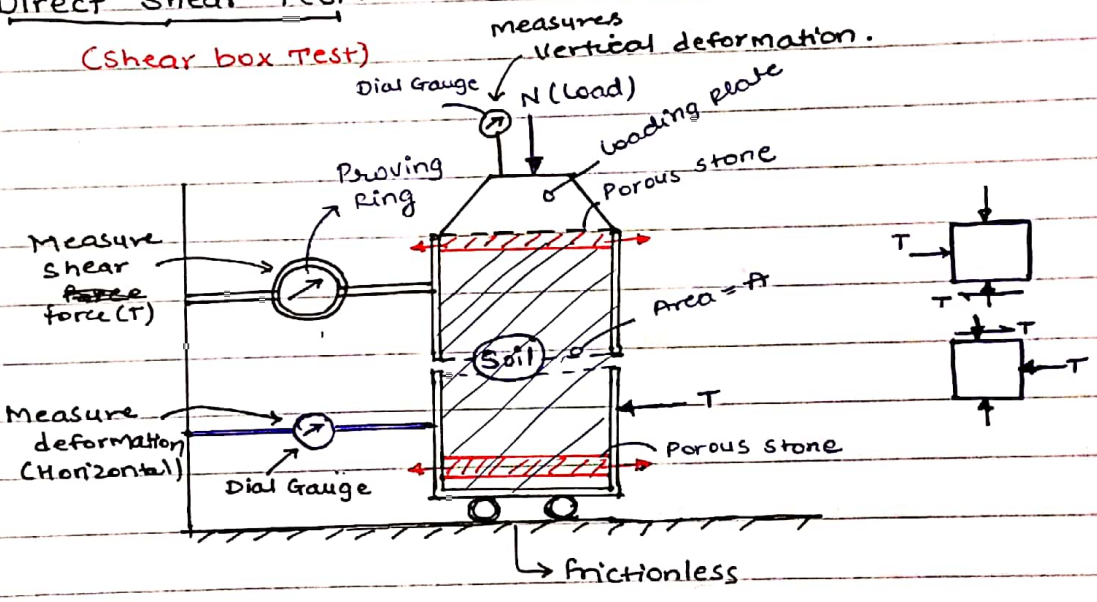
⊙ In Heavy OC clay (OCR>2) and in dense Sand under undrained condn, there is dilation tendancy and hence (-ve) excess pore pressure develops.

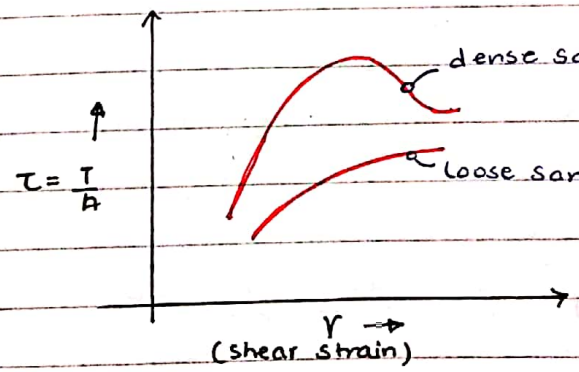
Various lab Test for find c & ϕ value

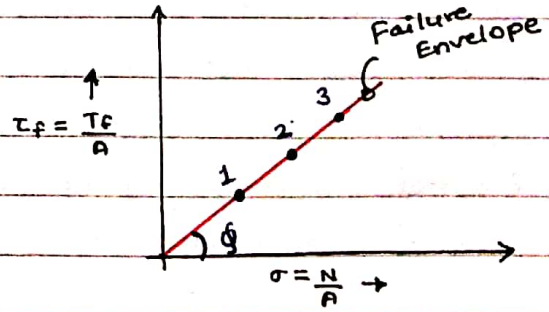
① Direct Shear Test: → Always take Effective (ϕ', $\bar{\sigma}$, c')

(Shear box Test)



For a Particular ($\sigma = N/A$)





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Note: ⊙ Normal load in the Shear Box test simulates the effective vertical stress in the field.

⊙ Vertical load is applied through vertical plate and soil is sheared gradually by applying horizontal force.

⊙ Shear is normally applied at constant rate of strain strain control test is better because stress-strain characteristic can be easily obtained in these tests as the shape of stress-strain curve beyond peak point can be observed only in the strain controlled Test, and strain controlled test is also easy to perform.

⊙ The shear deformation and vertical deformation are measured during the test using dial-Gauge.
Magnitude of shear load is measured using proving ring.

⊙ Normal stress (σ) = $\frac{N}{A}$ and Shear Stress (τ) = $\frac{T}{A}$

Where A is the Nominal area of c/s of sample and
N is the Normal load
T is the shear load.

⊙ As drainage can't be controlled in this test. Hence, rate of loading should be such that the pore water pressure doesnot develop.
i.e; we will be performing only drained (controlled consolidation) testing.

⊙ This test is thus good for free draining soil like Sand and Gravel.

⊙ This test can be used for performing drained test for clay also.

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- This is because sample is smaller in direct shear test. Hence, drain testing will take lesser time as compared to triaxial test.
- Further, it is more convinient to subject a sample of clay to large strains in the direct shear test.
- As the specimen fails along a pre-determined plane, this test is useful when the soil has predetermined fault or joint.

Disadvantages:

- Drainage condⁿ cannot be controlled and pore water pressure cannot be measured.
- failure plane is always horizontal which may not be the weakest plane.
- Non-Uniform stress-distribution on shear plane.
- Direcⁿ of principle planes are not known at every stage of test.
- It is only after we have determined the ϕ value, the principle plane would be known.
- Area under normal load doesnot remain constant during the test.

Ques→ A direct Shear Box test on a remoulded sample of sand gave the following observation at the time of failure:

Normal Load, N = 288 N, Shear Load, T = 173 N,

c/s Area of Sample = 36 cm².

Determine the angle of Internal friction (ϕ) and magnitude

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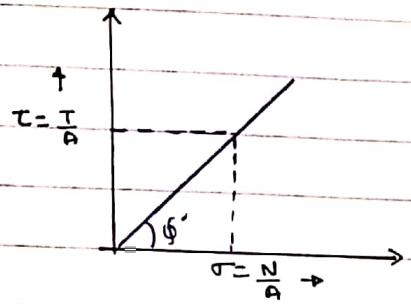
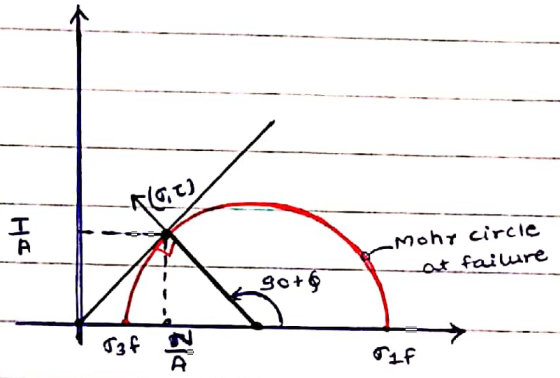
of principle stress and the principle plane.

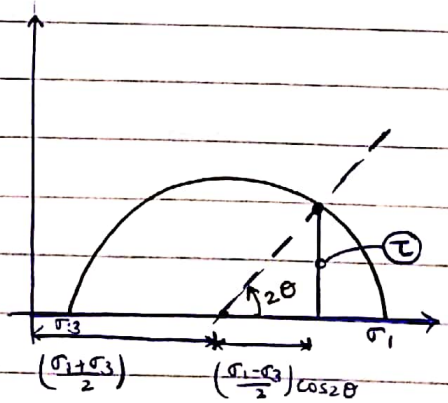
8 Soln:



$$\tan\phi' = \frac{\tau}{\sigma} = \frac{173}{288}$$

$$\Rightarrow \phi' = 31^\circ$$





$$\sigma = \frac{\sigma_1+\sigma_3}{2} + \frac{\sigma_1-\sigma_3}{2}\cos 2\theta$$

$$\tau = \frac{\sigma_1-\sigma_3}{2}\sin 2\theta$$

288 → N/A → 36

$$\frac{N}{A} = \frac{\sigma_{1f}+\sigma_{3f}}{2} + \frac{\sigma_{1f}-\sigma_{3f}}{2}\cos(2)\left(45+\frac{\phi}{2}\right)$$ → 31°

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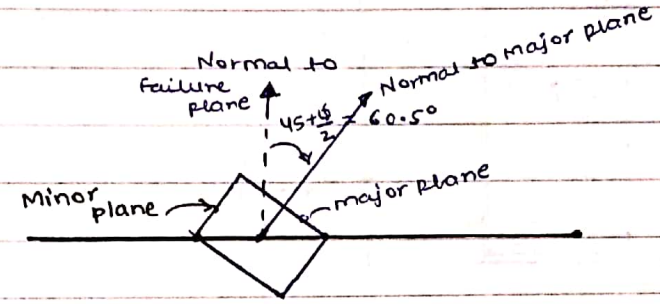
173

$$\frac{T}{A} = \frac{\sigma_{1f} - \sigma_{3f}}{2} \sin(2)\left(45 + \frac{\phi}{2}\right)$$

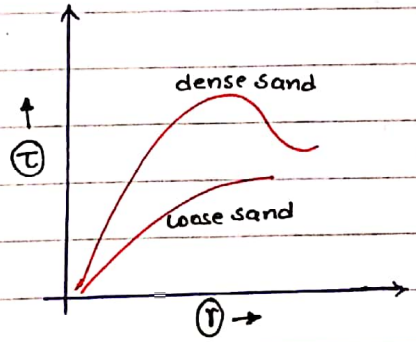
36 ↑ 31°

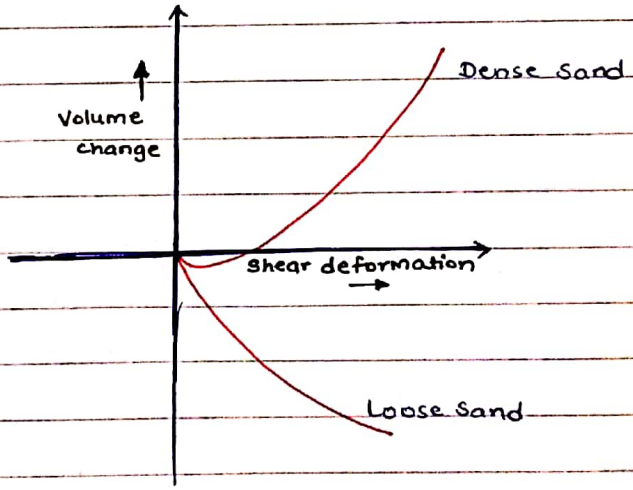
$\Rightarrow \sigma_{1f} = 16.48 \text{ N/cm}^2$

$\Rightarrow \sigma_{3f} = 5.28 \text{ N/cm}^2$



Other Results that can be obtained from direct Shear test:





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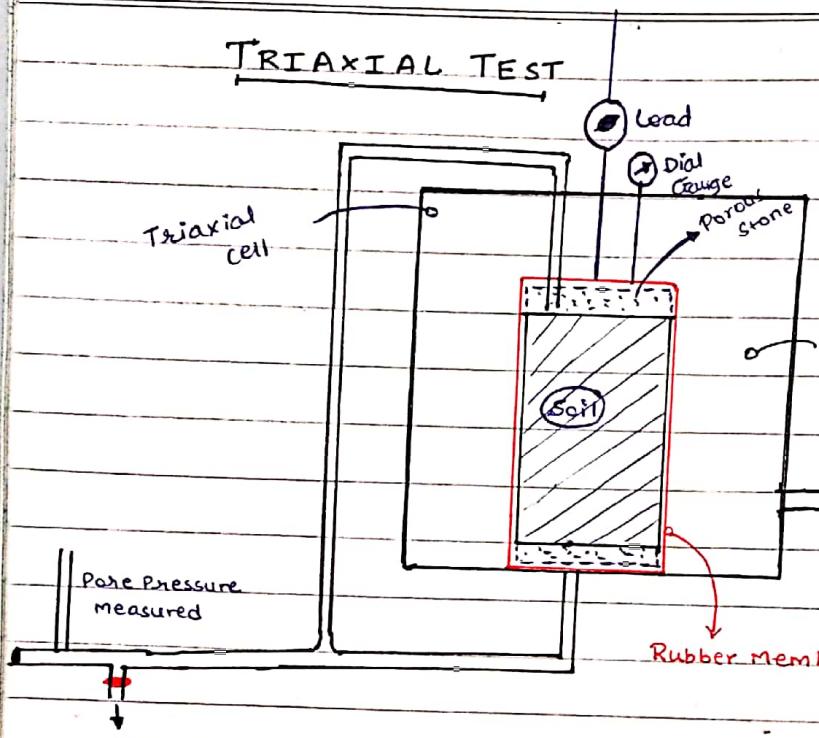
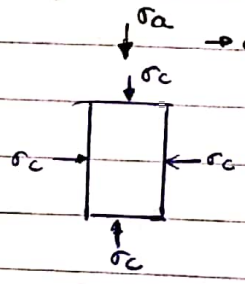
TRIAXIAL TEST

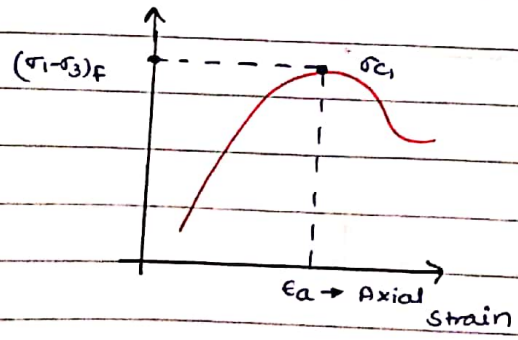


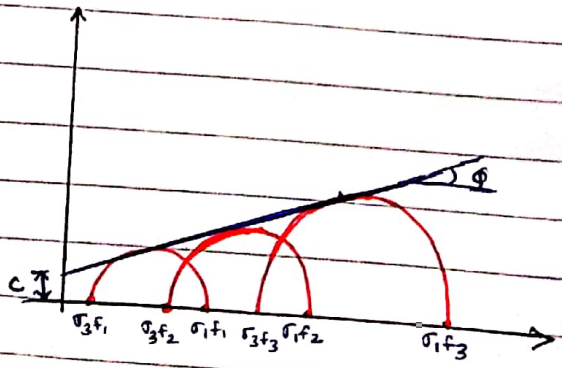
- σ_3 = Difference in Pressure b/w outside and inside
- Backwater Pressure → Applied to saturate the soil sample

$\sigma_a + \sigma_c = \sigma_1$

$\sigma_c = \sigma_3$







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⊙ Triaxial test is most widely used shear strength test and is suitable for all types of soil.

⊙ Drainage can be controlled whatever be the type of soil. i.e; Sand can be tested under undrained condⁿ and clay can be tested under drained condition.

⊙ Pore water pressure can be measured.

⊙ Volume changes can also be measured.
+ Volume change is measured in saturated soil by measuring the volume of water that flows out or flows into the specimen.

Note: ⊙ In Triaxial test, at any time, either pore pressure measurement can be done under undrained condⁿ or volume change can be measured under drained condition.
Both of them cannot be measured together.

⊙ failure plane is not pre-determined.

⊙ At every stage of testing, principle planes and principle stresses are known.

⊙ Stress distribution on failure plane is fairly uniform.

⊙ Triaxial cell is filled with water and the specimen is sealed inside a rubber membrane and cell pressure is applied. It is called confining pressure (σ_c).

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⊙ Confining Pressure is applied to simulate Effective Vertical Stress in the field.

⊙ With cell pressure held constant, additional axial stress is gradually applied.

This additional axial stress (σ_a) is called Deviator Stress and it produces shear in the soil, on all planes except the horizontal and vertical plane.

⊙ At the time of failure, major principle stress, $(\sigma_{1f} = \sigma_a + \sigma_c)$, and minor principle stress, $(\sigma_{3f} = \sigma_c)$ are noted by plotting deviator stress v/s axial strain curve.

⊙ During the application of loading, c/s area of the specimen will change. Hence, to calculate the deviator stress at the time of failure (σ_{af}), we need to know the c/s area of the specimen at the time of failure.

$$\sigma_{af} = \frac{P}{A}$$

P → Deviator load at the time of failure

A → c/s area of the specimen at the time of failure.

$$A(h_o + dh) = V_o + dV$$

$$A(h_o + dh) = V_o\left(1 + \frac{dV}{V_o}\right)$$

$$h_o A\left(1 + \frac{dh}{h_o}\right) = A_o h_o\left(1 + \frac{dV}{V_o}\right)$$

$$A = \frac{A_o(1 + dV/V_o)}{1 + dh/h_o}$$

For volume reduction & height reduction

$$A = \frac{A_o(1 - \epsilon_V)}{(1 - \epsilon_a)}$$

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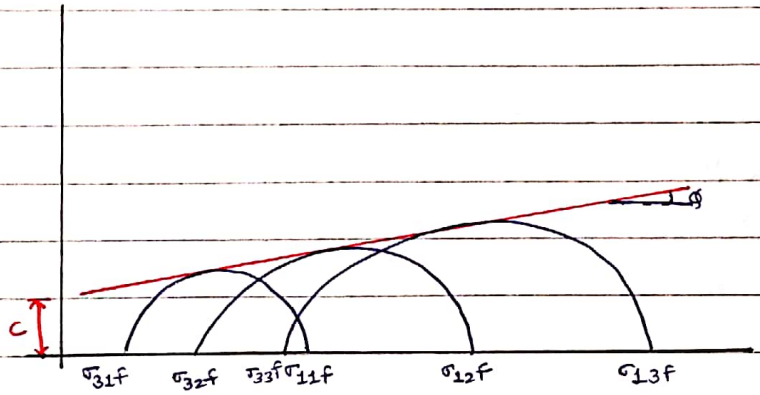
for undrained cond^n : $\epsilon_v = 0$

$$A = \frac{A_0}{1-\epsilon_a}$$

$(1-\epsilon_a)$ → Area correction factor.

→ Thus, performing the test at minm of 3 confinements, we can plot 3 Mohr circles.



Ques→ Following are the results of a drained test performed on two identical samples / of specimen of 38 mm dia. & 76 mm height

Sample No.	cell Pressure (kN/m^2)	Deviator Load at failure	change in vol. (cc)	Axial deformation (mm)
1	50	0.0711	−0.9	5.1
2	150	0.0956	−1.6	9.1

Determine shear strength parameter for the soil.

Soln: $(\sigma_{3f})_1 = 50\ kN/m^2$ $(\sigma_{3f})_2 = 150\ kN/m^2$

$$\sigma_{1f} - \sigma_{3f} = \frac{\text{deviator load}}{\text{Area}} = \frac{\text{deviator load}}{A_0\left(\frac{1-\epsilon_v}{1-\epsilon_a}\right)}$$

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For 1st Sample : $\frac{1-\epsilon_v}{1-\epsilon_a} = \frac{1 - \frac{0.9}{\pi/4 \times 3.8^2 \times 7.6}}{1 - \frac{0.51}{7.6}} = 1.0607$

For 2nd Sample: $\frac{1-\epsilon_v}{1-\epsilon_a} = \frac{1 - \frac{1.6}{\pi/4 \times 3.8^2 \times 7.6}}{1 - \frac{0.91}{7.6}} = 1.1149$

Sample-1: $\sigma_{1f} - \sigma_{3f} = \frac{0.0711}{\frac{\pi}{4} \times (0.038)^2 \times 1.0607} = 59.056 \Rightarrow \sigma_{1f} = 50 + 59.056 = 109.056 \text{ kN/m}^2$

Sample-2: $\sigma_{1f} - \sigma_{3f} = \frac{0.0956}{\frac{\pi}{4} \times (0.076)^2 \times 1.149} = 75.646 \Rightarrow \sigma_{1f} = 150 + 75.646 = 225.646 \text{ kN/m}^2$

$$\sigma_{1f} = \sigma_{3f} N\phi' + 2c'\sqrt{N\phi'}$$

$$109.056 = 50 N\phi' + 2c'\sqrt{N\phi'}$$

$$N\phi' = \frac{1+\sin\phi}{1-\sin\phi} = \tan^2(45 + \phi/2)$$

$\Rightarrow c' = 23.5 \text{ kN/m}^2$

$\phi = 4.389^\circ$

Note: ① c and ϕ value obtained from the drained test will be taken as effective stress shear strength parameters.

Hence, in the field:

$$\tau = c + \bar{\sigma} \tan\phi$$

$$\downarrow$$

$$(\sigma - u_h)$$

② Calculation of shear strength in above formula is approximation b/c shear strength of soil can be calculated as the shear stress on failure plane at failure but we are calculating vertical effective stress.

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⊙ UD test is similar to CD test.

Types of Triaxial Tests:

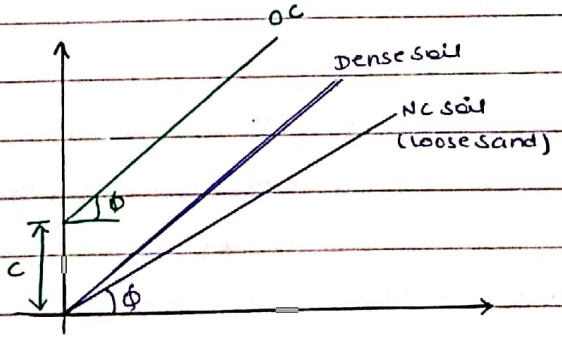
Triaxial tests are performed in 2 stages:

(a) Drainage allowed (consolidated) } 1st stage
(b) Drainage not allowed (unconsolidated) } (confining stress application)

(a) Volume change allowed (drained) } 2nd stage
(b) Volume change not allowed (undrained) } (Deviator stress application)

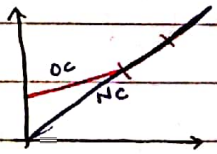
CD → consolidated drained test (takes long time)
CU → consolidated undrained test (1st stage → 24 Hrs, 2nd stage → 2 hrs)
UU → Unconsolidated undrained test (can be done in 15 min.)
UD → X not Exist → B/c it is similar to CD test.

① CD testing: → Drainage allowed in both the stages
↳ loading rate is slow, expell out water and therefore pore water pressure do not build up.
↳ we get Effective stress parameter.



→ OC will have some value of c due to cold welding.

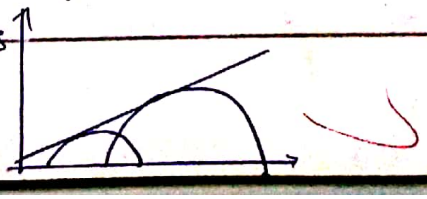
NC clay ⇒ (c=0)
OC clay ⇒ (c≠0)



⊙ Progressively, bigger Mohr circle is obtained if confinement is being done at greater confining stress. This is because during consolidation stage, soil would have become more dense and hence higher deviator stress is require to cause the failure and will go on increasing as the confining pressure is being increased.
(b/c void ratio is will ↓ as more and more higher confining pressure is applied)

$\tau_{field} = c' + \bar{\sigma} \tan\phi'$

Where, $\bar{\sigma} = \sigma - u_{hydrostatic}$



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drained → c doesnot mobilise
undrained → ϕ does not mobilise

Use of CD testing:

(i) Almost all cases involving coarse grain or gravel whether saturated or not except when exposed to rapid loading such as earthquake, bomb blast ... etc.

(ii) Cases involving fine sand & silt if the field loading is reasonably slow.

(iii) Long term stability check of any soil.

(iv) Long term loading of any soil.

Examples under which CD testing is done:

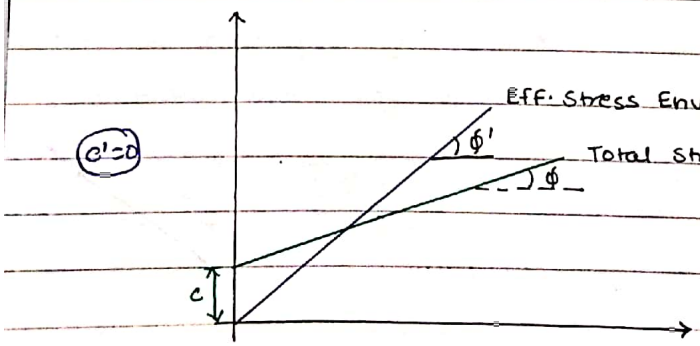
(a) Cut slopes ~~after~~ several years after consolidation/Excavation.

(b) Embankment construction very slowly in layers over soft clay deposits.

(c) Earth dam with steady seepage.

(d) Foundation on clay long term after construction.

② CU Testing: In this case, during confining stress application soil is allowed to consolidate but during deviator stress application, the soil is under undrained condn.



1st stage → drainage Permitted

2nd stage → drainage not Permitted

i.e; Volume change not allowed.

→ We can measure both total stress parameter and Eff. stress parameter b/c pore pressure can be measured at failure.

⇒ c' will exist in case of OC.

• Eff. stress failure envelope pass through origin. c'=0

Note: ① Under drained condn, cohesion doesnot mobilise and under undrained condn, friction (ϕ) doesnot mobilise.

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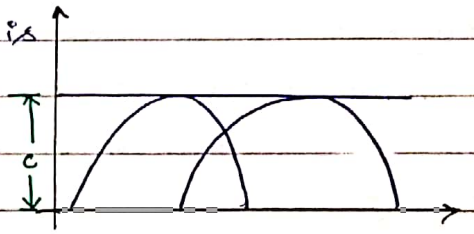
⊙ UU is used for sudden loading condⁿ such as rapid constrn
Eg: rapid constrn of earthern dam, rapid constrn of buildings, strength of soil in excavation. Immediately after cut is made.

Use of Cu CU test:

(i) Most cases involving short term strength i.e; strength of relatively impervious soil deposits (clays or clayey soils) that are to be loaded following initial consolidation under existing stress.

Examples: Building foundations, highway embankment, dams, highway foundⁿ, unloading problems like rapid drawdown, prediction of strength variation with depth from a uniform soil deposits from which sample can only be retrive from near the ground.

③ UU Testing: → Drainage is not permitted during 1st and 2nd stage.
→ Since, ϕ is not mobilised during undrained condⁿ, hence, in this case, failure envelope is horizontal.
→ Each Mohr circle is of same diameter.
→ This type of testing is suitable for low permeability soil under undrained condⁿ.
→ After application of confining pore pressure, excess pore pressure develops which is not allowed to dissipate.



Uses OF UU Testing

→ Most cohesive soil of relatively poorer drainage where the fill load is applied very rapidly.
Example: Compacted fill in an earthern dam that is being rapidly constructed.
→ Strength of foundation soil which is loaded rapidly.
→ Strength of soil in excavation immediately after cut is made.

Note: $\frac{C_u}{\sigma_z} = 0.11 + 0.0037\,(I_p\%)$ Cu can vary with depth also.

Sometimes, we prefer to perform UCS test instead of UU test.

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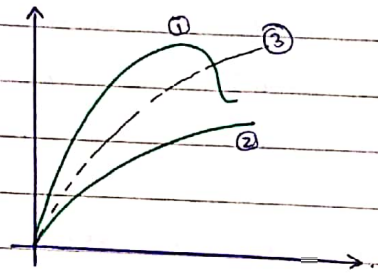
⊙ Generally, the equality of shear strength parameter in CD and CU test is true for remoulded clay only but for undisturbed clay, ϕ' for NC clays can be assumed to be same in the two test but not in case of OC clays.

Comparison of CU & CD test: → In reality, results of CU and CD test can vary slightly.

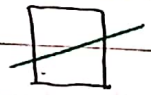
→ Generally, it is assumed that effective stress shear strength parameter determined by CU test is same as that obtained in CD test (in clays)

Stress-Strain & Volume Change Relationship for Clayey Soil



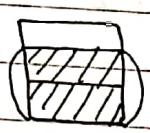
① → Undisturbed cohesive clay (sensitive)
② → Remoulded sensitive clay
③ → Insensitive clay.

Curve ①: Sharp peak at low strain, hence brittle failure. Specimen shear along a well defined plane.



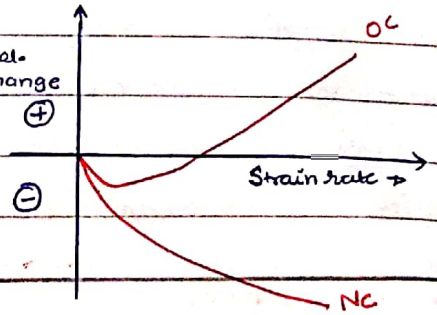
Failure is known as Brittle failure.

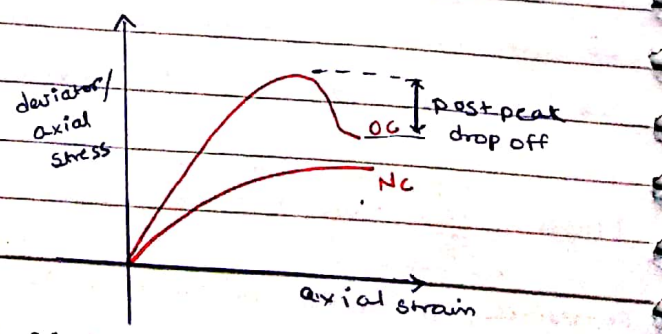
Curve ②: Plastic or barreled failure, results in bulging of specimen & failure is decided on the basis of some define strain.



Curve ③: Failure is occuring corresponding to the horizontal portion of the curve.

Results of CD test:





⊙ Post peak drop off is more pronounced as degree of OC increases.

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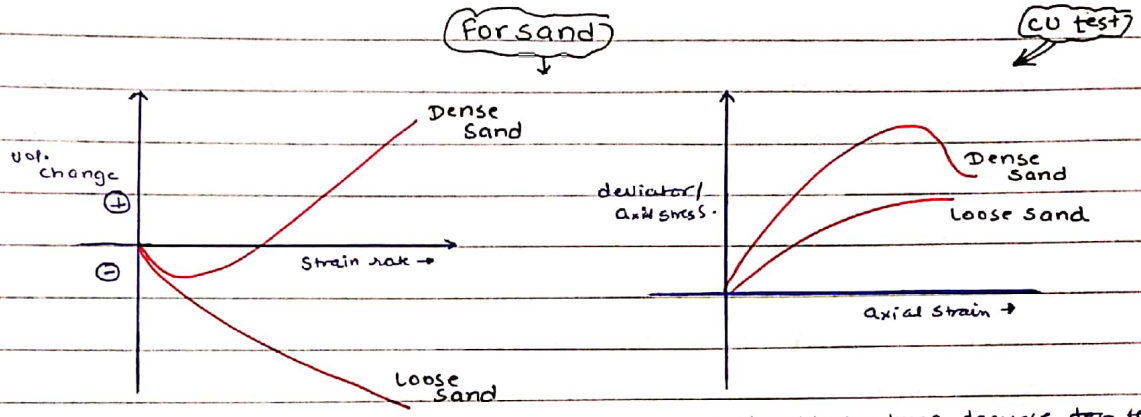
c' and ϕ' of CD in oc clays is > c' and ϕ' for CU test.

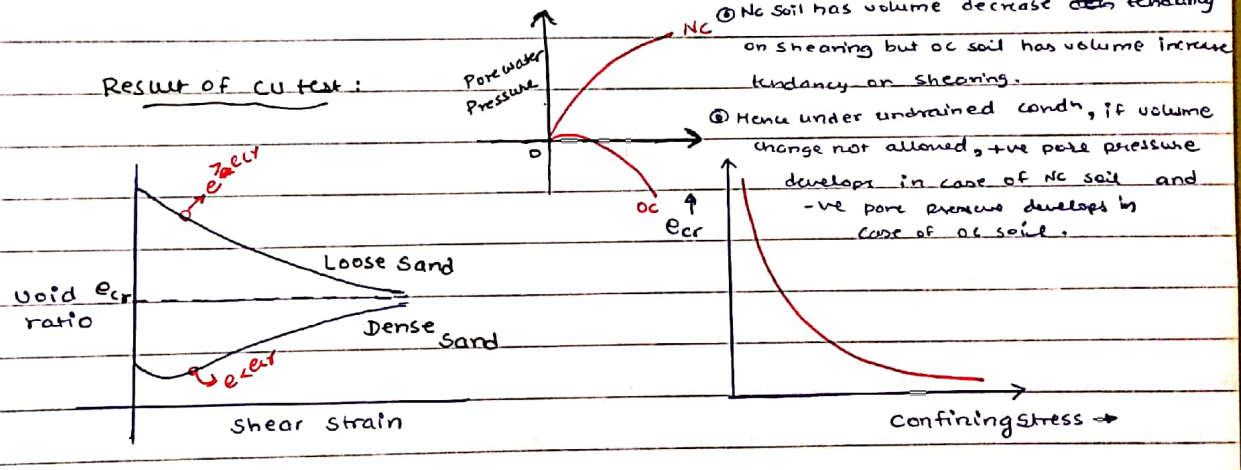
$c'_{CD} > c'_{CU}$ } → (oc clays)
$\phi'_{CD} > \phi'_{CU}$ }

Peak value of deviator stress is generally called compressive strength of soil.

→ Post peak drop off is due to breakage of adhesive bond and re-orientation of clay particles into dispersed arrangement. (in OC)



Result of CU test :



① NC soil has volume decrease ~~dec~~ tendancy on shearing but OC soil has volume increase tendancy on shearing.

② Hence under undrained condn, if volume change not allowed, +ve pore pressure develops in case of NC soil and -ve pore pressure develops in case of OC soil.

Note: ⊙ At $e = e_{cr}$, there is no volume change tendancy. Hence, drained & undrained strength will be same.

⊙ At $e > e_{cr}$ then there is tendancy of volume decrease hence under undrained condn, passive pore pressure develops and hence strength falls because eff. stress decreases.

Thus, **undrained strength < Drained strength.**

⊙ At $e < e_{cr}$, there is tendancy of volume increase hence under undrained condn, -ve pore pressure develops and thus effective stress increases hence shear strength increases.

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Hence, undrained strength will be greater than drained strength.

Soil Liquifaction

- Loose sand has a tendancy to compress when loaded. If rate of loading is large & the soil is saturated positive pore pressure develops. This will reduce the effective stress and hence strength.

- If effective stress ↓ reduces to zero, loose sand will loose all of its shearing strength. This phenomenon is called liquifaction.

- Liquifaction occurs in saturated loose sand.

- Liquifaction occurs during pile driving, vibration of machine, explosive blasting, earthquake shocks--- etc.

- Chances of liquifaction is greater in higher frequency & higher amplitude of vibration.

- There is cumulative increase in pore water pressure under successive cycle of loading.

Note: - Strength degradation of clay due to cyclic loading follows similar pattern as that of in sand, but loss of strength is not much in case of clay.

- Loss of strength in clay increases with PI.
- In case of quick clay, there is significant loss of strength. (high sensitivity)

- UC soil prone to greater loss of strength as compared to OC and NC clay due to cyclic loading

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Pore Pressure Coefficient / Skempton Pore Pressure Coefficient

→ Pore pressure coefficients are used to express the response of pore water pressure to change in total stress under undrained condⁿ and thus, enables the determination of initial excess pore water pressure.

⊙ The volume of coefficients can be determined in lab. & can be used to predict the pore water pressure in the field.

$$\Delta u = B[\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)]$$

$$\Delta u = B\Delta\sigma_3 + AB(\Delta\sigma_1 - \Delta\sigma_3)$$

$$\Delta u = B\Delta\sigma_3 + A_f(\Delta\sigma_1 - \Delta\sigma_3)$$

$A_f = AB$

$\Delta u = \Delta u_1 + \Delta u_2$ — A, B → Skempton's pore pressure coeff.

$\Delta u_1 = B\Delta\sigma_3$

$\Delta u_2 = AB(\Delta\sigma_1 - \Delta\sigma_3)$

UU Test:

1st stage testing: $\Delta u_1 = B(\sigma_3 - 0)$ → find B from 1st stage

B is found

$B = 1$ for saturated soil.

for unsaturated soil, B is very small.

for dry soil ⇒ $B = 0$ → even at 50% saturation

value of B varies with stress range.

