Geotechnical Engineering.

1) → Introduction
   → Index properties of soil.

2) → permeability
   → Effective stress & seepage through soil.

3) → Stress Distribution in soil.
   → compaction

4) → Consolidation

5) → Shear strength of soil.

father of soil Mechanics → 'Karl Terzaghi'

÷ Geotechnical Engineering ÷

: UNIT - I :

Introduction: Soil formation :-

↳ Soil are formed by weathering of rock and decomposition of organic matter.

↳ Based on source of soils are classified into two types.

↳ Organic soil

↳ Inorganic soil.

*→ Organic soils are called as Cumulose soils

Ex: peat, Humus, muck.

Geological Cycle :-

↳ Weathering of rocks - Transportation - Deposition - upheaval.

Types of Weathering :-

↳ 1, physical weathering 2, chemical weathering.

* **physical weathering :-**

↳ It is due to physical effects like, temperature, abrasion, wedging action of ice, penetration of plant roots.

↳ physical weathering results in no change in chemical composition of particles.

↳ It produces coarse grained and non cohesive soil.

Ex: Gravel, sand,

Chemical weathering:-

↳ It is due to chemical action oxidation, hydration carbonation, solution, leaching, hydrolysis. etc.

↳ Original rock minerals are transformed into clay minerals.

↳ It results in fine grained and cohesive soils.

Ex: clays.

Sedimentary soils:-

↳ These soil particles created at one location, transported and finally deposited in another location.

Source of transportation/Deposition Type of soil:

↳ River ⟶ Alluvial soil Ex: river, sand

↳ Lake ⟶ Lacustrine soil Ex: Varied clay

↳ Sea ⟶ Marine soil

↳ Wind ⟶ Aeoline soil Ex: Sand dune, Loess

↳ Gravitation ⟶ Colluvial soil, Ex: Talus.

↳ Glacier ⟶ Glacier deposited soil
Ex: Drift, till, outwash.

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Residual soil:

↳ These are soils which remain on the parent rock. without getting transported.

Ex: Black. Cotton soil, Laterite soils.

Soil Structure:-

↳ Soil structure is a geometrical arrangement of soil. particle in a soil mass.

↳ The behaviour of soil depends on the soil structure

Important type of soil structure:-

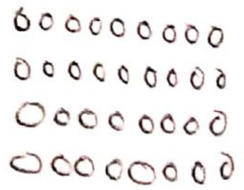
↳ The soil structure are clasified in to based on. Depending upon the particle size and mode of formation

↳ The following soil structure are:

1. → Single grained
2. → Honey- comb
3. → flocculated.
4. → Dispresed
5. → Composite structure.

Single grained structure :-

↳ It is present in soil like gravel, and sand.



Loosest paking

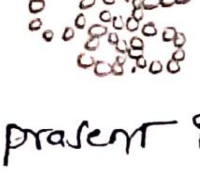
↳ if particles are assumed as spheres, the loosest and densest packing.

Densest packing:

↳ The void ratio for the loosest state is 0.91

↳ The void ratio for the densest packing 0.35

Honey - comb structure :-



↳ present in fine sand or silts to be deposited such that particles when settle develop a particle to particle contact. It voids lie b/w 0.002mm & 0.02mm.

Ex: Sands, silts.

Flocculated structure :-

↳ the flocculated soil structure are occurs in clays.

↳ formed when there is net attractive force b/w particles.

↳ Edge to face orientation

↳ Has high shear strength, low compressibility, high permeability.

Structure:

Dispersed structure :-

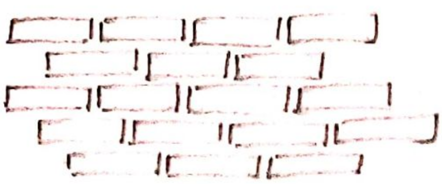
↳ the dispersed structure occurs in remoulded clays.

↳ face to face orientation

↳ formed when there is net repulsive force b/w particles.

↳ Has low shear strength, high compressibility, and low permeability.

Structure:



Composite structure:-

↳ When the soil contains different type of particles, a composite structure is formed.

Remoulding:

↳ Remoulding causes a loss of strength in cohesive soils.

Thixotropy:

↳ The phenomenon of regaining of lost strength with the passage of time, with no change in water content is known as thixotropy.

Clay mineralogy:

→ It deals with the structure of clay minerals.

→ The most significant properties of clay depends upon the types of minerals.

Basic structural units of clay minerals:-

→ Tetrahydral unit.

→ Octahydral unit.

Tetrahydral unit:-

→ It consist of one silicon ion surrounded by '4' oxides forming tetrahydral unit.

Octahydral unit:-

→ It consist of '6' hydroxides forming a configuration of octahydra and having alluminium atom at the centre.

Isomorphus:-

→ It is possible when the one atom in a base unit may be replace by another atom. It is known as Isomorphus substitution.

Types of clay minerals:-

1) → Kaolinite

2) → Illite

3) → Montmorillonite

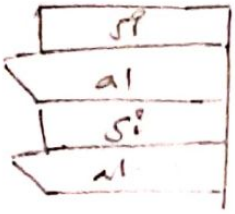
Kaolinite:-

↳ In this type of clay the basic unit is formed by atomic bond of silica sheet and alluminium sheet.

↳ stable mineral

↳ Causes no swelling and no srinkage present

↳ predominantly in china clay.



Illite clay:-

↳ In this Illite clay the basic structure of this clay consists of silica sheet, alluminium sheet and silica sheet and potasium ions are present in b/w.

↳ Illite clay causes due to medium swelling and srinkage

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Montmorillonite:-

↳ In this type of clay is similler to Illite clay where water molecules and Contains are present. in b/w the layer of clay.

↳ Montmorillonite clay causes due to high Shrinkage & high swelling.

↳ High percentage is present in Bentonite followed by Block cotton soils.

Hallosyte:-

↳ In this Hallosite clay properties due to Kaolinite.

Specific surface area:-

↳ In this total surface area of soil particles per unit weight or per unit volume of soil.

↳ montmorillonite clay minerals has largest specific surface area (about 800m²/gm).

Range of particle size

1. Gravel: → 80mm to 4.75mm
2. Sand → 4.75mm to 0.075mm
3. silt → 0.075mm to 0.002mm
4. clay → $<$0.002mm.

→ The majority of Naturally occuring collapsing soil
→ Aeolian

Properties of soils:-

i, soil phase system:-

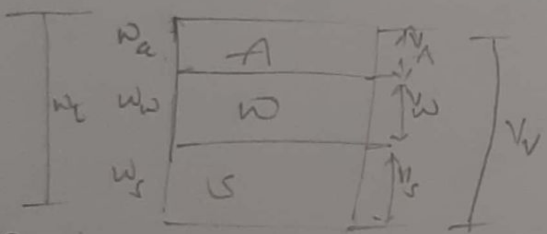
↳ It consist 3 types:

1, Un saturated
2, fully saturated
3, fully Dried:

1, Un-saturated:-

↳ Un saturated soil consist of solids + water + air.

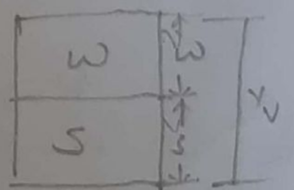
↳ it is also Termed as three phase system.



2, fully saturated:-

↳ it consist of solids + water

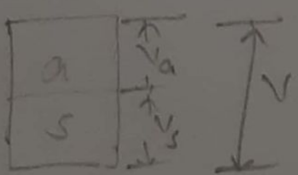
↳ it is known as Two-phase system.



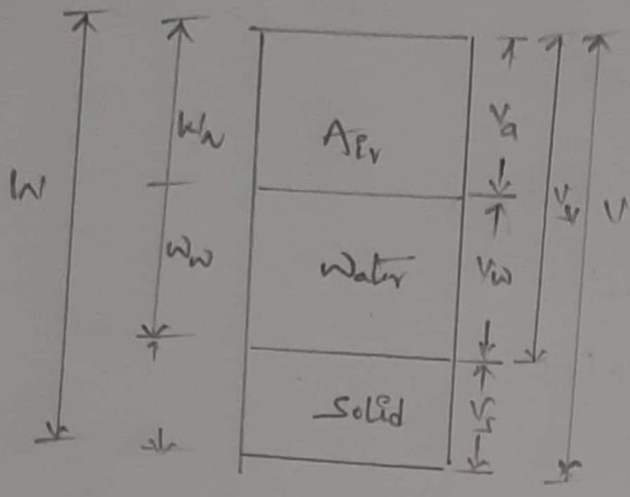
3, fully Dried:-

↳ it consist of solids + air.

↳ it is known as 2-phase system.

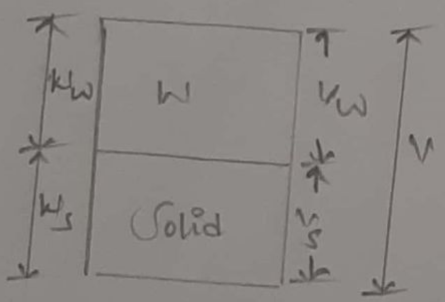


Phase Diagram:—



3-phase diagram. → partially saturated

2-phase System:—



Saturated

Dry

- V → Total volume
- W → Total weight of soil
- V_a → volume of air
- V_s → volume of solid
- V_w → volume of water
- W_a → weight of air
- W_w → weight of water
- W_s → weight of solid.

Void ratio:(e) ÷

↳ void ratio is defind as the ratio b/w Volume of Voids to Volume of Solids

$$e = \frac{\text{Vol. of Voids}}{\text{Vol. of Solids}} = \frac{V_v}{V_s}$$

↳ Range: Can have any Value grater then zero.

↳ Some times 'e' may also be grater then one

Porosity: ÷ (n)

↳ porosity is defind as the ratio b/w Volume of voids to total volume soil.

$$n = \frac{\text{Vol. of Voids}}{\text{Total. Vol. of Soil}} \times 100 = \frac{V_v}{V} \times 100.$$

↳ it is also Called as percentage Voids.

↳ Range: $0 < n < 100\%$

↳ Relation between 'n' & 'e' is $n = \frac{e}{1+e}$ $\therefore e = \frac{n}{1-n}$.

Water content: (w) ÷

↳ it is also called as moisture content and it is defined as ratio of weight of water to weight of solids in given soil

$$w = \frac{W_w}{W_s} \times 100.$$

Degree of Saturation: (S_r) ÷

↳ it is defined as volume of water to volume of voids

$$S_r = \frac{\text{Vol. of water}}{\text{Vol. of voids}} \times 100$$

$$= \frac{V_w}{V_v} \times 100$$

↳ Range: $0 \leq S_r \leq 100\%$

↳ for fully saturated $S_r = 1$ or 100%.

↳ for dry soil, $S_r = 0$.

Air Content (a_c) ÷

↳ it is defined as volume of air to volume of voids

$$a_c = \frac{\text{Vol. of air}}{\text{Vol. of voids}} \times 100$$

↳ for saturated soil, $a_c = 0$

↳ Dry soil, $a_c = 100\%$

↳ Range: $0 \leq a_c \leq 100\%$.

↳ $a_c + S_r = 100\%$

Percentage air voids: (n_a):-

↳ It is defind as the rato b/w Volume of air to total volume

$$n_a = \frac{\text{Vol. of Air}}{\text{total volume}} \times 100$$

↳ for a saturated soil $n_a = 0$

↳ for dry soil $n_a = n$

↳ Range: $0 \le n_a \le n$

↳ $n_a = n \cdot a_c$

Water Content (or) moisture Content (w):-

↳ It is defind as the ratio b/w weight of water to weight of solids

$$w = \frac{\text{wt. of water}}{\text{wt of solids}} \times 100 = \frac{W_w}{W_s} \times 100$$

↳ Range: Can have any value grater than ≥ 0

Some time it can be > 100%.

Bulk unit weight of soil (γ):-

↳ It is the ratio b/w total weight to total volume of soil.

$$\gamma = \frac{\text{total wt}}{\text{total vol. of soil}} = \frac{W_w + W_s + W_a}{V}$$

Dry unit weight of soil (γ_d):-

↳ It is the ratio b/w weight of solids to total volume of soil.

$$\gamma_d = \frac{\text{wt of solids}}{\text{total vol. of soil}} = \frac{W_s}{V}$$

Unit weight of solids (γ_s):-

↳ It is the ratio b/w weight of solids to volume solids

$$\gamma_s = \frac{\text{Wt. of solids}}{\text{volume of solids}} = \frac{W_s}{V_s}$$

↳ while γ_s is constant for a given soil the γ_d is not constant.

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Saturated unit weight: (γ_{sat}) :-

↳ It is bulk unit weight when soil is fully saturated.

↳ It is also equal to total weight of soil when fully saturated per unit of total volume

Submerged unit weight (γ_{sub} or γ') :-

↳ It is the submerged weight of soil solids per unit of total volume of soil

$\gamma' = \gamma_{sat} - \gamma_w$

↳ It is based on Archimedes principle.

Specific Gravity of soil solids or soil (G or G_s) :-

↳ It is the ratio of weight of given volume of soil solids at a given temperature to the weight of an equal volume of distilled water at that temperature

↳ As per IS code standard temperature for measuring 'G' is 27°c

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

for a given soil, G remain constant Generally 'G' for
↳ Inorganic solid lies between 2.60 - 2.85
↳ Organic solids → 1.2 to 1.4.

Apparent or Mass or Bulk Specific Gravity of soil :-

$G_m = \frac{\gamma}{\gamma_w}$ partially saturated soil

$G_m = \frac{\gamma_d}{\gamma_w}$ for dry soil

$G_m = \frac{\gamma_{sat}}{\gamma_w}$ for saturated soil

G_m is not constant

$G_m < G$.