2nd stage Testing: $\Delta u_2 = AB(\Delta\sigma_1 - \Delta\sigma_3)$, $\Delta\sigma_3 = 0$

find A=?

A is found.

$\Delta\sigma_1$ → Deviator stress at failure

[since, confining stress is constant during 2nd stage testing.]

Δu_2 → Pore pressure developed during deviator stress application.

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→ The value of A and B can be calculated from UU test.

→ If soil is saturated (B=1) then 2nd stage of the CU test can also be used to find out A.

Material (100% Saturation)	A at failure
① Very loose Sand	2-3
② Sensitive clay	1.5-2.5
③ NC clay	0.7-1.3
④ Slightly OC clay (OCR<2)	0.3-0.7
⑤ Heavily Consolidated clay (HCC) [OCR>2]	-0.5-0

Note: ⊙ Pore Pressure Parameter, A>1 is associated with loose structure either in sand or in clay which collapses under load application.

⊙ When the structure collapses then the deviator stress increment plus some of the effective stress acting on the soil could be transferred to the pore pressure & hence A can be greater than 1.

Ques→ An embankment is built of a soil whose effective stress shear strength parameters, c' & ϕ' are 100 kN/m² & 20°. The unit wt. of soil is 17 kN/m³. The pore pressure parameters are A=0.6 & B=0.8. The height of embankment has just been raised from 5m to 8m. Determine the shear strength of soil at the base of embankment. Assume that lateral pressure at any point is $\frac{1}{2}$ of the vertical pressure.

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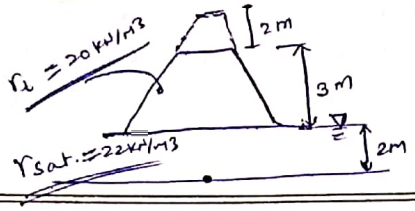
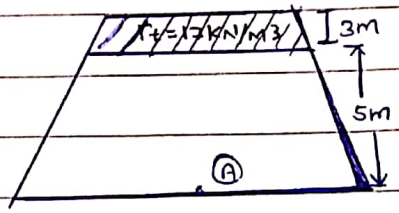


Soln:



$c' = 100\ kN/m^2$

$\phi = 20°$

$\tau = c' + \bar{\sigma} \tan\phi'$

$\sigma = 17 \times 8 = 136\ kN/m^2$

Assume initial pore pressure at (A) = 0

$\Delta u = u - 0 = u$

$\Delta u = 0.8 [\Delta\sigma_3 + 0.6 (\Delta\sigma_1 - \Delta\sigma_3)]$

$= 0.6 \left[17 \times 3 \times \frac{1}{2} + 0.6 \left(17 \times 3 - 17 \times 3 \times \frac{1}{2}\right)\right]$

$\Delta u = u = 32.64\ kN/m^2$

$\bar{\sigma} = 136 - 32.64 = 103.36\ kN/m^2$

$\tau = 100 + 103.36 \tan 20° = 137.619\ kN/m^2$

Unconfined Compression Test

($\phi = 0$ Always)

UCS → only for clay
↳ only 1 Mohr circle obtained.

- No confining pressure is applied.
- It is a special case of triaxial test ($\sigma_3 = 0$)
- Use for cohesive soil.
- Load is rapidly applied, hence it is an undrained test & ϕ_u does not get mobilised. ($\phi = 0$)
- only one Mohr circle will be obtained.
  As there is only one confining pressure i.e; ($\sigma_3 = 0$)

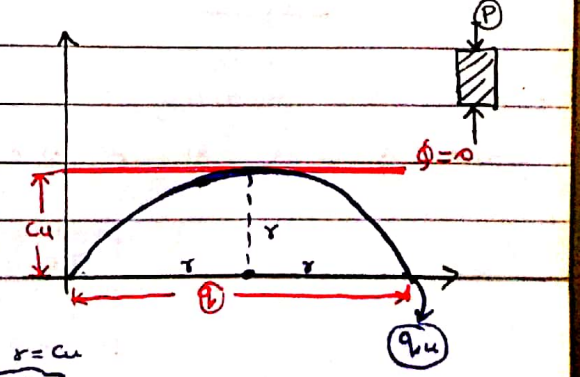
$\sigma_1 = q_u$ = unconfined compressive strength

$c_u = \frac{q_u}{2}$

$q_u = \frac{P}{A_f}$

$\Rightarrow q_u = \frac{P}{A_o \left(\frac{1}{1 - \epsilon_a}\right)}$ [$\epsilon_v = 0$ as it is undrained cond^n]

$\epsilon_a = \frac{\Delta h}{h}$



$2r = q_u$ and $r = c_u$

$\Rightarrow c_u = \frac{q_u}{2}$

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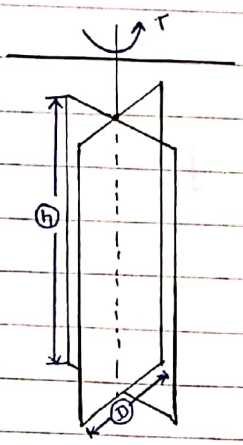
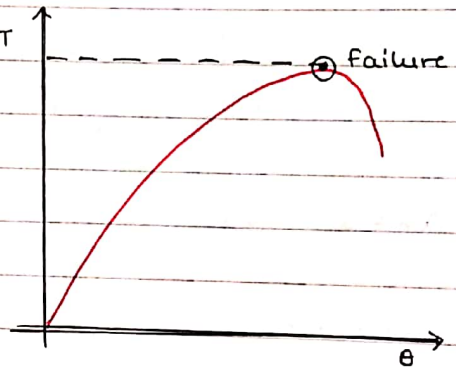
⊙ As no lateral pressure. i.e; confining pressure exist, sand / coarse grained soil can't stand in the equipment and hence, UCS test is not done for sand / coarse grained soil.

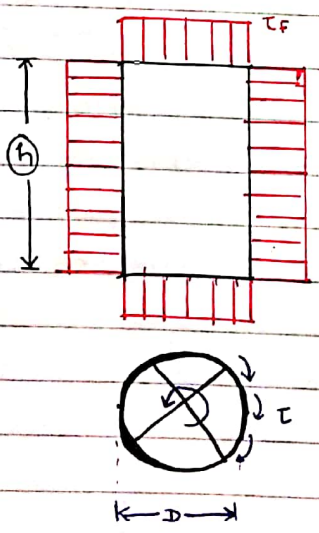
Vane Shear Test:

⊙ In plastic cohesive soil which is very sensitive obtaining the undisturbed specimen is difficult. Hence, shear strength of such soil will be significantly affected by sampling and handelling.

for such soil, vane shear test is done in the field.







$$T = \tau_f \pi D h \cdot \frac{D}{2} + 2\left[\int_0^{D/2} \tau_f \cdot 2\pi r \cdot dr \cdot r\right]$$

$$\tau_f = \frac{T}{\pi D^2 \left(\frac{h}{2} + \frac{D}{6}\right)}$$

$\tau_f = c + \bar{\sigma} \tan\phi$; $\phi = 0$

$\tau_f = c$

⊙ When the shearing is done such that the top end on the vane doesnot shear the soil then:

$$\tau_f = \frac{T}{\pi D^2 \left[\frac{h}{2} + \frac{D}{12}\right]}$$

$\tau_f = c$

→ c is the undrained cohesion.

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⊙ This test is also used sometimes to find the LL of soil in lab.
⊙ The m/c at which the shear strength is negligible is called a liquid limit m/c.

→ Vane shear test can be used to find out the sensitivity of the soil.

→ After the initial test, if vane is rotated several times, the soil becomes remoulded & strength of remoulded clay can be calculated.

$$\text{Sensitivity} = \frac{2\,C_u\ (\text{undisturbed})}{2\,C_u\ (\text{remoulded})} = \frac{\tau_f\ (\text{undisturbed})}{\tau_f\ (\text{remoulded})}$$

→ Vane shear test can also be used to find out liquid limit in the lab.

Note: ⊙ $\phi_{\text{Plane strain}} > \phi_{\text{Triaxial}}$

⊙ In drained test, ϕ decreases with increase in plasticity of soil.
⊙ Higher the OCR, higher is the ~~modulus~~ modulous of elasticity, and poisson's ratio.
⊙ Greater is the plasticity, lower is the modulous of elasticity & greater is the modulous of shearing strain.
⊙ Modulous of elasticity of dense sand is greater than that of OC clays.
⊙ Increasing the rate of at which saturated soil is sheared increases the undrained strength.

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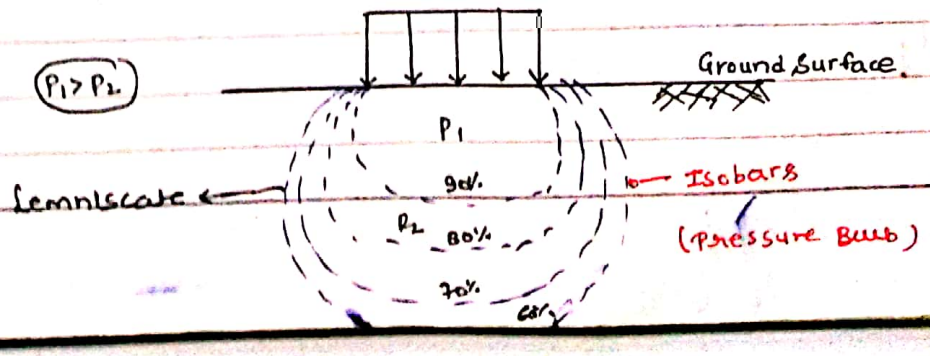
PART - 2

* Vertical Stress
* Earth Pressure Theory
* Stability of Slope
* Bearing Capacity
* Deep Foundation
* Soil Exploration.

VERTICAL STRESS

- The topic relates to the stress distribution within the soil due to the applied load.
- The stress-strain distribution is complex for a soil and the behaviour depends upon the drainage conditions.
- Soil is not a purely elastic material and can have poisson's ratio (>0.5). But for analysis, it is considered as Elastic Material in which for undrained cond^n, ($\mu = 0.5$) and for drained cond^n, it is (< 0.5)
- The two major theories in the vertical stresses are :

(i) Bousinessq's (ii) Westergard's



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Isobar: ⊙ It is a curve or contour, connecting all points below the Ground surface having equal vertical stress, (σ_z).

⊙ The isobars are arranged layer by layer representing lemniscate formation.

⊙ Generally, the influence due to the external load in the soil is considered upto a depth of 1.5 B to 2B below the loading point.

⊙ P

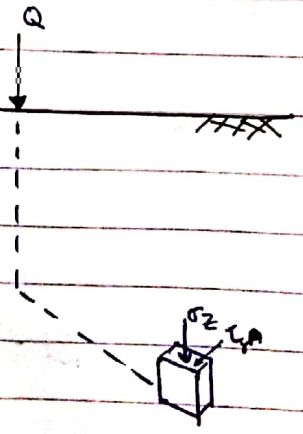
Pressure Bulb: ⊙ It is the zone of the soil in which there is significant distribution and beyond that zone, the stresses are less in value.

Boussinesq's Theory

Assumptions:

⊙ Soil is considered as Elastic medium, homogenous, isotropic, semi-infinite (one side infinite), weightless, initially unstressed

Case-I: Point load



$$\sigma_z = \frac{3}{2\pi} \cdot \frac{Q}{z^2} \left[\frac{1}{1+\left(\frac{r}{z}\right)^2} \right]^{5/2}$$

$$\tau_r = \sigma_z \cdot \frac{r}{z}$$

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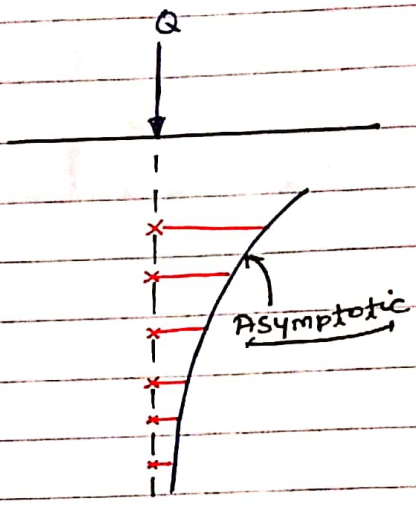
(i) Directly under the load:

$r = 0$

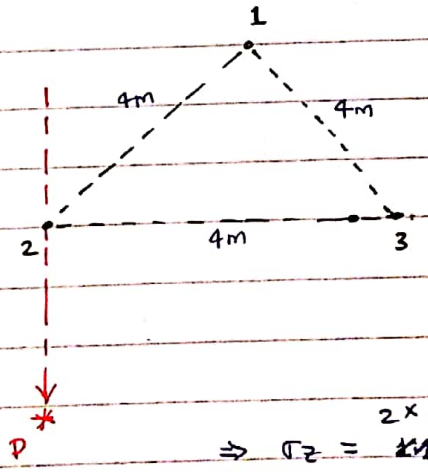
$$\sigma_z = \frac{3}{2\pi} \cdot \frac{Q}{z^2} = 0.47 \frac{Q}{z^2}$$

$$\sigma_z \propto \frac{1}{z^2}$$



Ques→ A circular tank (elevated) transferring a total load of 300 tonne by means of 3 no. of columns in an Equilateral triangle form having a c/c distance of 4m. The column foundation is at a depth of 1m below the ground level. Find out the Vertical Stress at any point located 6m below the Ground level under one of the column.

Soln:



$z = 5m$, $r = 4m$

$$\sigma_z @ = \frac{3Q}{2\pi(5)^2}\left[\frac{1}{1+\left(\frac{4}{5}\right)^2}\right]^{5/2} \times 2 + \frac{3Q}{2\pi(5)^2}$$

$\Rightarrow Q = 100$ tonne $\left(\frac{300}{3} = 100 \text{ tonne}\right)$

$$\Rightarrow \sigma_z = \frac{2 \times 0.554}{} + 1.910 = 3.019 \text{ tonne/m}^2$$

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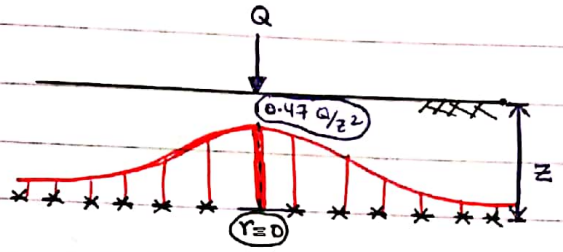
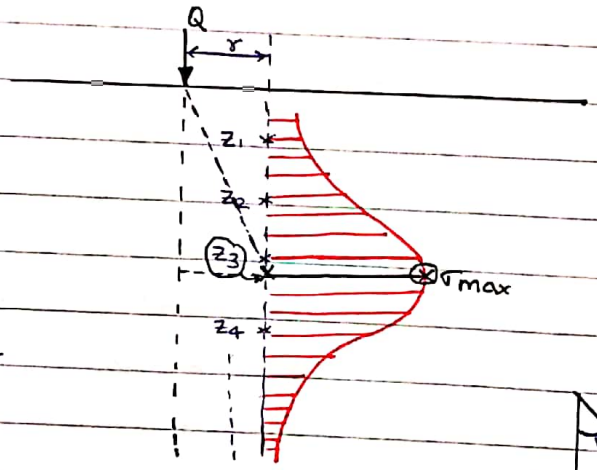
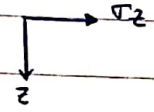
(ii) $r \neq 0$, z = constant



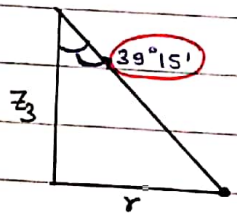
$\frac{r}{z} > 5$

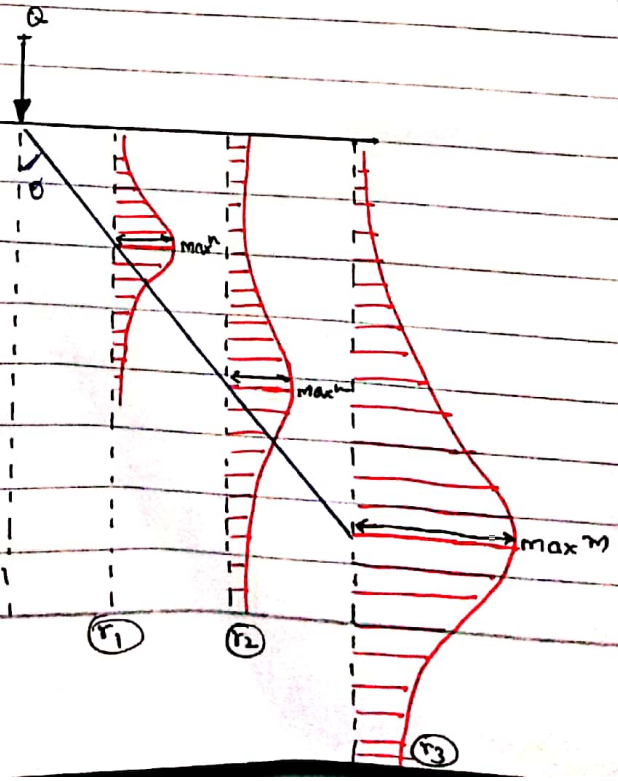
⊙ for $\frac{r}{z} > 5$, the stress curve not considered as the value of σ_z is quite small.





$$\sigma_z = \frac{3}{2\pi} \frac{Q}{z^2} \left[\frac{1}{1+(\frac{r}{z})^2} \right]^{5/2}$$





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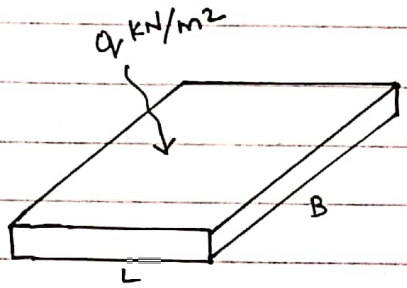
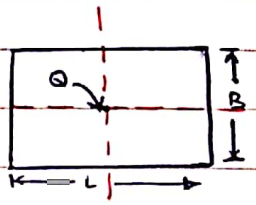
Case-II: Equivalent Point Load





$Q = q \times B \times L$

⊙ In this method, the pressure loading over an area is converted into a single point load which is applied at the centroid of the area. and after which the Bossiqnaus eqn for point load can be used.

Case-III: Vertical Stress at a point below one corner of a rectangular loaded area.

⊙ The factors used in the formula are derived based on Boussinesq's theory and it is given as:

$$\sigma_z = q \cdot I = q \cdot K_B$$

⊙ I or K_B → Influence factors based on the parameters m and n

m, n are the ratios of length and width w.r.t. depth and these values are **interchangable**.

$m = \frac{L}{z}$ $n = \frac{B}{z}$

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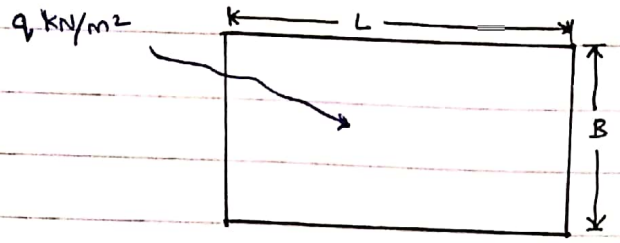
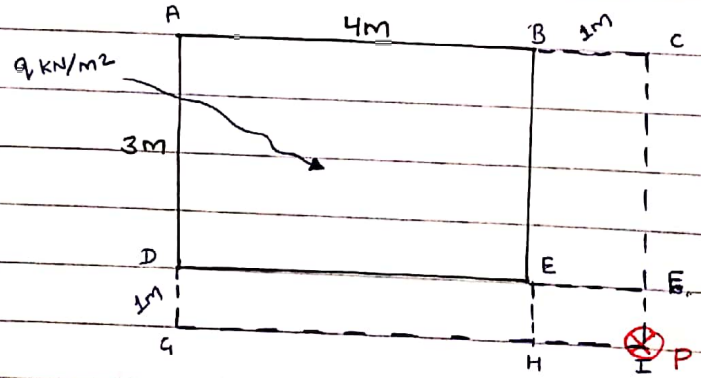


m	0.4	0.4
n	0.2	0.4
I	—	—

① If point P is outside the loaded Area:



⊕ ACIG = σ_{z1}

⊖ BCIH = σ_{z2}

⊕ EFIH = σ_{z3}

⊖ DFIG = σ_{z4}

$\Rightarrow \sigma_z$

σ_{z1} → $m = \frac{5}{z}$, $n = \frac{4}{z}$

σ_{z2} → $m = 1/z$, $n = 4/z$

σ_{z3} → $m = 1/z$, $n = 1/z$

σ_{z4} → $m = \frac{5}{z}$, $n = \frac{1}{z}$

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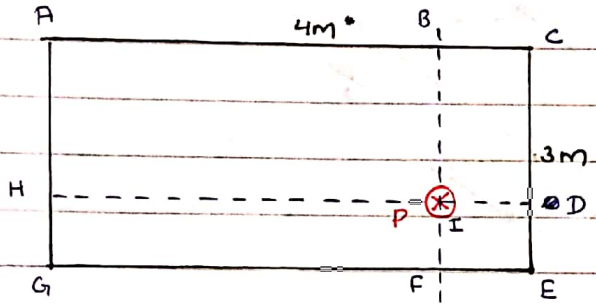
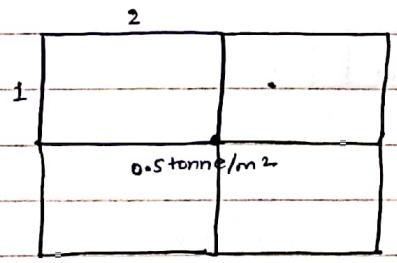
② If point P is Inside the loaded Area:



ACEG = ABIH + BCDI + DEFI + HIFG

Ques→ for a 2x1 rectangular area, the intensity of the stress at a point 4m below, one of the corners is 0.5 tonne/m². Find the intensity of the stress at the point below the centre of 2x4m rectangular area.

Solun:



$\sigma_z = 0.5 \times 4 = 2$ tonne/m²

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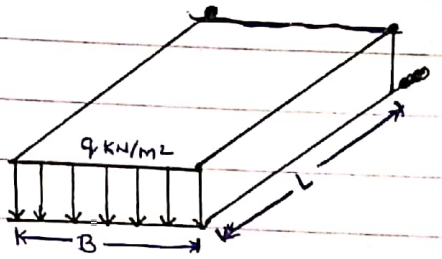
Case-IV

Strip loading / continuous foundation

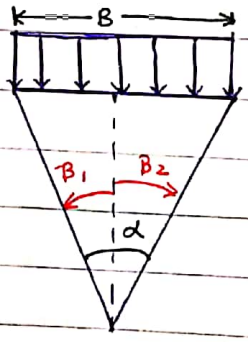
L >> B

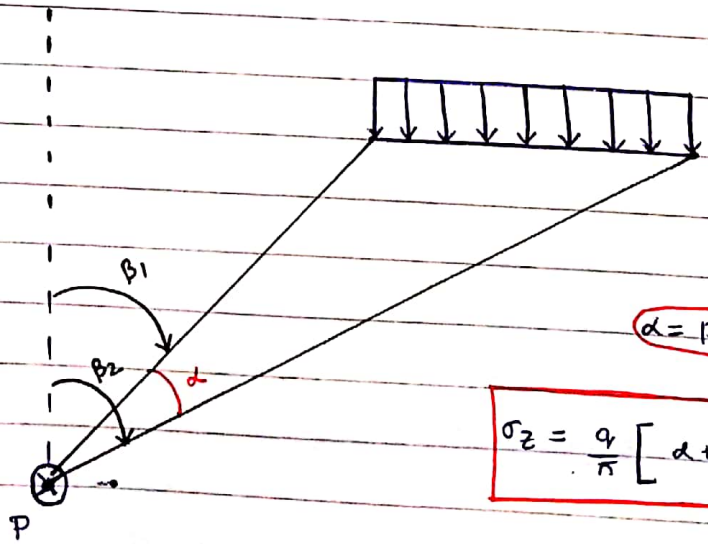


$$\sigma_z = \frac{q}{\pi}\left[\alpha + \sin\alpha\right]$$

Radian

Degree





$$\alpha = \beta_2 - \beta_1$$

$$\sigma_z = \frac{q}{\pi}\left[\alpha + \sin\alpha \cdot \cos(\beta_1 + \beta_2)\right]$$

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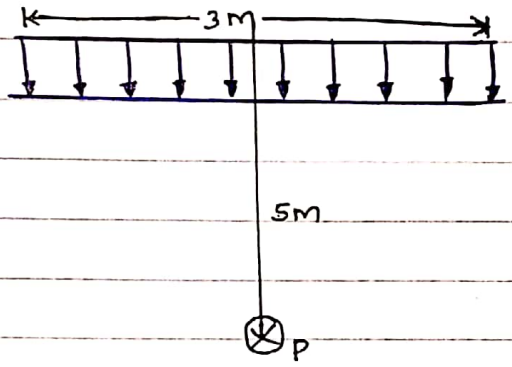
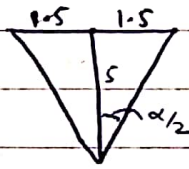
Ques→ A loading intensity of 10 tonne/m² is applied over a strip foundation having a width of 3m. Find the vertical stress at a point located 5m below the centre.

Soln:





Sol σ_z

$$\Rightarrow \sigma_z = \frac{10}{\pi}\left[0.58 + \sin 33.4^\circ\right]$$

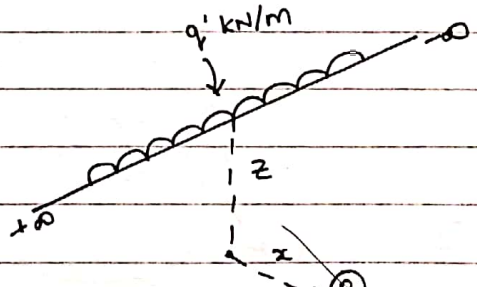
$$= 3.6 \text{ tonne/m}^2$$

$$\frac{\alpha}{2} = \tan^{-1}\left(\frac{1.5}{5}\right)$$

$$\Rightarrow \alpha = 33.4^\circ$$

$$= 0.58 \text{ radian}$$

Case-V : Line Loading



⊙ Based on Boussiqness theory, the vertical stress due to loading on any point P at a ⊥r distance x from the loading in the horizontal plane is given as:

$$\sigma_z = \frac{2q'}{\pi z}\left[\frac{1}{1+\left(\frac{x}{z}\right)^2}\right]^2$$

⊙ This loading is due to boundary wall, railway line.

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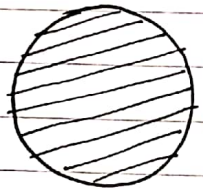
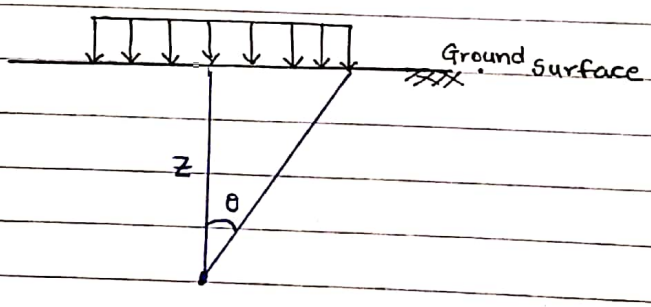
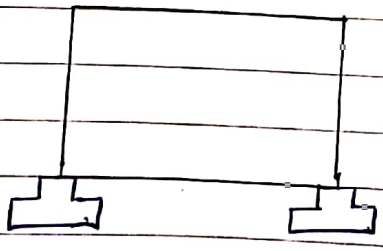
Case-VI : Circular loaded Area

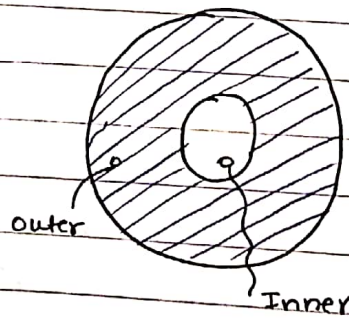
- The stress intensity at any point P located below the centre of a circular loaded area is given based on Bousiqness eqn as:

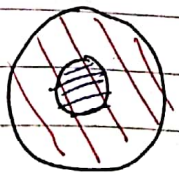
$$\sigma_z = q(1-\cos^3\theta)$$











$$\sigma_z = \sigma_{z\,outer} - \sigma_{z\,inner}$$

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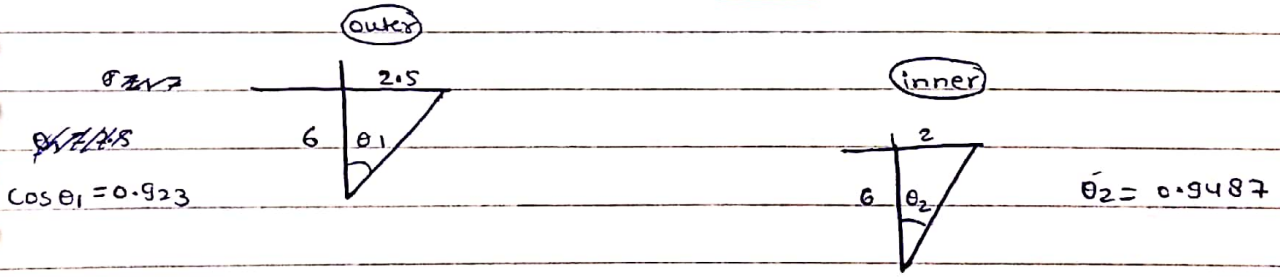
Ques→ A circular tank having a load 300 tonne has a ring wall foundation of outer dia. 5m and inner dia. 4m. The load is transfer by means of this ring foundation. Calculate the vertical stress at a point 6m below the centre of loaded area.

Soln:

$$q = \frac{Q}{\text{Area transferring the load}}$$

$$= \frac{Q}{\frac{\pi}{4}[D_o^2 - D_i^2]} = \frac{300t}{\frac{\pi}{4}[5^2 - 4^2]}$$

$$q = 42.4\ t/m^2$$



$\cos\theta_1 = 0.923$

$\theta_2 = 0.9487$

$\sigma_{z_1} =$

$\sigma_{z_2} =$

$\sigma_z = 2.85\ t/m^2$

Case-VII: Newmark's Chart → used to determine:
- Vertical Stress
- Horizontal Stress
- Shear Stress.

(under any shape.)

⊙ Newmark's chart is a set of concentric circles divided into different sectors, by means of radial lines. This chart is prepared based on Bousinesq's theory and is utilized to calculate the vertical stress intensity at any point due to a irregular loaded area and the intensity is given as:

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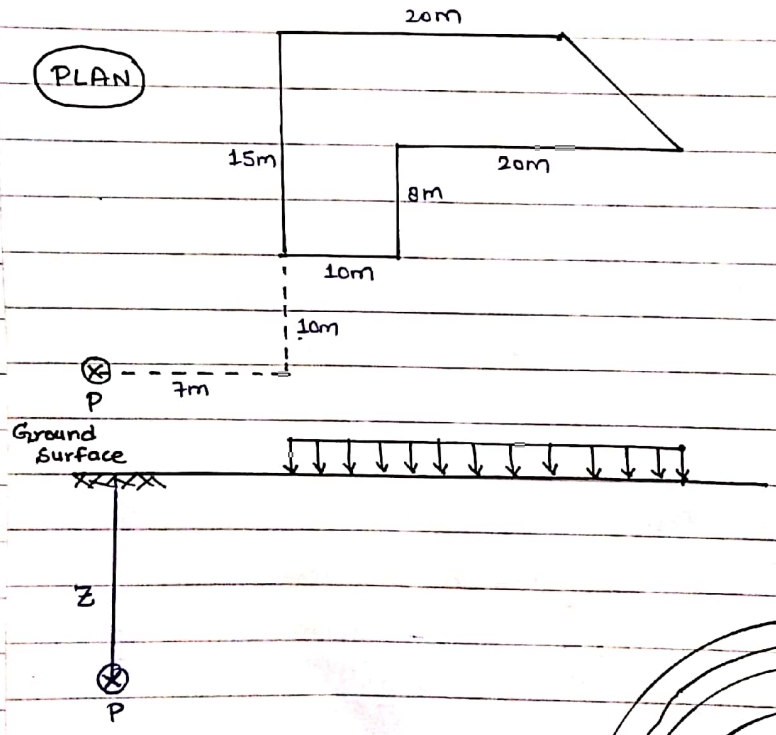
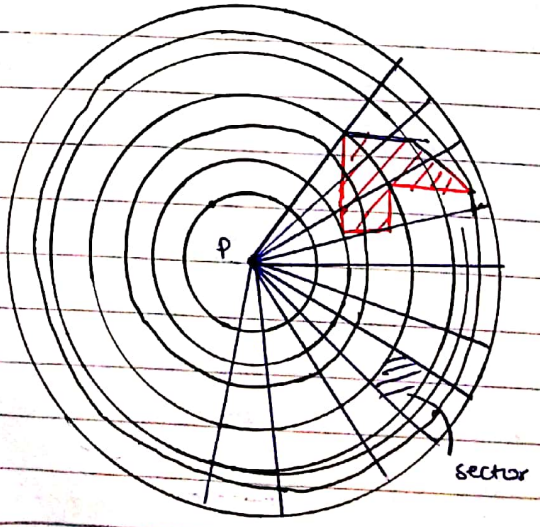
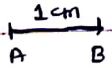
$\sigma_z = q \cdot n \cdot I$

- Where n is the no. of sectors which are covered or overlapped by the projected loaded area on the chart paper.

- I is the influence factor which depends on the total no. of sectors in which the Newmark Chart has been divided.







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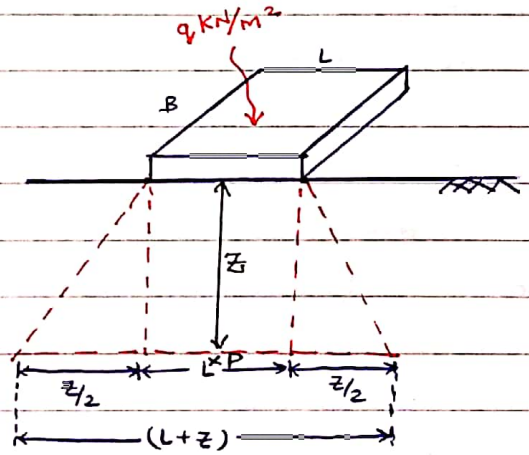
- The influence of every sector is same whatever may be its location from the centre of the chart.

$$I = \frac{1}{\text{Total no. of sectors}}$$

Total no. of sectors = no. of concentric circles × no. of radial lines

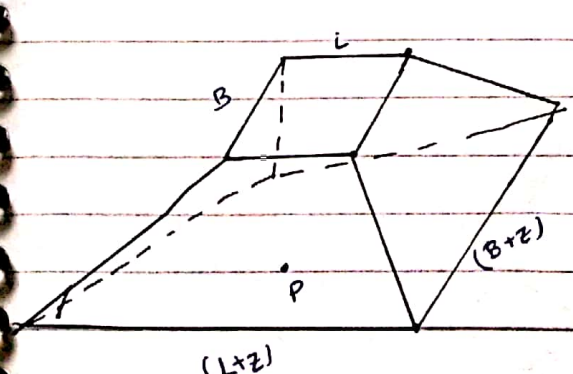
A ——— 1cm ——— B

Case - VIII : Load dispersion. Theory.



Vertical stress :

Total load = $Q = q(L \times B)$



$$\sigma_z = \frac{Q}{(B+z)(L+z)} = \frac{qLB}{(B+z)(L+z)}$$

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If $L >> B$ → continuous footing

$$\sigma_z = \frac{q \cdot L \cdot B}{(L+z)(B+z)} = \frac{q \cdot B}{(B+z)}$$

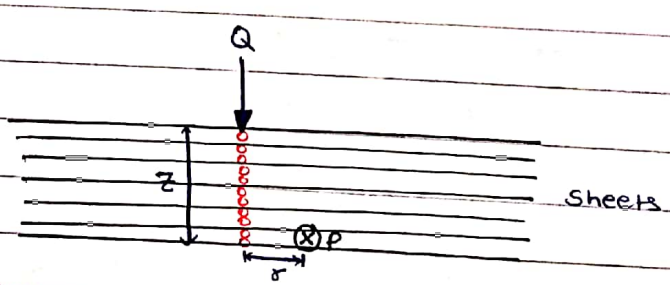
(σ_z; L; 0)

Westergard's Theory (for Anisotropic soils) (Not considered μ)

① Westergard assumed the soil to be in thin sheets, layered over one another.

② The poisson's effect is not considered and the lateral strain development is taken zero.

③ His analysis is best suitable for stratified deposits like Varved clay. and the vertical stress at any point below the loading is given by the eqn.

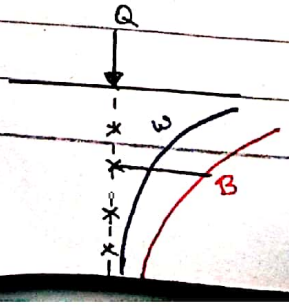


$$\sigma_z = \frac{Q}{\pi z^2}\left[\frac{1}{1+2\left(\frac{r}{z}\right)^2}\right]^{3/2}$$

Case-I $r = 0$ z = variable

$$\sigma_z = \frac{1}{\pi}\frac{Q}{z^2}$$

$$= 0.318\frac{Q}{z^2}$$



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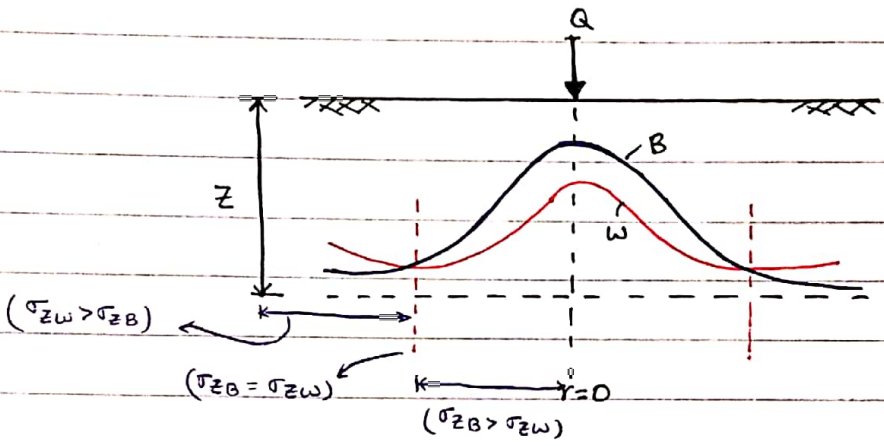
Case-② r= variable z= constant



⊙ The limiting point at which stresses due to Bousinesq and westergard are same is represented by ratio approx.

$$\Rightarrow \frac{r}{z} = 1.5$$

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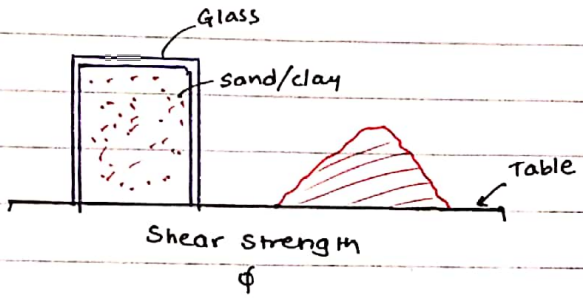
EARTH PRESSURE THEORY

Earth Pressure Theory



- In fluids, the hydrostatic pressure acts equally in all directions at any given point.

  i.e; $\sigma_x = \sigma_y = \sigma_z$.

  The horizontal and vertical pressure are same because water doesnot have any shear strength.

- In case of soil, the vertical and the horizontal stresses are not equal. It is because the soil has shear strength due to either friction, cohesion or both.

  Hence, $\sigma_H \neq \sigma_V$

  - Exceptions can be there based on different theories.

- Earth pressure can be defined on the basis of 3 types. i.e; active pressure, passive pressure, rest pressure.

- Terzaghi has performed various tests based on which he defined the above three earth pressures.

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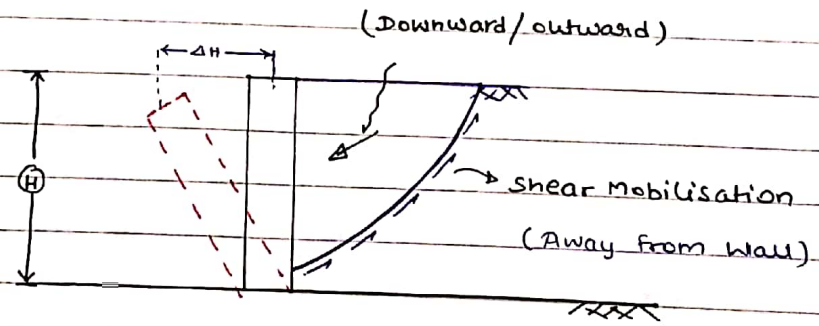
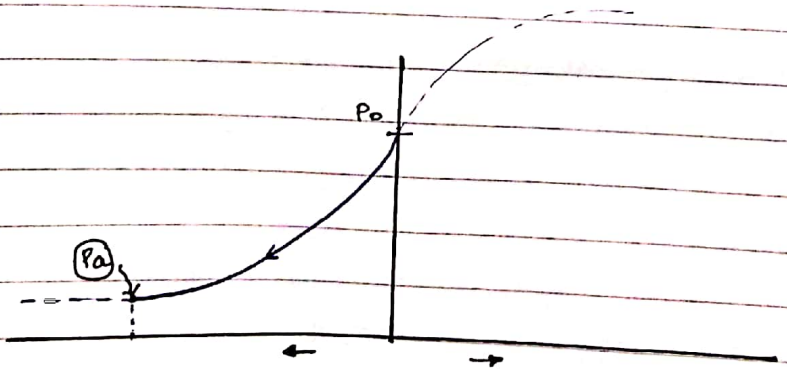
(i) Rest Pressure: If the walls of a rigid structure are unyielding, that means there is no movement of the wall then the soil is said to be in a state of rest

(ii) Active Pressure: ⊙ If the soil exerts the force on the wall due to which the walls move away from the soil. Then the soil is said to be in the state of active pressure.

⊙ The yielding of wall is given by $\left(\frac{\Delta H}{H}\right)$ ratio and due to the wall movement, the soil try to expand and shear planes are developed with the mobilization of shear away from the wall.

⊙ The soil mass which is failing tries to move downward and outward





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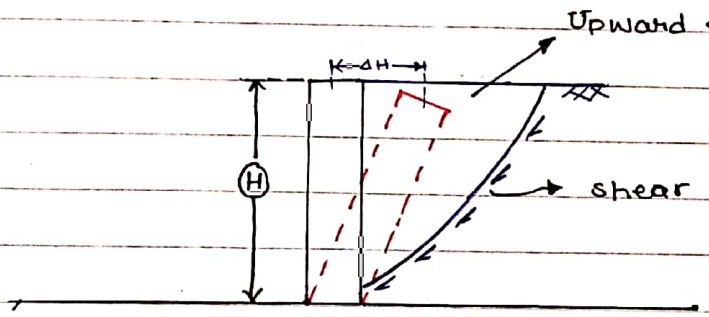
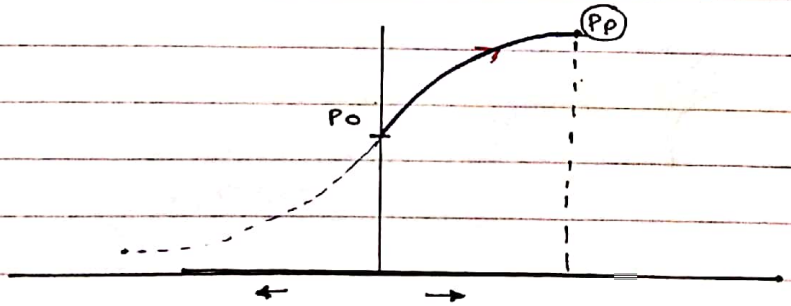
⊙ The movement of top of retaining wall to generate active earth pressure condⁿ for dense soil is given as 0.2% and for loose soil, it is given as 0.5% → $\left(\frac{\Delta H}{H} \times 100\right)$

$\left(\frac{\Delta H}{H} \times 100\right)$

(iii) Passive Earth Pressure: When the wall is pushed towards the backfill, the soil is getting compacted and the shear resistance which is mobalised is towards the wall. The soil tries to move upward and inward and the pressure exceeds the rest pressure.





⊙ To generate a passive condⁿ for the dense soil, the yielding shall be 2% and for loose soil, it should be 5%.

can vary (5% → 10%)

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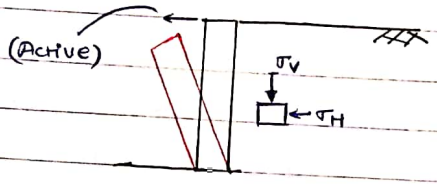
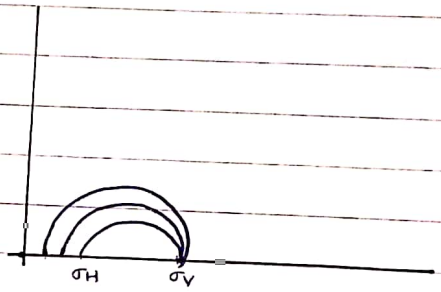
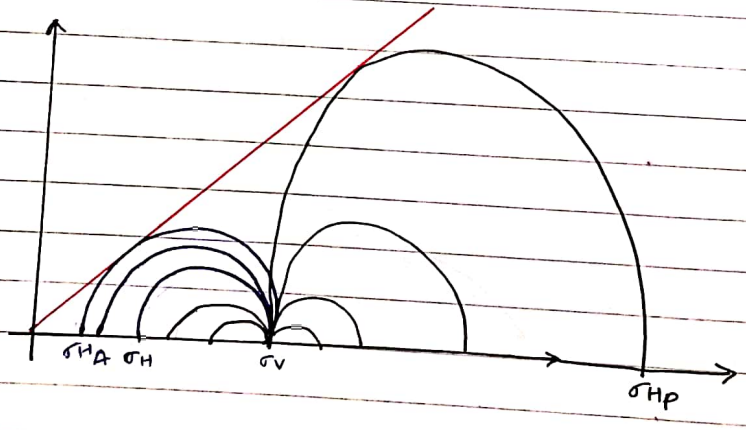
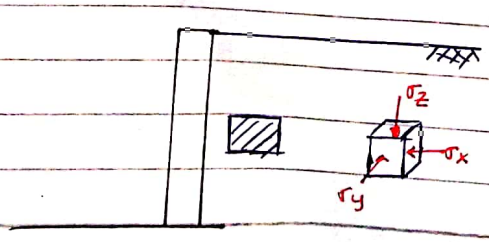
MOHR CIRCLE







Rest Pressure



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$$\epsilon_x = \frac{\sigma_x}{E} - \mu\left(\frac{\sigma_y}{E} + \frac{\sigma_z}{E}\right)$$

$$\epsilon_x = 0$$

$$(\sigma_x = \sigma_y)$$

$$0 = \frac{\sigma_x}{E} - \mu\frac{\sigma_x}{E} - \mu\frac{\sigma_z}{E}$$

$$\mu\sigma_z = \sigma_x(1-\mu)$$

$$\sigma_H = \sigma_v\left(\frac{\mu}{1-\mu}\right)$$

$$\Rightarrow K_o = \frac{\sigma_H}{\sigma_v} = \frac{\mu}{1-\mu}$$

⊙ If the soil is considered Elastic, Homogenous, isotropic, Semi-infinite, then the value of earth pressure coefficient based on elastic theory is given as:

$$K_o = \frac{\mu}{1-\mu}$$

μ is considered as a constant but in actual, its value changes depending on the drainage condition.

⊙ Based on the field measurements, the Earth pressure coefficients given by different Scientists are:

Ⓗ ALPAN → $K_o = 0.19 + 0.233 \log I_p$ (for NC clay)

$K_o = K_{o(NC)}\sqrt{OCR}$ (for OC clay)

Ⓗ JACKY → $K_o = 1 - \sin\phi$ (for sand)

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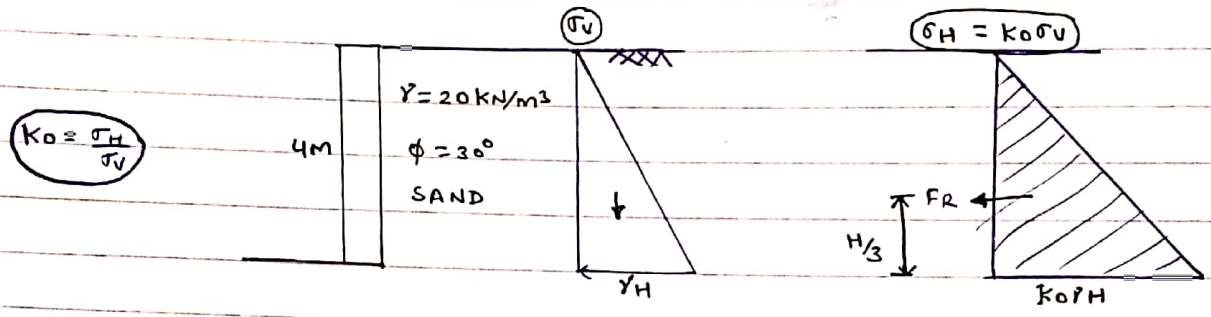
Ques → find out the force exerted by the soil on the wall if the wall is non-yielding.



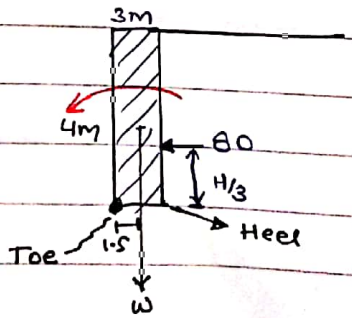
$$F_R = \frac{1}{2} \times (K_o \gamma H) \cdot H = \frac{1}{2} K_o \gamma H^2 \ \frac{KN}{m^3} \times m^2$$

$$K_o = 1 - \sin 30° = \frac{1}{2} \qquad = \frac{1}{2} K_o \gamma H^2 \left(\frac{KN}{m}\right)$$

$$F_R = \frac{1}{2} \times \frac{1}{2} \times 20 \times 4 \times 4 \qquad = 80 \ KN/m$$

Ques → In above question, calculate FOS against overturning if the retaining wall has a dimension of 3M x 4M and is made of concrete having density 25 KN/m3

Soln:



$$wt. = 3 \times 4 \times 25 = 300 \ KN/M$$

$$M_R = 300 \times 1.5 = 450 \ KN/M$$

$$M_O = F_R \times \frac{H}{3} = 80 \times \frac{4}{3}$$

$$FOS = \frac{M_R}{M_O} = \frac{450}{80 \times \frac{4}{3}} = \frac{450 \times 3}{80 \times 4}$$

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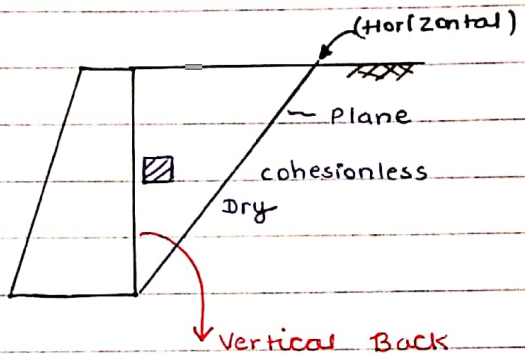
Rankine's Theory



Assumptions

- It considers the stress in a state of plastic Eqm.
- The back of the retaining wall is vertical and smooth and the backfill is horizontal.
- The soil is homogenous, dry, cohesionless (later, this theory was modified to consider the effect of water table & cohesion).
- The rupture surface is a plane surface.

Active Earth Pressure

Plastic Eqm Condn:

$$\sigma_1 = \sigma_3 \tan^2\left(45+\frac{\phi}{2}\right) + 2c\tan\left(45+\frac{\phi}{2}\right)$$

$\because c = 0$ (Assumption)

$$\sigma_1 = \sigma_3 \tan^2\left(45+\frac{\phi}{2}\right)$$

$$\Rightarrow \sigma_V = \sigma_H\left(\frac{1+\sin\phi}{1-\sin\phi}\right)$$

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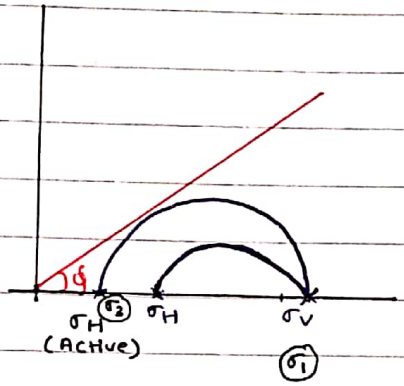
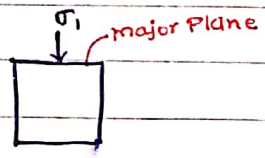
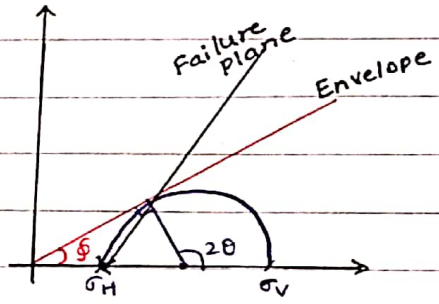
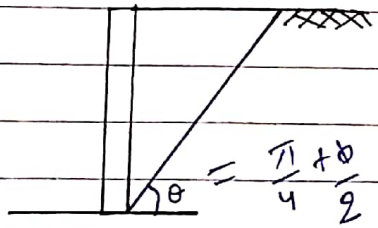


$$\frac{\sigma_H}{\sigma_V} = K_a = \frac{1-\sin\phi}{1+\sin\phi}$$

$K_a < 1$



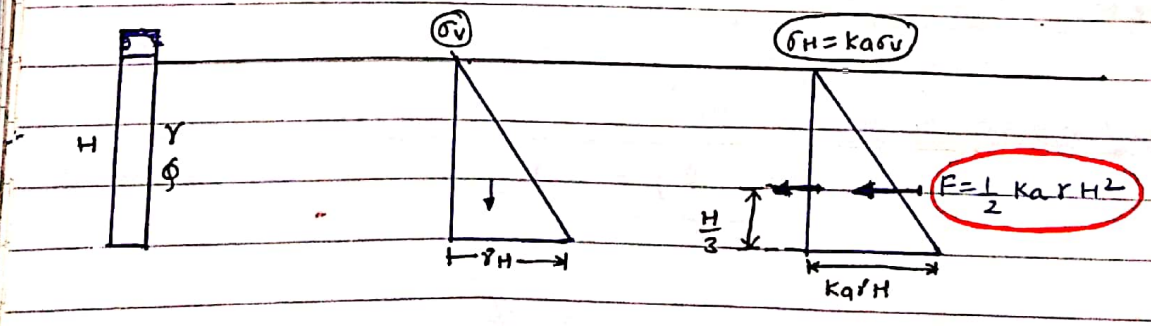




$$\theta = \frac{\pi}{4} + \frac{\phi}{2}$$

Analysis:

① No Water Table : (single Soil Mass) :



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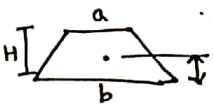
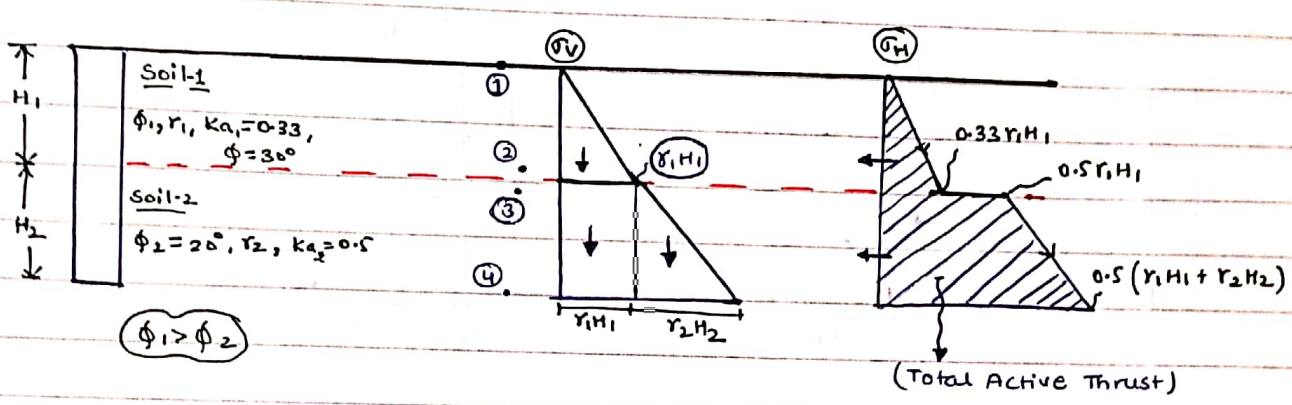
Trapezoid centroid : 

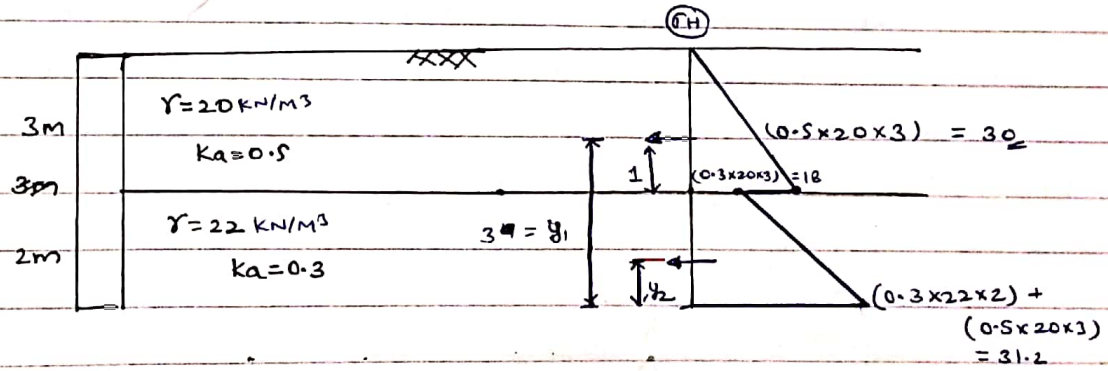
$\frac{H}{3}\left(\frac{2a+b}{a+b}\right)$

② No Water Table (Stratified Soil)



Ques→ For the figure, find the value of total active thrust and the location of thrust from the bottom.

Soln:



$A_1 = \frac{1}{2} \times 0.5 \times 20 \times 3 \times 3 = 90 \times 0.5 = 45$ KN

$A_2 = \frac{1}{2} \times [18 + 31.2] \times 2 = 49.2$ KN

$y_2 = \frac{2}{3}\left[\frac{2 \times 18 + 31.2}{18 + 31.2}\right]$

$= 0.91$ m

Total active thrust = $45 + 49.2 = 94.2$ KN/m

Line of thrust = $\frac{45 \times 3 + 49.2(0.91)}{94.2} = 1.908$ m from the Bottom

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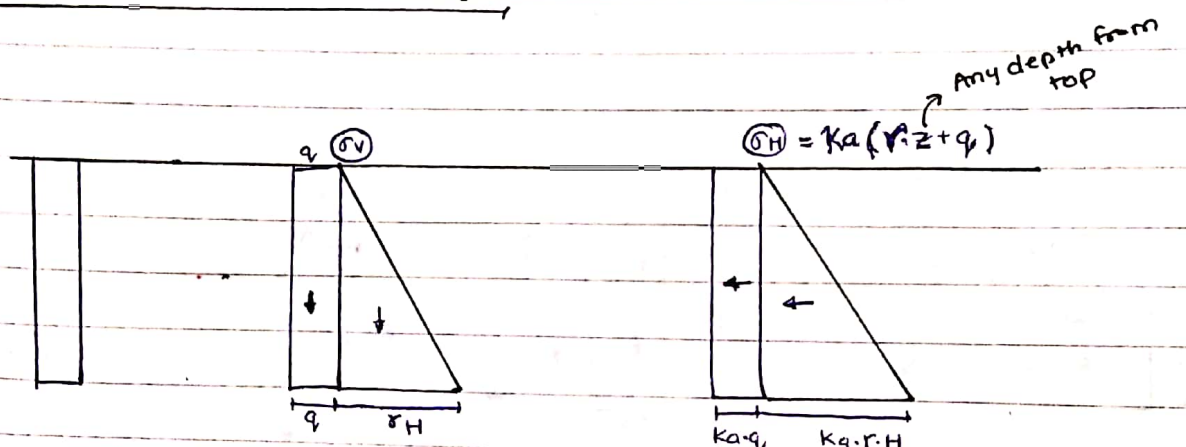
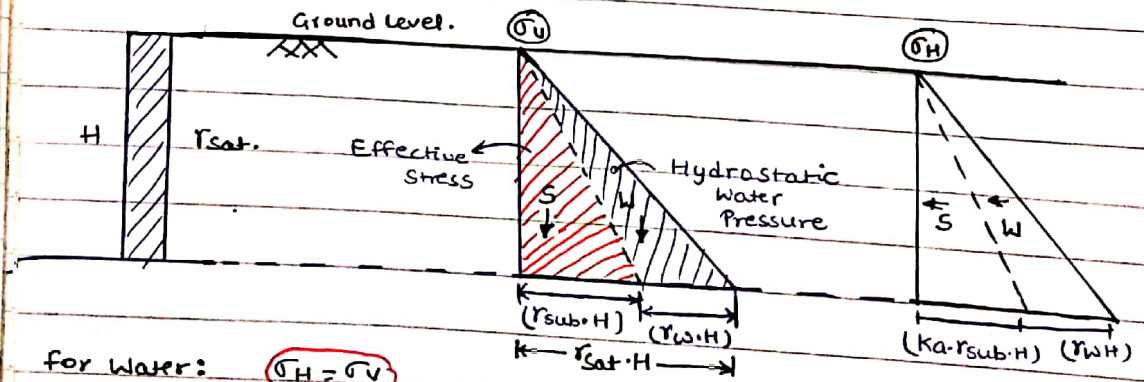
③ No Water table (Surcharge):



BELL THEORY

- The theory proposed by rankine was modified by Bell to consider the effect of water table & cohesion.

④ Water Table (upto Ground Surface):



for Water: $\sigma_H = \sigma_V$

for Soil: $\sigma_H = K_a \sigma_V$

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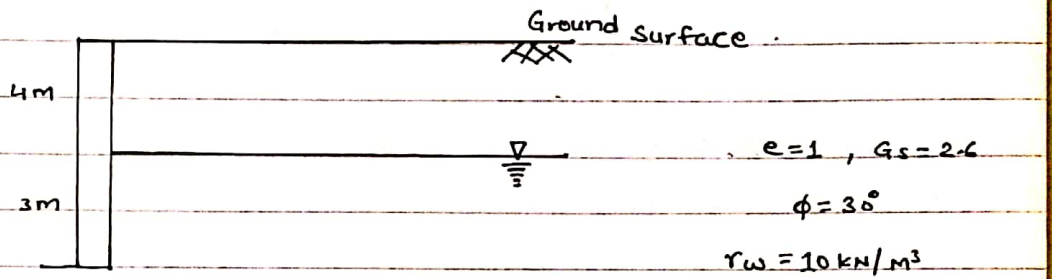
Ques→ For the soil system, calculate the total active thrust.

$\gamma_w = 10\ kN/m^3$

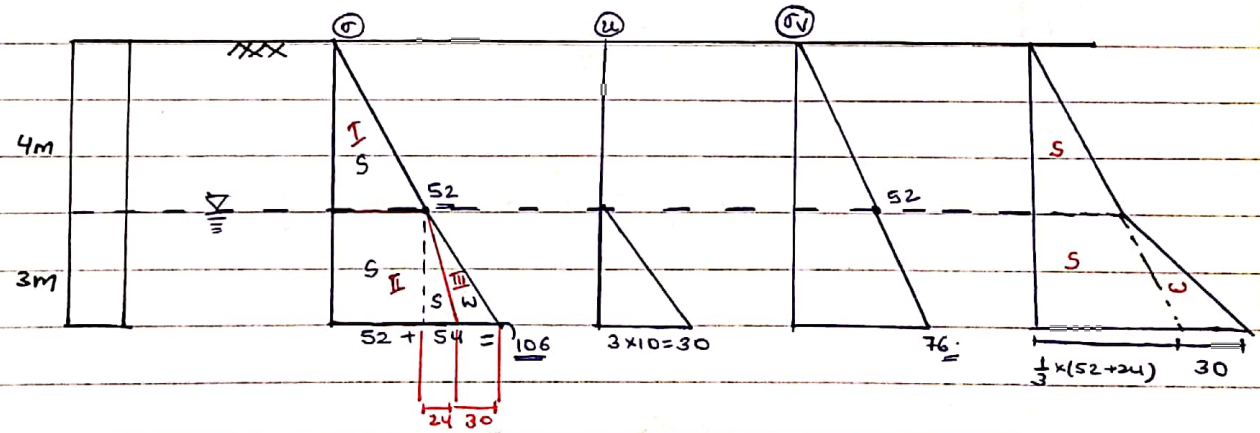


$Se = wG$

$\gamma_d = \frac{G_s \gamma_w}{(1+e)} = 13\ kN/m^3$

$\gamma_{sat.} = \frac{(G_s+e)\gamma_w}{1+e} = \frac{3.6 \times 10^5}{2} = 18.0\ kN/m^3$

$\gamma_{sub.} = 8\ kN/m^3$



Total thrust = $\frac{1}{2} \times \frac{52}{3} \times 4 + \frac{1}{2} \times 3 \left[\frac{52}{3} + 55.33 \right]$

($\frac{I}{3}$) ($\frac{I}{3}$) ($\frac{II}{3}$)+III ($\frac{52+24}{3}$)+30

⑤ c-ϕ soil (No-water table)

under plastic Eqm Eqn:

$$\sigma_1 = \sigma_3 \tan^2\left(45 + \frac{\phi}{2}\right) + 2c \tan\left(45 + \frac{\phi}{2}\right)$$

$$\sigma_v = \sigma_H \left(\frac{1+\sin\phi}{1-\sin\phi}\right) + 2c \sqrt{\frac{1+\sin\phi}{1-\sin\phi}}$$

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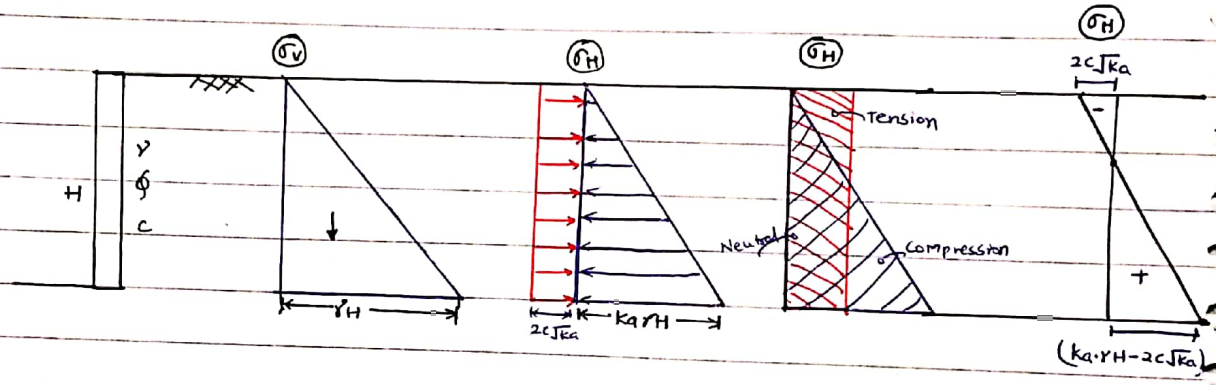
$$\sigma_H = \sigma_V \left(\frac{1-\sin\phi}{1+\sin\phi}\right) - 2c\sqrt{\frac{1-\sin\phi}{1+\sin\phi}}$$

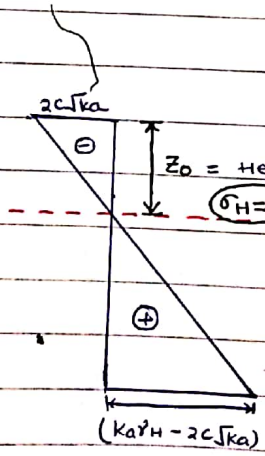
$$\sigma_H = \sigma_V \cdot K_a - 2c\sqrt{K_a}$$

$2c\sqrt{K_a}$ → independent of Height. (z)

① The ~~reduce~~ / ef Effect of cohesion, is to reduce the value of active earth pressure.



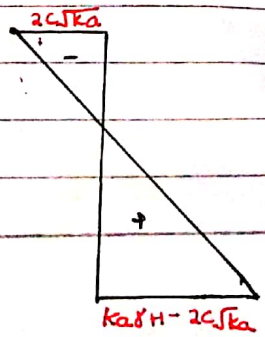
Total Active Thrust : $= (-2c\sqrt{K_a} \cdot H) + \left[\frac{1}{2} K_a \gamma H^2\right]$



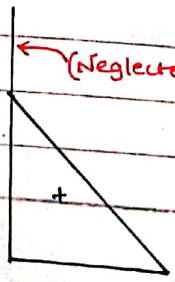
$$\sigma_H = K_a(\gamma \cdot z_0 + q) - 2c\sqrt{K_a}$$

$$z_0 = \frac{2c}{\gamma\sqrt{K_a}}$$

When Tension crack not occured.



When Tension crack has occured.



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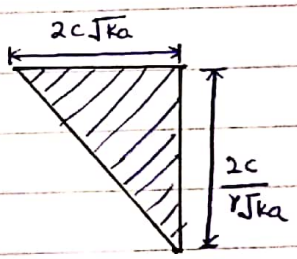
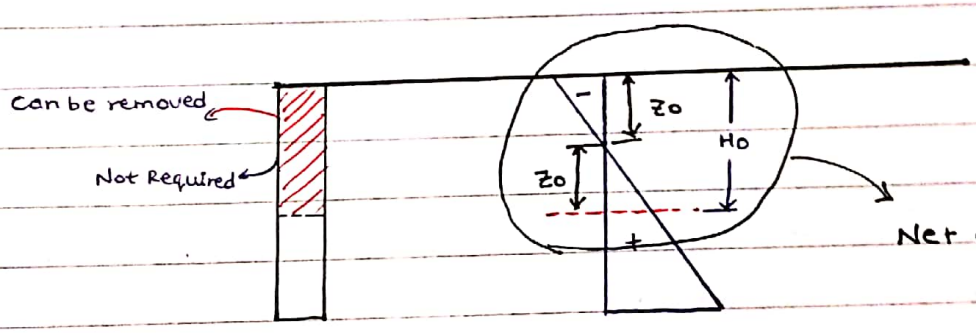
Due to this, increase in force,



Extra force = $\frac{2c^2}{\gamma}$

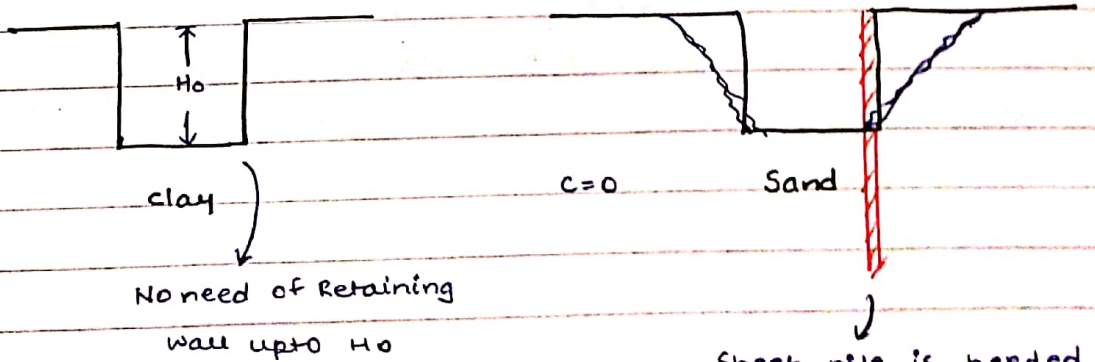


Net active thrust = 0

$H_0 = 2z_0$

$\Rightarrow H_0 = \frac{4c}{\gamma\sqrt{K_a}}$

= critical depth.