Some important relationship :-

↳ 1, $e = \frac{w \cdot G}{S_r}$

↳ 2, $\gamma = \frac{\gamma_w (G + e \cdot S_r)}{1+e}$

↳ 3, $\gamma_{sat} = \frac{\gamma_w (G+e)}{1+e}$

↳ 4, $\gamma_d = \frac{\gamma_w \cdot G}{1+e}$

↳ 5, $\gamma_d = \frac{\gamma}{1+w}$

↳ 6, $\gamma_d = \frac{(1-n_a) \cdot \gamma_w \cdot G}{1+w \cdot G}$

↳ 7, $\gamma = \gamma_d + S_r (\gamma_{sat} - \gamma_d)$.

→ $\gamma_{sub} = \gamma_{sat} - \gamma_w$ → Archimede's principle.

Apparatus: Uses:

→ Casagrande's → Determination of Consistency limit

Hydrometer → Determination of Grain size distribution

plate load test → Determination of safe load bearing Capacity

oedometer → Consolidation characteristic.

Equipment: property of soil:

pycnometer → Specific Gravity

permeameter → Co-efficient of permeability

Vane shear → Cohesion.

pippete → Grain size.

Problems:-

1) The specific gravity of a soil sample is 2.7 & it void ratio is 0.945. if it is fully saturated what will be the moisture content of soil?

Sol: $G = 2.7 \quad e = 0.945$

fully saturated soil $S_r = 1$

using the relation. $e = \frac{wG}{S_r}$

$0.945 = \frac{w \cdot 2.7}{1}$

$\frac{0.945}{2.7} = w$

$w = 0.35$ or 35%.

A sample of dry soil weights 120gm and its volume 80ml. if the specific gravity is 2.80 the void ratio of the sample is

$G = 2.80$

Dry wt, $W_s = 120$ gm

Volume, $V = 80$ml or 80 cc

$\gamma_d = \frac{120}{80} = 1.5$ gm/cc

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

$1.5 = \frac{1 \times 2.86}{1+e} = e = 0.86.$

→ 1 cubic meter of wet soil weight 20kN. and it's Dry weight is 18kN. Specific Gravity of Solids is 2.67. Determine the water content porosity void ratio and Degree of Saturation?

Sol: Given data.

Volume $= 1 m^3$

Weight of water $= 20 kN$,

Dry weight $= 18 kN$

Sp. Gravity $= 2.67$.

1. Water content $w = \frac{W_w}{W_s} \times 100$

2. porosity $n = \frac{V_v}{V_s} \times 100$

3. void ratio $= e = \frac{V_v}{V}$.

4. Degree of saturation $S = \frac{V_w}{V_v} \times 100$

$W_w = W - W_s$

W_w = wet wt of solids − Dry wt of solids

$= 20 - 18$

$W_w = 2 kN$

$$\boxed{V_v = V - V_s}$$

$V_s = \frac{W_s}{G_s \cdot \gamma_w} =$ $\gamma_w = \frac{W_w}{V_w}$

$V_w = \frac{W_w}{\gamma_w} = \frac{2}{9.8} = 0.204 m^3$

$$\boxed{\begin{aligned} &= \frac{2}{\cancel{10}} \times \cancel{100} \\ &W = 10 kN. \end{aligned}}$$

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$V_s = \frac{18}{2.67 \times 9.81} = \frac{kN}{kN/m^3}$

$V_s = 0.68 m^3$

$\boxed{V_v = V - V_s}$

$V_v = 1 - 0.68$

$V_v = 0.32 m^3$

2. porosity $(n) = \frac{V_v}{V} \times 100$

$n = \frac{0.32}{0.68} \times 100 \quad \frac{\cancel{m^3}}{\cancel{m^3}}$

$n = 46.72$

3. Void ratio $(e) = \frac{V_v}{V_s} \quad \frac{\cancel{m^3}}{\cancel{m^3}}$

$= \frac{0.32}{0.68}$

$e = 0.4672$

4. Degree of saturation $(S) = \frac{V_w}{V_v} \times 100$

$V_w = \frac{W_w}{\gamma_w} \quad \frac{kN}{kN/m^3}$

$= \frac{2}{9.81} m^3$

$S = \frac{0.203}{0.32} \times 100 \quad \frac{\cancel{m^3}}{\cancel{m^3}}$

$S = 0.634$

The weight of moisture soil is 25 kN and its volume is $0.01 m^3$. after drying the soil in an even weight reduces to 16 kN. Take $G = 2.71$ calculate water content Dry Density void ratio porosity and degree of saturation

Sol: Given that

Volume $(V) = 0.01 m^3$

wt of moist soil = 25 kN

$G = 2.71$

Dry wt = 16 kN

1. water content $(w) = \frac{W_w}{W_s} \times 100$

2. Dry Density

3. Void ratio $(e) = \frac{V_v}{V_s}$

4. porosity $(n) = \frac{V_v}{V_s} \times 100$

5. Degree of saturation $S = \frac{V_w}{V_v} \times 100$

W_w = wt of solids – Dry wt. of soil

$= 25 - 16$

$W_w = 9 kw$

1. Water Density $(w) = \frac{W_w}{W_s} \times 100$

$= \frac{9}{\cancel{25}} \times \cancel{100}$

$= 36 kN$

2. Void ratio $(e) = \frac{V_v}{V_s}$

$V_v = V - V_s$

$V_s = \frac{W_s}{G_s \cdot \gamma_w}$

$= \frac{16}{9.81 \times 2.71}$

$V_s = 0.601 m^3$

$V_v = V - V_s$

$= 1 - 0.601$

$V_v = 0.39 m^3$

2. porosity $(n) = \frac{V_v}{V_s} \times 100$

$= \frac{0.399}{0.601} \times 100$

$n = 66.3$

3. Void ratio $(e) = \frac{V_v}{V_s}$

$= \frac{0.399}{0.601}$

$e = 0.663$

4. Degree of saturation $S = \frac{V_w}{V_v} \times 100$

$V_w = \frac{W_w}{\gamma_w} = \frac{9}{9.81}$

$V_w = 0.9177$

$= \frac{0.917}{0.399} \times 100$

$S = 229.8.$

→ The mass moisture soil is 20kg and it volume 0.011m, find the mass reduce to 16.5kg after drying determine the Dry density. Density of moist soil. water content void ratio porosity and degree of saturation and take specific gravity 2.67

Solt Given

wt moist soil = 20kg

Volume (V) = $0.011 m^3$

Dry. wt = 16.5 kg

$G_s = 2.67$

$\gamma_w = 1$

i) Dry Density:

* $\frac{\text{wt Dry Density}}{\text{volume}}$

* moist soil Density = $\frac{\text{Soil Density}}{\text{Volume}}$

ii) water content (w) = $\frac{W_w}{W_s} \times 100$

iii) Void ratio (e) = $\frac{V_v}{V_s}$

iv) porosity (n) = $\frac{V_v}{V_s} \times 100$

v) Degree of saturation (S) = $\frac{V_w}{V_v} \times 100$

W_w = wt. of solids − Dry wt. of soil

= 20 − 16.5

W_w = 3.5 kg

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$$= \frac{8.5}{16.5} \times 100$$

$w = 21.21\,kg$

$$\frac{\text{wt of Dry density}}{\text{volume}}$$

$$= \frac{16.5}{0.011} = 1500\ kg/m^3$$

$$\frac{\text{wt of Dry density of soil}}{\text{volume}}$$

$$= \frac{20}{0.011} = 1500\ kg/m^3$$

1. porosity $(n) = \frac{V_v}{V_s} \times 100$

$V_v = V - V_s$

$$V_s = \frac{W_s}{G_s \cdot \gamma_w}$$

$$= \frac{16.5}{2.67 \times 1}$$

$V_s = 6.17\ kg/m^3$

$= 0.11 - 6.17$

$V_v = -6.06\,kg$

Void ratio $(e) = \frac{V_v}{V_s}$

$$= \frac{-6.06}{6.17}$$

$e = -0.98\,m^3$

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Degree of saturation $(S) = \frac{V_w}{V_s} \times 100$

$V_w = \frac{W_w}{\gamma_w} = \frac{3.5}{1}$

$V_w = 3.5\ kg$

$= \frac{3.5}{-6.06} \times 100$

$S = -57.75$.

↳ If the volume of voids is equal to the volume of solids in a soil mass then the values of porosity & void ratio.

Given: $V_v = V_s$

porosity $= n = \frac{V_v}{V} = \frac{V_v}{V_s + V_v} = \frac{1}{2} = 0.5$

Void ratio $e = \frac{V_v}{V_s} = \frac{1}{1} = 1$

$= 0.5$ to 1.0.

↳ The porosity of a soil sample is 30% and sp. Gravity of it's particle is 2.75. Then the saturated unit wt is equal to.

$\gamma_{sat} = \frac{(G + e)\gamma_w}{1+e}$ $\quad = \quad e = \frac{n}{1-n} = \frac{0.3}{1-0.3}$

$e = 0.43$

$= \frac{(2.75 + e)\gamma_w}{1+e}$

$= \frac{(2.75 + 0.43) \times 1}{1 + 0.43} = 2.22\ g/cm^3$

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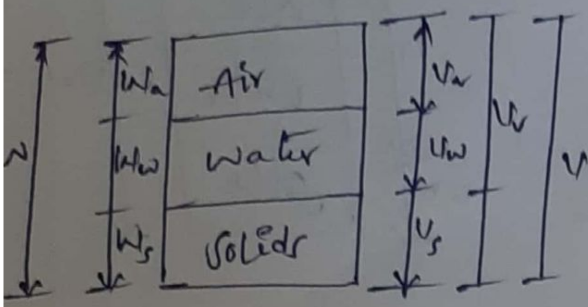
Relations:



$$V = V_a + V_w + V_s \quad \therefore V_v = V_a + V_w$$

$$W = W_a + W_w + W_s$$

$$\boxed{W = W_w + W_s}$$

Relation b/w W, W_s, w:—

$$W = W_a + W_w + W_s$$

$$W = W_w + W_s \qquad \therefore \frac{W_w}{W_s} = w$$

$$W = W_s\left(1 + \frac{W_w}{W_s}\right)$$

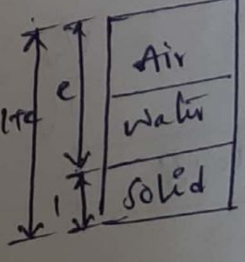
$$W = W_s(1 + w)$$

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

Relation b/w e & n:—

Porosity: $(n) = \frac{V_v}{V}$

$$n = \frac{V_v}{V_v + V_s} = \frac{\frac{V_v}{V_s}}{\frac{V_v + V_s}{V_s}} = \frac{e}{1+e}$$



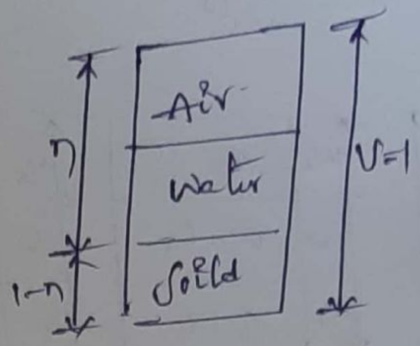
$V_v = e$

$V_s = 1$

$V = 1+e$.

$$\boxed{n = \frac{e}{1+e}}$$

Void ratio: $e = \frac{V_v}{V_s}$



$$e = \frac{V_v}{V - V_v}$$

$$= \frac{V_v/V}{V - V_v/V} = \frac{n}{1-n}$$

$$\boxed{e = \frac{n}{1-n}}$$

Relation b/w n, n_a, a_c :-

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

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

$$\boxed{n_a = n \cdot a_c}$$

Relation b/w a_c & S :-

$$a_c + S = \frac{V_a}{V_v} + \frac{V_w}{V_v}$$

$\because a_c = \frac{V_a}{V_v}$ → Air Content

$\because S = \frac{V_w}{V_v}$ → Degree of Saturation.

$$= \frac{V_a + V_w}{V_v}$$

$$= \frac{V_v}{V_v} = 1$$

$$\boxed{a_c + S = 1}$$

Relation b/w n_a, e & s:-

$$n_a = \frac{V_a}{V} = \frac{V_v - V_w}{V_v + V_s} = \frac{V_v\left(1 - \frac{V_w}{V_v}\right)}{V_s\left(1 + \frac{V_v}{V_s}\right)}$$

$$\boxed{n_a = \frac{e(1-s)}{(1+e)}}$$

Relation b/w γ_d, γ & w:-

$$\gamma = \frac{W}{V} = \frac{W_s + W_w}{V}$$

$$\gamma = W_s\left(1 + \frac{W_w}{W_s}\right) / V$$

$$\gamma = \frac{W_s(1+w)}{V}$$

$$\gamma = \gamma_d (1+w)$$

$$\boxed{\gamma_d = \frac{\gamma}{1+w}}$$

$\frac{W_w}{W_s} = w$ → water content.

$\frac{W_s}{V} = \gamma_d$ → dry unit weight of soil.