$\phi = 0$

$c = c$

$K_a = 1$

$H_0 = \frac{4c}{\gamma}$

No need of Retaining Wall upto H_0

Sheet pile is needed

Because $c = 0$

So; $H_0 = 0$

To maintain verticality of soil

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1 tonne = 10,000 N = 10 kN
or = 9.81 kN = 9810 N

- For design purpose, the loss of contact is assumed to have occured.
- In practice, tension can't be taken on the wall since tension crack tends to develop within the soil and it may not remain adhered to the wall.
- In calculation of the total thrust, usually tension zone is ignored which results in a larger value of the active earth pressure or thrust.
- At a theoretical concept, no tension crack condn will result in the net active thrust value for a region upto a depth Ho as zero (0).

  Hence, a vertical cut without any lateral support can be theoritically done upto a depth (Ho).

Ques → ① $d = 4.5\,m$ $\quad \gamma = 1.85\ t/m^2$

$\phi' = 16°$

$c = 1.91\ t/m^2$

Determine the force at the bottom of vertical cut after the formation of tension crack.

Solⁿ:

Force; $F = \frac{1}{2} K_a \gamma H^2 - 2c\sqrt{K_a} H + \frac{2c^2}{\gamma}$

$K_a = \frac{1-\sin 16°}{1+\sin 16°} = 0.567$

≈ 0.57

$$F = \frac{1}{2} \times 0.57 \times 1.85 \times 4.5^2 - 2 \times 1.91 \sqrt{0.57} \times 4.5 + \frac{2 \times 1.91 \times 1.91}{1.85}$$

$F = 10.67 - 12.978 + 3.94$

$= 1.635\ t/m = 16.04\ kN/m$

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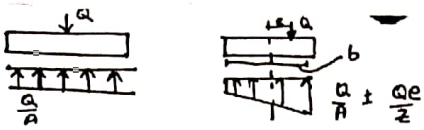
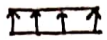
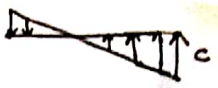


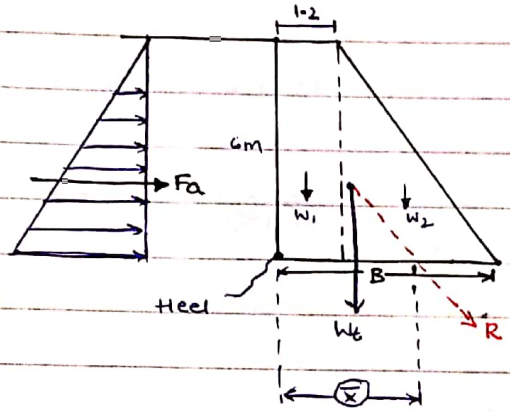
$e = 0$ 

$e < \frac{b}{6}$

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

$e > \frac{b}{6}$ 

Ques → (5)



for No tension; ~~B=~~ $e = \frac{B}{6}$

and ~~$\bar{x}$~~ $e = (\bar{x} - B/2)$

$$\bar{x} = \frac{\Sigma M_{Heel}}{\Sigma V}$$

$Wt. = W_1 + W_2$

$$W_1 = 6 \times 1.2 \times 20 = 120 \times 1.2 = 144 \text{ KN/M}$$

$$W_2 = \frac{1}{2} \times (B-1.2) \times 6 \times 20 = 60(B-1.2) \text{ KN/M}$$

$$= (60B - 72) \text{ KN/M}$$

$$Wt. = W_1 + W_2 = 144 + 60B - 72 = 72 + 60B$$

$$Ka = \frac{1-\sin 35^\circ}{1+\sin 35^\circ} = 0.27$$

$$\Sigma V = (72 + 60B)$$

ΣM about Heel $= Fa \cdot \frac{H}{3} + W_1 \cdot x_1 + W_2 \cdot x_2$

$$= \left(\frac{1}{2} \times 0.27 \times 16 \times 6 \times 6 \times 2\right) + 144 \times 0.6 + (72+60B) \times 1.2 + \frac{(B-1.2)}{2}$$

$$= \frac{242.49 + 60(B/3 + 0.8)(B-1.2)}{144 + 60(B-1.2)}$$

$$e = \bar{x} - \frac{B}{2}$$

$$\frac{2B}{3} = \frac{242.49 + 60(B/3 + 0.8)(B-1.2)}{144 + 60(B-1.2)} - \frac{B}{2} = \frac{B}{6}$$

$$\Rightarrow B(96 + 40(B-1.2) = 242.49 + (20B + 48)(B-1.2)$$

$$96B + 40B^2 - 48B = 242.49 + 20B^2 - 24B + 48B - 57.6$$

$$\Rightarrow B = 2.5$$

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Ques-8

$\sigma_1 = K_a(\gamma \cdot z + q) - 2c\sqrt{K_a}$

$K_a q = 2c\sqrt{K_a} \Rightarrow q = \frac{2c\sqrt{K_a}}{K_a}$

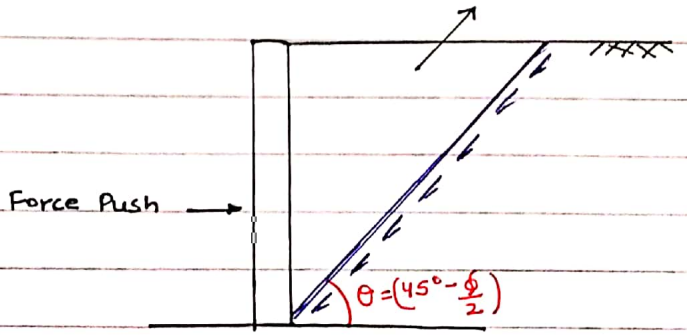
Tip $\Rightarrow z = 0$

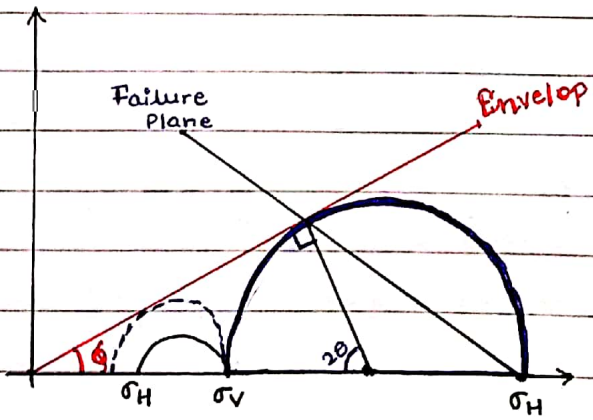
$= \frac{2c}{\sqrt{K_a}} = 2c\cot\alpha$

$K_a = \frac{1-\sin\phi}{1+\sin\phi} = \tan^2\alpha$

q

Passive Earth Pressure





$2\theta = 90 - \phi$

$\theta = (45° - \frac{\phi}{2})$

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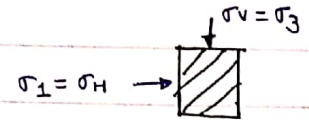


Plastic Eqm Eqn:

$$\sigma_1 = \sigma_3 \tan^2\alpha + 2c \tan\alpha$$

$$\alpha = \left(45° + \frac{\phi}{2}\right)$$

$c = 0$ (Assumption)

$$\sigma_H = \sigma_v \left(\frac{1+\sin\phi}{1-\sin\phi}\right)$$

$$K_p = \frac{\sigma_H}{\sigma_v} = \left(\frac{1+\sin\phi}{1-\sin\phi}\right)$$

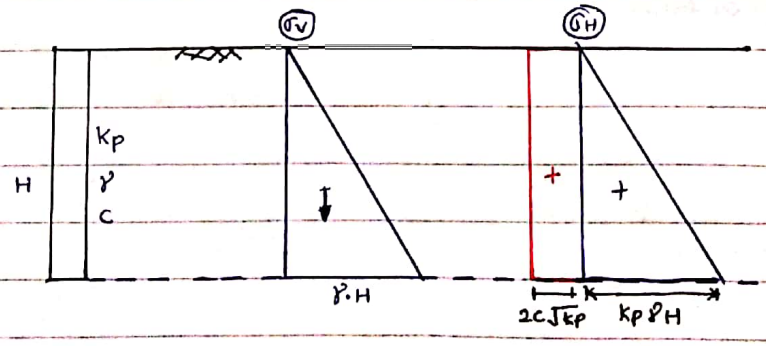
c-ϕ soil :

$$\sigma_H = \sigma_v \cdot K_p + 2c\sqrt{K_p}$$

Note: ⊙ The Effect of cohesion is to increase passive earth pressure.

⊙ There is no tension region in the Passive Pressure theary.

(1)



$$F_p = \frac{1}{2} K_p \gamma H^2 + 2c\sqrt{K_p} \cdot H$$

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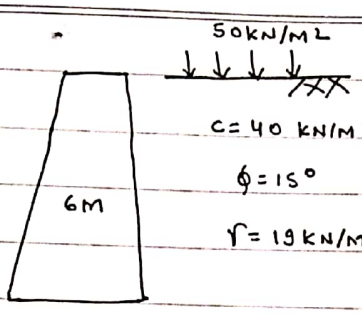
Ques → (2)

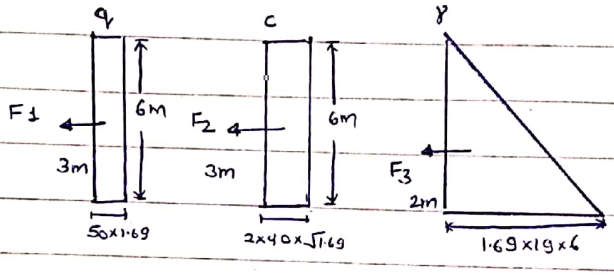


$c = 40$ kN/m²

$\phi = 15°$

$\gamma = 19$ kN/m³

$$K_p = \left(\frac{1+\sin 15°}{1-\sin 15°}\right) = 1.6984$$



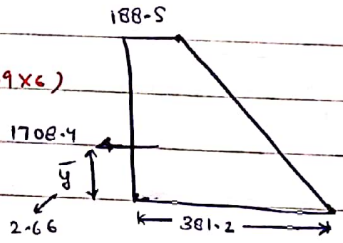
~~Moment = F1×3 + F2×3 + F3×2~~

~~= (50×1.69)×~~

$f = \frac{1}{2} K_p \gamma H^2 + 2c\sqrt{K_p} H + q \cdot K_a \cdot H$

$= \left(\frac{1}{2} \times 1.69 \times 19 \times 6 \times 6\right) + (2 \times 40\sqrt{1.69} \times 6) + (50 \times 1.69 \times 6)$

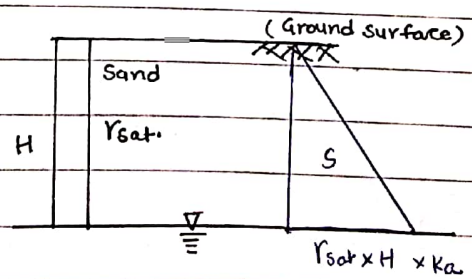
$= 1708.98$ kN/m

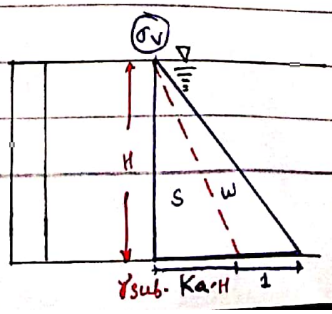


⇒ Moment = 1708.4×2.66

$= 4544.344$ kN-m/m

Ques → In the given fig., if water table rises upto the ground surface, what will happen to the active earth pressure on the wall.





Ka < 1

⇒ Value of Fa will increase.

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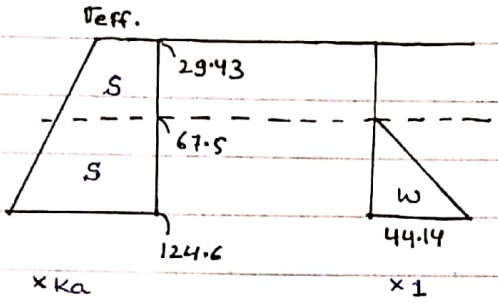
Ques → (4)

$\sigma_{total} = \sigma_{eff} + u$



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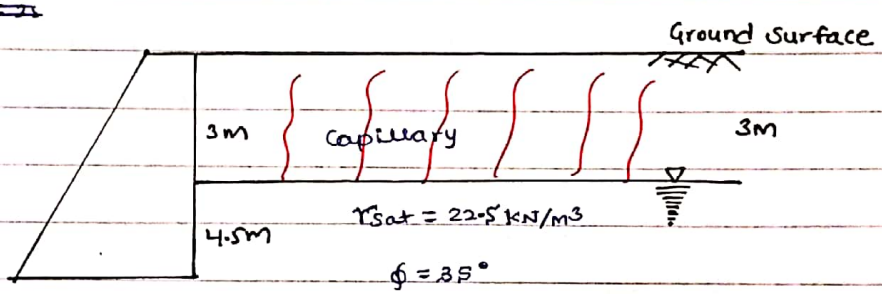
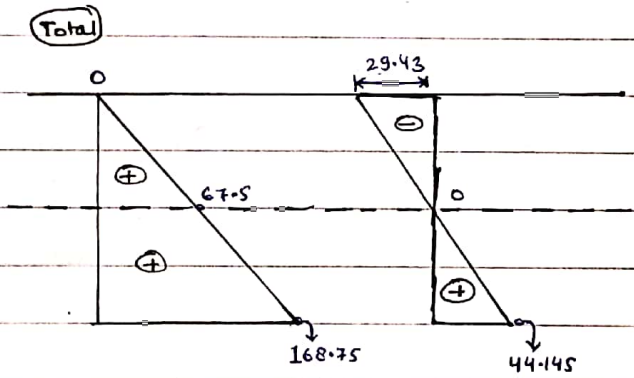
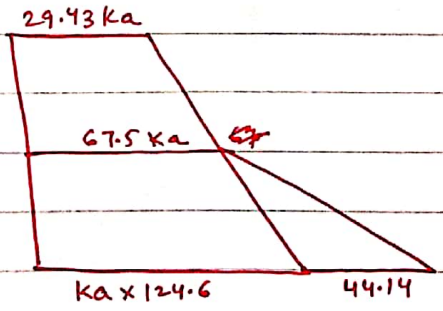




$F = 124.6 \times Ka + 44.14 \times 1 = 258$

$Ka = 0.278$



$$F = \frac{1}{2}\left[(29.43 + 67.5) \times 3\right] + \left[\frac{1}{2}\left\{67.5\,Ka + [124.6\,Ka + 44.14]\right\} \times 4.5\right]$$

$$= 145.395 + 219.47 = 364.86 \text{ KN/m}$$

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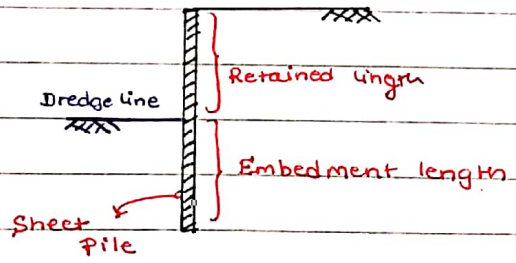
Flexible in Nature

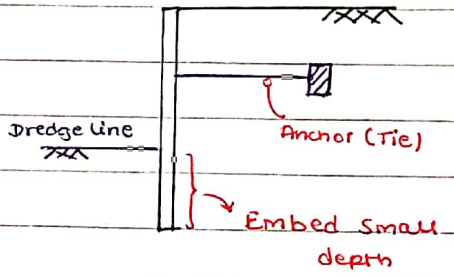
Sheet Piles

- Sheet piles are also known as flexible retaining walls, that are used to support soil temporarily after the excavation. They are driven into the soil and act as a continuous wall which is retaining the earth mass.

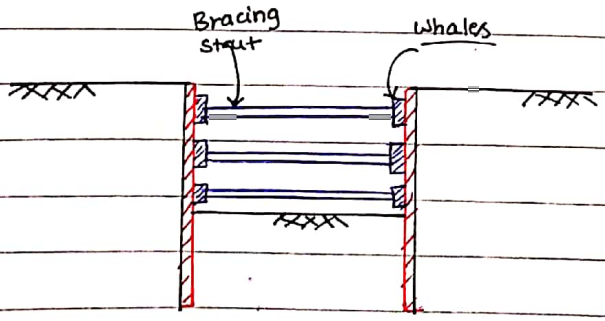
- These are generally made of steel, which has high strength, it is having a very light weight and is thin.



Cantilever Sheet Pile



Anchored Sheet Pile



Braced sheet Pile

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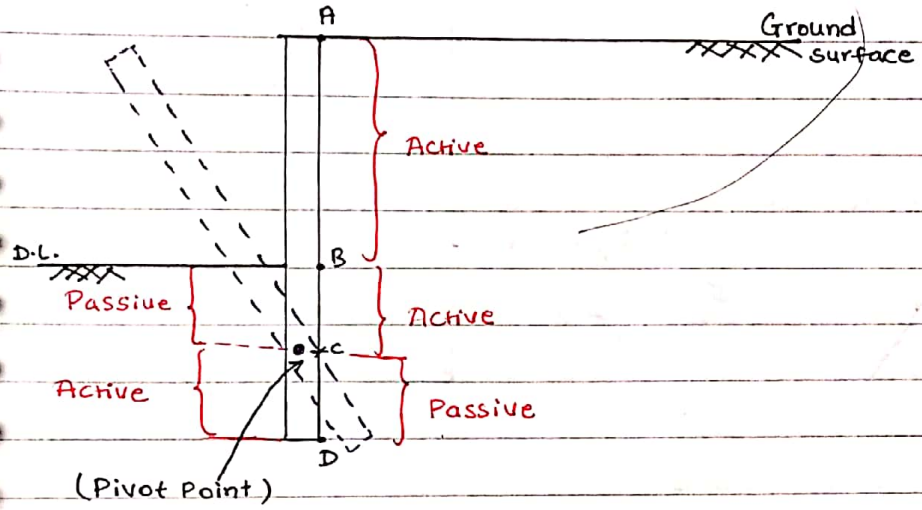
ANALYSIS

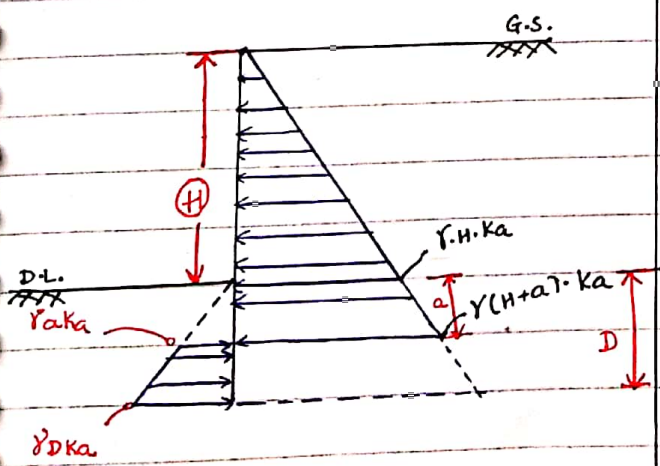


⊙ For cohesionless soil, following Active and Passive earth pressure diagrams are as follows:

Active Pressure diagram

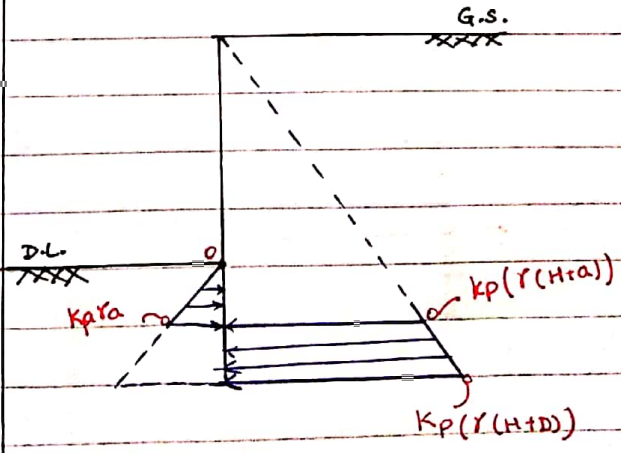
$\sigma_H = \sigma_v \cdot K_a$

$\sigma_v = \gamma \cdot z$



Passive Pressure diagram

$\sigma_H = \sigma_v \cdot K_p$



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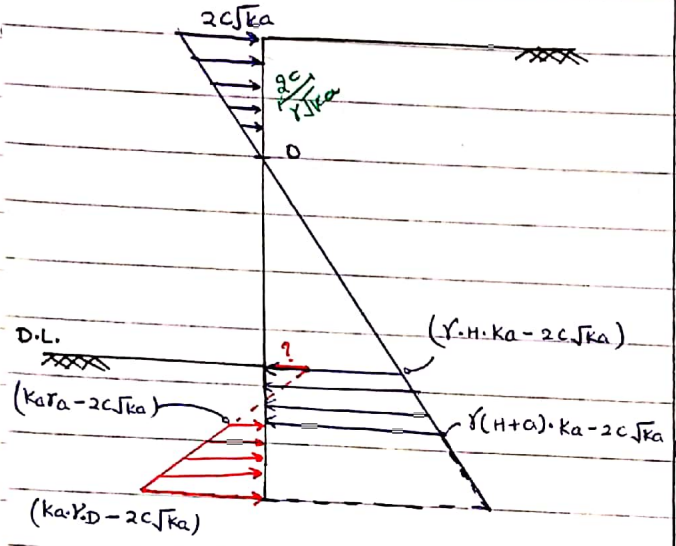
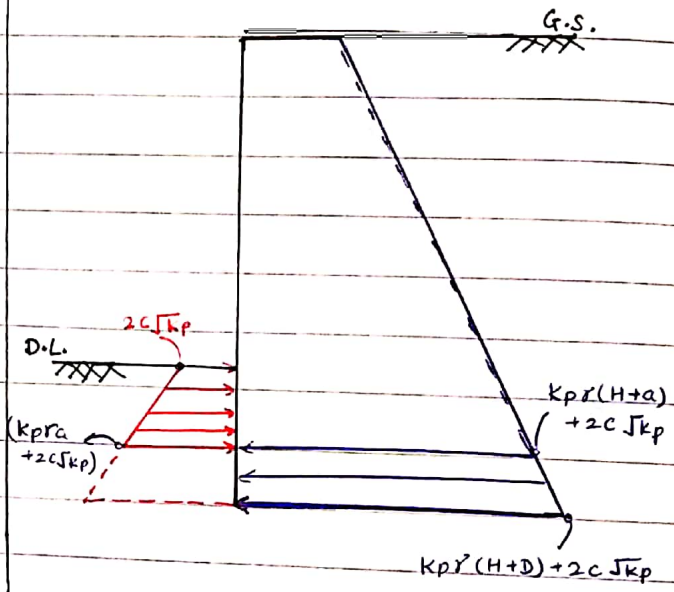
C-ϕ Soil

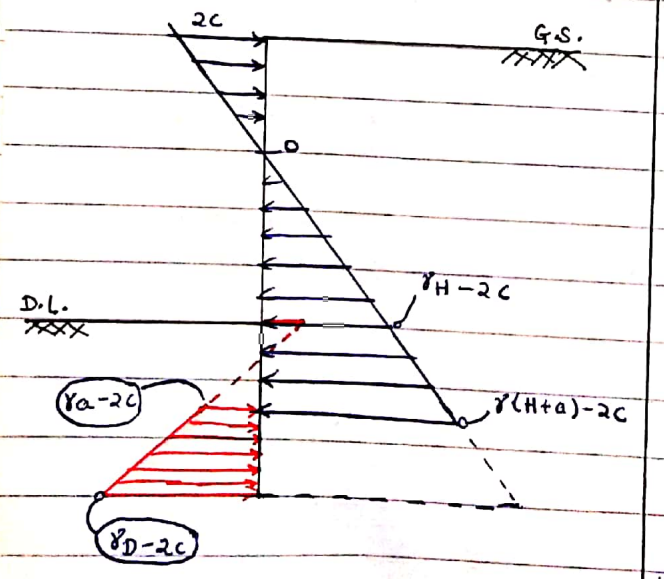
$\sigma_H = K_a \gamma z - 2c\sqrt{K_a}$

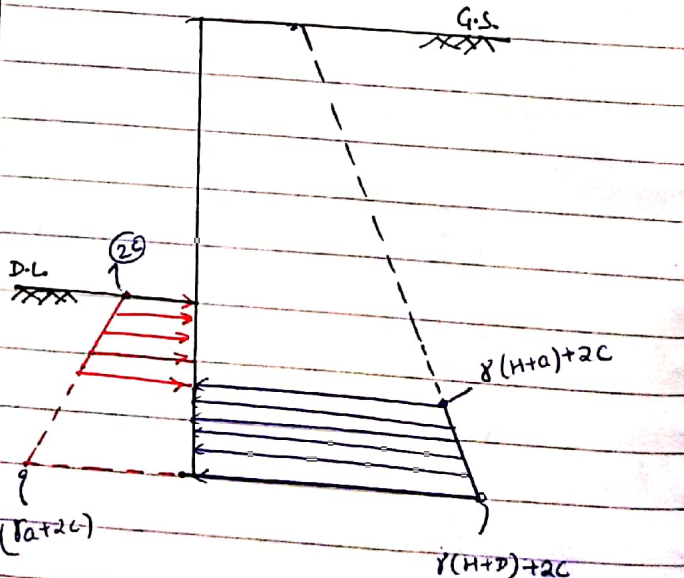


$\sigma_H = K_p \gamma z + 2c\sqrt{K_p}$



◉ For purely cohesive soil, $\phi = 0°$. Hence, $K_a = K_p = 1$





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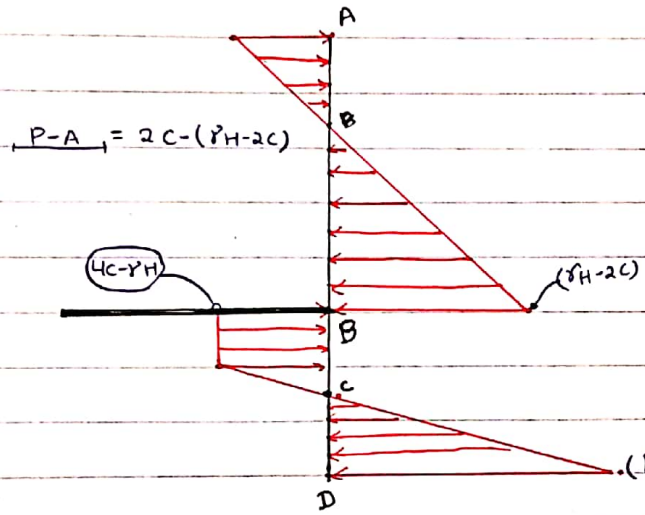
⊙ The resultant diagram is based on assuming that there will be some consolidation over a period of time which will take place. Also, the pile is of flexible nature. Hence, it will get deformed due to overturning and there can be shrinkage, swelling, loss of contact with the pile or development of tension cracks.

Resultant Diagram

(For the stability of sheet pile)

⇒ Net trust is always zero

[Active Force = Passive force]

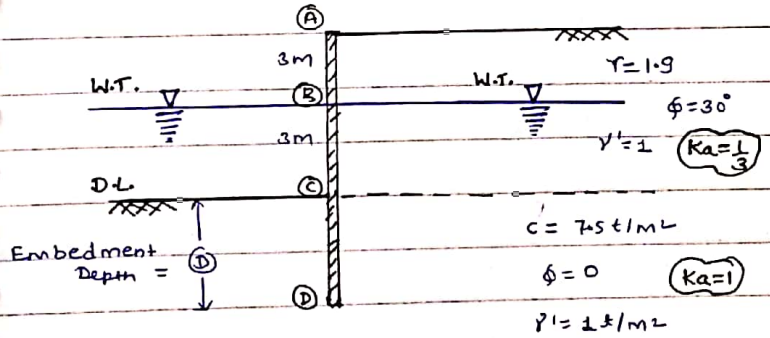


⇒ $\Sigma M_D = 0$ ⇒ $\Sigma F_x = 0$

(Eqm cond^n's)

(A-P)

Ques→



Find out the earth pressure at point A, B, C and D.

Soln:

Sand: $\sigma_H = K_a \gamma' z$ Clay: $\sigma_H = K_a \gamma' z - 2c\sqrt{K_a}$

$\sigma_H = K_p \gamma' z$ $\sigma_H = K_p \gamma' z + 2c\sqrt{K_a}$ $K_a = \frac{1}{3}$

Since; WT is on same level on both sides. So, Hydrostatic pore water pressure will cancel each other from both sides.

So, the Net pressure will be due to effective stress only

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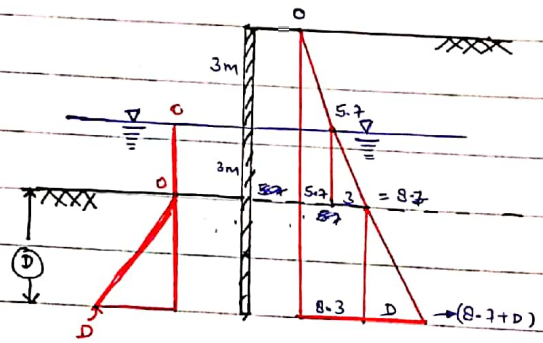
At (A): $(\sigma_H)_A = 0$ $\quad (\sigma_H)_P = 0$

At (B): $(\sigma_H)_A = \frac{1}{2} \times K_a \times \gamma H^2$

$= \frac{1}{2} \times \frac{1}{3} \times 1.9 \times 3 \times 3$

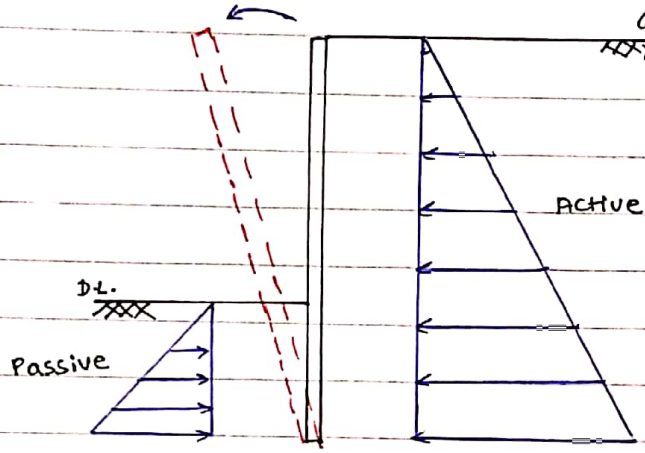
$= 1.9$ t/m²



		Active	Passive	Resultant
A		0	×	0
B		1.9 (←)	×	1.9 (←)
C	Above	2.9 (←)	×	2.9 (←)
	Below	$1 \times 8.7 - 2 \times 7.5 \times \sqrt{1}$ $= (6.3)$ (←)	$1 \times 0 + 2 \times 7.5 \sqrt{1}$ $= 15$ (→)	(→) 8.7
D		$(1 \times D) - 2 \times 7.5 \sqrt{1}$ $= [D - 15]$ (→)	$1 \times (8.7 + D) + 2 \times 7.5 \times \sqrt{1}$ $= [23.7 + D]$ (←)	38.7 (←)

Approximate Diagram for Analysis

Approximate diagram for Analysis



Ques: A 6m deep excavation in sandy soil is performed and a cantilever sheet pile ~~in~~ is provided, $\gamma = 18\ KN/M^3$, $\phi = 35°$, $FOS = 2$

Using the simplified analysis is find out the depth of Embedment.

Soln:

$$FOS = \frac{\text{Resisting Moment}}{\text{Disturbing Moment}}$$

$K = 0.27$

$K_p = 3.69 \simeq 3.7$

$$FOS = \frac{\frac{1}{2} \times 3.7 \times 18 \times D^2 \times \frac{D}{3}}{\frac{1}{2} \times 0.27 \times 18 \times (6+D)^2 \times \left(\frac{6+D}{3}\right)} = 2$$

$$\Rightarrow = \frac{D^3}{3} \times 3.7 = 0.27 \times \frac{(6+D)^3}{3} \times 2$$

$$\Rightarrow (6+D)^3 = (6.85)^3 D^3$$

$$\Rightarrow 6+D = 1.9\, D^{3}$$

$$\Rightarrow \boxed{6.7\,m}$$

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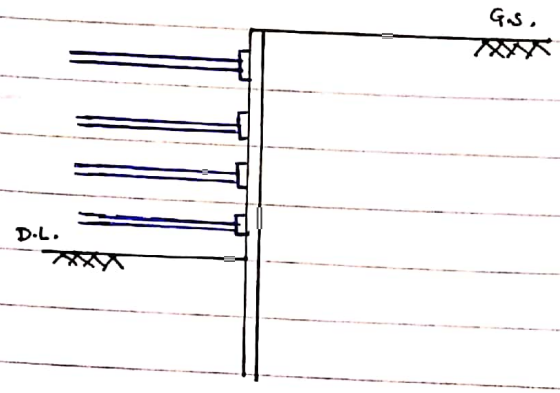
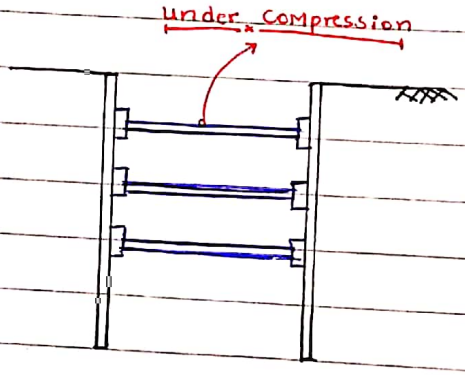
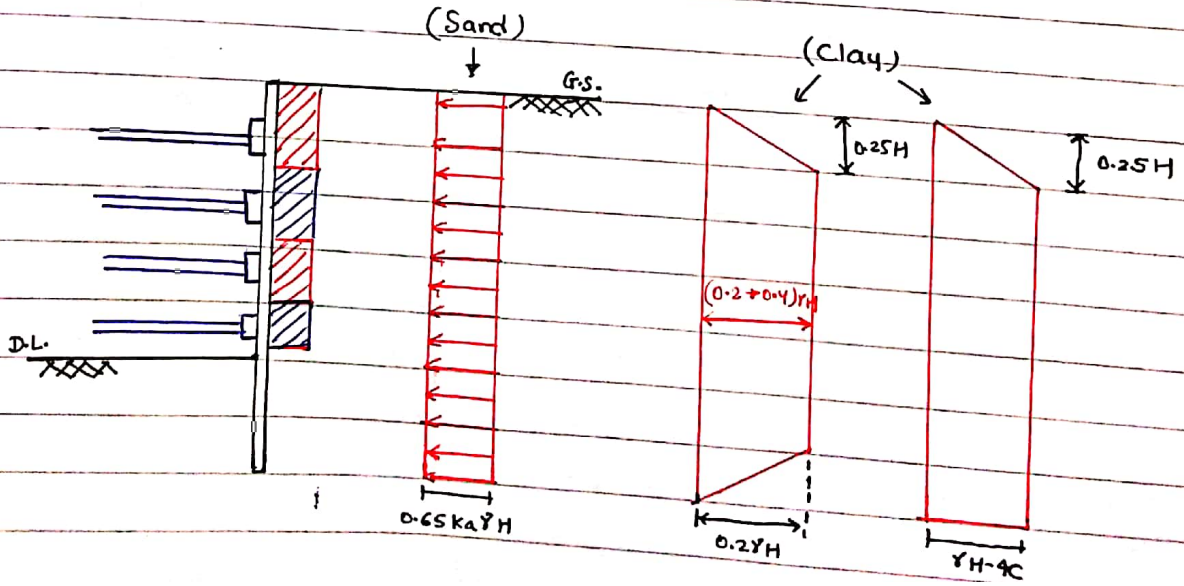
Braced System







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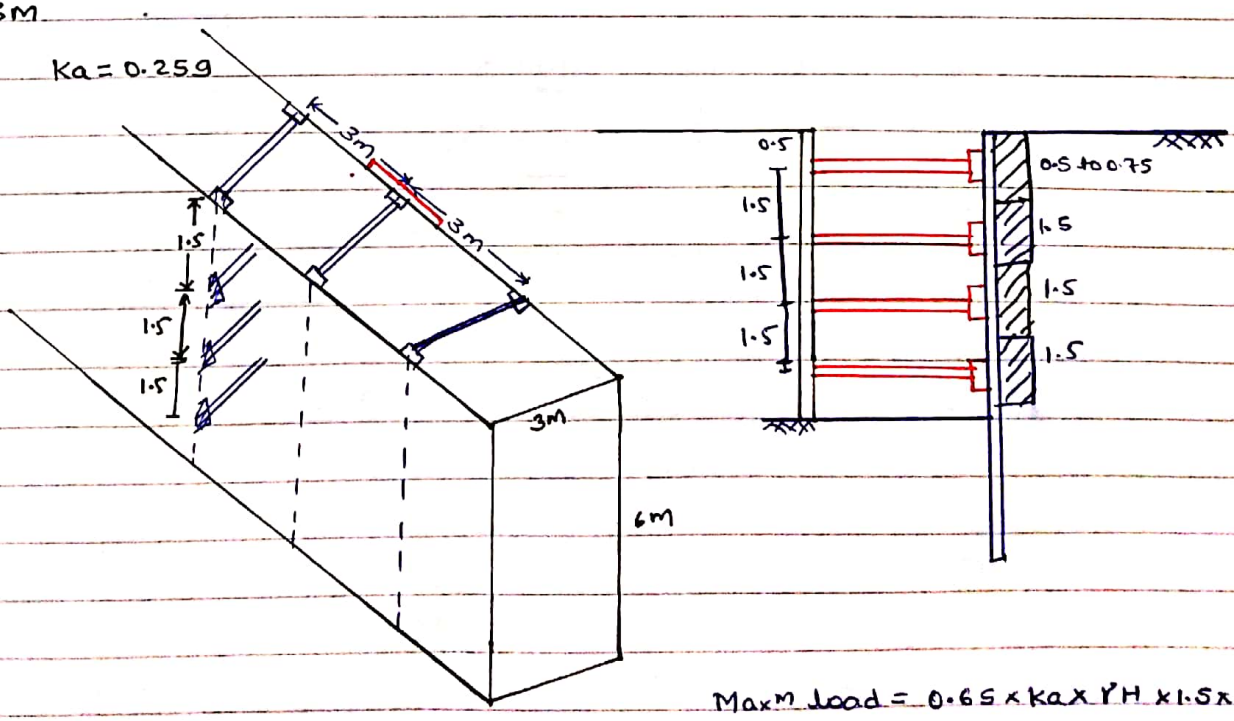
⊙ In a braced Excavation, the theory proposed by Peck, Hanson and Thornburn suggest that the earth pressure distribution on the back filled side of the sheet pile is represented by an apparent earth pressure diagram which is as shown in previous figure.

⊙ An imaginary strut is assumed at the dredge line for the analysis. The load carried by each strut is based on the tributary area which is taken upto half of the distance b/w the struts above and below the strut level.

Ques→ An Excavation is made 3m wide, 6.5m deep and is proposed in a sandy soil having $\phi = 36°$. Assuming the Ist row of strut to be located 0.5m below the Ground surface and spacing b/w them as 1.5 m. Calculate the maxm strut load. $\gamma = 20\ kN/m^3$, Horizontal spacing of the strut = 3m.

Soln: Ka = 0.259



Maxm load = $0.65 \times Ka \times \gamma H \times 1.5 \times 3$

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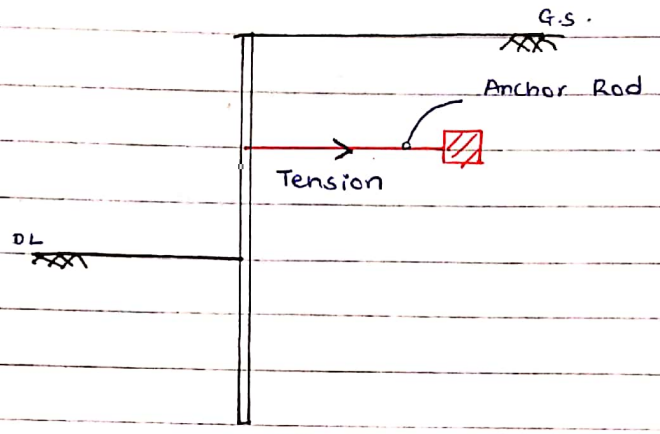
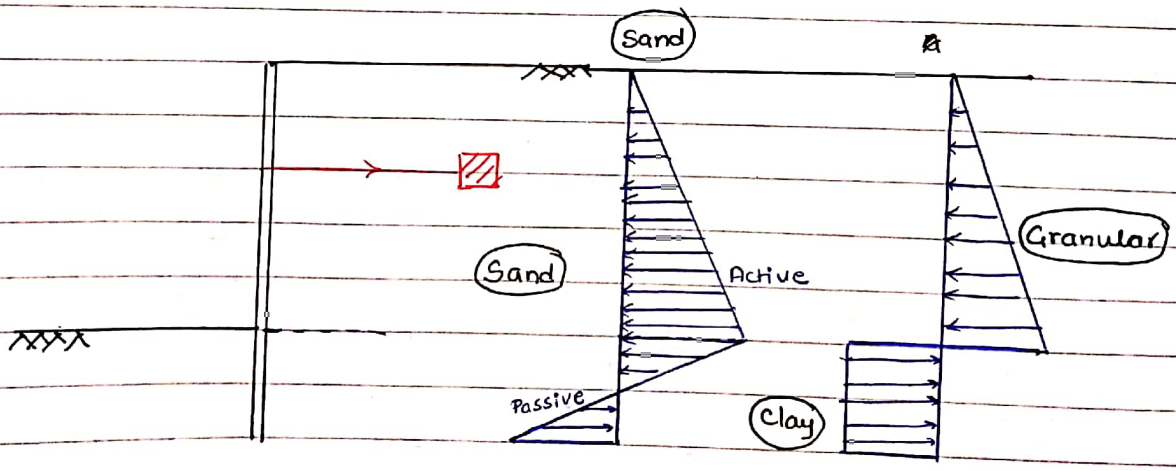
Anchored sheet Pile



⊙ In anchored bulk head, the amount moment Eqn is considered about the anchor level and the resultant earth pressure diagram is as follows:



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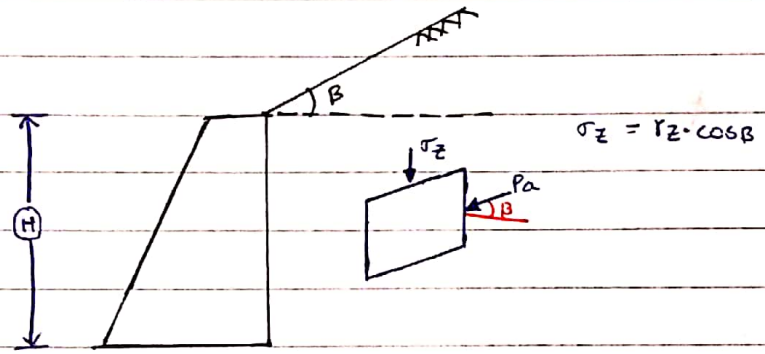
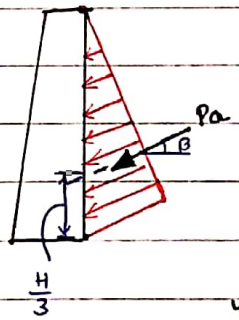
Modifications in Rankines Assumption

① Inclined Backfill :



$\sigma_z = \gamma z \cdot \cos\beta$



$$P_a = \frac{1}{2} K_a \cdot \gamma \cdot H^2 \cos\beta$$

@ $\frac{H}{3}$ from bottom

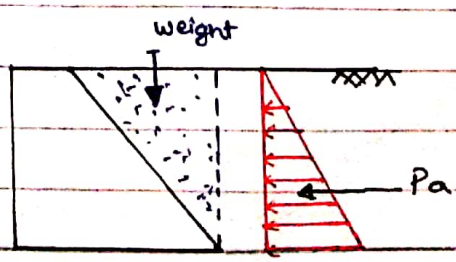
$$K_a = \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}$$

Gate:

$$K_a = \left(\frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}} \right) \cdot \cos\beta$$

⊙ The vertical stress and lateral pressure are considered to be conjugate - i.e; the direcn of one is parallel to the plane in which the other acts.

② Inclined Back of Wall :



Total = (Wt. + Pa)

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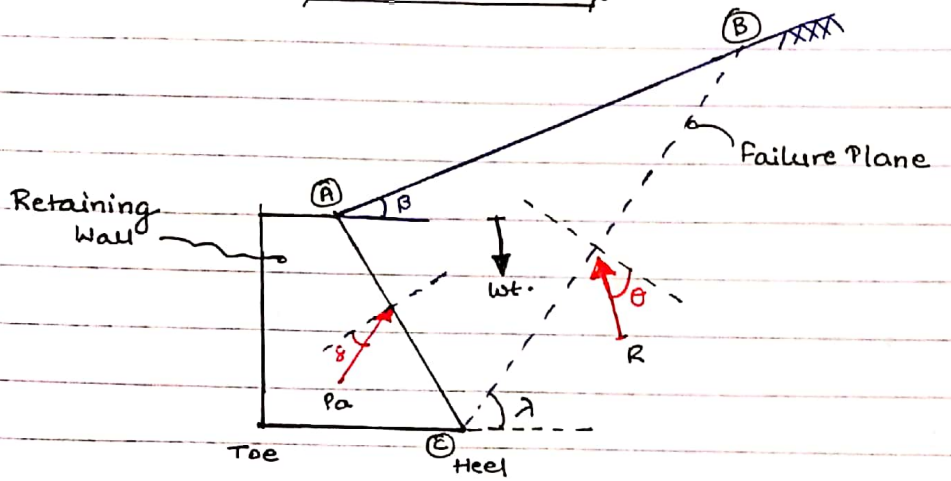
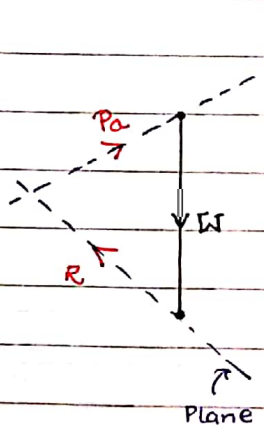
Columb's Theory



Force Triangle

Pa = ?
R = ?



Infinite No. of planes can be taken.

	(Pa)	(Pp)
Plane-1	50	300
Plane-2	60	250
Plane-3	100	280
Plane-4	120 → maxm	200
Plane-5	90	120 → min.

Critical

Min. Value of force at which soil get destablize

(Active force)$_{Rankine}$ > (Active Force)$_{Columb}$

(Passive Force)$_{Rankine}$ < (Passive force)$_{Colum}$

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Assumptions:

① The backfill is assumed dry, cohesionless and isotropic
② The back of the wall and the backfill can be inclined.
③ The friction b/w the wall and the soil is considered.
④ Actual failure plane is curve in the bottom region but assumed plane for analysis.
⑤ The failure plane is assumed to pass through the heel.
⑥ The sliding wedge is considered as a rigid body and λ is the angle of failure plane with the horizontal.
⑦ At the failure condⁿ, the trial wedge is in Eqᴹ under the action of 3 forces that are W, R and Pa.
W (wight), R (Reacⁿ force at the failure plane which is inclined at an angle ϕ from the normal to the failure plane). P (earth pressure force which is inclined at an angle δ with the normal to the back of the wall.

The value of δ is assumed as follows:

① for smooth walls → $\delta = \frac{\phi}{3}$

② for ordinary retaining wall made of concrete → $\delta = \frac{2}{3}\phi$

③ For retaining walls where provision of drainage of soil provided through weep holes
↳ $\delta = \frac{3}{4}\phi$

④ If the soil is subjected to vibration, $\delta = 0$

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⑤ The direcn of all the 3 forces are known and the value of W are also known. Hence, using this data, a force Δ (triangle) can be drawn and the value of force Pa can be found out

⑥ For active earth pressure force, the highest among all the trial planes is considered as the critical value and for the passive earth pressure force, the min. of all the trial planes is considered as the critical.

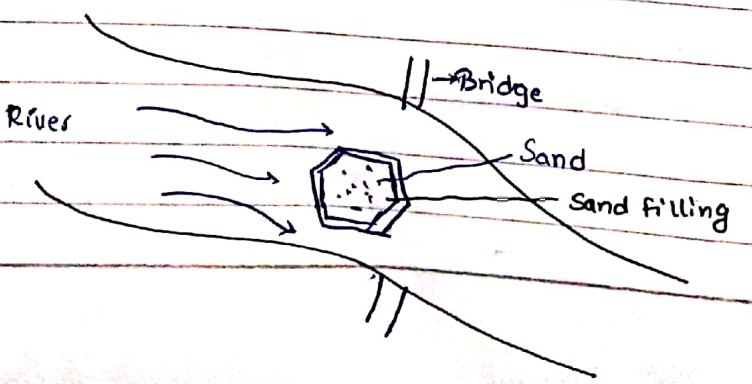
⑦ ~~Rankine's~~ *

⊙ Rankine's theory over estimates the active earth pressure force and underestimates the passive earth pressure force.

Note: ⊙ Apart from the Emperical methods given by Rankine and Columb, there are some Graphical methods also, which are given by → REHBAN'S and CULMAN'S.

These methods are based on Columb's theory and out of these, CULMAN'S method is more generalised as it considers the effect of Surcharge, effect of line loading, Layered Backfill.

Coffer Dam



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STABILTY OF SLOPES

- Slopes are provided in Embankments for Railways, roadways, earthern dams --etc. or they can be a part of mountains and hills.

- The failure of the slope is due to gravitational forces, Earthquake, Erosion, Undercutting, Sudden drawdown, Seepage of water, Excavation near the edges----etc.

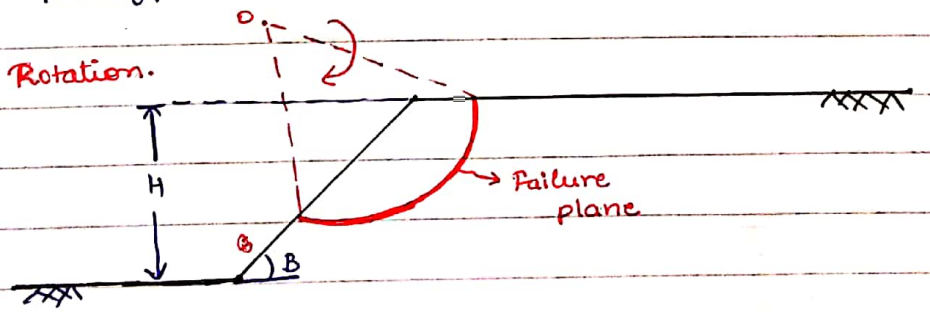
- The analysis of a slope is a 2D problem assuming that the shear parameters c and ϕ are constant.

- The slope can be classified in 2 types:

(i) Finite slope (ii) Infinite slope.

- Finite slopes are the ones having a smaller sloping length which is bounded at top and bottom.

For eg: Embankments, Earthern Dams, Canal Slopes



- The failure on the basis of observation done for various structure is of rotational type and the failure plane is a curve.

The rotation is assumed to occur about an imaginary centre.

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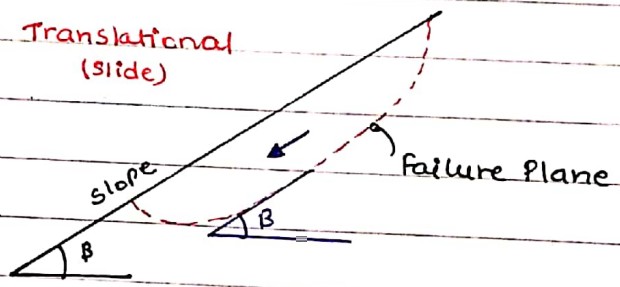
- By Infinite slope, we mean the slopes of Mountains, Hills where the sloping length is very large and its upper boundary can't be exactly defined.

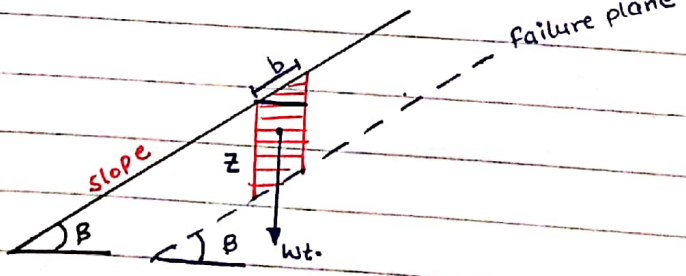
- Based on various observation, it has been found that the failure is of translational type.

  That means the soil mass slides down along the slope and the failure plane is assumed to be parallel to the slope.

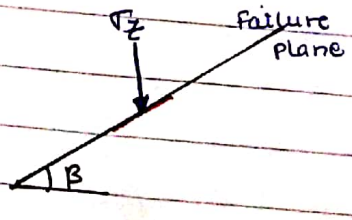


Analysis of Infinite Slope

Wt. ⇒ $\gamma z b \cos\beta$



$$\sigma_z = \frac{\gamma \cdot z \cdot b \cos\beta}{b \cdot 1} = \gamma z \cos\beta$$



$\sigma_z = \gamma z \cos\beta$

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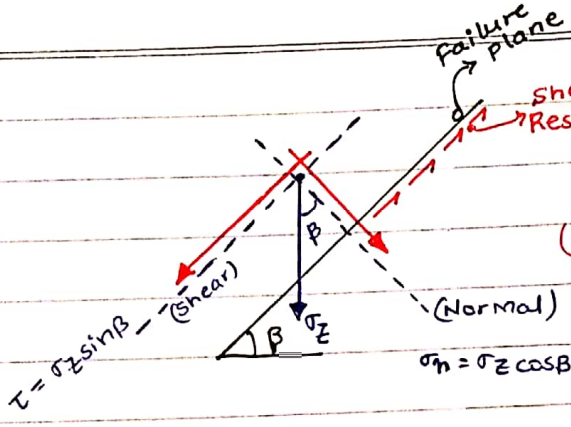


$\sigma_n = \gamma \cdot z \cdot \cos^2\beta$

(Disturbing) $\tau = \gamma \cdot z \cdot \cos\beta \cdot \sin\beta$

$S = c + \sigma_n \cdot \tan\phi$ (Resistance)

$$FOS = \frac{S}{\tau}$$

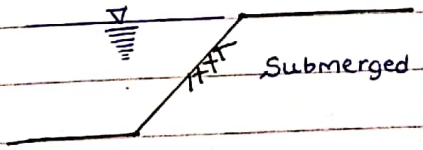
For Cohesionless Soil

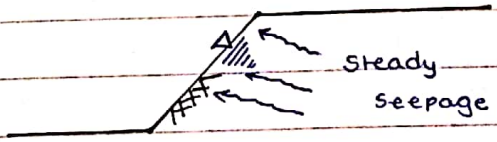
$c = 0$, $S = \sigma_n \tan\phi$

$$FOS = \frac{\gamma z \cos^2\beta \cdot \tan\phi}{\gamma z \cos\beta \sin\beta}$$

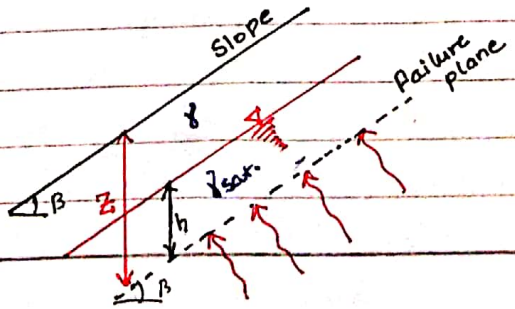
$$FOS = \frac{\tan\phi}{\tan\beta}$$

- Submerged
- Dry
- Non-Seepage condn.





Steady Seepage Condition



$$\gamma_{eff.} = \frac{\gamma(z-h) + \gamma_{sat.}(h)}{z}$$

$\sigma_n = \gamma_{eff.} z \cos^2\beta - \gamma_w h \cos^2\beta$

$\Rightarrow [\gamma_{eff.} z - \gamma_w \cdot h] \cos^2\beta$

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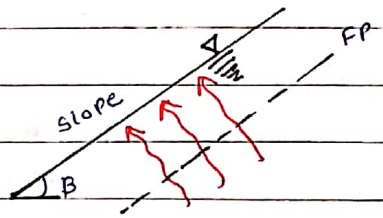
$$\tau = \gamma_{eff.} \cdot z \cdot \cos\beta \sin\beta$$

$$FOS = \frac{[\gamma_{eff.} \cdot z - \gamma_w h] \cos^2\beta \cdot \tan\phi}{\gamma_{eff.} \cdot z \cdot \cos\beta \sin\beta}$$

$$FOS = \left[1 - \frac{\gamma_w}{\gamma_{eff.}} \cdot \frac{h}{z}\right] \frac{\tan\phi}{\tan\beta}$$

Remark

⊙ If the seepage takes place upto the ground surface as an approx., we can say that the FOS reduces to $\frac{1}{2}$ of the previous value, hence, making the soil much more unstable.



$\frac{1}{2}$ approx.

$$FOS = \left[\frac{\gamma_{sub.}}{\gamma_{sat.}}\right] \frac{\tan\phi}{\tan\beta}$$

$$\rightarrow FOS = \frac{1}{2} \frac{\tan\phi}{\tan\beta}$$

For C-ϕ soil

If steady seepage condn

$$\tau = \gamma \cdot z \cos\beta \sin\beta$$

$$s = c + \sigma_n \tan\phi$$

$$\Rightarrow \frac{\gamma_{sub.}}{\gamma_{sat.}}$$

$$FOS = \frac{c + \gamma z \cos^2\beta \cdot \tan\phi}{\gamma z \cos\beta \sin\beta}$$

dry $\Rightarrow \frac{\gamma_d}{\gamma_d}$

Sub. $\Rightarrow \frac{\gamma_{sub.}}{\gamma_{sub.}}$

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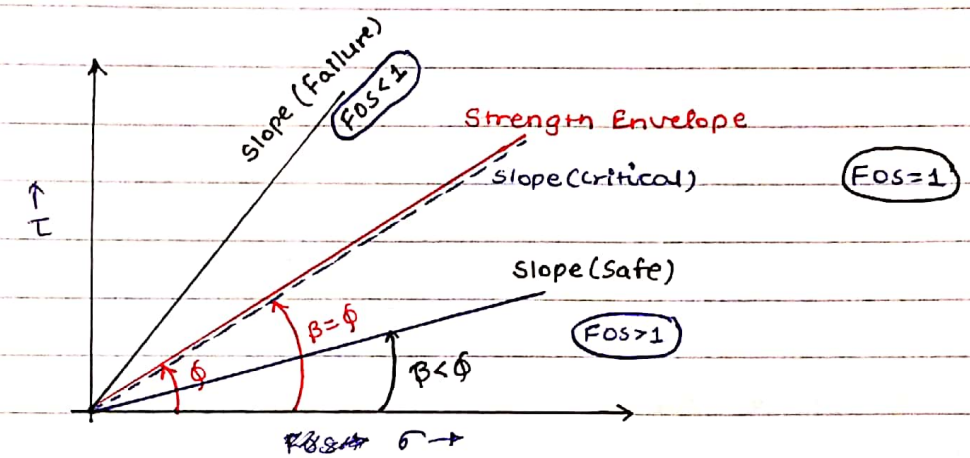
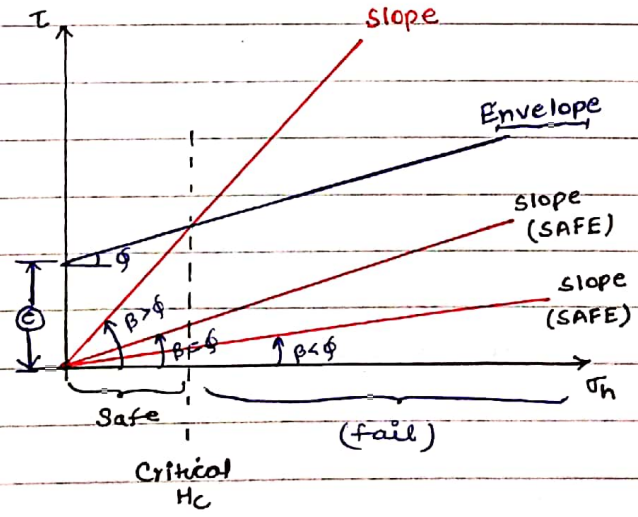
Graph

Cohesionless Soil



C-ϕ soil:



Critical Condn:

$\tau = S$ i.e; $FOS = 1$

$\gamma H_c \sin\beta \cos\beta = c + \gamma' H_c \cos^2\beta \tan\phi$

$\left[\sin\beta\cos\beta - \cos^2\beta \tan\phi\right] = \frac{c}{\gamma H_c} = S_n$

Taylor's stability No.

(Test Result)

$(S_n)_{max.} = 0.5$ (Theoritical)

$(S_n)_{max} = 0.261$ (Actual)

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Ques→ A slope having $c=0$ and $\phi=36°$. It is assumed that water level may occasionally reach upto the ground surface due to steady seepage. Find the slope angle β if FOS is 1.5 and the saturated density is $19\,KN/m^3$ and $\gamma_w = 10\,KN/m^3$

Soln:

$$FOS = \frac{\gamma_{sub.}}{\gamma_{sat.}} \times \frac{\tan\phi}{\tan\beta} \Rightarrow 1.5 = \frac{9}{19} \times \frac{\tan 36°}{\tan\beta}$$

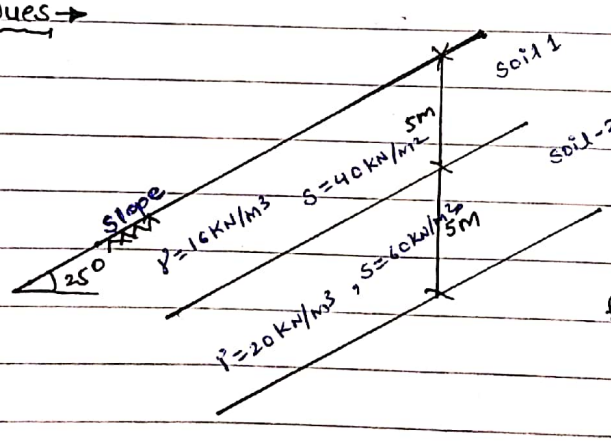
$$\Rightarrow \beta = 12.89° \approx 12.9°$$

Ques→ If the water table well below the surface, find the FOS.

Soln:

$$FOS = \frac{\tan\phi}{\tan\beta} = \frac{\tan 36°}{\tan 12.89°} = 3.17$$

Ques→

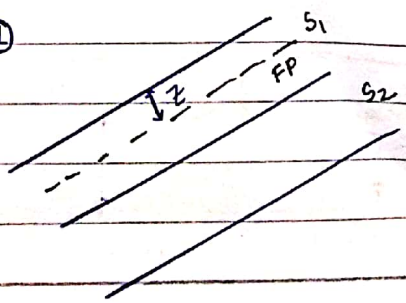


For a stratified soil having a slope angle of 25° as shown in the diagram, assuming an infinite slope failure, what will be the location of the failure plane from the sloping surface.

Soln: Critical condn ⇒ $FOS = 1 = \frac{S}{\tau} = 1$

$$\Rightarrow S = \tau$$

①



$$S = \tau$$

$$40 = 16Z \cos 25 \times \sin 25$$

$$\Rightarrow (Z = 6.5 > 5)$$

Our Assumption is wrong.

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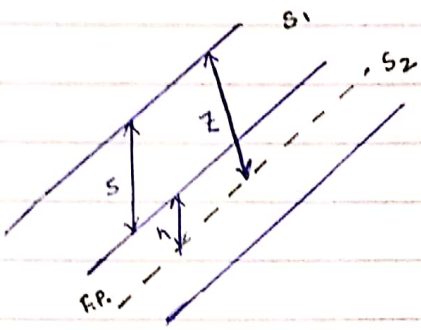
(2)



$S = \gamma_{eff} \cdot z \cos\beta \sin\beta$

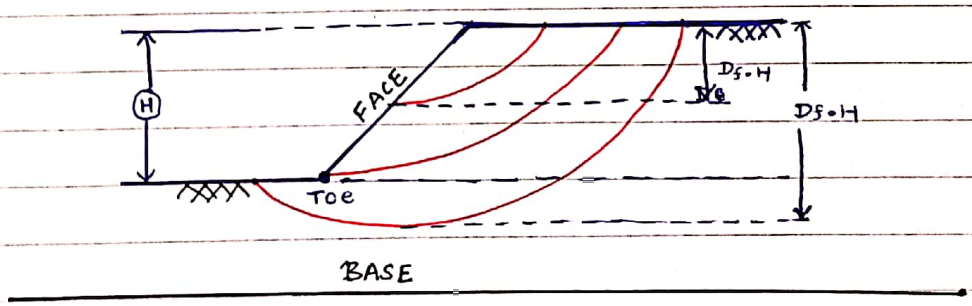
$\gamma_{eff} = \frac{(16 \times 5 + 20 \times h)}{(5+h)}$

$h = 3.83$

⇒ location from the sloping surface

$= 3.83 + 5 = 8.83$

Finite Slope



① The failure may be observed in various modes such as face failure, Toe failure, Base failure.

face failure occurs when the failure surface passes through the sloping face. This type of failure is observed in steep slopes where the soil below the toe is hard and rigid compared to the soil above the toe.

② The toe failure is observed when in steep slopes, the failure plane passes through the toe.

In this failure, the soil above the toe and below

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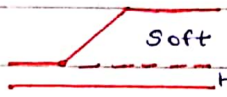
the toe is having same property.

⊙ Base failure is mostly seen in flatter slopes where the soil below the toe is comparatively softer than the soil above the toe.

Face: 

$D_f < 1$

Toe:

$D_f = 1$

Base:

$D_f > 1$

D_f = Depth factor.

⊙ It is defined as the ratio of the depth of lowest point of the failure plane wrt to the height of the slope.

Factor of Safety

F_ϕ, F_c, F_H

$$F_\phi = \frac{\tan\phi}{\tan\phi_m}$$ → Mobilized.

$$F_H = \frac{H_c}{H}$$ (H_c: critical; H: Safe Height)

$$F_c = \frac{c}{c_m}$$

All the factors mobilize in such a way that:

$$F_\phi = F_c = F_H$$

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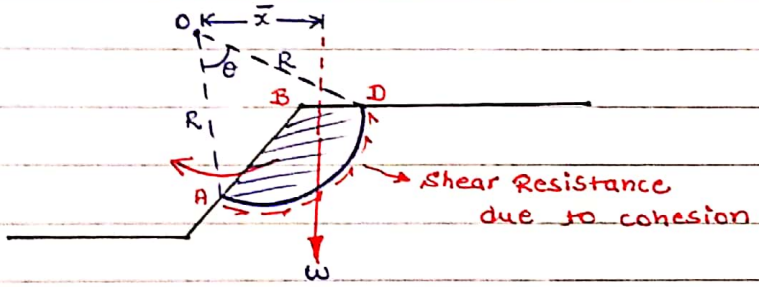
⊙ All the FOS will have the same value since it is considered that friction, cohesion, shear, Height -- all are Mobilized to the same Proportion of their ultimate value.

Finite Slope Analysis

① $\phi_u = 0$ (undrained) Analysis



Destabilising moment = $W \cdot \bar{x}$

$\overset{\frown}{AD}$ = curved length $\hat{L}$

Force on curved length = $\overset{\frown}{AD} \times C_u$

Resisting moment $\Rightarrow \overset{\frown}{AD} * C_u \times R$

$$FOS = \frac{\hat{L} \cdot C_u \cdot R}{W \cdot \bar{x}}$$

$$\hat{L} = \frac{\theta^\circ}{360^\circ} \times 2\pi R$$

$$W = \gamma \times (\text{Area})_{\text{shaded}}$$

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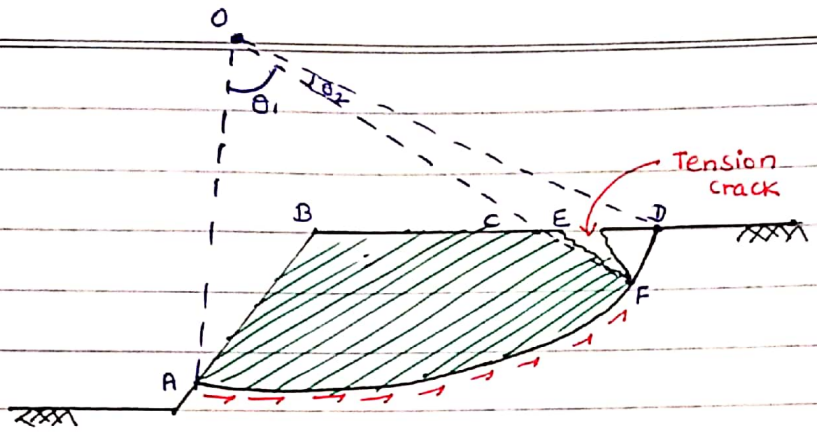


Tension crack

$\theta = \theta_1$

$\hat{L} = \overline{AF}$

Area = ABEFA

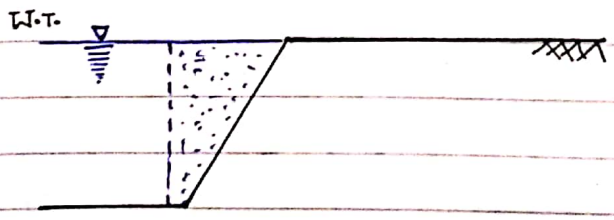
No Tension Crack

$\hat{L} = \overline{AD}$

$\theta = \theta_1 + \theta_2$

Area → ABEDFA

Canal Running Full



$\gamma = \gamma_{sub.}$

⊙ Due to the submergence of the slope when the canal is running full, the water resting on the slope will provide an additional stability and thereby the FOS will increase.