Relation b/w e, S, w & G :-

Void ratio : (e) = $\frac{V_v}{V_s}$

$= \frac{V_v}{V_w} \cdot \times \frac{V_w}{V_s}$

$\gamma_s = \frac{W_s}{V_s}$ ⇒ ; $\gamma_w = \frac{W_w}{V_w}$

$V_s = \frac{W_s}{\gamma_s}$ $V_w = \frac{W_w}{\gamma_w}$

$\because \frac{V_w}{V_v} = S$
Degree of saturation

$e = \frac{1}{S} \cdot \left(\frac{(W_w/\gamma_w)}{(W_s/\gamma_s)} \right)$

$= \frac{1}{S} \cdot \left(\frac{W_w}{\gamma_w} \times \frac{\gamma_s}{W_s} \right)$

$= \frac{1}{S} \cdot \left(\frac{W_w}{W_s} \cdot \frac{\gamma_s}{\gamma_w} \right)$

$\because G = \frac{\gamma_s}{\gamma_w}$

$\therefore \gamma_s = G \cdot \gamma_w$

$e = \frac{1}{S} \times w \times G$

$\boxed{e \cdot S = w \cdot G}$

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Relation b/w γ_t, G, e, w & γ_w :-

$$\gamma_t = \frac{W}{V} = \frac{W_s + W_w}{V_s + V_v}$$

$$= \frac{W_s\left(1 + \frac{W_w}{W_s}\right)}{V_s\left(1 + \frac{V_v}{V_s}\right)}$$

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

$\because G = \frac{\gamma_s}{\gamma_w}$

$\therefore \gamma_w \cdot G = \gamma_s$

$S.e = w.G$

$w = \frac{S.e}{G}$

$$\gamma_t = \frac{G.\gamma_w\left(1 + \frac{S.e}{G}\right)}{1+e}$$

$$\gamma_t = \frac{G.\gamma_w\left(\frac{G + Se}{G}\right)}{1+e}$$

$$\gamma_t = \gamma_w \frac{(G + S.e)}{1+e}$$

if soil is fully saturated $\gamma_t = \gamma_{sat}$ & $S = 1$

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

if soil is fully dry $\gamma_t = \gamma_{dry}$ & $S = 0$

$$\gamma_{dry} = \frac{\gamma_w \cdot G}{1+e}$$

if soil is fully submerged $\gamma_t = \gamma' = \gamma_{sat} - \gamma_w$

$$\gamma' = \gamma_w \frac{(G+e)}{1+e} - \gamma_w$$

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

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

$$\boxed{\gamma' = \frac{\gamma_w (G-1)}{1+e}}$$

Relation b/w γ_d, G, w & n_a :-

$$V = V_a + V_w + V_s$$

$$1 = \frac{V_a}{V} + \frac{V_w}{V} + \frac{V_s}{V}$$

$$1 = n_a + \frac{V_w}{V} + \frac{V_s}{V}$$

$$1 - n_a = \frac{V_w}{V} + \frac{V_s}{V}$$

$$\gamma_s = \frac{W_s}{V_s} \Rightarrow V_s = \frac{W_s}{\gamma_s} = \frac{W_s}{G.\gamma_w}$$

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

$$\therefore G\gamma_w = \gamma_s$$

$$\gamma_w = \frac{W_w}{V_w} \Rightarrow V_w = \frac{W_w}{\gamma_w} = \boxed{\frac{W_w}{\cancel{G}.\frac{\gamma_s}{w}}}$$

$$V_w = \frac{W_w}{\gamma_w}$$

$$= \frac{W_s}{\gamma_w} \cdot \frac{W_w}{W_s}$$

$$\therefore \frac{W_w}{W_s} = w$$

$$\boxed{V_w = \frac{W_s}{\gamma_w} \cdot w}$$

$$1-n_a = \frac{V_s}{V} + \frac{V_w}{V}$$

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

$\gamma_s = G.\gamma_w$

$$= \frac{1}{V}\left[\frac{W_s}{\gamma_s}\right] + \frac{1}{V}\left[\frac{W_s}{\gamma_w}\right].w$$

$$= \frac{1}{V}\left[\frac{W_s}{G.\gamma_w}\right] + \frac{W_s}{V}\left[\frac{w}{\gamma_w}\right]$$

$$= \frac{W_s}{V}.\frac{1}{G.\gamma_w} + \frac{W_s}{V}.\frac{w}{\gamma_w}$$

$$= \frac{\gamma_d}{G.\gamma_w} + \frac{\gamma_d.w}{\gamma_w}$$

$$1-n_a = \frac{\gamma_d}{\gamma_w}\left(\frac{1}{G} + w\right)$$

$$1-n_a = \frac{\gamma_d}{\gamma_w}\left(\frac{1+G.w}{G}\right)$$

$$\frac{\gamma_d}{\gamma_w} = \frac{G(1-n_a)}{1+w.G}$$

$$\boxed{\gamma_d = \frac{\gamma_w.G.(1-n_a)}{1+wG}}$$

if $n_a = 0$ soil becomes fully saturated at given w

$$\gamma_d = \frac{G.\gamma_w}{1+w.G}$$

Relation b/w S, w, G, γ_t, & γ_w :-

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

$$\frac{\gamma_t}{\gamma_w} = \frac{G+Se}{1+e}$$

But $S.e = w.G$

$$e = \frac{w.G}{S}$$

$$\frac{\gamma_t}{\gamma_w} = \frac{G+w.G}{1+\frac{w.G}{S}}$$

$$\frac{\gamma_t}{\gamma_w} = \frac{G(1+w)}{\left(1+\frac{w.G}{S}\right)}$$

$$\left(1+\frac{w.G}{S}\right) = G(1+w).\frac{\gamma_w}{\gamma_t}$$

$$\frac{\gamma_w}{\gamma_t}(1+w) = \frac{1}{G}\left(1+\frac{w.G}{S}\right) = \frac{1}{G}+\frac{w}{S}$$

$$\frac{1}{G}+\frac{w}{S} = \frac{\gamma_w}{\gamma_t}(1+w)$$

$$\frac{w}{S} = \frac{\gamma_w}{\gamma_t}(1+w)-\frac{1}{G}$$

$$\boxed{S = \frac{w}{\left(\frac{\gamma_w}{\gamma_t}(1+w)-\frac{1}{G}\right)}}$$

A oven dry soil sample has mass sp. gravity of 1.5g/cc if bulk density of soil in it natural state is 2g/cc then wc of soil in natural soil be.

$G = 1.5g/cc$

$\rho_d = 2g/cc$

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

$\gamma_d = G_m . \gamma_w$

$\gamma_d = 1.5 . (\gamma_w = 1)$

$\gamma_d = 1.5g/cc$

$\gamma = 2g/cc$

$\gamma_d = \frac{\gamma}{1+w} = \frac{2}{1+w} = 1.5.$

→ A soil sample has a porosity of 40%. The sp.G. of solids is 2.70. Calculate a) void ratio. b) dry density. c) unit weight if the soil is 50% saturated d) unit weight if the soil is completely saturated.

Given $n = 40\% = 0.4$, $G = 2.70$

a) $e = \frac{n}{1-n} = \frac{0.4}{1-0.4} = 0.667$

b) $\gamma_d = \frac{G.\gamma_w}{1+e} = \frac{2.7 \times 9.81}{1+0.667} = 15.89 \text{ kN/m}^3$ $\because \gamma_w = 9.81 \text{ kN/m}^3$.

c) $e = \frac{w.S}{G}$ (or) $w = \frac{e.S}{G} = \frac{0.667 \times 0.5}{2.70} = 0.124$

$\gamma_d = 15.89 \text{ kN/m}^3$

$\gamma = \gamma_d(1+w) = 15.89(1+0.124) = 17.85 \text{ kN/m}^3$

$\gamma = \frac{\gamma_w(G+e.S)}{1+e} = \frac{9.81(2.70+0.667 \times 0.5)}{1+0.667} = 17.85 \text{ kN/m}^3$

d) The soil is fully saturated $e = w_{sat} \cdot G$

$w_{sat} = \frac{e}{G} = \frac{0.667}{2.70} = 0.247$

$\gamma_{sat} = \gamma_d(1+w_{sat}) = 15.89 \times 1.247 = 19.87 \text{ kN/m}^3$

$\gamma_{sat} = G\gamma_w(1-n) + \gamma_w n = 2.7 \times 9.81(1-0.4) + 9.81 \times 0.4$

$= 15.89 + 3.92 = 19.81 \text{ kN/m}^3$.

The in-situ % voids of a sand deposit is 34%. for determining the density index, dried sand from the stratum was first filled loosely in a 1000 cm³ mould and was then vibrated to give maximum density. The loose dry mass in the mould was 1610 g & the dry dense mass at maximum compaction was found to be 1980 g. Determine the density index if the sp. gravity of the sand particle is 2.67.

Given $n = 34\% = 0.34$

$$e = \frac{n}{1-n} = \frac{0.34}{1-0.34} = \frac{0.34}{0.66} = 0.515$$

$$\gamma_d = \frac{G\gamma_w}{1+e} = \frac{2.67 \times 9.81}{1+0.515} = 17.289$$

$$(\gamma_d)_{max} = \frac{1980}{1000} \times 9.81 = 19.42\ kN/m^3 \quad (\gamma_d)_{min} = \frac{1610}{1000} \times 9.81 = 15.79\ kN/m^3$$

$$e_{min} = \frac{G\gamma_w}{\gamma_{d(max)}} - 1 \Rightarrow \frac{2.67 \times 9.81}{19.42} - 1 = 0.349$$

$$e_{max} = \frac{G.\gamma_w}{\gamma_{d\,(min)}} - 1 = \frac{2.67 \times 9.81}{15.79} - 1 = 0.659$$

$$I_D = \frac{e_{max} - e}{e_{max} - e_{min}} = \frac{0.659 - 0.515}{0.659 - 0.349} = 0.465 = 46.5\%$$

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→ The sp. gravity of soil equals. 1.64. The sp. gravity of solids is 2.70. Determine the void ratio under the assumption that the soil is perfectly dry. what would be the voids ratio, if the sample is assumed to have a water content of 8%.

When the sample is dry $G_m = \frac{\gamma_d}{\gamma_w} = 1.64$

$\gamma_d = 1.64\,\gamma_w$

$\gamma_d = 1.64 \times 9.81 = 16.09\ kN/m^3$

$e = \frac{G.\gamma_w}{\gamma_d} - 1 = \frac{2.7 \times 9.81}{16.09} - 1 = 0.646$

The sample has the water content $w = 8\%$. $G_m = \frac{\gamma}{\gamma_w} = 1.64$.

$\gamma = 1.64\,\gamma_w = 1.64 \times 9.81 = 16.09\ kN/m^3$

$\gamma_d = \frac{\gamma}{1+w} = \frac{16.09}{1+0.08} = 14.9\ kN/m^3$

$e = \frac{G.\gamma_w}{\gamma_d} - 1 = \frac{2.7 \times 9.81}{14.9} - 1 = 0.78.$

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↳ Calculate the unit weights & sp. Gravities of solids

a) a soil composed of pure quartz

b) a soil composed of 60% quartz, 25% mica & 15% iron oxide.

- Assume that both solids/soils are saturated and have void ratio of 0.63 Take the avg G for quartz = 2.66 for mica = 3.0 & iron oxide = 3.8.

a) pure quartz: $G = 2.66$

$$\gamma_{sat} = \frac{G+e}{1+e}\gamma_w = \frac{2.66+0.63}{1+0.63} \times 9.81 = 19.8 \text{ kN/m}^3$$

b) for the composite soil:

$$G_{avg} = (2.66 \times 0.6) + (3.0 \times 0.25) + (3.8 \times 0.15) = 1.60 + 0.75 + 0.57$$
$$= 2.92$$

$$\gamma_{sat} = \frac{2.92+0.63}{1+0.63} \times 9.81 = 21.36 \text{ kN/m}^3$$

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↳ one cubic meter of wet soil weights 19.80 kN if the sp. Gravity of soil particles is 2.70 & water content is 11%. find the void ratio, dry density & degree of saturation.

Bulk unit weight, = 19.80 kN/m³

Water content $w = 11\% = 0.11$

Dry unit weight, $\gamma_d = \frac{\gamma}{(1+w)} = \frac{19.80}{(1+0.11)}$ kN/m³ $= 17.84$ kN/m³