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Ques → (22)

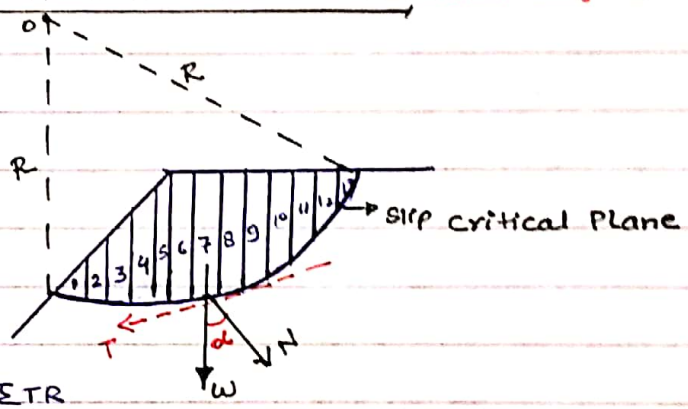
$$FOS = \frac{[c_1 L_1 + c_2 L_2] \times R}{\gamma A \bar{x}}$$

$$\Rightarrow FOS = \frac{[100\times84 + 50\times16]\,(21.6)^2 \times (\pi/180)}{(10.5) \times (156\times18)}$$

$$= 2.54$$

② Swedish Circle Method / Method of Slices (for c-ϕ soil)

(By Fellenius)



$N = W\cos\alpha$

$T = W\sin\alpha$

Disturbing Moment = T·R

Total disturbing Moment = $\Sigma T.R$

$$S = \frac{c.\Delta L + N\tan\phi}{\text{Total Resistive Moment}} = \Sigma S.R$$

$$FOS = \frac{\Sigma S.R}{\Sigma T.R} = \frac{[\Sigma c.\Delta L + \Sigma N\tan\phi]R}{\Sigma T.R}$$

$$FOS = \frac{c\hat{L} + \Sigma N\tan\phi}{\Sigma T}$$

No. of slices in this method is considered b/w 6-8 no. & each slice behaves as an independent column that means the friction b/w adjacent slices is neglected.

Out of various trial slip surfaces, critical is the one which will behave least FOS.

$$\left[FOS = \frac{c\hat{L} + (\Sigma N - \Sigma U)\tan\phi}{\Sigma T}\right]$$

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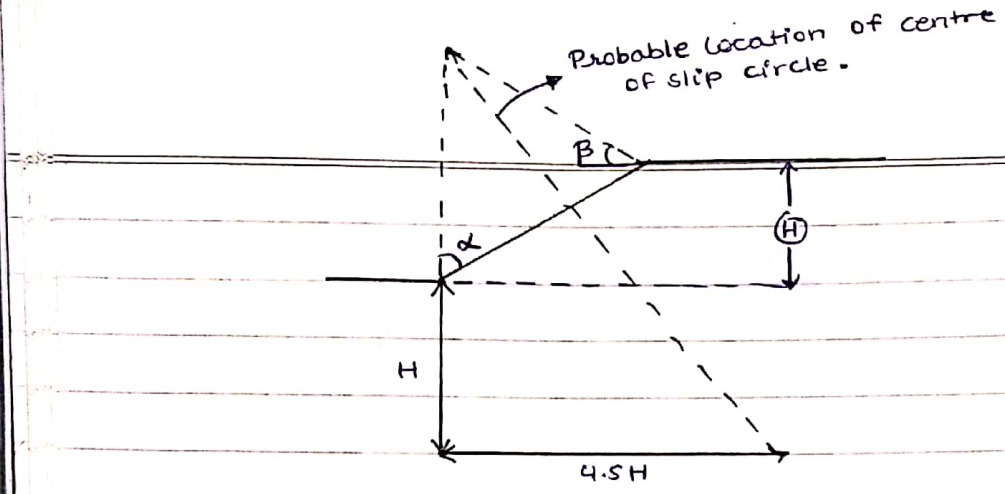
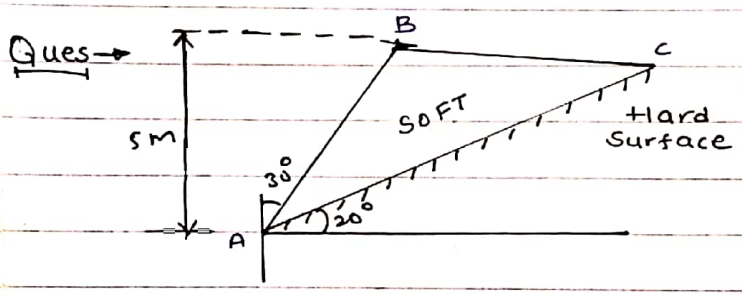


α, β → Directional Angle.

Ques →



$c = 15\ kN/m^2$

$\phi = 20°$

$\gamma = 18\ kN/m^3$

For the diagram as shown, find the FOS against the possible wedge failure at the interface AC.

Soln:

$$\text{Area of } ABC = \frac{1}{2} \times 5 \times (AC\cos 20° - AB\cos 60°)$$

$$= \frac{5}{2}\left[\frac{5}{\sin 20°} \times \cos 20° - \frac{5}{\sin 60°} \times \cos 60°\right] = 27.12\ m^2$$

Now; Disturbing force $= W\sin 20°$

Resisting force $= (AC) \times c + (W\cos 20°)\tan\phi$

$$FOS = \frac{\left(\frac{5}{\sin 20°}\right) \times 15 + [(\text{Area})\gamma]\cos 20° \tan 20°}{[(\text{Area})\gamma]\sin 20°} = 2.31$$

(c → 15, ϕ → 0, γ → 18)

$W = 488.3$ FOS = 2.31

$AC = 14$

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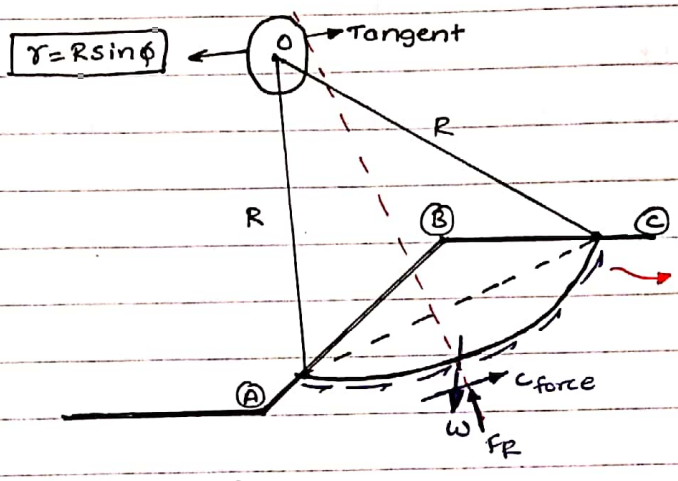
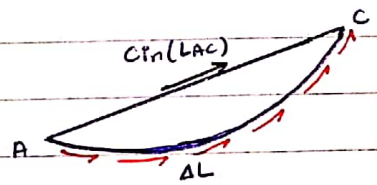
③ Friction circle Method:



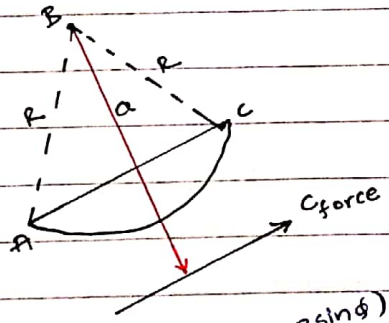


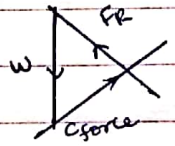
R.M. due to element = $C_m \Delta L \cdot R$

Total R.M = $\Sigma C_m \cdot \Delta L \cdot R$

$C_m(LAC) \cdot a = C_m \widehat{L_{AC}} \cdot R$

$\Rightarrow \boxed{a = \frac{\widehat{L_{AC}} \cdot R}{L_{AC}}}$

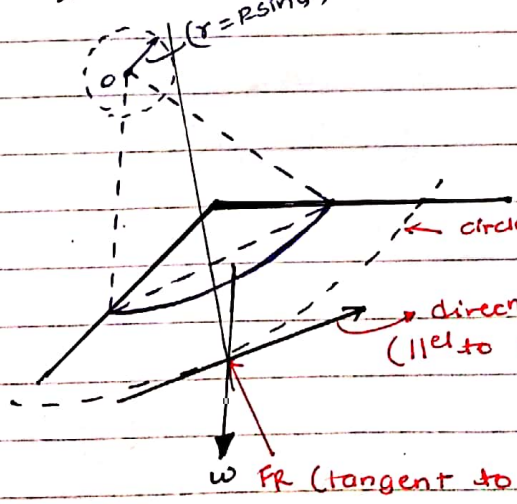




using force Δ find:

C_{force}, $\left[C_m = \frac{C_{force}}{L_{AC}}\right]$

& $\left[F_c = \frac{C}{C_m}\right]$



& assumption, $\left[f_\phi = \frac{\tan\phi}{\tan\phi_m} = 1\right]$

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④ This method is based on assumption that the resultant rxn force 'FR' on the rupture surface is tangential to the circle of radius r = Rsinϕ & this circle which is also known as friction circle is concentric with the trial slip circle.

The Eqm of the sliding soil mass is under the action of following 3 forces that are:

(i) weight force (W)

(ii) cohesive force (C)

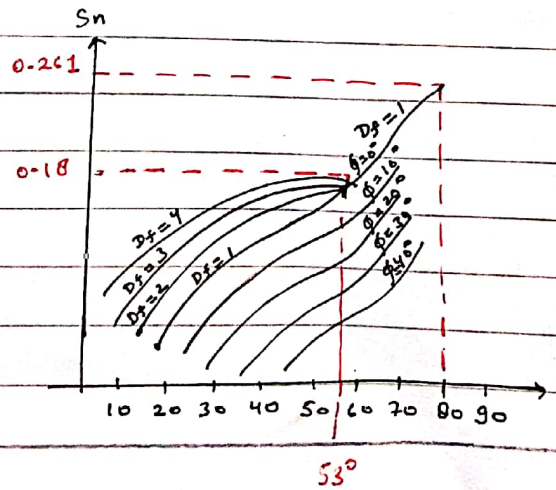
(iii) Soil Reaction (FR) which is tangential to friction circle.

In this method, FOS is calculated based on mobilised cohesion value by assuming initially that the friction is fully mobilised.

i.e; $F\phi = 1$

If the value of $F\phi$ & Fc doesnot Match, the iterations need to be done meaning that the $F\phi$ value will be somewhat >1 which has been represented in the Taylor's Stability Analysis.

TAYLOR'S



Assumption:

$F\phi = 1$ i.e; $(\phi_m = \phi)$

$$S_n = \frac{C}{\gamma H_c}$$

Now, $F_c = \frac{C}{C_m}$

& $F_H = \frac{H_c}{H}$

& $F_c = F_H$

$$\Rightarrow S_n = \frac{C_m}{\gamma H}$$

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Reiteration:

Eg: Given c-ϕ soil $\phi = 20°$, $\beta = 30°$, $c = ---- KN/m^2$

① $F\phi = \frac{\tan\phi}{\tan\phi_m} = 1$ Assume $\phi = \phi_m = 1$ find Sn from table for $\phi = 20°$ & $\beta = 30°$

Now, $S_n = \frac{C_m}{\gamma H}$ find C_m then

$F_c = \frac{c}{C_m} = 2.2$ (let)

② Assume $F\phi$ b/w 1 and F_c earlier (2.2)

$F\phi = \frac{\tan\phi}{\tan\phi_m} = 1.5$ (let) find ϕ_m then find Sn' for ϕ_m and β.

find C_m then F_c

& repeat till $[F_c \approx F\phi]$

⊛ Taylor's stability chart / curve is used to read the Taylor's stability no. (Sn) which is a dimensionless parameter & is used to Analyse the stability of the slope.

⊛ Taylor has utilized the friction circle analysis for preparation of chart.

(i) From the Graph, we can infer that for $\phi = 0$ and $\beta < 53°$, the probable failure is base failure.

(ii) for $\phi = 0°$ & $\beta > 53°$, probable failure is toe failure.

(iii) For $\phi = 0°$ and β from 0 to 90°, probable failure is toe failure.

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⊙ The stability no. (Sn) is read from chart based on mobilised friction angle

⊙ Analysis is an iterative process which starts assuming $F_\phi = 1$ & iterations are done till F_c and F_ϕ have same values.

→ Analysis is not valid for cohesionless soils.

→ Value of unit wt. of soil is to be referred based on following:

(i) For dry soil, $\gamma = \gamma_d$

(ii) For saturated & submerged soil, $\gamma = \gamma_{sub.}$

(iii) If soil is sat. by capillary, $\gamma = \gamma_{sat.}$

(iv) If sudden drawdown condⁿ is encountered, $\gamma = \gamma_{sat.}$ & Sn is calculated based on the wetted friction Angle

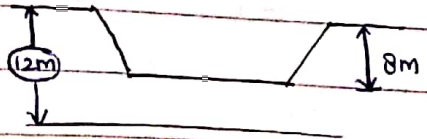
$$\left[\phi_w = \frac{\gamma_{sub.}}{\gamma_{sat.}}\phi\right]$$

SQ21: $0.181 = \dfrac{C \to 115}{20.2 \times 15 \times FOS} \Rightarrow FOS = 2.096$

SQ→23 $\phi_{wetted} = (20) \times \dfrac{[20 - 9.81]}{20} = 10°$

So, $Sn = 0.108 = \dfrac{C \to 50}{20 \times 12 \times FOS} \Rightarrow FOS = 1.929$

S.Q→24



$Sn = \dfrac{C}{\gamma H} = \dfrac{28}{20 \times 8} = 0.175$

& $D_f = \dfrac{12}{8} = 1.5$

then; $i = 45°$

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$$\gamma_{sat.} = \frac{(2.65+1)}{2} 9.8 = 17.885$$

SQ → (25) :

$$\phi_{weighted} = 15 \times \frac{\gamma_{sub.}}{\gamma_{sat.}} = 6.781^\circ \approx 7^\circ$$

$$\gamma_{sub.} = 8.08$$

Now,

$$0.126 = \frac{18}{(17.885) \times 6 \times FOS} \Rightarrow FOS = 1.33$$

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FOUNDATION

⊙ Foundations are part of structural element which transfers load from superstructure to soil.

Design involves sizing and placement so that neither soil fails in the shear nor structure fails due to excessive settlement. Hence, min. of the 2 criteria i.e; whichever occurs first is used as bearing capacity of soil.

There are diff. types of foundations such as shallow & deep. As per Terzaghi:

If $\frac{D_f}{B} \leq 1$ ⇒ Shallow foundation.

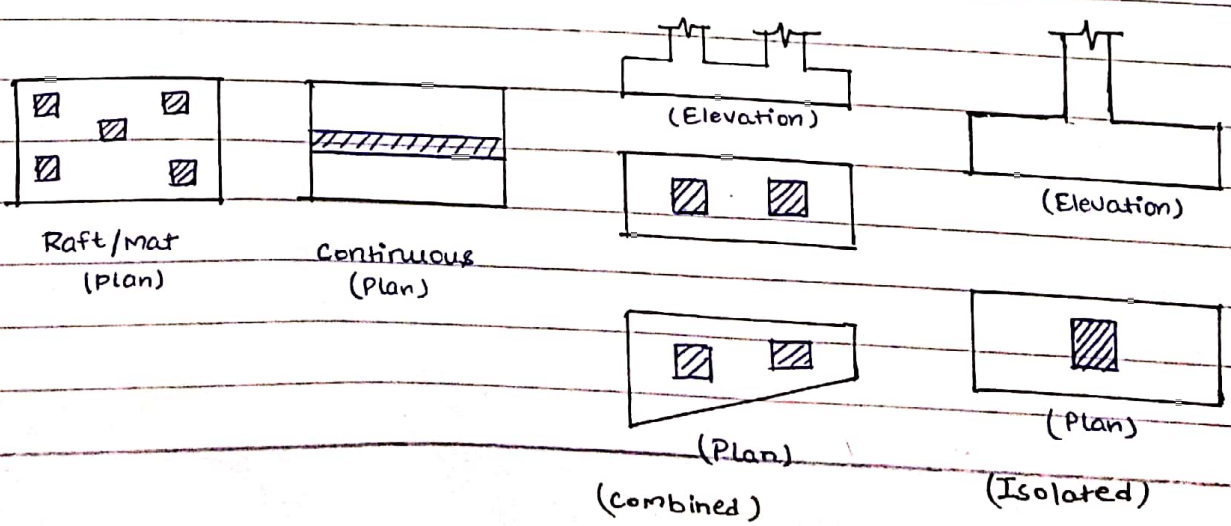
If $\frac{D_f}{B} \geq 1.5$ ⇒ Deep foundation.

As per Skempton:

If $\frac{D_f}{B} \leq 2.5$ ⇒ Shallow foundation

If $\frac{D_f}{B} > 2.5$ ⇒ Deep foundation.

⊛ Shallow foundation consist of isolated footing, combined footing, continuous footing (strip foundation), Raft/mat foundation, strap beam foundation.



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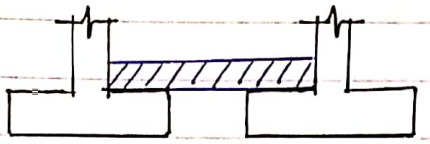
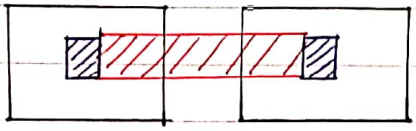
Deep Foundation

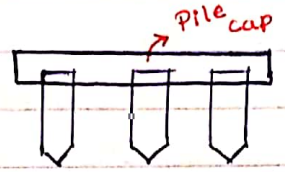


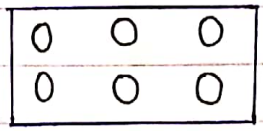
(Elevation)



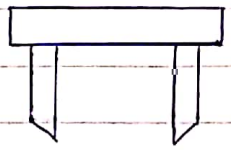
(Plan)

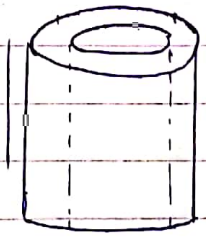
Strap foundation





Pile





Cassion / Well

- Isolated footings are provided when loading is of smaller value & columns of structure are seperated by large distribution.

- Combine foundation are provided when 2 columns are close to each other.

- If loading on 2 columns is not of large difference, rectangular shape is provided, else, if one of the column is

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having heavy load, trapezoidal Geometry is adopted.

⊙ When one of the column is subjected to large horizontal forces, then it is joined by means of strap beam with the adjacent foundation system, so as to transfer a part of its load.

⊙ If multiple no. of columns are coming for a structure, then instead of providing individual isolated footing for all of them, we prefer to provide a single raft or Mat.

⊙ If the soil is weak & the load from superstructure is of large value then the foundation is taken upto deeper depth & pile foundation is provided.

⊙ In riverfront structure such as for supporting the piers of a bridge. ~~of~~ well foundation also known as cassion, are provided.

Pressure Distribution

⊙ If the resultant of the load is concentric with the centroid of foundation system, then pressure below the footing will be uniform.

for (e=0)

$$a + \bar{x} = z$$

$$\bar{x} = \frac{P_2 \times c_1}{P_1 + P_2}$$

$$z = \frac{1}{3}\left[\frac{2B_2 + B_1}{B_1 + B_2}\right]$$

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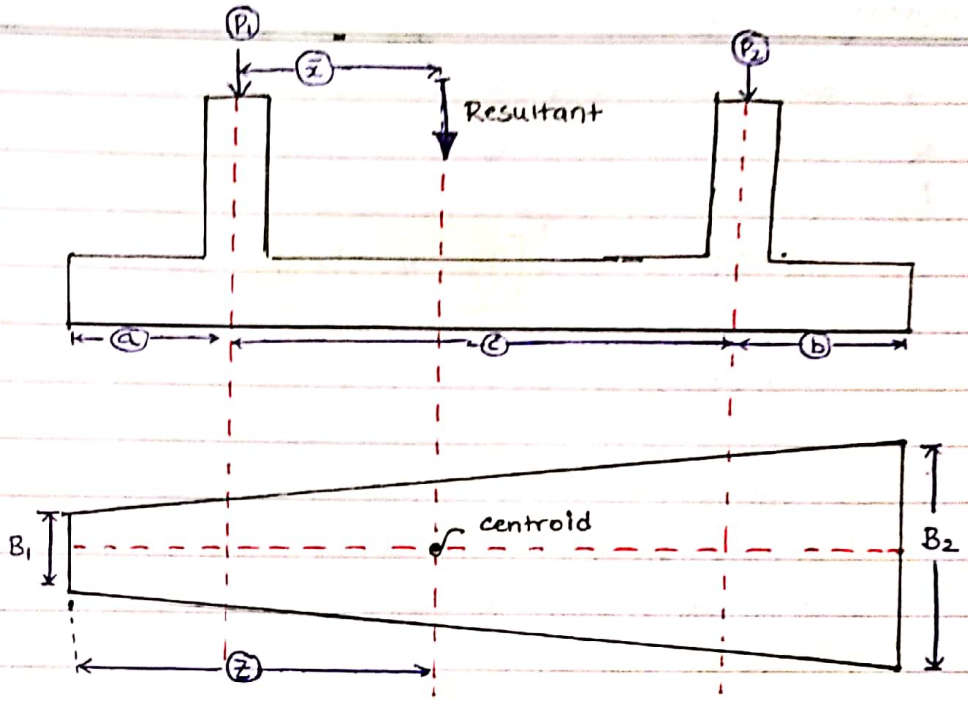

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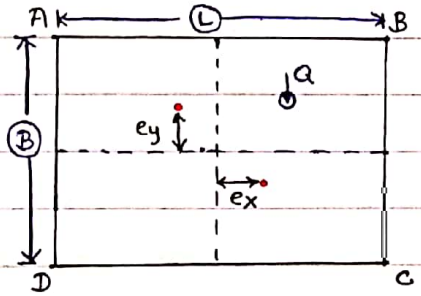

$$\sigma_B = \frac{Q}{LB} + \frac{Q \cdot e_x}{\left(\frac{BL^2}{6}\right)} + \frac{Q \cdot e_y}{\left(\frac{LB^2}{6}\right)}$$

$$\sigma_D = \frac{Q}{LB} - \frac{Q \cdot e_x}{\left(\frac{BL^2}{6}\right)} - \frac{Q \cdot e_y}{\left(\frac{LB^2}{6}\right)}$$

Important definition based on shear Criteria

① Gross Pressure (q_g): Total Pressure at the base of the footing due to the load from superstructure, load of foundation system, self wt of soil backfill.

$$q_g = \frac{\text{Total load}}{\text{Base Area}} = \frac{Q_{structure}}{A} + \frac{\text{Self wt. of Backfill + foundation}}{A}$$

∵ Density of concrete and density of soil doesnot have large difference, hence they can be treated as same &

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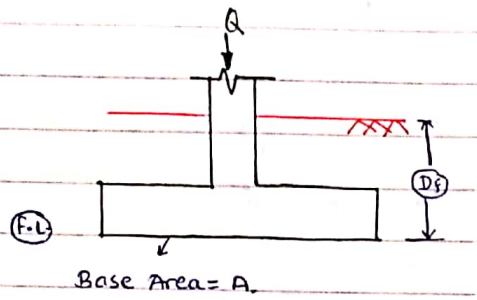
$\gamma_{concrete} \approx \gamma_{soil}$

for analysis purpose, we consider it as the density of soil.



$$\left[q_g = \frac{Q}{A} + \gamma D_f\right]$$

② Net Pressure (q_n): (due to wt. of structure only) It is the pressure exerted at the base of foundation due to load in excess of the overburden pressure.

$$\left[q_n = \frac{Q}{A} = q_g - \gamma D_f\right]$$

③ Ultimate Bearing Capacity (q_u): Maxm value of Gross pressure which is acting at the foundation base without causing any shear failure in the soil

(OR)

⊙ It is the min. value of gross pressure at which soil just fails in shear.

for design: $q_n < q_{ns}$

$q_g < q_s$

④ Net Ultimate Bearing Capacity (q_{nu}): It is the maxm value of net pressure which is applied at the base of foundation without causing any shear failure.

⊙ It is the minm value of net pressure at which the soil just fails in shear.

$$q_{nu} = q_u - \gamma D_f$$

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⑤ Net Safe Bearing Capacity (q_{ns}): It is that net pressure value which can be safely applied at foundation base without causing any shear failure.

$$q_{ns} = \frac{q_{nu}}{FOS}$$

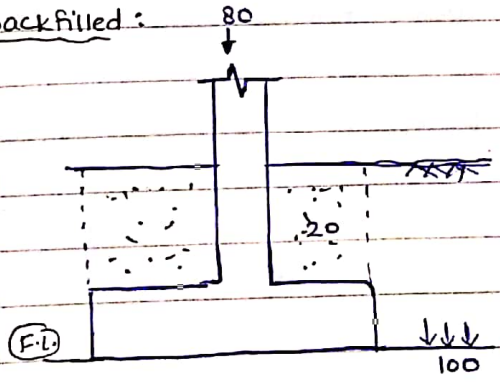
⑥ Safe bearing Capacity (q_s): It is that maxm value of gross pressure which can be safely applied at the foundation base without causing any shear failure.

$$q_s = q_{ns} + \gamma D_f = \frac{q_{nu}}{FOS} + \gamma D_f$$

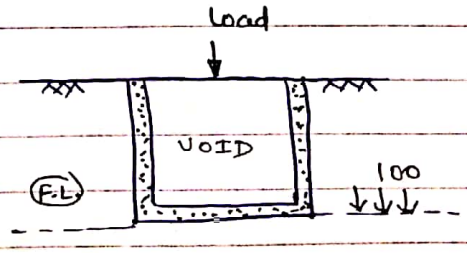
* FOS is not applied on γD_f because its pressure is acting from historical period.
FOS is only applied on q_{nu} which represents increase in pressure due to recent loading.

Non-Backfill condition

Backfilled:



Non-Backfilled:



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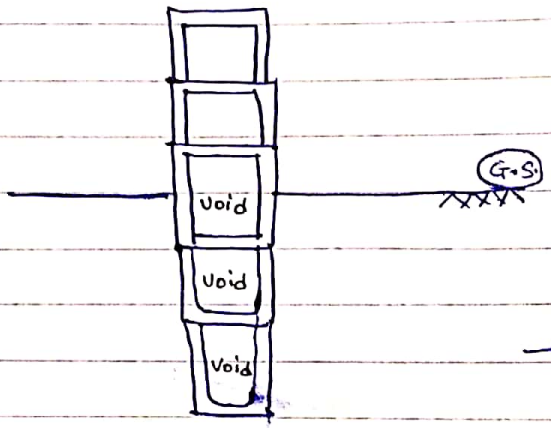
Floating Raft / compensated Raft



→ It is a type of raft found^{n} in which wt. of soil excavated is equal to wt. of structure & any applied load,

→ It means that net increment of pressure on the soil will be negligible & thus settlement will also be Negligible.

- Load carrying capacity is significantly increased in non-backfilled condition.
- when loads are heavy and soil deposit is of clayey nature, then this concept is advantageous.
- For a fully compensated raft, net pressure will be zero.

i.e; $q_n = \frac{Q}{A} - \gamma D_f$ → for non-backfilled.

$q_n = 0$

$\Rightarrow \boxed{\frac{Q}{A} = \gamma D_f}$

Settlement criteria

- Footing should be designed such that max^{m} settlement is rejected below the permissible values which are provided by various codal agencies or equipment manufacturers.

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- As already discussed, the values specified by codal agencies are permissible or safe. FOS is not required for them.

  Acc. to settlement ⇒ q_{np}, q_{ps}

Allowable Bearing capacity:

- q_t is the min^n of net safe due to shear criteria & the permissible bearing capacity due to settlement.

$$q_a = \min.[q_{ns}, q_{np} \text{ or } q_{ps}]$$

Methods to Determine Bearing capacity

1. Analytical Methods
2. Codal Provisions
3. Field Tests.

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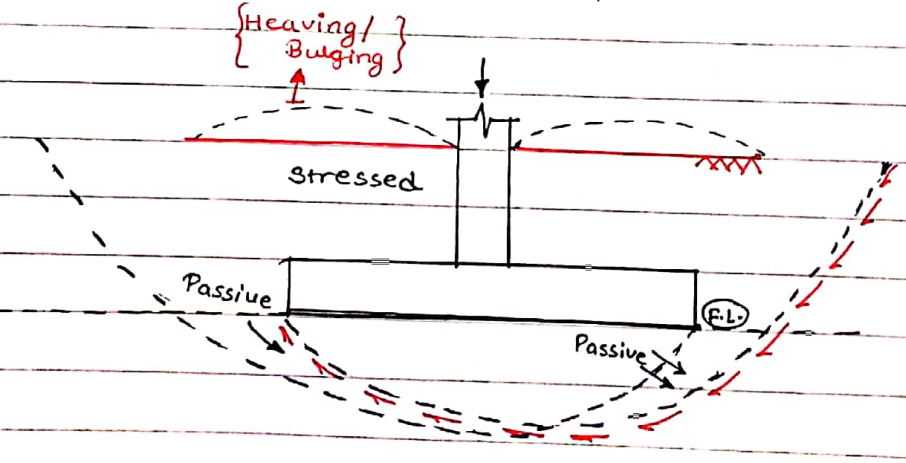
Analytical Methods / Static Methods

① In this method, elastic theory was proposed by Scheilchr's.
② Earth Pressure theory given by Rankine, Bell Packer.
③ Plastic theory given by Terzaghi, Skempton, Prandtl, Meyerhoff, Vesic, Teng's.

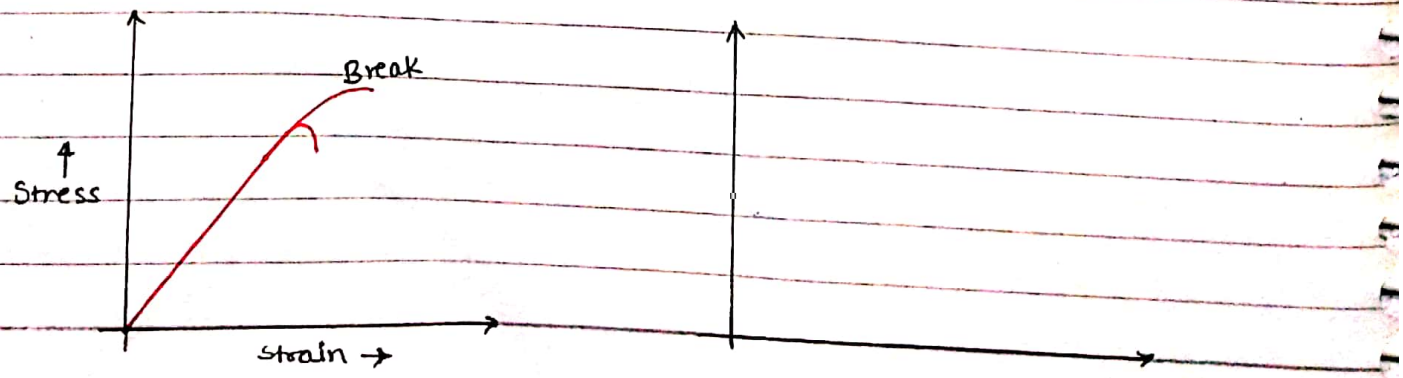
Shear failure

Shear failure in 3 modes: (i) General shear failure
(ii) Local shear failure
(iii) Punching Shear failure.

General Shear failure



⑥ It occurs in shallow footing in soils having Brittle type of shear stress curve i.e; Medium dense sand, silt, or clay represent General shear failure.



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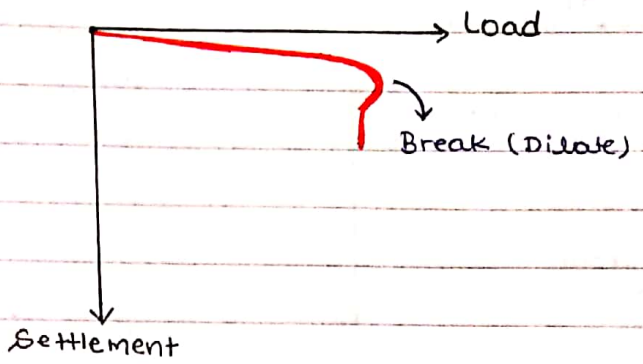

- These soils have low compressibility.
  At the time of failure, foundation gets tilted on one side.
- Before failure, settlement will be negligible & stress zone will be extended upto the ground level.
- State of plastic Eqm is reached initially around the edges & it spreads downward & outwards.
- Portion of soil below foundation exerts passive pressure on the surrounding soil.
- Shear planes developed are well defined.
- Bulging and heaving can be noticed in the surrounding of foundation.

Density Index > 70%

SPT value (N) > 30

Angle of friction resistance $\phi > 36°$

Void ratio < 0.55

UCS $q_u = 100\ KN/m^2$

Local shear failure

- Load settlement curve for a soil subjected to local shear failure represents the stress curve of a plastic material.

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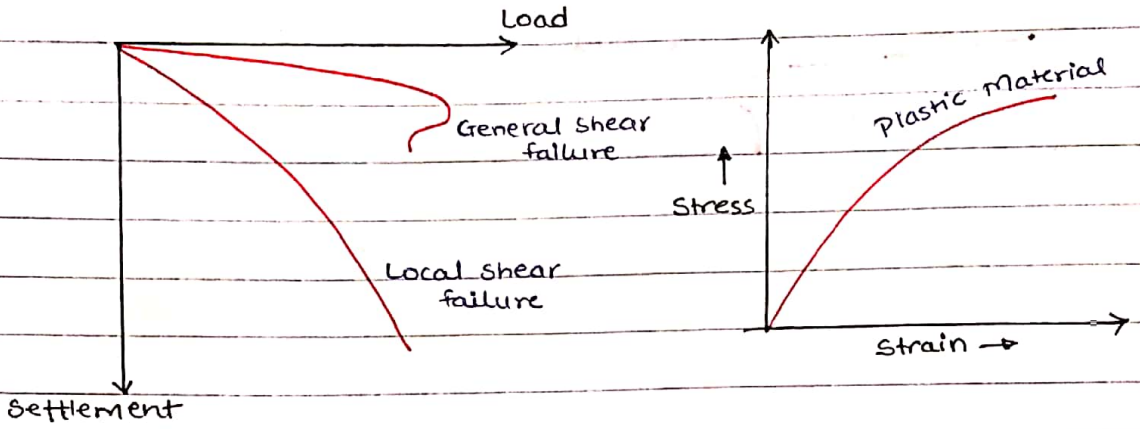


- Considerable compression of the soil occurs.
- Failure surface doesnot reach ground surface & only a slight heaving is observed due to excessive load.
- This failure occurs in loose sand & soft clay.
- No sudden failure is observed.
- A well defined slip surface is only observed below the foundation.

Density Index : 30% - 70%

$\phi \leq 28°$

SPT (N) < 5

$e > 0.75$

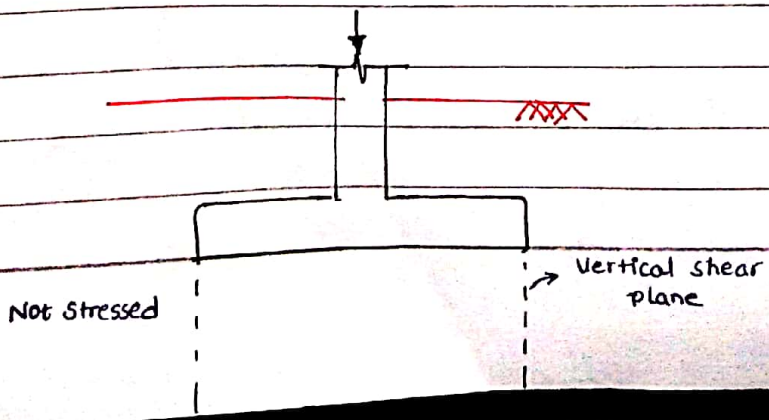
UCS, $q_u < 80$ KN/M^2

$$\left[\begin{array}{l} c_m = \frac{2}{3} c \\ \tan\phi_m = \frac{2}{3} \tan\phi \end{array}\right]$$

i.e; for local shear, Mobilise c & ϕ

i.e; c_m & ϕ_m are to be taken.

Punching shear failure



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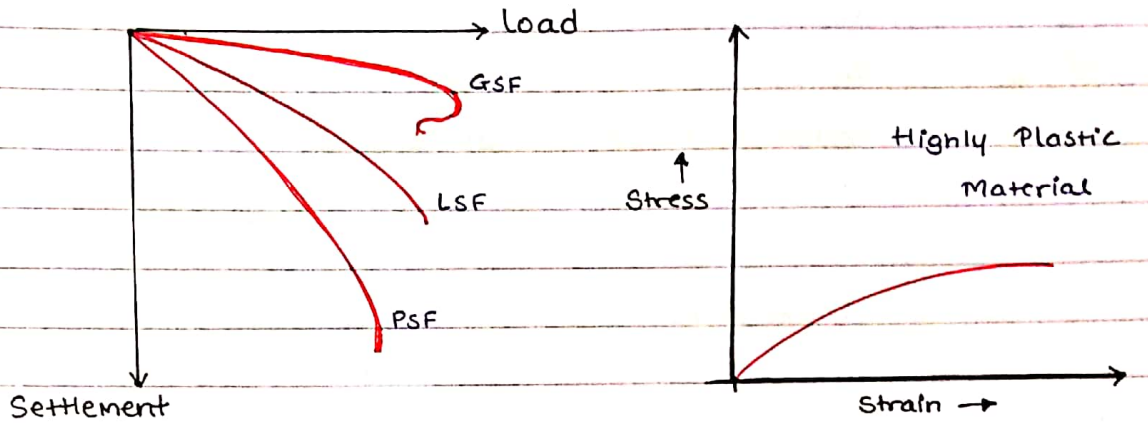
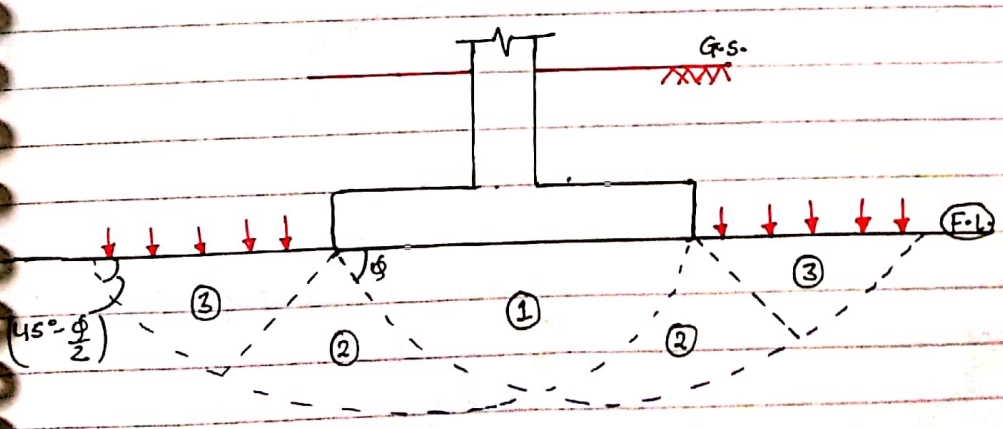


- This failure results in high compression of soil accompained by shearing in vertical direcn which is around the edges of footing.
- No heaving or bulging is observed in adjacent soil.
- Load settlement curve resembles the stress curve of a highly plastic material.
- This failure occurs in very loose sand & very soft clays.
- Zones beyond vertical shear plane are not stressed.
- Density index of soil (< 30%)

① Terzaghi's Theory



- Foundation considered is a shallow foundation if $\frac{D_f}{B} < 1$

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- Base of foundation is rough.
- General shear failure is assumed.
- Footing considered is a continuous one.
- Footing is assumed when soil reaches plastic state.
- Footing is assumed as surface footing, hence shear planes are considered only upto foundation level (F.L.)
- Weight of soil above found^{n} level is considered as surcharge.
- Resistance from edges of footing (sides) is ignored.
- Soil below the footing is divided into 3 zones

  i.e; Zone-1 ⇒ Zone of Elastic Eqm

  Zone-2 ⇒ Radial Shear Zone

  Zone-3 ⇒ Rankine's Passive Zone

Zone-1 ⇒ It is considered as integral part of found^{n} system & it remains in elastic eqm. Soil present in this zone is prevented from moving laterally.

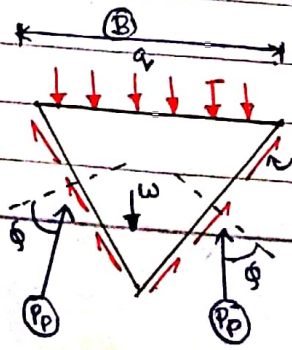
This zone pushes zone 2 & 3 giving a passive cond^{n} & making the zone 2 & 3 to come in a plastic state.

Zone-2 ⇒ It has shear plane of circular or spiral nature. for clayey and sandy soil respectively.

Zone-3 ⇒ Shear planes are straight lines and they make an angle of $\left(45° - \frac{\phi}{2}\right)$ with horizontal.

- Bearing capacity of soil is calculated by using eqm of soil in zone-1.

$P_p = P_{pc} + P_{pq} + P_{pr}$



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Strip / continuous

Bearing capacity factor

$$q_u = cN_c + (\gamma D_f) N_q + \frac{1}{2} \gamma B N_\gamma$$

① Surface footing: $D_f = 0$

② Soil is cohesive: $N_c = 5.7$, $N_q = 1$, $N_\gamma = 0$

Acc to Terzaghi's theory (for Rough Bottom)

If Bottom is smooth:

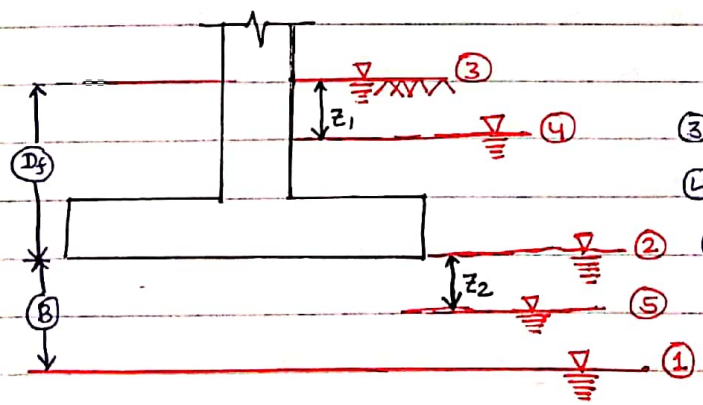
then by Prandtl: $N_c = 5.14$, $N_q = 1$, $N_\gamma = 0$ (cohesive)

So; $q_u = 5.7c + \gamma D_f$ → Independent of 'B' i.e; width of footing.

$q_{nu} = q_u - \gamma D_f = 5.7c$ → Independent of D_f & B & γ

i.e; q_{nu} is independent of water table (for cohesive soil)

③ Effect of Water Table:



① $q_u = cN_c + (\gamma D_f) N_q + \frac{1}{2} \gamma B N_\gamma$

② $q_u = cN_c + (\gamma' D_f) N_q + \frac{1}{2} \gamma_{sub.} B N_\gamma$

③ $q_u = cN_c + (\gamma_{sub.} D_f) N_q + \frac{1}{2} \gamma_{sub.} B N_\gamma$

④ $q_u = cN_c + (\gamma_{avg.} D_f) N_q + \frac{1}{2} \gamma_{sub.} B N_\gamma$

$$\gamma_{avg.} = \frac{\gamma \cdot z_1 + \gamma_{sub.} (D_f - z_1)}{D_f}$$

⑤ $q_u = cN_c + (\gamma D_f) N_q + \frac{1}{2} \gamma_{avg.} B N_\gamma$

$$\gamma_{avg.} = \frac{\gamma \cdot z_2 + (B - z_2) \gamma_{sub.}}{B}$$

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for cohesionless soil ; $c = 0$

So, for ① ⇒ $q_u = (\gamma D_f) N_q + \frac{1}{2} \gamma B N_\gamma$

& for ③ ⇒ $q_u = (\gamma_{sub.} D_f) N_q + \frac{1}{2} \gamma_{sub.} B N_\gamma$.

$\because \gamma_{sub.} \simeq \frac{1}{2} \gamma$ ⇒ $q_{u①} \simeq q_{u③}$

⇒ If w.t. rises to ground level for cohesionless soil, then q_u reduces to almost half.

Approximate Values (q_u) based on w.t. correcⁿ factor

$$q_u = cN_c + R_{w_1} (\gamma D_f N_q) + 0.5 \gamma B N_\gamma R_{w_2}$$

$$R_{w_1} = 0.5 \left[1 + \frac{z_1}{D_f}\right] \qquad R_{w_2} = 0.5 \left[1 + \frac{z_2}{B}\right]$$

$0.5 \le \frac{z_1}{D_f}$, $\frac{z_2}{D_f} \le 1$

for ⑤ :

$R_{w_1} = 1$ $\quad q_u = cN_c + (\gamma D_f) N_q + 0.5 \gamma B N_\gamma (0.5)$

$R_{w_2} = 0.5$

for ③ :

$R_{w_1} = 0.5$ $\quad q_u = cN_c + [0.5] (\gamma D_f) N_q + 0.5 \gamma B N_\gamma (0.5)$

$R_{w_2} = 0.5$

④ Modified Bearing capacity eqⁿ for diff. shape of foundⁿ:

$$q_u = cN_c + R_{w_1} (\gamma D_f N_q) + 0.5 \gamma B N_\gamma R_{w_2}$$

	cN_c	qN_q	$\frac{1}{2} \gamma B N_\gamma$
Circular	1.3	1	0.6
Square	1.3	1	0.8
Rectangle	$(1 + 0.3 \frac{B}{L})$	1	$(1 - 0.2 \frac{B}{L})$

B → Shorter dimension.

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SQ → (30) $q_u = (1.3) \times 72 \times 0 + (1.17 \times 1.5) \times 64 + \frac{1}{2}(0.8)(20-10) \times 2 \times 95 \times \frac{1}{2}$

square

760

rect

$$\frac{q_u - \gamma D_f}{3} + \gamma D_f = \frac{q_u + 2\gamma D_f}{3}$$

$$= \frac{17 \times 96 + 760 + 2 \times 17 \times 1.5}{3} = \frac{2443}{3} = 814$$

SQ → (31) for short term stability, ϕ will be used (as undrain)

So, for $\phi = 0 \Rightarrow N_c = 5.7, N_q = 1, N_\gamma = 0$

$q_u = 5.7 \times 40 + (\gamma_{avg.} \times 1.5) \times 1 \Rightarrow q_u = 253$

$$\gamma_{avg.} = \frac{1 \times 20 + 0.5(20-10)}{1.5} = \frac{25}{1.5}$$

SQ → (32) Total load = 880 KN

$$\text{Area} = \frac{880}{300} \Rightarrow \text{size} = \sqrt{\frac{8.8}{3}} = 1.71 \text{ m}$$

Ques → A raft foundation is provided for a building which has $\gamma = 20$ KN/m^3 for soil. If total pressure at base of raft is 150 KN/m^2, find depth of foundation of the compensated raft.

Soln: $150 = D_f \times 20 \Rightarrow D_f = 7.5$ M

Ques → 2 footings → 1 circular & other square are founed on the surface ($D_f = 0$) of a purely cohesionless ($c = 0$) soil. Dia. of circular footing is same as that of side of square footing. Find out the ratio of their ultimate bearing capacity.

Soln: $\frac{q_u \text{ circle}}{q_u \text{ sq.}} = \frac{0.6}{0.8}$

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Ques→ A strip footing is to be designed to carry gross load of 300 KN/m at depth of 1m in a clayey soil. Shear strength parameters are $c = 60\ KN/m^2$, $\phi = 0°$. Determine width of footing considering FOS = 3. Water table is found to be at foundation level with $\gamma = 18\ KN/m^3$ above W.T. & $\gamma_{sat.} = 20 KN/m^3$.

Soln:

$$q_u = 5.7 \times 60 + 1(18 \times 1) = 342 + 18 = 360\ KN/m^2$$

Now, $$q_s = \frac{q_u - \gamma D_f}{3} + \gamma D_f \geq q_g$$

$$\Rightarrow \frac{360 - 18\times1}{3} + 18\times1 \geq \frac{300}{B}$$

$$\Rightarrow B = \frac{300 \times 3}{396} = 2.27m$$

$q_u = \left\{\frac{CN_c}{FOS}\right\} + (\gamma D_f + N_q)$

$q_u = \frac{Load}{Area} = \frac{300}{B \times 1}$

② Skempton's Theory

⊙ It is applicable for purely saturated clays & can be applied for shallow as well as deep foundation. (i.e; we are considering stress zone above F.L. also)

⊙ $\phi_u = 0$ & value of undrained cohesion is obtained from unconfined compression test (UCS)

i.e; $$c_u = \frac{q_u}{2}$$

q_u = Unconfined compression strength

⊙ Skempton's theory considers shear strength of soil above F.L which was neglected in Terzaghi's theory.

⊙ Bearing capacity factor N_c depends on shape of foundation, size, depth, which was a constant in

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Terzaghi's theory for cohesive soil.

$[q_{nu} = cN_c]$ & $\frac{D_f}{B} \leq 2.5 \Rightarrow$ Shallow

N_c values for different foundation.

① Strip:

$$N_c = 5\left[1 + 0.2\frac{D_f}{B}\right]$$

for shallow, $\left(\frac{D_f}{B}\right)_{min.} = 0 \Rightarrow N_c \text{ min.} = 5$

$\left(\frac{D_f}{B}\right)_{max.} = 2.5 \Rightarrow N_c \text{ max.} = 7.5$

② Square / circular:

$$N_c = 6\left[1 + 0.2\frac{D_f}{B}\right]$$

$(N_c)_{min.} = 6$, $(N_c)_{max.} = 9$

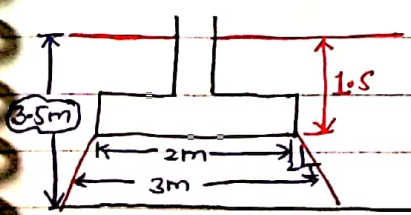
③ Rectangle:

$$N_c = 5\left[1 + 0.2\frac{B}{L}\right]\left[1 + 0.2\frac{D_f}{B}\right] \quad (B \leq L)$$

SQ→(33): $q_{nu} = cN_c = 5 \times 6\left[1 + 0.2 \times \frac{1.5}{2}\right] = 11.5 \times 3 = 34.5$

with $FOS = \frac{34.5}{3} = 11.5$

SQ→(34) $C_c = 0.009(w_L - 10) = 0.18$



$\Delta\bar{\sigma} = \frac{11.5 \times 4}{9} = 5.11$

$\Delta H = \frac{(0.18) \times 2}{2.08} \log\left[1 + \frac{5.11}{0.82 \times 2.5}\right]$

$= 94mm$

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③ - Meyerhoff's Theory

⊙ Meyerhoff's theory is most generalised theory in which Shape factor, depth factor & inclination factor are used to account for shape of footing, variation in depth of footing, inclination of load as well as ground surface & eqn is given as:

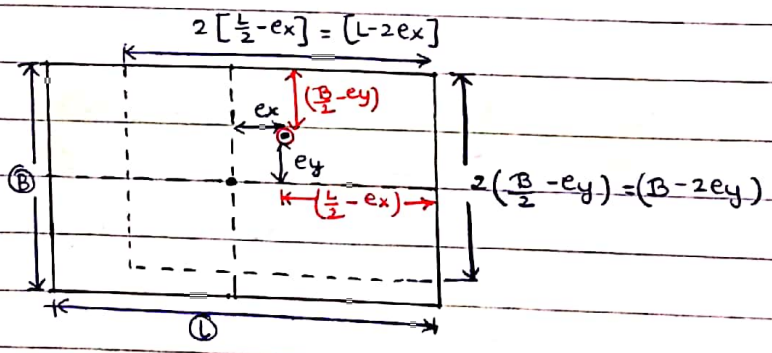
(depth factor) (Inclination factor)

$$\left[q_u = cN_c s_c d_c i_c + (\gamma D_f) N_q s_q d_q i_q + (0.5 \gamma B N_\gamma) s_\gamma d_\gamma i_\gamma\right]$$

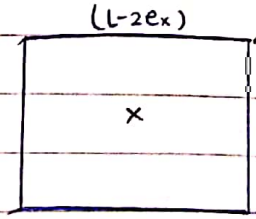
(Shape factor)

⊙ Stress Zone is considered to extend upto the ground level, hence this theory can be applied to deep foundation also.

⊙ Meyerhoff has also given the concept of reduced area for eccentric loading condition.



⇒



Pressure is uniform ⇒ $\left[Pr. = \frac{Q}{(L-2e_x)(B-2e_y)}\right]$

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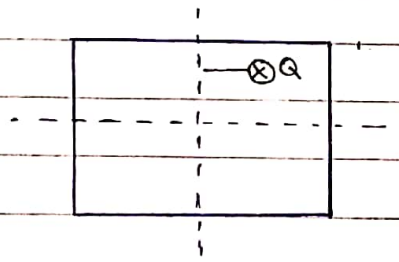
⊙ If load 'Q' on footing acts eccentrically. i.e; line of action of force is not passing through the centroid then width & length are reduced such that load acts at centre of reduced area.

Ques→ A footing 2m x 2m is founded 1m below the Ground level in a sandy soil (c=0) having $\phi = 36°$. Compute the allowable load at the base of footing considering FOS against shear = 3

$N_c = 50$, $N_q = 42$, $N_\gamma = 46$, $e_x = 0.25mm$ $e_y = 0.25mm$

$\gamma_{sat.} = 2\ t/m^3$ $\gamma_d = 1.6\ t/m^3$. Water table is 1m below the Ground level.

Solⁿ:



Using Elastic Theory:

$$q_u = \gamma D_f N_q + \frac{1}{2} B \gamma N_\gamma \quad \text{(Sq. footing correction)}$$

$$q_u = 1.6 \times 42 + 0.5 \times 0.8 \times 2 \times 2 \times 46 \times \frac{1}{2}$$

$$= 104\ t/m^2$$

then $q_{nu} = 104 - \gamma_{D_f} = 102.4\ t/m^2$ ($\gamma = 1.6$, $D_f = 1$)

then allowable Maxm Stress $= \frac{102.4}{FOS} = 34.13\ t/m^2$ (FOS = 3)

$$34.13 = \frac{Q}{4} + 2 \times \frac{Q \times (0.25)}{\frac{(2)^3}{6}} = Q\left[\frac{1}{4} + \frac{1.5}{4}\right]$$

$$Q = 54.61\ t$$

Using Meyerhoff's theory:

$$\frac{Q}{(2 - 2 \times 0.25)^2} \leq 34.13$$

$$\Rightarrow Q = 76.79\ t$$

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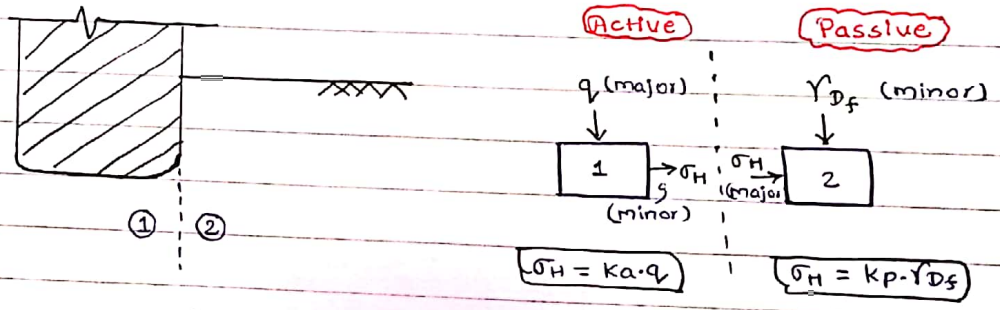
④ IS code Method

⊙ Based on Meyerhoff's eqn & is given in IS-6403:1981 & it utilises some additional water table correction factors.

⑤ Rankine's Theory (cohesionless)



$\Rightarrow K_a \cdot q = K_p \cdot \gamma_{D_f}$

$\Rightarrow \boxed{q = \gamma_{D_f} K_p^2}$

⊙ Theory is applicable for cohesionless soil & it considers eqm of 2 elements adjacent to each other at corner of footing. Element ① is assumed to be in active state & element [2] is in passive state.

⊙ If $\boxed{D_f = 0}$, bearing capacity from formula is also 0 (zero) which is not practical. This theory doesnot consider the effect of shape as well as size of foundation & hence is obsolete

Also, it is not valid for clayey soil.

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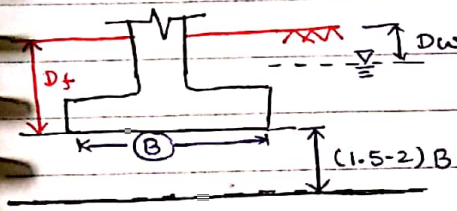
Bearing Capacity Based on settlement Criteria

⊙ Bearing capacity as proposed by various scientists is as follows:

$$q_{ps} = q_a = q_{a(net)} = 0.44\, N S_c C_w$$

(Peck, Hensen & Thorburn)

N = Corrected value from SPT test for soil upto a region of 1.5B - 2B from base of foundation.



S = Permissible settlement in "mm"

C_w = water-Table correction factor

$$= 0.5\left[1 + \frac{D_w}{D_f + B}\right] \leq 1$$

D_f = Depth of foundation.

B = width of foundation.

D_w = depth of W.T. from Ground surface.

⊙ Bearing capacity for raft foundation based on Is-code proposed by Peck ~~modified~~ modifying the TENG's formula is given as:

$$q_{ps} = q_a = q_{a(net)} = 0.88\, N S_c C_w$$

Note: ⊙ If width of foundation > 6m, it is treated as a Raft.

Ques → For a footing 2m x 2m, founded 1m deep in a sandy soil, the maxm settlement of the footing is not to exceed 50mm. W.T. is 1m below the Ground level & allowable soil pressure for a 50mm settlement

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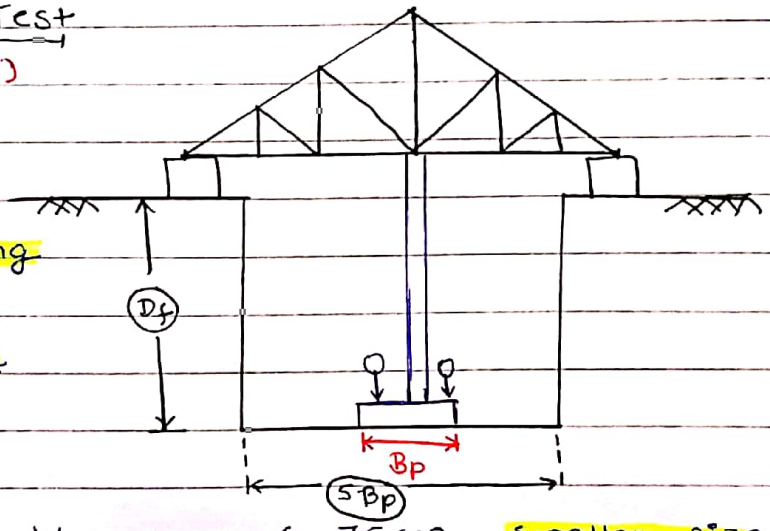
with w.t. at a very great depth is 50 t/m². Find the allowable value of bearing capacity for the actual soil condition.

Soln: $q = (0.44)(NS)(C_w)$ → $0.5\left(1+\frac{1}{3}\right)$

$\Rightarrow 50 = (0.44)(NS)$ → ①

$\frac{q}{50} = \frac{4}{3\times 2} \Rightarrow q = \frac{200}{3\times 2} = \frac{66.67}{2} = 33.33\ t/m^2$

Plate Load Test (PLT)



⊙ It is performed to calculate ultimate bearing capacity of soil.
Pressure v/s settlement graph is plotted.

⊙ Size of plate is kept b/w 30cm & 75cm. Smaller size plate is used for Granular soil and for clayey soil, larger size are used.

Min. thickness of plate should be 25mm. Width of test pit should be > 5 times plate width.

⊙ Initially, a pressure of 70 kg/cm² which is also known as seating pressure is applied on the plate and removed before test starts. This is to establish a perfect contact b/w plate and soil.

⊙ Atleast 2 no. of dial Gauges arranged diametrically opposite on the plate are used to measure settlement.

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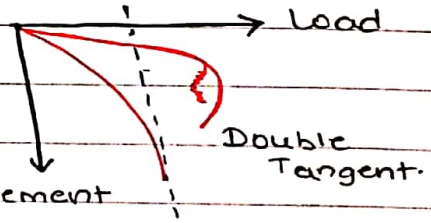
- Test is performed at an assumed foundation level & the plate is loaded by means of hydraulic jet or sand bags over a platform.

- Water table is kept below the test level so as to avoid any pore pressure developments.

- The test is performed in the increment of $\frac{1}{5}^{th}$ of assumed ultimate load upto the breaking point or upto a total settlement of 25 mm whichever occurs first.

- If a marked breakpoint is not visible, then approx. value of ultimate bearing capacity is determined by double tangent method.



Limitations of Plate Load Test

1. **Size Effect:** Results of the test, they reflect the soil behaviour within the pressure bulb.
   Pressure bulb depends upon size of loaded area & ∵ $B_p < B_f$, test doesnot represent a true value.

2. **Scale Effect:** Ultimate Bearing capacity 'q_u' of saturated clay is independent of size whereas for a granular soil $q_u \propto B$

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③ Time Effect: Plate load test is a short duration test hence settlements calculated are for short term loading condition, which is good representation when considered for granular soil. But in case of clayey soil, major part of ultimate settlement is due to 1° consolidation, which accounts for nearly 90% of ultimate settlement.

This is a long term effect, hence, settlement from Plate load test are not good representation.

④ Interpretation of Load: In General shear failure, ultimate load is easily identified from load settlement curve, but in other cases, interpretation may lead to error.

⑤ Water Table Effect: It affects bearing capacity of sandy soil & needs to be lowered below the footing. Practical observations state that the WT should be lowered by 1 to 1.5 m. below the test level so that it eliminates the effect of capillary action which will lead to an increase in effective stress of sandy soil.

Correction

Sandy

$$\left[\frac{q_{uf}}{q_{up}} = \frac{B_f}{B_p}\right]$$

$$\frac{S_f}{S_p} = \left[\frac{B_f\,(B_p + 0.3)}{B_p\,(B_f + 0.3)}\right]^2$$

B_f, B_p → metre (m)

Clayey

$$[q_{uf} = q_{up}]$$

$$\frac{S_f}{S_p} = \frac{B_f}{B_p}$$

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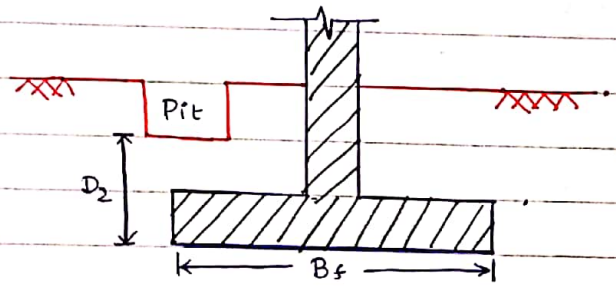
⊙ If actual founding level is below the PLT level then corrections need to be performed for settlement.

$$S_{f\,(corrected)} = S_f \left[\frac{1}{1 + \frac{D_2}{B_f}} \right]^{\frac{1}{2}}$$

(only for sandy soil)



Ques→ Following data was obtained from a plate load test carried on a 60cm square plate at a depth of 2m below ground level on a sandy soil which extends upto a large depth. Determine settlement of foundation 3m x 3m carrying a load of 110 ton.

foundation is located at a depth of 3m below ground surface. WT is at large depth.

Load Intensity (t/m²)	5	10	15	20
Settlement (mm)	2.0	4.0	7.5	11.0

Soln:

$$q = \frac{110}{9} = 12.22 \text{ t/m}^2$$

Sp for $q = 12.22 \Rightarrow 4 + \frac{2.22}{5}(3.5)$

$\Rightarrow S_p = 5.554$ mm

then; $\frac{S_f}{5.554} = \left[\frac{3\,(0.6 + 0.3)}{0.6\,(3 + 0.3)} \right]^2$

$\Rightarrow S_f = 10.328$ mm

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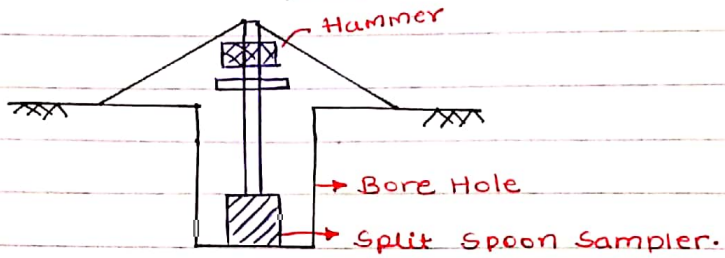
$$(S_f)_{corrected} = 10.328 \left[\frac{1}{1 + \frac{D_2 \to 1}{B \to 3}} \right]^{1/2} = 8.94 \text{ mm}$$

Standard Penetration Test (SPT)



⊙ In this test, a split spoon sampler is inserted into soil by means of free falling wt. of 65 kg from height of 75 cm. Test is performed in the bore Hole at the level of foundation.

⊙ No. of blows are observed for every 150mm penetration upto a total of 450mm. For the first 150mm, the no. of blows is ommited as it is considered a seating drive. 'N' value is the sum of last 300mm penetration at a particular level.

⊙ In a bore hole, SPT is performed at a vertical interval of 0.75m which can be taken upto 1.5m if depth of bore is quite large

Following values are refused i.e;

① If for successive 10 no. of blows, there is no advancement.

i.e; no penetration of Sampler.

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② If for any 150 mm penetration, the no. of blows is ~~>100~~ >50, it is refused.

③ If for any 300 mm penetration, no. of blows >100 it is refused.

→ It is best suited for Granular soil.

⊙ Out of no. of tests performed at diff. bore location, the (Sum / Avg.) is taken for the values within the range of ±50% of avg.

'N' value calculated needs to be corrected for overburden & Dilatency.

Overburden Correction

⊙ At shallow depth, N value is underestimated & actual values are higher, this is due to lateral confinement.

⊙ At deeper depth, N value is overestimated & actual value is lower & the value needs to be corrected.

$$N' = N\left[\frac{350}{70+\bar{\sigma}}\right]$$

$\bar{\sigma}$ = Eff. Stress in KN/m^2

N' = Corrected Value due to overburden.

⊙ This correcn is applied for soils having $\bar{\sigma} \leq 280$.

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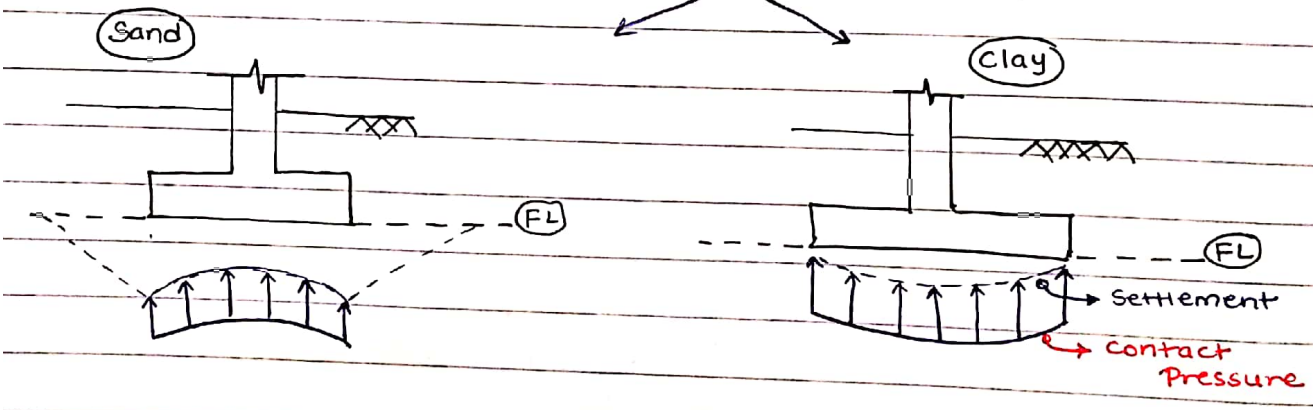
Dilatency correction

⊙ This correction is performed after overburden where water table is above or at the level of test. Correction is done only if $N' > 15$ & $\left[N_c\phi = 15 + \frac{1}{2}(N'-15)\right]$

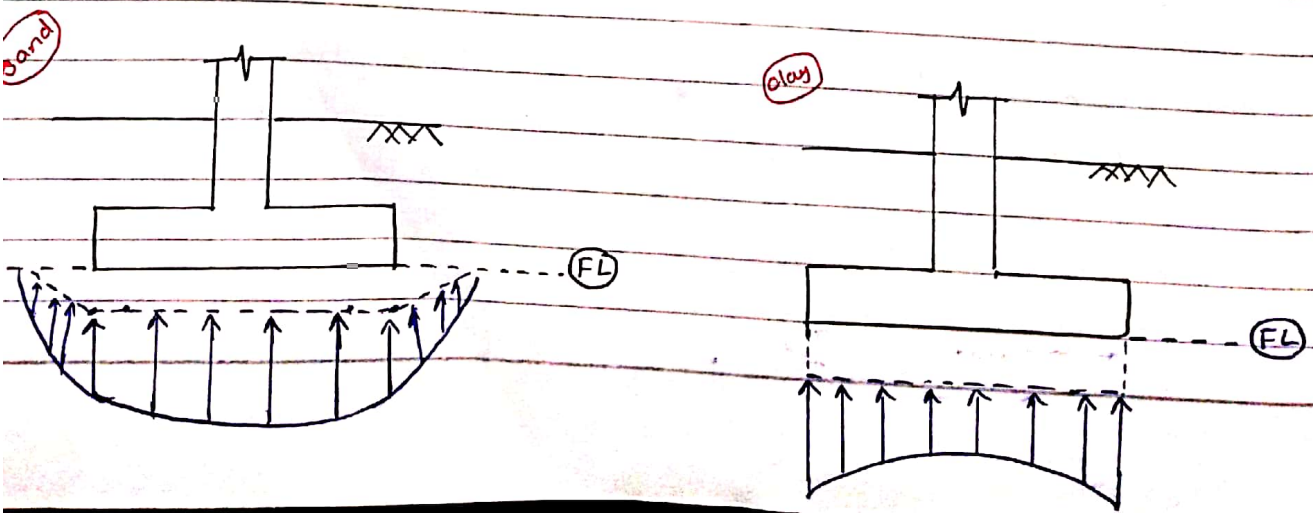
⊙ This correction is applied as the soil of sandy & silty nature has a tendancy to dilate in an undrained cond^n due to penetration resistance. Hence, if water table is below test level, dilatency correction are not applied.

Settlement / Pressure distribution

(Flexible footing)



(Rigid footing)



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Immediate settlement / Elastic Settlement

$$S_i = \frac{q B I (1-\mu^2)}{E}$$

E → Mod. of Elasticity of soil
μ → Poisson's ratio
B → width of footing
q → Net Pressure Intensity
I → Influence factor.

Note: ⊙ In flexible footing, contact pressure is uniform over the foundation width whereas in rigid footing, settlement is uniform along foundation width.

⊙ To increase settlement at any particular point, contact pressure at that point needs to be increased.

⊙ In case of Granular soil, E varies along the loaded area with lesser value at corner compared to central point.

PILE & FOUNDATION

⊙ Pile foundations are the deep foundations having small dia. shaft which can be driven or installed by boring into the ground.

⊙ Well foundation also known as Cassions are also known as deep foundation which are large dia. shaft which are installed by excavation & sinking.

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Classification of Pile

Piles can be classified on following basis:

① Material ② Method of load Transfer
③ Method of Installation ④ Purpose / function
⑤ Displacement of soil.

① → Material: Steel, concrete & timber are used as pile material. Concrete is provided where found^{n} is subjected to very high compressive forces.