Specific Gravity of soil particle $G = 2.70$

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

unit wt of water $\gamma_w = 9.81$ kN/m³

$$17.84 = \frac{2.70 \times 9.81}{(1+e)}$$

$$(1+e) = \frac{2.70 \times 9.81}{17.84} = 1.485$$

$$1+e = 1.485$$

$$e = 1.485 - 1$$

$$e = 0.485$$

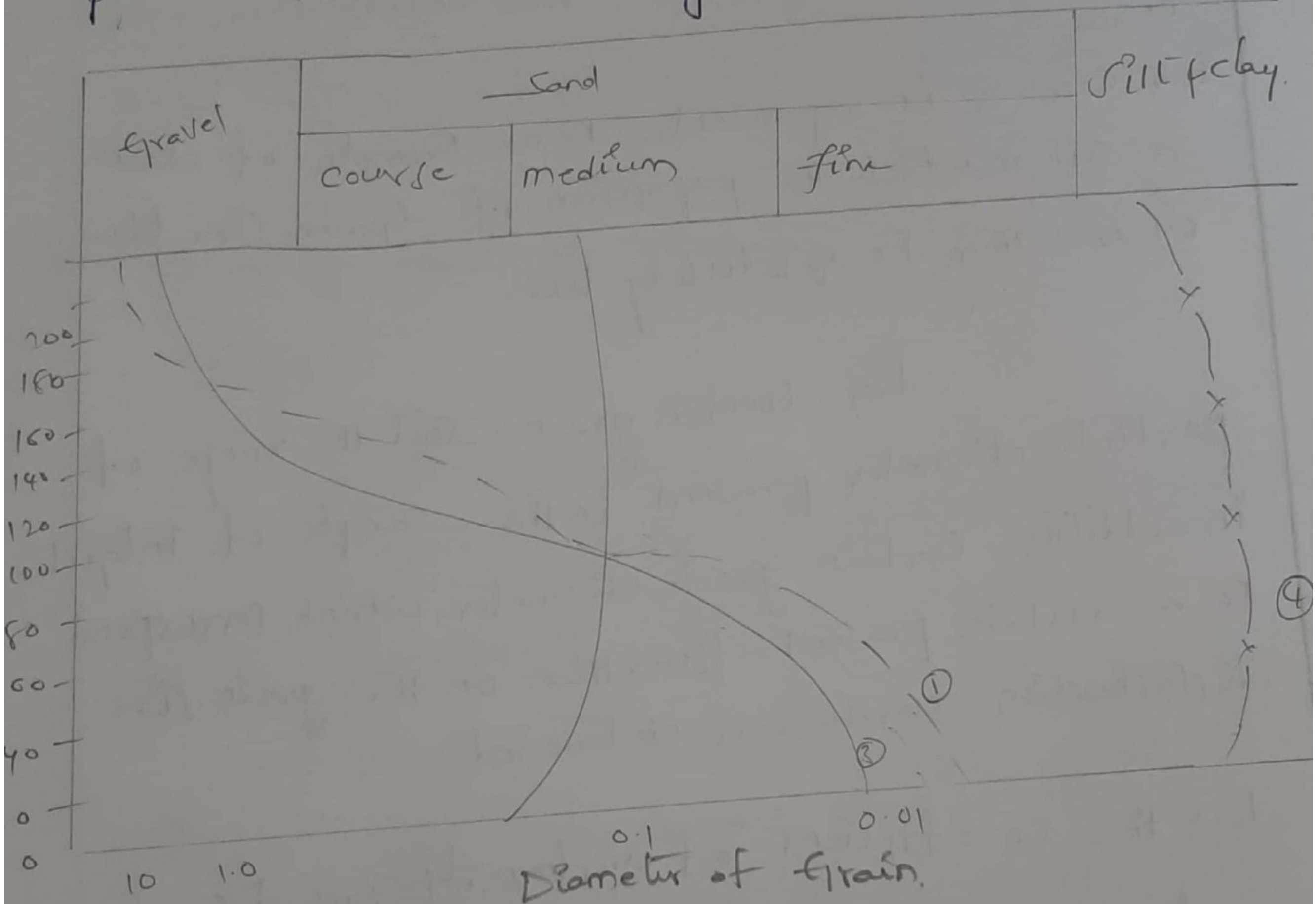
Degree of saturation $S = \frac{wG}{e}$

$$S = \frac{0.11 \times 2.70}{0.485} = 0.6124$$

Degree of Saturation = 61.24%

↳ Grain size distribustion. curves:—

↳ the result of Grain size analysis are usually represented in the form of agraph the agrigate are cumulete weight as a percentage of total weight is plotted on the ordinate using and arthametic scale.



↳ A well graded soil as agood representation of grain size over a void range and its Gradiation curve is smooth.

Curve 1:

↳ curve '1' is represented a well graded soil, with particle ranging from Gravel to fines aggrigate.

curve: 2

↳ Curve '2' is represented a poorly graded sand of uniform Gradiation.

curve: 3:

↳ Curve '3' is represented to an Example of such a soil in which the proposition of Grain size b/w 0.1 and 1mm is relatively low.

for course grain soil the range of particle diameter present in the sample of interest in addition certain grade diameter which correspond to a certain present finer then on the grain size distribution curve are determind

↳ the co-efficient of uniformity (Cu) = $\frac{D_{60}}{D_{10}}$ is a shape of parameter is defind as

$$C_u = \frac{D_{60}}{D_{10}}$$

D_{60} → Grain size diameter in mm.

Another shape parameter the co-efficient of curvature 'Cc' defined

$$C_c = \frac{D_{30}^2}{D_{10} \times D_{60}}$$

Where:

D_{30}: → Grain diameter in mm corresponding to 30% finner

C_u → Co-efficient of Curvature.

for well graded soil C_c lies b/w '1 & 3' and C_u must be grater then '4' for gravel & grater then '6' sand.

Generally Grain size diameter is classified by '2' types:

↳ 1) Sieve Analysis.
2) Hydrometer Analysis (Wet Mechanical Analysis).

Sieve Analysis:

↳ Sieves are made by waving two sets of voids at right angle to each other. Sieve sizes are given in terms of no. of openings per unit length.

According indian standard (sieve number is meshwidth in mm (or micron (μ) = 0.001 mm).

Analysis is done by sieving of dry sample of down weight through series of sieves sample place one below other in decreasing order of size.

↳ whole set is horizontally taken for 10mn till weight of soil retaind in each sieve term constant.

↳ By determining weight of soil sample a retaind on each sieve the following are made.

↳ percentage retain on sieve

= Weight of soil retaind on the sieve / weight of total soil.

Cumulative percentage retain on any sieve:

= sum of percentages retaind on all — courser sieve

percentage finer then only sieve opening =

100 – Cumulative percentage retaind

Hydrometer Analysis:-

In Hydrometer Analysis the soil sample about 40 to 60 gram of dried soil, whose particle size is less than 0.075 mm (or) 75 micron is accurate weighed and mixed with distilled water to form a paste.

The contents is transform to a cup of mechanical mixer using a jet of distilled water and then stirred well for 15 mnt then transfer the jar to make 1000 cc suspension.

It is then vigorously shaken and kept verticle on a solid base.

Stop watch is started simultaneously. The hydrometer inserted slowly and reading are taken at 0.5 m ($\frac{1}{2}$ m), 1 m, 2 m, 3 m, 4 m, 5 m, 6 m, etc.

A stable correction is applied on these reading the diameter of particle is calculated according to stokes law.

Terminal Velocity, $V_s = \frac{1}{18} d^2 \frac{(\gamma_s - \gamma_w)}{\mu}$ (or) $V_s = \frac{g}{18}[G-1]\frac{d^2}{\nu}$.

d = diameter of particle, $\gamma_s \rightarrow$ unit weight of particle

μ = dynamic viscosity of water, $\nu \rightarrow$ Kinematic viscosity of water $= \mu/\rho$

$G \rightarrow$ specific gravity of solids

Approximate Version of Stokes law

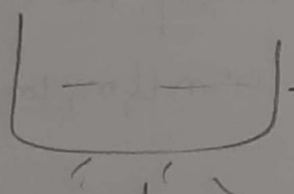
$$V_s = 900d^2$$

V = velocity in mm/sec

d = Diameter of particle in mm.

Soil:

- C.G.S
  - → Gravel
  - → Sand
- F.G.S
  - → silt
  - → clay
  - → organic soil
- Highly organic soil
  - → ~~Black~~ colour Dark
  - → Bad odour
  - Ex: peat.

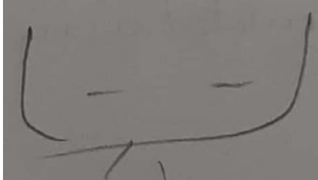


→ 75μ → if more 50% of soil retain → C.G. soil

→ " " " " passing → f.G. soil

↓

fine fractions



→ 4.75 mm sieve

→ if more 50% soil retain → Gravel

" " 50% " passing → Sand

Consistency limit
(or)
Limits of consistency (Atterberg's limit):—

↳ Mostly used for fine grained soil.

↳ Atterberg divided the entire range from liquid to solid state into four states.

1) Liquid state
2) plastic state
3) Semi solid state
4) solid state.

Atterberg limit are the water contents at which soil mass passes from one state to another state.

Liquid Limit: (W_L) :-

↳ It is the minimum water content at which the soil is still in the liquid state.

↳ But has a small shearing strength against flowing.

↳ Casagrande's apparatus is used to determine liquid-limit. The shear strength of the soil at liquid limit is 2.7kN/m

Plastic Limit: (W_P) :-

↳ It is the minimum water content at which the soil can be rolled into a thread of 3mm in diameter without crumbling

↳ It is determined by plastic limit test (or) thread test

Shrinkage Limit (W_S) :-

↳ It is the moisture content at which further loss of moisture content does not cause a decrease in the volume of soil

↳ Shrinkage limit is lowest water content at which a soil can be still completely saturated.

Liquidity Index (I_L):-

$$I_L = \frac{W - W_P}{I_P}$$

↳ The liquidity Index is also known as water plasticity ratio.

Consistency Index, or Relative consistency:-

$$I_C = \frac{W_L - W}{I_P}$$

It is useful in the study of the field behaviour of saturated fine grained soil.

Range:- It can be greater than or less than 1.

if $I_C = 1$, soil is at it's plastic limit.

$I_C = 0$, soil is at liquid limit

$I_C > 1$ soil is in semi solid state and is stiff

* Important Indices: ÷

a) plasticity Index (I_p): ÷

I_p = (Liquid Limit − plastic Limit)

$I_p = (w_L - w_p)$

↳ It is relatively more for plastic clays.

I_p (%)	plasticity
0 →	Non plastic
<7 →	Low plastic
7–17 →	Medium plastic
>17 →	Highly plastic

* Shrinkage Index (I_s): −

I_s = (plastic limit − shrinkage Limit)

$I_s = (w_p - w_s)$

* Toughness Index (I_T) ÷

$I_T = I_p / I_f$ = plasticity Index / flow Index.

permeability:-

↳ It is the property of the soil which allows passage of fluid through it.

permeable soil:-

↳ when a soil has continuous voids

→ A soil is said to be highly permeable when

$k > 10^{-1}$ cm/sec

Ex: Gravel, sand

→ soil said to be impermeable when

$k < 10^{-7}$ cm/sec.

Ex: stiff clay.

Seapage:-

→ Seapage is the flow of water under Gravitational force in a permeable medium.

Laminar flow:-

→ the flow is termed as laminar flow when all the particles of water move in parallel path without corresponding the other particle.

Turbulent flow:-

→ In there type of flow all the particle of water in move zig-zag.

Soil water:-

↳ The water which is in present in the void space of soil mass is known as soil water.

↳ The soil water may be in two types:

*→ Held water.

*→ Gravitational water.

Held water:-

↳ The water held in the void spaces of soil mass due to some forces of attraction is called as held water.

↳ It is further classified into 2 types.

*→ Adsorbed water.

*→ Structural water.

Adsorbed water:-

↳ It is a thin layer of water that surrounded the surface of clay particle due to certain physical force of attraction.

Structural water:-

↳ It is defined as the water which is combined chemically to the crystaline structure of the clay particle.

The structural water is impossible to separate even at the temperature of 110° Celsius.

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Gravitational water:-

↳ the water which moves freely through the pore space of soilmass water the influence of gravitational force is termed as gravitational water

↳ they are two types:

* → free water.
* → Capil ry water.

Free water:-

↳ the water which translate under the influence of gravitational force is called as free water.

↳ it is divided two types:

* → Surface water.
* → Ground water.

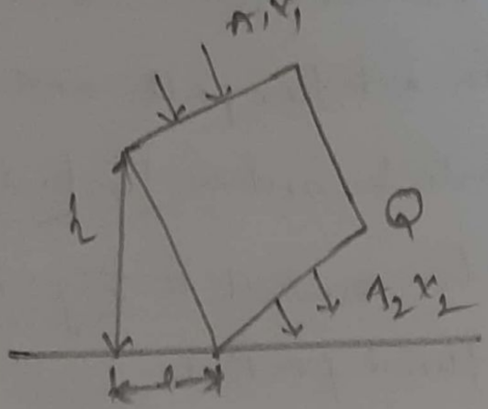
Capillary water:-

the water which is held by the force of capillarity action is called capillary water.

Capillary rise:-

A rise in a liquid above the level of zero pressure due to net upward force produced by attraction of the water molecules to a solid surface

Darcy's law:-



↳ In 1856 Darcy demonistrated expermentally. that for laminar-flow in a homogeneous soil the velocity of flow is given by

$$V = ki \Rightarrow \frac{Q}{A} = ki \Rightarrow Q = kiA$$

$$Q = kiA \quad (\because Q = AV)$$

$$(\because \frac{Q}{A} = V \quad \because \frac{Q}{V} = A)$$

Where:

k = Co-efficient of permeability

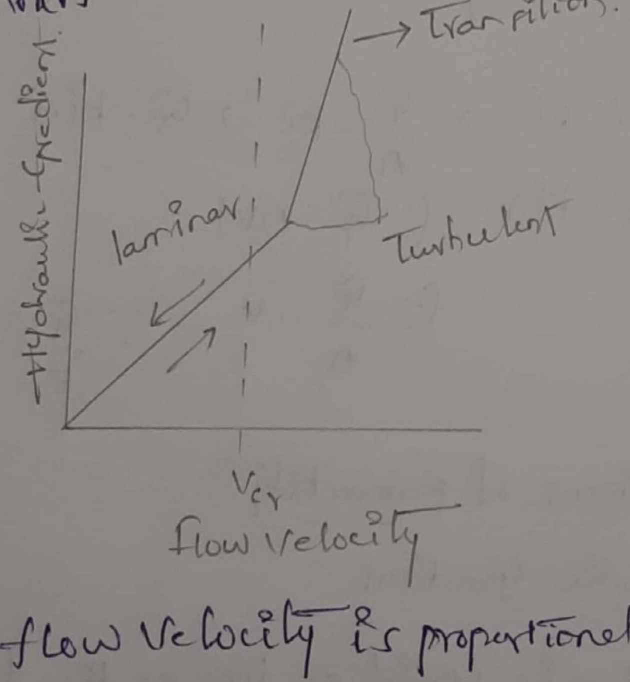
i = Hydraulic Gradient.

↳ Darcy's law is valid as long as the flow is laminer. it is applied to the soil fraction. finner than the gravel

Flow of water through Soil:-

↳ Flow of fluid is described as laminer if a fluid particle flows in a define path and doesnot loss the path of other particle. and as Turbulent where Random velocity fluctation, result in zig-zag and criss-cross party of fluid particle

↳ the fundamental low the determine weather the flow is laminer are turbulent where formulated by Reynold's



↳ the flow velocity is proportional to the hydraulic Gradient as long as the flowest Laminer.

↳ As a velocity increase a critical state is reach at which eddies begin to form nearly wall of tube and the relationship b/w flow velocity and hydraulic gradient become uncertain.

↳ A further increasing velocity again leads to definite relationship this time is represented by continuous curved line

↳ The velocity at which laminar flow changes to turbulent flow a change occurs and it is called as lower critical velocity (V_{cr}).

↳ As a below lower critical velocity V_{cr} the flow is always laminar.