⊙ Steel in the form of circular tubes or H-Pile is provided, where speedy work is needed to be done.

② Method of Load Transfer: Pile can provide resistance either by end bearing, friction or combination of both.

End bearing is considered at the pile tip & friction also known as skin friction acts on the periphery of the pile.

③ Method of Installation:

① Bored ② Driven

⊙ In bored pile, a borehole is excavated in soil which is either filled with cast-in-situ pile or a prefabricated pile.

⊙ Driven piles are installed within the soil by means

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of hammering.

④ → Purpose / Function & Location:

(i) Compression Pile
(ii) Tension Pile
(iii) Anchor Pile
(iv) Fender / Dolphin Pile
(v) Batter Pile
(vi) Compaction Pile.

⊙ Fender / Dolphin piles are provided in the Periphery of foundation of hydraulic structure.

⑤ → Soil displacement: Driven piles create larger disturbance in soil and hence are known as displacement pile whereas ~~boring~~ boring operation creates least disturbance. Hence, it is known as non-displacement pile.

⊙ In case of loose Sand, displacement pile can influence a zone of 3.5 times the pile dia. whereas in clayey type of soil, soil can get remoulded upto a region of 2 times the dia.

Pile Load Capacity

⊙ Load Capacity can be ~~strate~~ determined by, Static formula, dynamic formula, pile load test, penetration data given by various agencies.

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Static formula:

$$q_u = cN_c + (\gamma D_f) N_q + 0.5 \gamma B N_\gamma$$

1. ⊙ for piles, B is small. so, $0.5 \gamma B N_\gamma$ is neglected.

$$\gamma D_f = \bar{\sigma}_v$$

Clay

$$Q_{end\ Bearing} = A_b \cdot q_u = (cN_c) A_b$$

($\bar{\sigma} N_q$ is less)

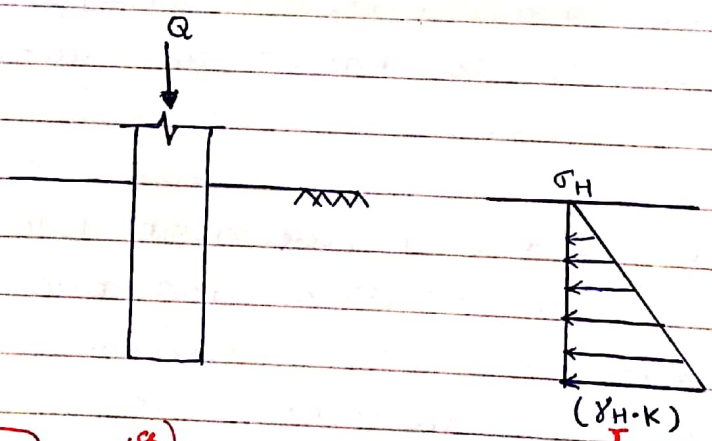
$N_c = 9$

Sand

$$Q_{end\ Bearing} = A_b \cdot q_u = (\bar{\sigma}_v N_q) A_b$$

A_b → Base Area.

Skin friction



$Q_s = f_s \cdot A_s$ (f_s)

$= [(\sigma_{v\,avg} \cdot K) \tan\delta] A_s$ → (Sand) [Effective]

(σ_v)

Clay: $f_s = c \times \alpha$ & $Q_s = f_s \cdot A_s$

α = adhesion factor.

$Q_u = Q_B + Q_s$

$= f_b A_b + f_s A_s$

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Sandy soil:

$$Q_u = \left[(\bar{\sigma}_{v\,avg.} \times K)\tan\delta\right] A_s + \left[\bar{\sigma}_{v\,(Base)} \cdot N_q\right] A_b$$

(L/2)

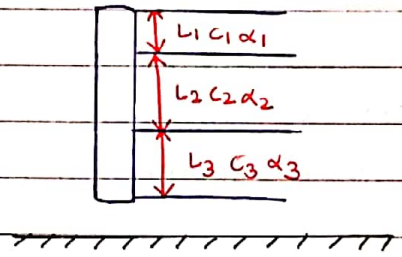
Clay:

$$Q_u = (c_{avg.} \cdot \alpha) A_s + \left[c N_c\right]_{(base)} A_b \qquad N_c = 9$$

Eg: ① Clayey:

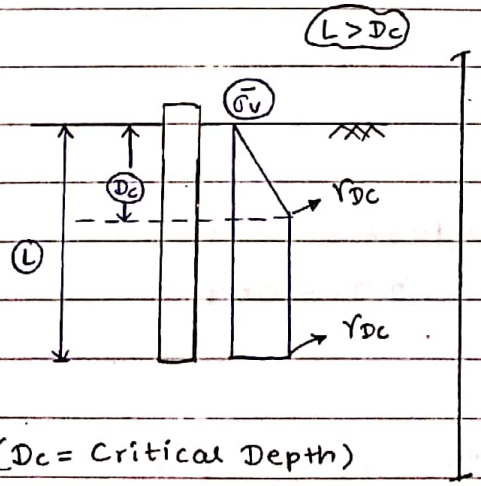
$$Q_B = (c_3 N_c)\frac{\pi}{4} d^2$$

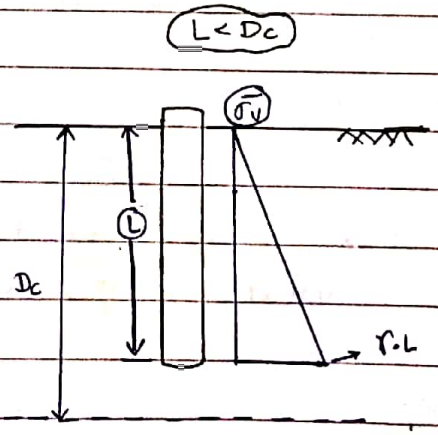
$$Q_s = \pi d \left[L_1 c_1 \alpha_1 + L_2 c_2 \alpha_2 + L_3 c_3 \alpha_3\right]$$



$\Rightarrow$ $Q_u = Q_B + Q_s$

Eg: ② Sand: In case of sandy soil, due to arching effect, effective vertical stress increases (↑) upto certain depth, after which it remains constant.





(Dc = Critical Depth)

- Critical depth in case of dense sand is 20 × Pile dia.
- Loose sand → Dc = 15 × Pile dia.

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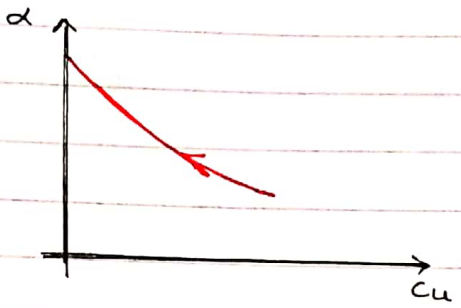
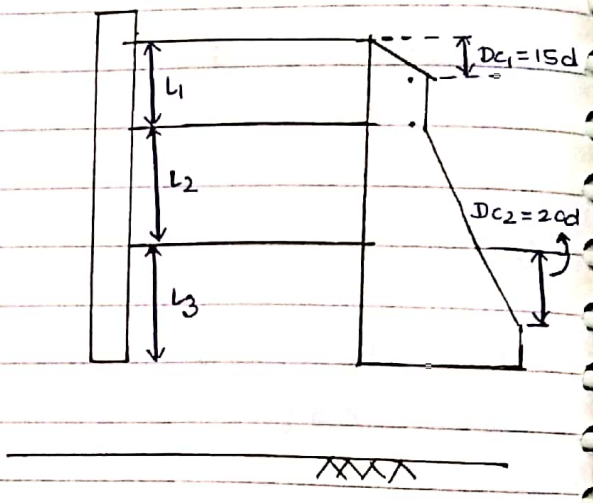
Eg:③ Stratification:

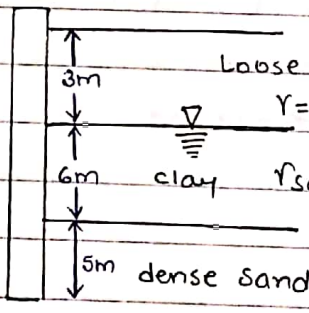




⊙ If we increase water consistency of soil then adhesion factor increases.

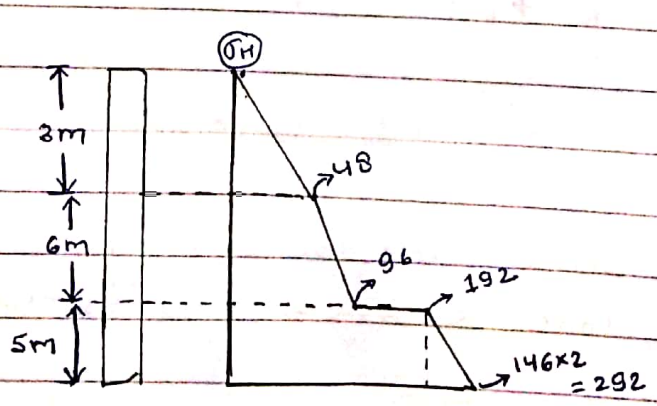
Ques→ Determine allowable pile load capacity of a 40cm dia. driven concrete pile having $N_q = 160$, FOS = 2.5, $\delta = \frac{3}{4}\phi$

K ⇒ Loose sand = 1
Dense sand = 2
$\alpha = 0.6$



Soln: $\overline{\sigma_{av}}_{(base)} = 16\times3 + (18-10)\times6 + (20-10)\times5 = 146\ KN/m^2$

$\sigma_b = 146 \times 160 \times \left(\frac{\pi}{4} \times 0.4^2\right) = 2935.504$



Dc loose = 15×0.4 = 6m

Dc dense = 20×0.4 = 8m

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$$Q_s = \left[(0.6\times15)\times6 + \left[\frac{1}{2}\times48\times3\tan\left(\frac{3}{4}\times30\right)\right]\times3 + \left[\left(192+\frac{(292-192)}{2}\right)\tan\times2\right.\right.$$

(Loose)

$$\left.\left.\tan\left(\frac{3}{4}\times40\right)\right]\times5\right]\times$$

(Dense)

$$= 0.4\pi\,[54 + 89.47 + 698.593] \qquad 0.4\pi$$

$$= 1058.168\ KN$$

$$Q_u = Q_s + Q_b = 3993.672$$

$$Q_{allowable} = \frac{Q_u}{2.5} = 1597.47$$

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09/11/12

For driven pile, the following Assumptions needs to be considered:

	δ	K Loose	K Dense
Steel	20°	0.5	1
Concrete	$\frac{3}{4}\phi$	1	2
timber.	$\frac{2}{3}\phi$	1.5	4

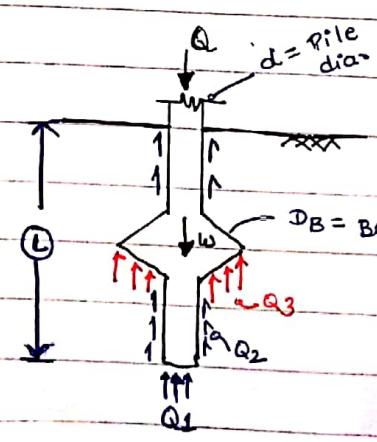
$\phi = 20°$

Bored Pile:

① The capacity of bored pile is b/w 50-30 % of driven pipe. This is due to loosening of the pile during the boring aberation.

The value of K varies b/w 0.3 to 0.7 and as an assumption, it is considered 0.5. The value of δ is equal to ϕ in case of dry soil and in case of slurry being used → $\delta < \phi$.

Under Reamed Pile



$Q + W$ = Resistance = $Q_1 + Q_2 + Q_3$

$Q_1 = f_B \cdot A_B$

(Base) { $\bar{\sigma}_v \cdot N_q$ → Sand ; $c \cdot N_c$ → clay

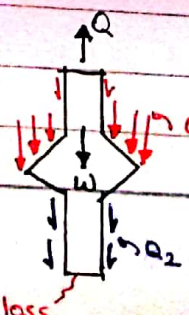
$Q_3 = f_{\text{Bulb level}} \cdot \frac{\pi}{4}\left[D_B^2 - d^2\right]$

clay ← cN_c

$Q_2 = f_s \cdot A_s$ → $\pi \cdot d \cdot L$

Sand: $\bar{\sigma}_{avg} \cdot K \cdot \tan\delta$

clay: $c \cdot \alpha$



$Q = W + Q_2 + Q_3$

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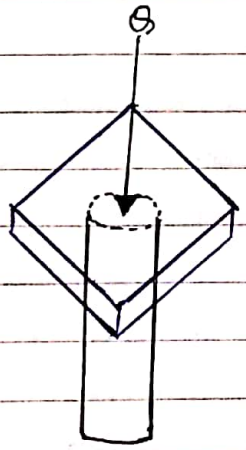
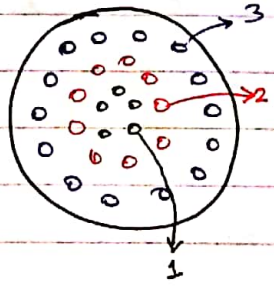
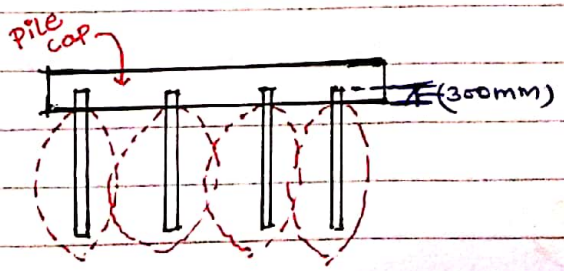
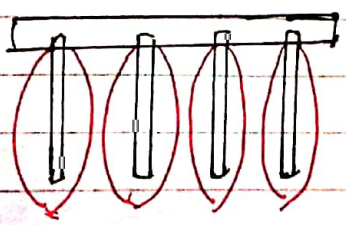
- These are provided on Expansive soils which are ~~bored~~ bored into the soil and are cast-insitu with the bulb.
- IS - 2911 : 1980, part-III recommends that a maxm of 2 no. of bulbs should be provided. Bulb helps in bearing and anchorage.
- The dia. of the bulb should be b/w 2-3 times the shaft dia.
- The vertical spacing of the bulbs should be > 1.5 times the bulb dia.
- The horizontal pile spacing should be > 2 x (bulb dia.)

Group Capacity









$\Sigma Q_{individual} > Q_{group}$

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Pile Group

⊙ A Min. of 3 no. of piles are to be provided as there is uncertainity in the vertical installation of the pile.

⊙ Bored Piles can have a single pile since in boring operation, verticality can be ensured.

⊙ Pile driving in sand should begin from the centre and proceed outwards to avoid the ground ~~tearing~~ tightening.

⊙ The Efficiency of the pile in a group reduces as the pressure bulbs overlap. Hence, $\eta < 100\%$ but in case of loose soil, since during the pile driving operation, the densification may occur resulting in a group Efficiency $> 100\%$.

⊙ The piles in a group behave independently if the spacing b/w them is > 8 (Shaft dia.). The min. spacing b/w the piles for end bearing is $2.5\,d$ and for friction pile, it is $3d$.

Minm spacing is provided so as to avoid any disturbance from the adjacent pile during the pile driving operation.

⊙ The Group capacity is considered as the min. of the individual behaviour and block behaviour. i.e;

$$(Qu)g = \min.[n\,Qui, Qu_B]$$

n = no. of piles in Group.

Qui = Individual pile capacity

Qub = Block capacity,

⊙ The Efficiency of Group η is calculated w.r.t. the individual pile capacities:

$$\eta g = \frac{Qug}{n.\,Qui}$$

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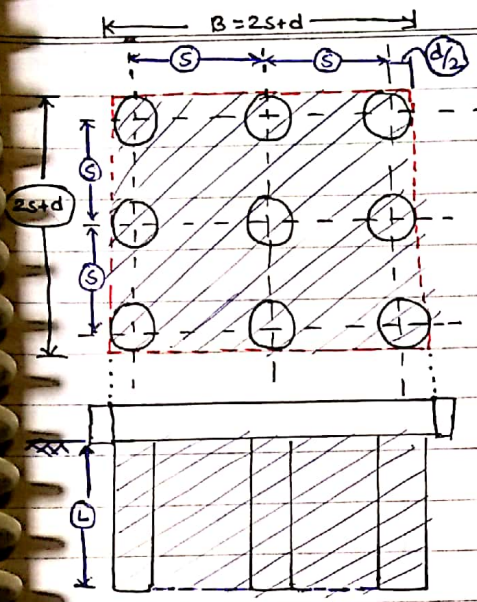


friction Effect

Bearing Effect

$Q_{u_B} = f_s A_s + f_B \cdot A_B$

Block Pile

B×B'×L

$A_s = 2(B+B')L$

$A_B = B \times B'$

In case of clays; $\alpha = 1.0$

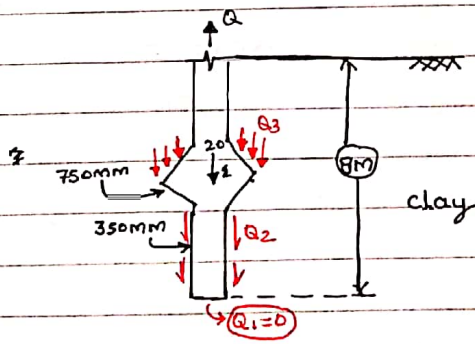
due to soil-soil interaction.

The value of α in case of pile group is always considered as '1' since in the block, the major contact is b/w soil and soil.

Ques→③ L = 8 mm, W = 20 KN, d = 350 mm, D_B = 750 mm

C_u = 50 kPa, α = 0.3, N_c = 9.

Soln:



C_u = 50 KN/m², ∴

clay $Q = Q_2 + Q_3 + W$

$Q_3 = 50 \times 9 \times \frac{\pi}{4}\left[(0.75)^2 - (0.35)^2\right]$

$= 155.548$ KN

$Q_2 = 0.3 \times 50 \times \pi \times 0.35 \times 8$

$= 131.95$ KN

W = 20 KN

Tensile load = $Q_2 + Q_3 + W$

= 307.4 KN

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$C_u = \frac{UCS}{2}$

Ques → ⑥ $\gamma = 20.2\ kN/m^3$ UCS = 200 kPa, pile dia. = 300mm (d)

L = 12m S = 750mm $\alpha = 0.6$

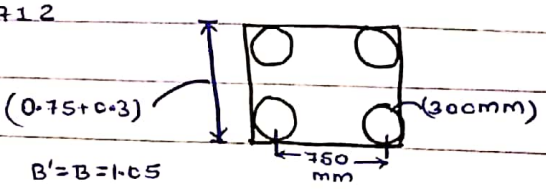
$C = \frac{UCS}{2} = \frac{200}{2} = 100\ kN/m^2$

$Q_{ug} = \min.[n Q_{ui}, Q_{uB}]$

$Q_u = f_s A_s + f_B A_B$ (→ 0)

↳ only friction pile so, taken zero.

$n \times Q_{ui} = 4 \times 0.6 \times 100 \times \pi \times 0.3 \times 12 = 2712$



$Q_{uB} = 2[1.05 + 1.05] \times 12 \times 1 \times 100$

↳ 5040

$B' = B = 1.05$

⇒ $Q_{ug} = 2712\ kN$

Ques → ④ A = 3×3 L = 12m, d = 250MM, UCS = 60 kN/M²

$\alpha = 0.9$ $\eta = 1$, s = ?

$C = \frac{60}{2} = 30$

Efficiency: $\eta = \frac{Q_{ug}}{n.Q_{uc}}$

$Q_{ui} = f_s A_s + f_B A_B$ (→ 0)

$= 0.9 \times 30 \times \pi \times 0.25 \times 12$

$= 254.469\ kN$

$9 \times Q_{ui} = 254.469 \times 9 = 2290.221$

$Q_{uB} = f_s A_s = (1 \times 30)[2 \times 2(2s + 0.25) \times 12] = 1440 \times (2s+d)$

$(2s+d) = 1.59m$

$s = 0.7m$

But min. spacing = 3 × dia = 3 × 0.25 = 0.75m = 750mm

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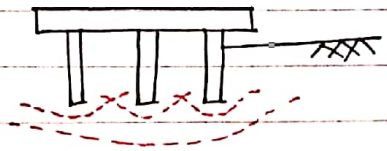
Settlement

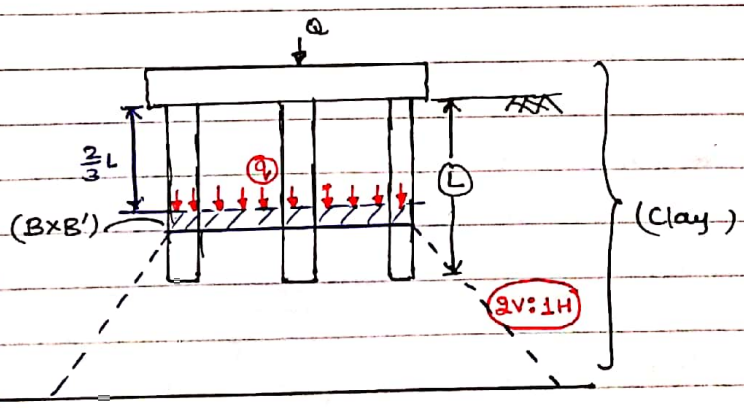
- The settlement of the pile group > that of individual pile. This is because a large volume of soil gets influenced due to the group behaviour.



- On clayey soil, since the major part of settlement is due to primary consolidation, hence the pile load test which is performed for a short duration does not give good result corresponding to the group effect.

  Hence, an equivalent raft theory is assumed for the calculation of group settlement:-

1.



$$q = \frac{Q}{B \times B'}$$

$$S_f = \frac{C_c \cdot H}{1+e_o} \log_{10} \frac{\bar{\sigma}_o + \Delta\bar{\sigma}}{\bar{\sigma}_o}$$

- An equivalent raft at a depth of $\frac{2}{3}^{rd}$ of embedment length of pile is considered to transfer the load on the pile.

  In case of friction pile and also the displacement pile.

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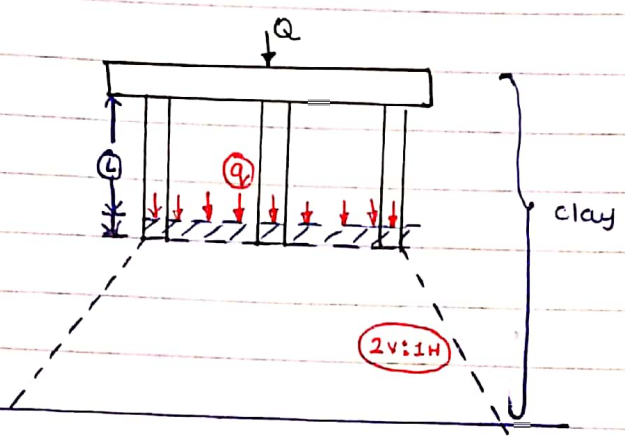
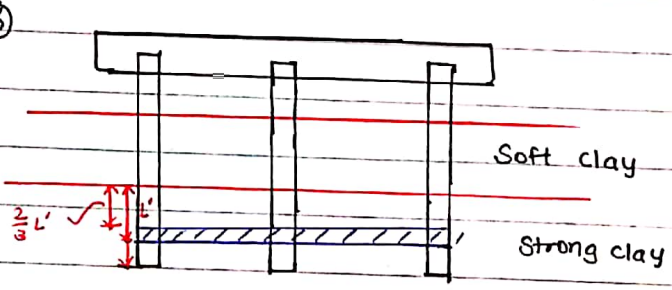
(2)

⊙ In case of Bearing pile, the equivalent raft is considered at the tip of the pile.



(3)



④ For Sandy soil, the settlement of the group is determined by the empirical relation in reference to the settlement obtained from pile load test.

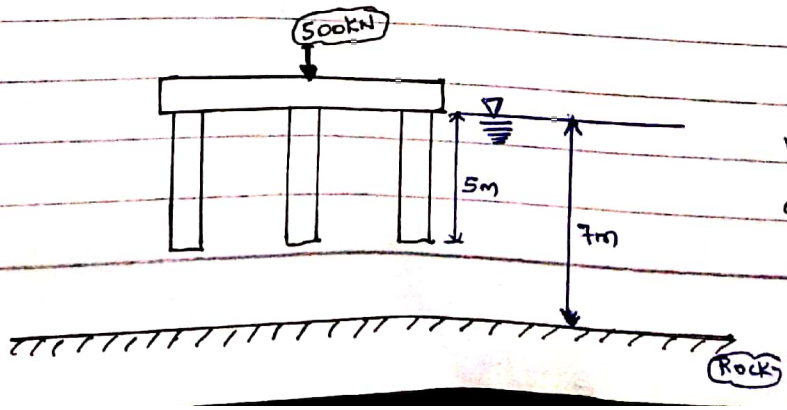
$$\frac{Sg}{Si} = \left[\frac{4B + 2.7}{B + 3.6}\right]^2$$

Sg → Group Settlement.

Si → Individual pile settlement

B → width of Block (m)

Ques → (5)



$\gamma_{sat.} = 20\ kN/m^3$

$C_c = 0.027$

$e_0 = 1.05$

$d = 200mm$

$S = 0.5\ m$

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Soln:

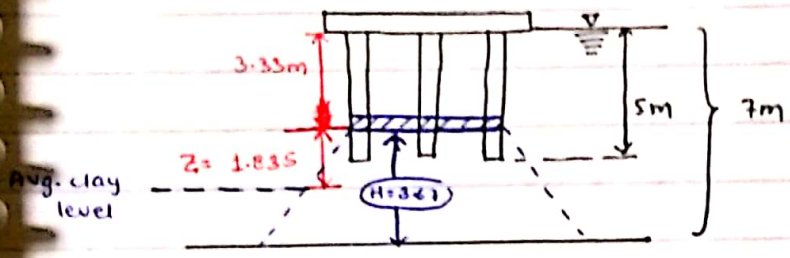
$$\frac{2}{3} \times S = \frac{10}{3} = 3.33\,m$$

$$S = \frac{C_c H_0}{1+e_0} \log\left(\frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_0}\right)$$

$H_0 = 7m$

$e_0 = 1.05$

$C_c = 0.027$



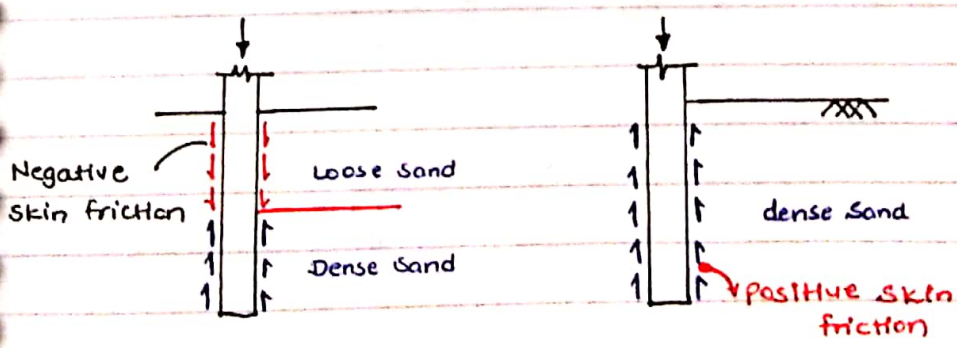
$$\bar{\sigma}_0 = 5.165 \times (20-10) = 51.65\ kN/m^2$$

$$B = B' = 2S + d = 2 \times 0.5 + 0.2 = 1.2$$

$$\Delta\bar{\sigma} = \frac{500}{(B+z)(B'+z)} = 54.28\ kN/m^2$$

$$S = 15.078mm$$

Negative Skin friction



If the settlement of the pile relative to the soil is in the downward direction then the frictional resistance on the pile surface will result in +ve skin friction.

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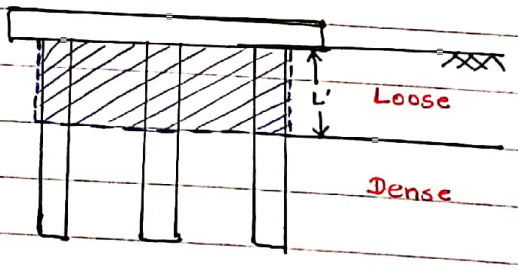
⊙ If the soil is a loose fill and the relative settlement of the adjacent soil is downward compared to the pile then Negative skin friction exists on the periphery of the pile.

Due to this -ve skin friction, the pile load capacity decreases.

⊙ For construction purpose, any loose deposit should be excavated and backfilled with a good quality Engineered soil.

Pile Group



Single

$Q_{Negative\ friction} = \alpha \cdot C \cdot L' \cdot \pi d$ ⇒ For clay

$Q_{Negative\ Friction} = \bar{\sigma}_{avg} \cdot K \tan\delta \cdot \pi d L'$ ⇒ For Sand

Group:

$Q_{Negative\ friction}$ = Qng at friction + wt. of Block in the Shaded Region

$\gamma \times B \times B' \times L$

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Pile Capacity Based on SPT Value

① For driven pile, the ultimate load is given as:

$$Q_u = 40\,N\,\frac{D_f}{B}\,A_B + 2\bar{N}\cdot A_S$$

$$40N\,\frac{D_f}{B} \not> 400N\ \left(\frac{KN}{m^2}\right)$$

H

② The same formula is used for displacement piles also

③ For H-Pile :

$$Q_u = 40\,N\,\frac{D_f}{B}\,A_B + \bar{N}\cdot A_S$$

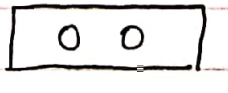
$$40N\,\frac{D_f}{B} \not> 400N$$

④ For Bared Pile, the value is $\frac{1}{3}^{rd}$ of the driven pile.

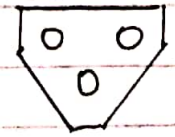
$$Q_u = \frac{1}{3}\left[40\,N\,\frac{D_f}{B}\,A_B + 2\bar{N}\cdot A_S\right]$$

Pile Group Efficiency on fields rule.

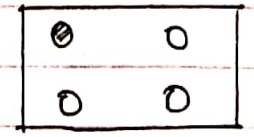
① A/c to this rule, for every adjacent pile, the group Efficiency is reduced by $\frac{1}{16}$



$$\eta_g = 1 - \frac{1}{16}$$



$$\eta_g = 1 - 2\times\frac{1}{16}$$



$$\eta_g = 1 - \frac{3}{16}$$

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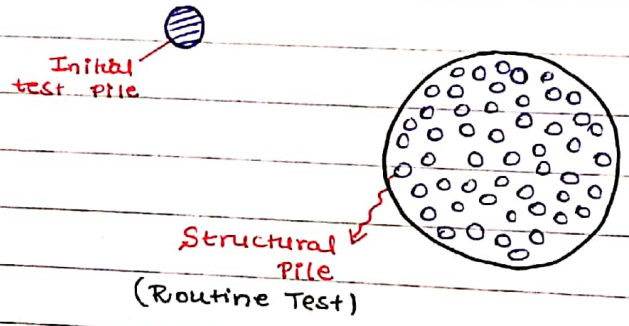
Group Efficiency Based on converse Lebbare's Theorum

$$\eta_g = 1 - \frac{\theta^\circ}{90^\circ}\left[\frac{m(n-1) + n(m-1)}{m.n}\right]$$

m = No. of pile rows

n = No. of Pile columns.

Pile Load Test



- It is a direct method to evaluate the pile capacity by doing insitu test.
- IS-2911 states that if the no. of piles are more than 200 then atleast two (2) no. of initial test needs to be perform.
- For routine test, approx. 0.5% - upto 2% of the total no. of piles are considered for testing.
- The initial test pile doesnot form the part of the structural system and is abandent after the test. Initial test is performed to obtain the allowable load and to check the settlement.
- Routine test is done on the piles which are part of the structural system to assess the soil densification and the suitability against the working load.

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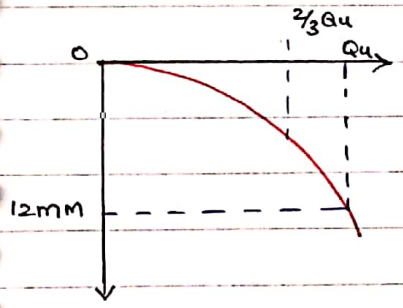
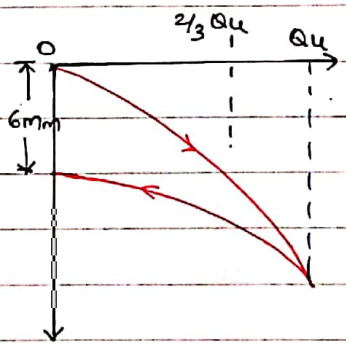
The safe load for the pile is taken as the min. of the following criteria:

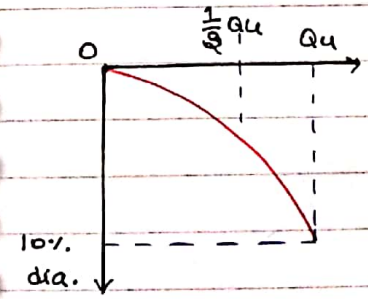
① $\frac{2}{3}$rd of the ultimate load value corresponding to a settlement of 12mm in total.

② $\frac{1}{2}$ of the load corresponding to a settlement of 10% of the pile dia. in total.

③ $\frac{2}{3}$rd of the load corresponding to a net settlement of 6mm.







Dynamic formula

$$W \cdot H = Q \cdot S$$

It is based on conservation of Energy.

$$\Rightarrow Q_{(\text{allowable})} = \frac{W \cdot H}{\text{Fos} \times (S + C)}$$

H = Height of drop (Free fall)
S = Settlement per blow (avg.)
drop → last 5 blows, steam → last 20 blows
C → Emp. const → drop = 2.5cm, → steam 0.25cm

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Modified, Engg. News formula:

$$Q_a = \frac{\eta \cdot W \cdot H}{FOS \times \left[S + \frac{C}{2}\right]}$$

- The pile capacity ~~is~~ based on the dynamic formula is based on principle that energy imparted on the pile equal to work done on the pile driven.

Engineering News formula

- In ENF, S represents Settlement per blow which is taken in cm which is the avg. for settlement of the last 5 no. of blows in case of drop hammer and 20 no. in case of steam Hammer.
- C is an emperical constant which is equal to 2.5 cm for the drop Hammer and 0.25 cm for Steam Hammer.

$$Q_a = \frac{W \cdot H}{FOS\,(S+C)}$$

Modified Hilley's formula

$$Q_a = \frac{\eta\, W \cdot H}{FOS\,(S + \frac{C}{2})}$$

η → Efficiency of the Hammer which is depend on W, e, p

$$\eta = \frac{W + e^2 P}{W + P}$$

W = Wt. of Hammer

P = Pile self wt.

e = Coeff. of Restitution.

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C= Total elastic compression per blow for pile, soil, packing.

S value is known as final settlement per blow.

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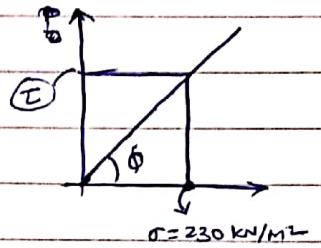
Ques:- A sample of dry sand tested in triaxial and direct shear test. In the triaxial test, the sample fails when the major and minor principle stresses are 960 KN/M² and 260 KN/M² respectively. What shear strength would be expected in the direct shear test when the normal stress is 230 KN/M².

Soln:

$$\sigma_1 = \sigma_3 \tan^2\left(45 + \frac{\phi}{2}\right)$$

$$960 = 260 \tan^2\left(45 + \frac{\phi}{2}\right)$$

$$\phi = 35°$$



$$\sigma_1 = \sigma_3 N\phi + 2c\sqrt{N\phi}$$

Assuming c=0

Taking the value of ϕ calculated in triaxial test to be equal to that in shear box test.

$$\tan 35.01 = \frac{\tau}{230} \Rightarrow \tau = 161.07 \text{ KN/M}^2$$

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