↳ Reynold's formed that lower critical velocity is inversely proportional to the diameter of the tube or pipe

↳ Reynold's develop a general formula which is applicable for any fluid & it is independent of system unit.

$$\frac{V_{cr} \cdot d \cdot \gamma_w}{\eta g} = 2000.$$

d = Tube diameter.

γ_w = unit weight of water

η = viscosity of fluid

g = gravitational force

V_{cr} = Critical velocity.

Importance of permeability :-

↳ The following important of permeability are

*→ Seepage through earth dam and cannels

*→ A uplift pressure under hydraulic structure and safty again it's piping.

Factors effecting permeability:-

1, Effect of grain size:
2, Effect of shape
3, Effect of void ratio.

* Effect of grain size:-

↳ The co-efficient of permeability of soil is proportional to the square of representative particle size

$$k = CD_{10}^2$$

where C – constant varies from 0.4 to 1.2 with avg value 1.

* Effect of shape:-

↳ for same void ratio the soil with angular particle are less permeable then those with rounded particles. k is inversely proportional to specific surface ar

c, Void ratio:-

↳ To predict for a given soil the value of k at a void ratio other then the one at which it is determin taylor (1948) recommends the following relationship

$$k_1 : k_2 = \frac{C_1 e_1^3}{1+e} : \frac{C_2 e_2^3}{1+e_2}$$

C_1 C_2 are shape-factor which depend on manner of paking of grains & the shape characteristic of the pores. for sands 'C' changes only slightly with void ratio i.e. $C_1 \simeq C_2$

$$k_1 : k_2 = \frac{e_1^3}{1+e_1} : \frac{e_2^3}{1+e_2}$$

- Another relationship (some time used is

$$k_1 : k_2 = C_1 e_1^2 : C_2 e_2^2$$

- Assuming 'C' is constant for sands.

Unit weight of water: (γ_w): ÷

$$k = C D_{10}^2 \frac{\gamma_w}{\mu} \cdot \frac{e^3}{1+e}$$

$C \rightarrow$ shape-factor it depends on shape of particle

$k \rightarrow$ Co-eff. of permeability.

Temperature: ÷

$k \propto$ Temperature.

Specific surface Area ÷

$$SSA \propto \frac{1}{size} \qquad (size\downarrow\ SSA\uparrow)$$

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

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Effect of permeant & (it is viscosity of temperature):

↳ the factor (γ_w/η) depends on the kind & physical state of pore fluid.

$$K \propto \frac{\gamma_w}{\mu}$$

$$K_1 : K_2 = \frac{\gamma_{w_1}}{\eta_1} : \frac{\gamma_{w_2}}{\eta_2}$$

Since both viscosity & unit weight of permeat vary with temperature K will be affected by changes in tem. grater viscosity lower the permeability it is common practice to note the temp & of water during permeability determination and reduce. the computed permeability volume corresponding to 27°C

$$K_{27} = \frac{\mu_T}{\mu_{27}} K_T$$

$K_{27} \rightarrow$ volume of K of 27°C

$K_T \rightarrow$ " " " of T°C

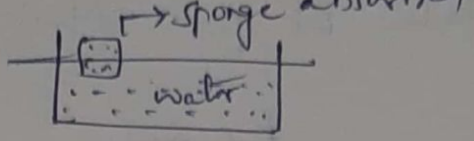
$\mu_{27} \rightarrow$ viscosity of permeant at 27°C

$\mu_T \rightarrow$ viscosity of " at T°C

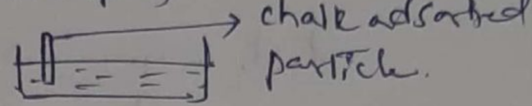
Adsorbed Water :-

$K \propto \frac{1}{\text{adsorption}}$

Absorption.



Adsorption



Permeability : 'K' :-

$$K = C D_{10}^2 \frac{\gamma_w}{\mu} \frac{e^3}{1+e}$$

$K \Rightarrow$ Co-efficient of permeability

$C \rightarrow$ shape factor. it depends on shape of particle

$D_{10} \rightarrow$ effective grain size

$\mu \rightarrow$ Viscosity of water.

$e \rightarrow$ void ratio

$\gamma \rightarrow$ unit weight of percolating water.

1) the co-efficient of permeability soil is 4×10^{-5} cm/s if the viscosity of fluid is reduced to ($\frac{1}{2}$) other other things are remaing constant, then find the co-efficient of permeability.

permeability↑ viscosity↓

$k\uparrow$ $\mu\downarrow$

Given:

$k_1 = 4\times10^{-5}$ cm/sec

$\mu_1 = \mu_1$

$k_2 = ?$

$\mu_2 = \frac{\mu_1}{2}$

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

$\frac{k_1}{k_2} = \frac{\mu_2}{\mu_1} \Rightarrow \frac{k_2}{k_1} = \frac{\mu_1}{\mu_2}$

$\frac{k_2}{k_1} = \frac{\mu_1}{\frac{\mu_1}{2}}$

$\frac{k_2}{k_1} = 2$

$k_2 = 2k_1$

$k_2 = 2\times4\times10^{-5}$

$k_2 = 8\times10^{-5}$ cm/sec

Measurement of permeability :-

↳ The co-efficient of permeability can be determined by three ways.

1. Laboratory 2, field test, 3, Empirical approach.

In the laboratory, it is possible to use either the constant head or the varible head test.

Constant head test :-

↳ It is based on the measurement of the quantity of water. That flows under a given hydraulic gradient through a soil sample of known length & cross-sectional area in given time.

↳ Constant head permeameters are specially suited to the testing of pervious coarse-grained soil.

↳ Measurable discharge is needed for the accurate determination of permeability by this method.

↳ The soil sample is contained in perspex cylinder. At the cylinder, a number of manometer connection point of are provided to enable pairs of pressure head reading to be taken water is allowable to flow through the sample from a reservior designed to keep the water level constant by over flow.

↳ The quantity of water flowing out of the soil or discharge Q during a given time 't' is collected in a vessel and weighed.

↳ The presence of entrapped air in the soil can affect the results seriously.

↳ To eliminate this possibility, firstly de-aired water is supplied to the reservoir and then vacuum is applied to the soil sample before commencing the test.

↳ The test is started by closing the value C and keeping values A and B open with value A being used to control the rate by constant level in the manometer tubes then the discharge is measured.

↳ Several such test at varying rates of flow can be performed and the avg value of 'k' determined.

$$q = kiA$$

$$k = \frac{q}{iA} = \frac{Q}{Aht} \text{ cm/s.}$$

k = Co-efficient of permeability (cm/s)

Q = discharge (cm³) collected in time t(s)

A = Cross-sectional area of sample (cm)²

h = difference in manometer level (cm)

L = distance between manometer tapping points (cm).

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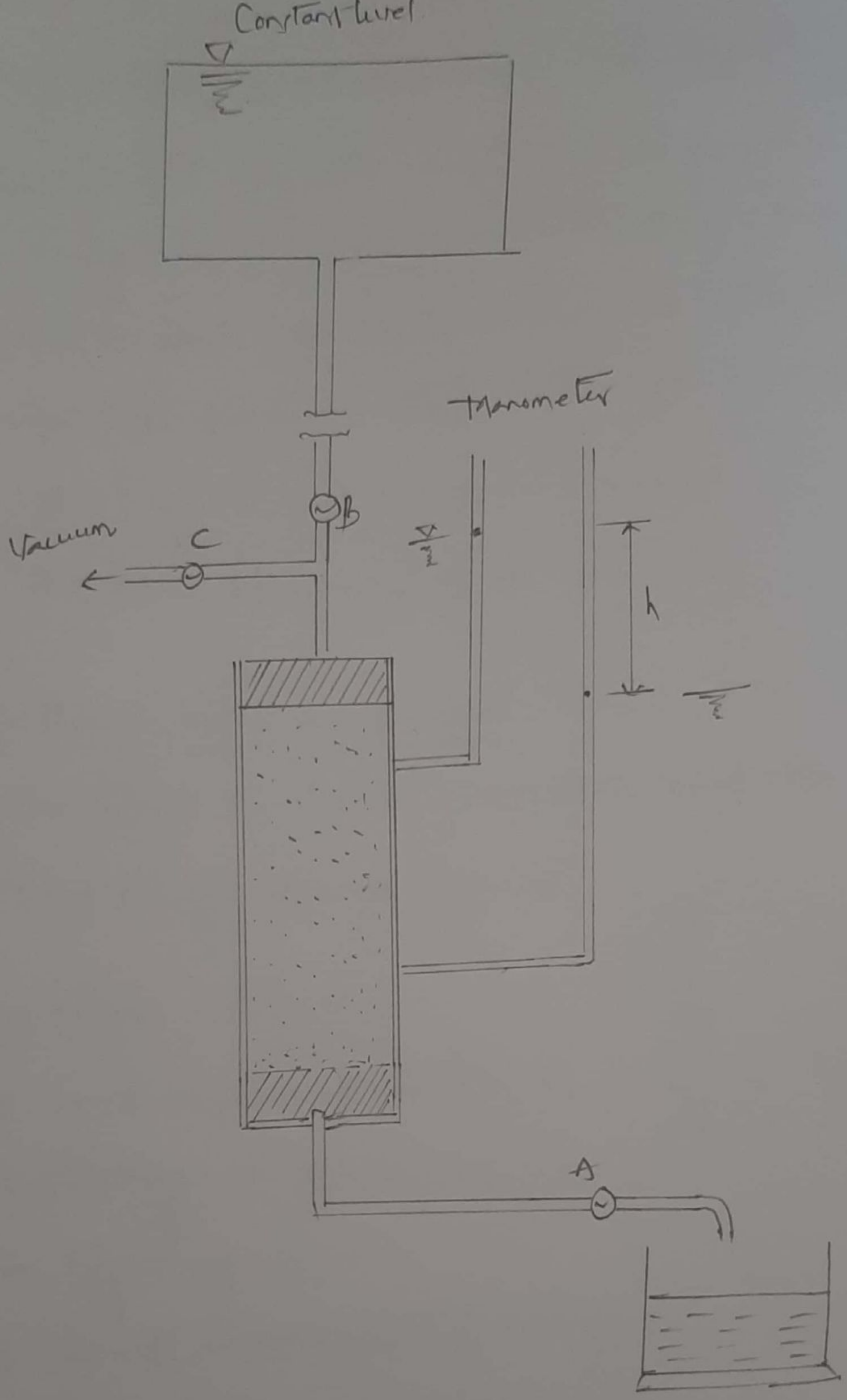


Constant head poremeter:

permeability in layered soil: ÷

↳ permeability of stratified soil deposite.

A starified soil deposite consist of of no. of soil layers having with different permeability the avg permeabity of deposite as whole parallel to the planes of stratification, and normal to the planes of stratification are presented below.

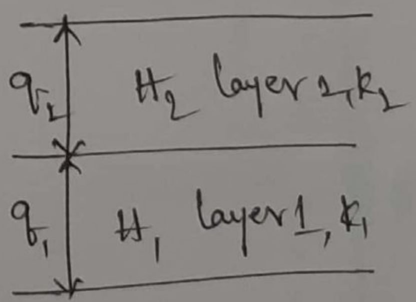
*→ flow through layers are parallel

*→ flow of a normal to the plane

flow through layer are parallel:

*→ Considerd two layers parallel to each other

let q_1 → discharge through layer 1



H_1 → height of layer 1

k_1 → Co-eff of permeability of layer 1

q_2 → discharge through layer 2

H_2 → hight of layer 2

k_2 → Co-eff of permeability of layer 2.

Total discharge $(Q) = q_1 + q_2$

we know: $Q = A \times V$

$$A \times V = A_1 V_1 + A_2 V_2$$

$$(H_1 + H_2) \times Ki = (H_1 \times 1) \times K_1 i + (H_2 \times 1) \times K_2 i$$

$$(H_1 + H_2) \times Ki = (H_1 K_1 i + H_2 K_2 i)$$

$$(H_1 + H_2) \times K\cancel{i} = (H_1 K_1 + H_2 K_2)\cancel{i}$$

$$\boxed{K = \frac{H_1 K_1 + H_2 K_2}{H_1 + H_2}}$$

Flow Normal to the plane of stratification:-

↳ Let us consider a soil deposite consisting of 2 layer of thickness. H_1 & H_2 in which flow hours Normal to the plane of stratification as shown fig.

from the fig it is $Q = Q_1 = Q_2$

$$k_v i_v = k_{v_1} \cdot i_{v_1} = k_{v_2} \cdot i_{v_2}$$

$$k_{v_1} = \left[\frac{i_v}{i_{v_1}}\right] k_v$$

$$k_{v_2} = \left[\frac{i_v}{i_{v_2}}\right] k_v$$

$$i_{v_1} = \left[\frac{k_v}{k_{v_1}}\right] i_v$$

$$i_{v_2} = \left[\frac{k_v}{k_{v_2}}\right] i_v$$

Total head $(h) = h_1 + h_2$

$$i_v H = i_{v_1} H_1 + i_{v_2} H_2$$

$$i_v H = \left[\left[\frac{k_v}{k_{v_1}}\right] H_1 + \left[\frac{k_v}{k_{v_2}}\right] H_2\right] i_v$$

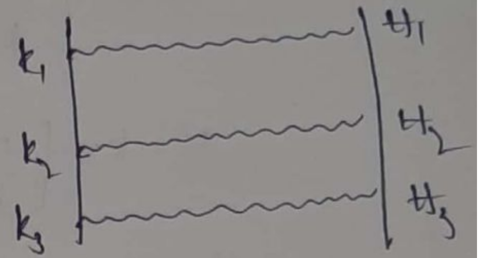
$$H = k_v \left[\frac{H_1}{k_{v_1}} + \frac{H_2}{k_{v_2}}\right]$$

$$k_v = \frac{H}{\left[\frac{H_1}{k_{v_1}} + \frac{H_2}{k_{v_2}}\right]}$$

$$k_v = \frac{H_1 + H_2}{\left[\frac{H_1}{k_{v_1}} + \frac{H_2}{k_{v_2}}\right]}$$

A stratum of soil consists of three layer of equal thickness. The permeability of top and bottom layer are 1×10^{-4} cm/sec and that of middle layer is 1×10^{-3} cm/sec then the value of horizontal co-eff of permeability for the entire soil layer in cm/sec is:

Given data:



The layer of thickness is same

i.e, $H_1 = H_2 = H_3 = H$

permeability of top & bottom layers:

$$k_1 = k_3 = 1\times10^{-4} \text{ cm/sec}$$

middle layer $k_2 = 1\times10^{-3}$ cm/sec

$$k_H = \frac{k_1H_1 + k_2H_2 + k_3H_3}{H_1 + H_2 + H_3}$$

$$= \frac{\cancel{H}(k_1 + k_2 + k_3)}{3\cancel{H}}$$

$$= k_1 + k_2 + k_3 / 3$$

$$= \frac{1\times10^{-4} + 1\times10^{-3} + 1\times10^{-4}}{3}$$

$k_H = 4\times10^{-4}$ cm/sec.

$$k_v = \frac{H_1 + H_2 + H_3}{\left[\frac{H_1}{kv_1} + \frac{H_2}{kv_2} + \frac{H_3}{kv_3}\right]}$$

$$= \frac{200 + 500 + 800}{\left[\frac{200}{5\times10^{-4}} + \frac{500}{2\times10^{-2}} + \frac{800}{3\times10^{-3}}\right]}$$

$$= \frac{1500}{\left[\frac{200}{5\times10^{-4}} + \frac{500}{2\times10^{-2}} + \frac{800}{3\times10^{-3}}\right]}$$

$$k_v = 1.90\times10^{-3} \text{ cm/sec}$$

$$\frac{k_H}{k_v} = \frac{0.11}{1.90\times10^{-3}}$$

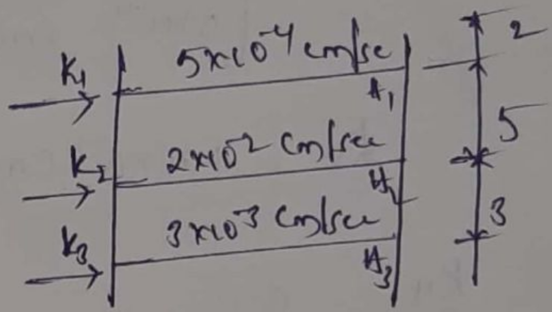
$$\frac{k_H}{k_v} = 57.89$$

The soil sample having permeability of 5×10^{-4} cm/sec for 1st 2m, 2×10^{-2} cm/sec for 2nd 5m, & 3×10^{-3} cm/sec for 3rd last 3m and the hydraulic gradient 0.3 in both Horizontal & vertical permeability then find.

a, find the ratio of $K_{H_2} : K_V$

b, find the discharge of the value & discharge velocity for each layer of Horizontal flow.

↳ Given data



$K_1 = 5\times10^{-4}$ cm/sec

$K_2 = 2\times10^{-2}$ cm/sec

$K_3 = 3\times10^{-3}$ cm/sec

$H_1 = 2m$

$H_2 = 5m$

$H_3 = 3m$

$i = 0.3$

$$K_H = \frac{K_1 H_1 + K_2 H_2 + K_3 H_3}{H_1 + H_2 + H_3}$$

$$= \frac{5\times10^{-4}\times(2\times100) + 2\times10^{-2}\times(5\times100) + 3\times10^{-3}(3\times100)}{200+500+300}$$

$K_H = 0.011$ cm/sec $\Rightarrow 1.1\times10^{-2}$ cm/sec.

Specific yield (S_y) :-

↳ Specific yield of an unconfined aquifer is the ratio of volume of water which will flow under saturated condition due to gravity effect to the total volume of aquifer (V)

$$S_y = \frac{V_{wy}}{V}$$

V_{wy} = Volume of water yielded under gravity effect
V = Total volume of water.

Specific retention :-

↳ The specific retention of an unconfined aquifer is the ratio of volume of water retained against gravity effect to the total volume of aquifer (V)

$$S_R = \frac{V_{wR}}{V}$$

V_{wR} = Volume of water retained under gravity effect.

$$\boxed{S_y + S_r = n} \rightarrow \text{porosity}$$

Kozney - Karman Equation:-

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

$C \rightarrow$ Shape co-efficient ~5mm for spherical particle

$S \rightarrow$ Specific surface area = $\frac{\text{Area}}{\text{Volume}}$

Allen Hazen Equation:

$$\boxed{K = C \cdot D_{10}^2}$$

D_{10} = effective size in cm.

K. is in cm/s.

$$\boxed{C = 100 \text{ to } 150.}$$

Values of permeability:-

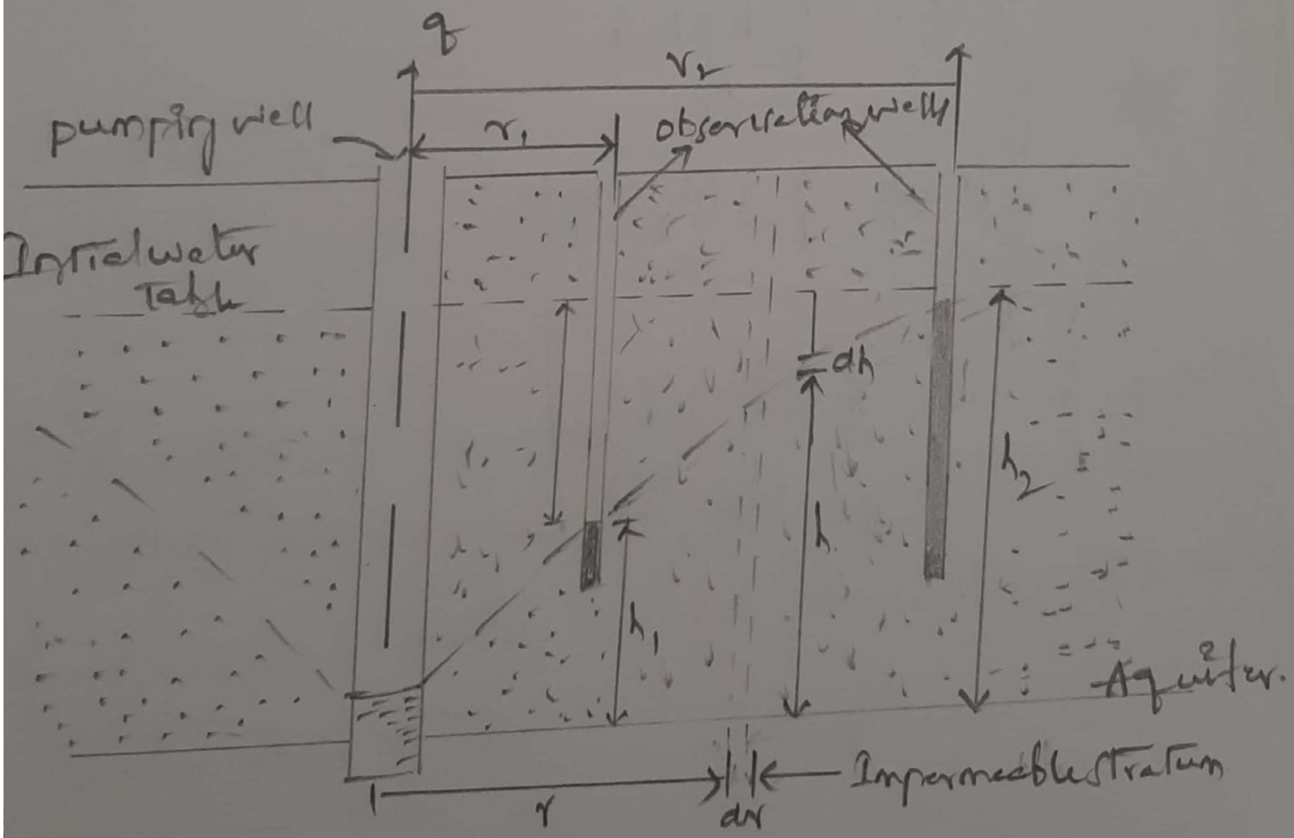
Soil.	K (cm/sec).	Degree of permeability:-
1. Course gravel.	>1	High
2. Fine gravel, fine sand	1 to 10^{-2}	Medium.
3. Silt-Sand admixture, loose silts, rock flour, and loose	10^{-2} to 10^{-4}	Low
4. Dense silt. clay silt admixture, non homogeneous. clay	10^{-4} to 10^{-5}	Very low
5. Homogenous clay	$<10^{-6}$	Impervious.

Aquifer:-

↳ A aquifer is an underground layer of water beyand the permeability rock. they are two. types of aquifer

1) un confined Aquifer.
2) Confined Aquifer.

un confined Aquifer:-



Consider the flow through an Elementary cylinder of soil having radius r, thickness dr, & height h, Hydraulic gradient.

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

Area of flow, $A = 2\pi r h$

from Darcy's Law $= q = kiA$

$$= k\frac{dh}{dr} \cdot 2\pi r h$$

$$\frac{dr}{r} = 2 \cdot \frac{\pi}{q} \cdot k \cdot h \cdot dh$$

Integrating on both side.

$$\int_{r_1}^{r_2} \frac{dr}{r} = \frac{2\pi}{q} \cdot k \int_{h_1}^{h_2} h \cdot dh$$

$$\log_e \frac{r_2}{r_1} = \frac{\pi}{q} \cdot k (h_2^2 - h_1^2)$$

$$\boxed{k = \frac{2.3 \frac{q}{\pi} \log_{10}\left(\frac{r_2}{r_1}\right)}{(h_2^2 - h_1^2)}}$$

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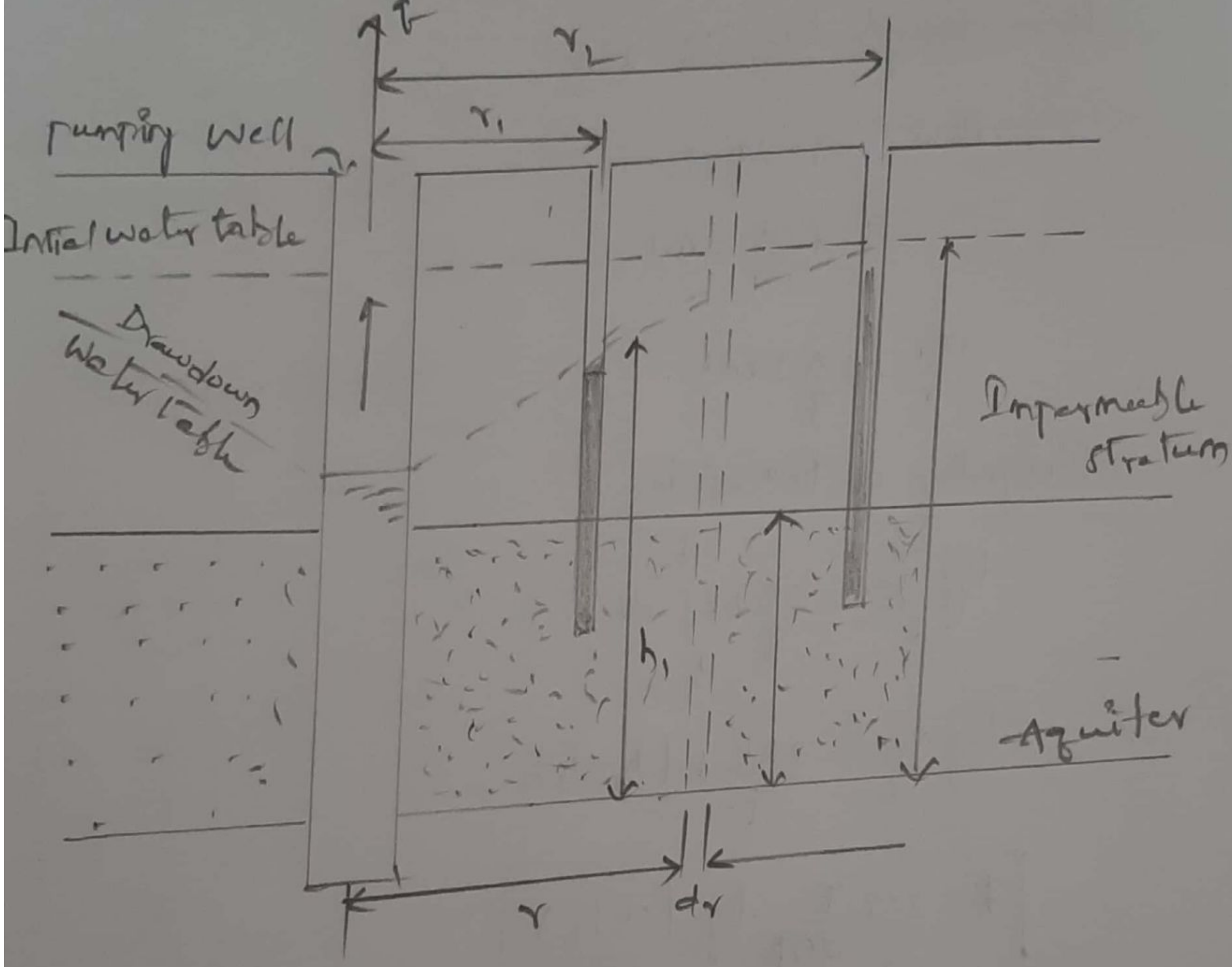
Confined Aquifer :-



↳ A confined flow condition occurs when the aquifer is confined both and above & below by impermeable strata.

↳ the drawdown surface is for all values of r, above the upper surface of the aquifer

↳ Consider the flow after steady state is obtained (in some cases pumping at a steady rate must be continued for many days before a steady flow is reached.

from darcy's law, $q = kiA$.

Here, $A = 2\pi r D$

$$q = k\frac{dh}{dr} \cdot 2\pi r D$$

$$\frac{dr}{r} = \frac{2\pi}{q} D k\, dh$$

Integrating on both side.

$$\log \int_{r_1}^{r_2} \frac{dr}{r} = \frac{2\pi}{q} D k \int_{h_1}^{h_2} dh$$

$$\log \frac{r_2}{r_1} = \frac{2\pi D}{q} k \cdot (h_2 - h_1)$$

$$\boxed{k = 2 \cdot 3 \frac{q}{2\pi D} \cdot \log_{10} \frac{(r_2/r_1)}{(h_2 - h_1)}}$$

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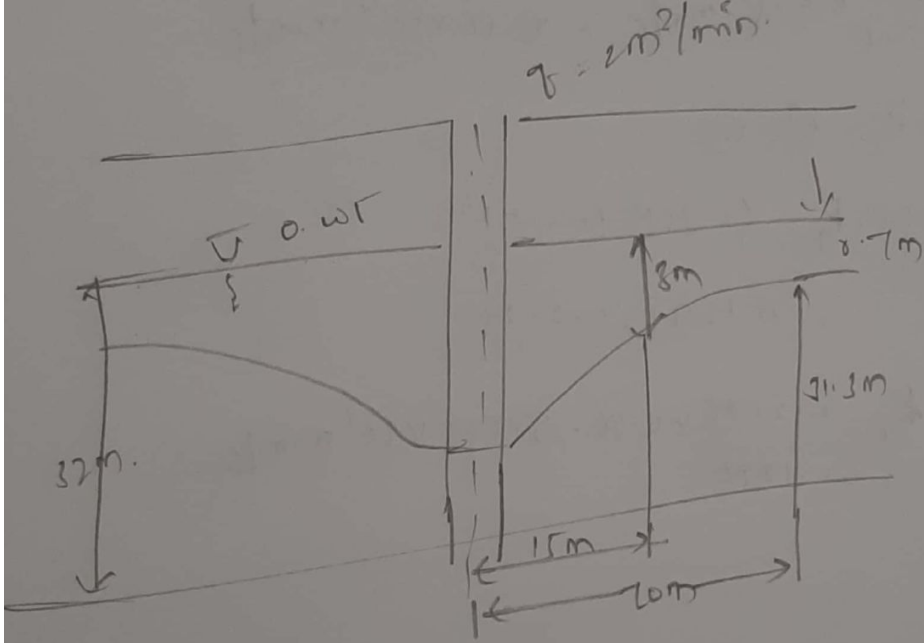
An un confind aquifer is known to be 32 m thick below the water table. A constant discharge of 2 cubic meter per minute is pumped out of the aquifer through a tubewell. till the water level. in the tube well becomes steady. two observation wells at distance of 15 m & 20 m, from the tube well shows falls of 3 m, & 0.7 m resp. from there static water levels. find the permeability of the aquifer.



$$K = \frac{q \cdot \log_e \cdot (r_2/r_1)}{\pi (z_2^2 - z_1^2)}$$

$$= \frac{2.303 \times 2 \log_{10} (20/15) \times 100}{60 \times \pi (31.3^2 - 29^2)} \quad \text{cm/sec}$$

$$= 1.15 \times 10^{-1} \text{ mm/sec}$$

A cohesionless soil has a permeability of 0.036 cm per sec. at a void ratio of 0.36. make prediction of the permeability of this soil, when it void ratio of 0.45 according to the two function of void ratio, that are proposed.

$$K_1 : K_2 = \frac{e_1^3}{(1+e_1)} : \frac{e_2^3}{(1+e_2)}$$

$$0.036 : K_2 = \frac{(0.36)^3}{1.36} : \frac{(0.45)^3}{1.45} = 0.546 : 1$$

$$K_2 = \frac{1}{0.546} \times 0.36 \text{ mm/s} = 6.60 \times 10^{-4} \text{ mm/s}$$

$$K_1 : K_2 = e_1^2 : e_2^2$$

$$0.036 : K_2 = (0.36)^2 : (0.45)^2$$

$$= 0.1296 : 0.2025$$

$$K_2 = \frac{0.2025}{0.1296} \times 0.36 = 5.625 \times 10^{-1} \text{ mm/sec}$$

The Discharge of water collected from a constant head permeameter in a period of 15min, is 500ml. The internal diameter of the permeameter is 5cm & the measured diff head b/w two gauging points 15cm vertically apart is 40 cm. Calculate the co-efficient of permeability.

If the dry weight of the 15cm long sample 4.86N & sp. gravity of the solids is 2.65, Calculate the seepage velocity.

$Q = 500ml$. $t = 15 \times 60 = 900 sec$

$A = \frac{\pi}{4} \times d^2 = \frac{\pi}{4} \times 5^2 = 6.25\pi \ cm^2$. $L = 15cm$, $h = 40cm$.

$K = \frac{QL}{Ath} = \frac{500 \times 15}{6.25 \times \pi \times 900 \times 40} \ cm/s = 0.106 \ mm/s.$

Superficial velocity: $v = Q/At = \frac{500}{900 \times 6.25\pi} \ cm/sec$

$= 0.0283 \ cm/s.$

$= 0.283 \ mm/s.$

Dry weight of sample: 4.86N

Volume of sample: $A \cdot L = 6.25 \times \pi \times 15 = 294.52 \ cm^3$

Dry density $= \gamma_d = \frac{4.86}{294.52} \ N/cm^3 = 16.5 \ kN/m^3$

$\gamma_d = \frac{G\gamma_w}{(1+e)} \Rightarrow 1+e = \frac{2.65 \times 10}{16.5} = 1.606$, $\because \gamma_w \simeq 10 \ kN/m^3$

$e = 0.606$

$n = \frac{e}{1+e} = 0.3773 = 37.3\%$. $\therefore$ Seepage velocity $= v_s = \frac{v}{n}$

$= 0.283/0.377 = 0.750 mm$

Determine the Co-efficient of permeability from the following data.

Length of Sand Sample : 25 cm

Area of the Cl/s of Sample : 30 cm²

Head of water : 40 cm

Discharge : 200 ml in 110 s.

$L = 25\ cm$

$A = 30\ cm^2$

$h = 40\ cm$ (assumed constant).

$Q = 200\ ml \quad t = 110\ sec$

$q = \frac{Q}{t} = \frac{200}{110}\ ml/sec = \frac{20}{11} = 1.82\ cm^3/sec$

$i = \frac{h}{L} = \frac{40}{25} = 8/5 = 1.60$

$q = kiA$

$K = \frac{q}{iA} = \frac{20}{11 \times 1.6 \times 30}\ cm/s.$

$= 0.03788\ cm/sec$

$= 3.788 \times 10^{-1}\ mm/s.$

In falling head permeability test, head causing flow, was initially 50 cm, & it drops 2 cm in 5 min. How much time require for the head to fall to 25 cm

falling head permeability:

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

Designating $2.303 \frac{aL}{At}$ as a constant c

$K = c \cdot \frac{1}{t} \cdot \log_{10}\left(\frac{h_1}{h_2}\right)$.

$h_1 = 50\,cm$, $h_2 = 48$, $t = 300\,s$.

$\frac{K}{c} = \frac{1}{300} \cdot \log_{10}\left(\frac{50}{48}\right)$

$h_1 = 50$. $h_2 = 25$.

$\frac{1}{300} \log_{10}\left(\frac{50}{48}\right) = \left(\frac{1}{t}\right) \cdot \log_{10}\left(\frac{50}{25}\right)$

$t = 300 \cdot \frac{\log_{10} 2}{\log_{10}\left(\frac{25}{24}\right)} = 5093.555 = 84.9\,m.$

A sample in a Variable head permeameter is 8cm. in dia & 10cm high. The permeability of the sample is estimated to be 10×10^{-4} cm/sec. If it is desired that the head in the stand pipe should from fall 24cm to 12cm in 3min. determine the size of the stand pipe which should be used.

Variable head permeameter:

Soil sample dia = 8cm.

height (length) = 10cm.

permeability = 10×10^{-4} cm/sec

$h_1 = 24$ cm, $h_2 = 12$ cm, $t = 180$ s.

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

$$10^{-3} = \frac{2.303\times a\times10}{\pi\times16\times180}\cdot\log_{10}(24/12)$$

$$a = \frac{\pi\times16\times180}{2.303\times10^{4}(\log_{10}2)}\ cm^2 = 1.305\ cm^2.$$

If the dia. of the stand pipe 'd' cm.

$$a = (\pi/4)d^2$$

$$d = \sqrt{\frac{4\times1.305}{\pi}} = 1.29\ cm.$$

Effective Stress:

↳ Generally the soils are subjected to 3 types

* → Total stress (or) Geostatic stress
* → Neutral (or) pore water pressure
* → Effective stress (or) Intergranular pressure

Total stress or Geostatic stress: (σ):

↳ Total stress developed by soil due to

* Self weight of soil (solid + water)
* Over burden load [Surcharge (q)]

Self weight of soil:

$$\sigma = \frac{\text{Self weight}}{\text{Area}} \quad \text{— (1)}$$

We know. $\gamma = \frac{\text{wt}}{\text{Volume}}$

$$\text{wt} = \gamma \cdot \text{Volume} \quad \text{— (2)}$$

(2) Sub in (1)

$$\sigma = \frac{\gamma \cdot \text{Volume}}{\text{Area}}$$

but Volume = Area × h

$$\sigma = \frac{\gamma \cdot \text{Area} \times h}{\text{Area}}$$

$$\boxed{\sigma = \gamma \cdot h}$$

σ = Total stress

h = height (or) depth of soil

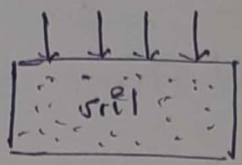
γ = unit weight of soil

l = length

b = bredth.

2) Over burden load:-

$$\boxed{\sigma = \gamma h + q}$$



2. Neutral or pore water pressure:(u)):-

↳ Neutral stress is developed due to only water prasent in the voids of the soil

Neutral stress (u) = $\dfrac{\text{weight of water}}{\text{Area}}$ — (1)

unit weight of water $(\gamma_w) = \dfrac{W_w}{\text{Volume}}$

$(\gamma_w) = \dfrac{\text{weight of water}}{\text{Volume}}$

weight of water (W_w) = Volume $\times \gamma_w$ — (2)

then (2) sub (1)

$u = \dfrac{\gamma_w \cdot \text{volume}}{\text{Area}}$

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$$u = \frac{\gamma_w \times \cancel{Area} \times h}{\cancel{Area}}$$

$$\boxed{u = \gamma_w \times h}$$

$\gamma_w \rightarrow$ unit weight of water

$u \rightarrow$ Neutral or pore water pressure

$h \rightarrow$ pressure head.

Effective stress or Intergranular pressure (σ') :-

↳ Effective stress developed due to particle to particle through them point of contact. which tends to decrease void ratio ↓ increase in strength ↑

increase in density ↑

decrease permeability ↓

$$\sigma' = \sigma - u$$

σ' = effective stress

σ = total stress

u = Neutral stress.

$\sigma' = (\gamma h - \gamma_w h)$

$\sigma = \gamma h$

$u = \gamma_w \cdot h$

$$\sigma' = \gamma h - \gamma_w h$$

$$\boxed{\sigma' = (\gamma - \gamma_w) h}$$

→ It is Equal to the total verticle reaction force transimitted at the point of contact of soil grains dived by the total area, including that occupied by water.

↳ In other words it is the pressure transimitted from particle to particle through their points of contact through soil mass.

↳ It has No physical meaning and it can not be directly measured.

↳ In can only be computed knowing the σ & u

↳ It is also called as Intergranular pressure

↳ It is not Equal to the actual 'contact stress' it much smaller then the actual 'contact stress'

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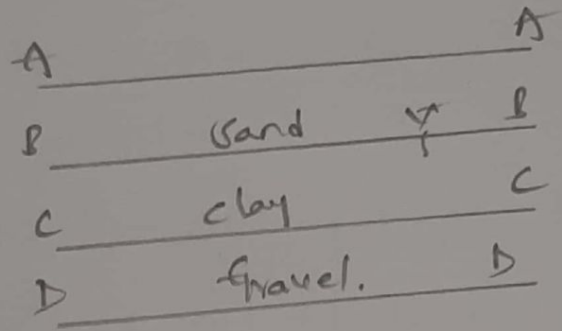
A soil profile consists of a surface layer of sand 4m thick ($\gamma = 1.6\ t/m^3$), an intermediate layer of clay 3.5m thick ($\gamma = 1.9\ t/m^3$) & the bottom layer of the gravel 4m thick ($\gamma = 1.925\ t/m^3$). The water table is the upper surface of the clay layer. Determine the effective stress at plane DD.



$\sigma' = \sigma - u$

At D-D plane

$\sigma' = \sigma - u$

$\sigma' = (1.6 \times 4) + (1.9 \times 3.5) + (1.925 \times 4) - (7.5 \times 1)$

$\sigma' = 13.25\ t/m^2$

* The water which is held in the soil against the gravity is divided into:

a) Structural water
b) Adsorbed water
c) Capillary water.

Structural water:-

↳ it is chemically combined in the crystalline structure of the soil.

↳ Cannot be removed by simple oven drying at 105°c

↳ it is integral part of the soil.

Adsorbed water:-

↳ water held by electrochemical forces existing on the soil surface.

↳ it is important only for clays.

↳ for course grained soil the adsorbed water is negligible or zero.

↳ Also called 'hydroscopic water' (the amount of water in an air dried soil).

↳ Can be removed by oven drying.

Capillary water:-

Capillary rise. $h_c = \frac{4\sigma_s \cos\alpha}{\gamma_w . d}$

σ_s = Surface tension of water

α = contact angle

d = diameter of tube.

↳ In the soil capillary height, $h_c = 0.3/d$

h_c = Capillary rise in cm.

d = dia of void in 'cm'

Note: $h_c \propto \frac{1}{d}$.

Seepage pressure:

↳ Total head = pressure head + velocity head + datum head:

↳ The velocity head in soil is neglected.

Total head = pressure head + datum head

$$\boxed{H = h_w \pm z}$$

* Hydraulic Gradient:-

↳ it is the loss of head per unit seepage distance

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

* Seepage pressure (P_s):-

↳ The pressure exerted by the water on the soil through which it percolates.

$$\text{Seepage pressure } P_s = \pm \gamma_w . h = i . z . \gamma_w$$

(+ → Downward flow; − → upward flow)

h = net head causing flow

i = hydraulic gradient.

z = seepage length.

* Seepage force per unit volume $i . \gamma_w$:

i) Seepage pressure always act in the direction of flow

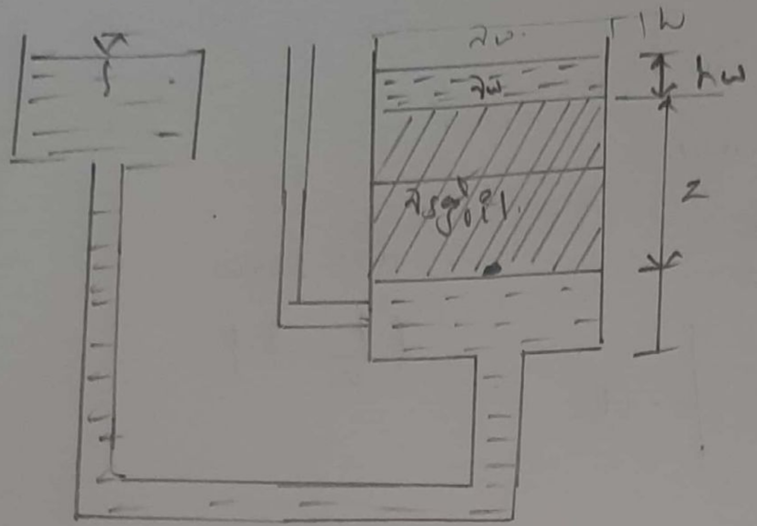
ii) Due to seepage pressure vertical effective pressure may be increased or decreased based on the direction of the flow

iii) Effective stress increase if flow is down ward direction.

iv) Effective stress decrease if flow is upwards

Derivation of critical Hydraulic gradient :-

↳ it is denoted by 'i_c'



↳ where the critical hydraulic gradient occurs when effective stress is equal to zero.

Total stress at point (A): ($\sigma = \gamma \cdot h$)

$\sigma = \gamma_{sat} \cdot z + \gamma_w \cdot h_w$

Neutral stress (u) = ($\gamma_w \cdot h$)

$u = \gamma_w \cdot z + \gamma_w \cdot h_w + \gamma_w \cdot h$

Effective stress ($\sigma' = (\sigma - u)$)

$\sigma' = \gamma_{sat} \cdot z + \cancel{\gamma_w \cdot h_w} - (\gamma_w \cdot z + \cancel{\gamma_w \cdot h_w} + \gamma_w \cdot h)$

$\sigma' = \gamma_{sat} \cdot z - \gamma_w \cdot z - \gamma_w \cdot h$

$\sigma' = (\gamma_{sat} - \gamma_w) z - \gamma_w \cdot h$

$$\boxed{\sigma' = \gamma' \cdot z - \gamma_w \cdot h}$$

Critical Hydraulic gradient occurs when ($\bar{\sigma} = 0$)

$\bar{\sigma} = \gamma' \cdot z - \gamma_w \cdot h$

$\bar{\sigma} = 0.$

$\gamma' \cdot z - \gamma_w \cdot h = 0$

$\gamma' \cdot z = \gamma_w \cdot h$

$\frac{\gamma'}{\gamma_w} = \frac{h}{z}$

we know

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

$$\boxed{i_c = \frac{\gamma'}{\gamma_w}} \quad — (1)$$

$\because \gamma' = (\gamma_{sat} - \gamma_w)$

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

$\gamma_{sat} = \gamma_w \left[\frac{G + e - S_r}{1+e}\right]$

$i_c = \frac{\gamma_{sat} - \gamma_w}{\gamma_w}$

$S_r = 1$

$i_c = \frac{\gamma_{sat}}{\gamma_w} - 1$

$\gamma_{sat} = \gamma_w \left[\frac{G+e}{1+e}\right]$

$S_r = 0.$

$\gamma_{dry} = \gamma_w \left(\frac{G}{1+e}\right].$

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

$i_c = \frac{\cancel{\gamma_w} \cdot \frac{(G+e)}{1+e}}{\cancel{\gamma_w}} - 1$

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

$\frac{G + \cancel{e} - (1 + \cancel{e})}{1+e}$

$i_c = \frac{G-1}{1+e} \quad — (2)$

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

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

We know that

$$n = \frac{e}{1+e} \Rightarrow n.(1+e) = e \Rightarrow n + ne = e \Rightarrow n = (e - ne)$$

$$\Rightarrow n = e(1-n) \Rightarrow \frac{n}{e} = (1-n).$$

$$= \frac{\frac{e}{1+e}}{\frac{e}{1}} \Rightarrow \frac{\cancel{e}}{1+e} \times \frac{1}{\cancel{e}} \Rightarrow \frac{1}{1+e}$$

$$\therefore 1-n = 1 - \frac{e}{1+e}$$

$$= \frac{1+\cancel{e}-\cancel{e}}{1+e}$$

$$\Rightarrow (1-n) = \frac{1}{1+e}.$$

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

$$i_c = (G-1) \times (1-n) \text{ ——— } (3)$$

from (1) & (2) & (3).

$$\boxed{i_c = \frac{\gamma'}{\gamma_w} = \frac{G-1}{1+e} = (G-1)(1-n)}$$

A coarse soil having voidratio: 0.7 Sp. Gravity 2.7 then find the critical Hydraulic gradient.

Given:

$e = 0.7$

$G = 2.7$

$$i_c = \frac{G-1}{1+e} = \frac{2.7-1}{1+0.7} = 1.$$

$$\boxed{i_c = 1}$$

Quick Sand Condition or Boiling Condition or Hydraulic Condition: ÷

↳ The upward seepage pressure become equal to submerged weight of soil, the effective pressure reduces to zero. This case sand looses all its shear strength soil particle move in upwardirection. This phenomenon is called Quick sand condition

↳ Quick sand condition is not a sand but flow condition occuring in cohesionless soil.

↳ The cohesive soil does not develop quick sand condition.

↳ it possesses some shear strength equal to its cohesive strength even when the effective stress is zero.

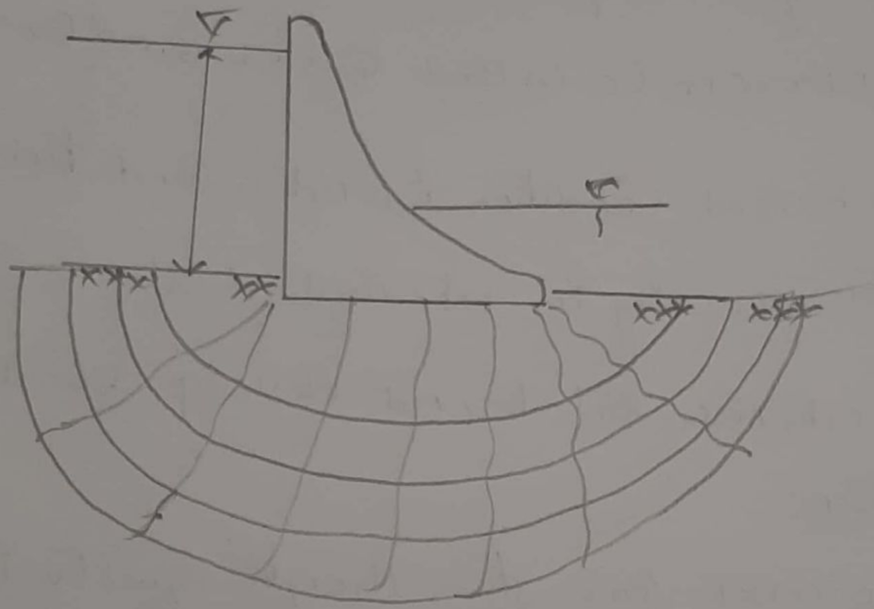
↳ The Quick sand condition is most likely to arise in silts and fine sand flow Net: ÷

↳ it combination of flow line and Equipotential lines

flow line: ÷

↳ it is a path along which a water particle travels

it is also called as stream line



Equipotential line: ÷

↳ it is a line joining points having equal total head

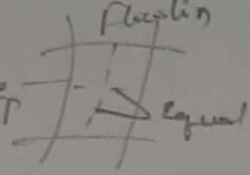
↳ if piezometer are inserted into the soil at different point along an equipotential line water would be rise to the same elevation in all these piezometer.

Note: Along the Equipotential line the total head is constant. while the pressure heads different. elevation heads are different.

Flow path:-

↳ The space between two adjacent flow lines

Also called as flow channel.



Field:

↳ The space b/w any two adjacent flow lines & equipotential lines.

Characteristic of flownet:-

↳ Flow lines & equipotential lines are orthogonal to each other. (i.e. ⊥er).

↳ The quantity of seepage in each flow channel is same

↳ Drop in head b/w adjacent equipotential lines is the same

↳ Two flow lines ~~or~~ two equipotential lines are can never meet or cross each other.

↳ Fields are kept approximately squre

↳ Flownet does not depend on permeability of the soil(k). & head causing flow.(H).

↳ Depends on boundary condition only.

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Uses of flownet:-

a) Seepage loss b) Seepage pressure

c) uplift pressure. d) Exit gradient.

Seepage Quantity:-

$$q = k \cdot H \left[\frac{N_f}{N_d}\right]$$

$k \rightarrow$ permeability of soil.

for an isotropic soil. $k = \sqrt{k_x \cdot k_y}$.

k_x = permeability in Horizontal direction

k_y = permeability in vertical direction.

H = net head causing flow (d/f b/w U/S & D/S.
water level),

N_f = No. of flow channel.

N_d = No. of potential drops.

↳ The ratio of N_f/N_d is called 'shape factor
of a flow net.

↳ if a flownet there are '4' flow channels & 15 equipotential graph, then the estimate the Quantity of seepage if head loss is 3m & $k = 2\times10^{5}$ m/sec

$N_f = 4$

$N_d = 15$

$k = 2\times10^{-5}$ m/s

$H = 3m$

$Q = k.H\left(\frac{N_f}{N_d}\right) = 2\times10^{-5}\times3\left(\frac{4}{15}\right)$

$Q = 1.6\times10^{-5}$ m³/sec

→ the flownet to determine the seepage through the soil which is an isotropic given 4 flow channels 16 equipotential graphs the co-efficient of permeability are 4×10^{-7} m/sec & 1×10^{-7} m/sec Horizontal & Vertical direction are. the find seepage of dam. help the storage head is 20m

$N_f = 4$

$N_d = 15$

$k_H = 4\times10^{-9}$ m/s.

$k_V = 1\times10^{-7}$ m/sec

$H = 20m$

$$K = \sqrt{K_h . k_v}$$

$$= \sqrt{4 \times 10^{-7} \times 1 \times 10^{-7}}$$

$$K = 2 \times 10^{-7} m/sec$$

$$Q = K . H \left(\frac{N_f}{N_d}\right)$$

$$= 2 \times 10^{-7} \times 20 \times \left(4/5\right)$$

$$Q = 1.067 \times 10^{-6} m^3/sec$$

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UNIT – III.

Shear distrubustion in soils: & Compaction: –

Compaction: –

↳ it is compression of soil mass by mechanical means to improve Engineering properties.

↳ it is due to escape & compression of air present in the soil mass.

↳ Volume reduction occurs due to escape of air under short term loading under constant water content.

Due to Compaction: –

- → permeability:
- → Void ratio:
- → Compressibility

} → Decrease

- → Shear strength → Increase.

Compaction test are done to determine: –

a) Amount of compactive energy.

b) The optimum moisture Content (OMC).

Indian Standard light weight Compaction test:-
(Similar to standard proctor test.)

- → The test results used for, highways, Embankments, Canal banks.
- → Mould volume 1 lit. soil in 3 layer each layer is given 25 hammer blows.
- → Hammer weight 2.60kg & hight of fall 31cm.

Indian Standard Heavy Compaction test:-
(Similar to Modified proctor test):

- → Results used for mordern Express highways & Runways.
- → Mould capacity 1 lit. soil in 5 layer each layer is given 25 hammer blows.
- → Hammer weight 4.90 kg & height of fall 45 cm.

\# The compactive effort in the modified proctor test is about 4.55 times then that in the standard proctor test.

OMC: (Optimum Moisture Content): ÷

↳ The water content at which the density of the soil maximum is called optimum moisture contain.

↳ The compaction effect in modified proctor test is 4.55 times the standard proctor test.

factor effecting the compaction ÷

*→ Water content
*→ Types of soil
*→ Amount of compaction.

*→ Water Content: ÷

↳ As a water content increase the dry density increases. reaches the maximum value then decrease

* Types of soil: ÷ Sandy soil. & gravel soil are more com[p]
to clay soil & silty soil.

* Amount of compaction: ÷

↳ As a load increase Dry density increase, moisture content decrease

Compaction Equipment:-

a) Tampers:-

↳ for compacting in confined areas like trench behind bridge abutments.

b) Smooth wheel rollers:-

↳ for granular soil provide smooth surface at the end of the days. work to quickly drain off rain w

c) pneumatic tyred rollers:-

↳ for cohesive & non-cohesive soils

d) Sheep foot rollers:-

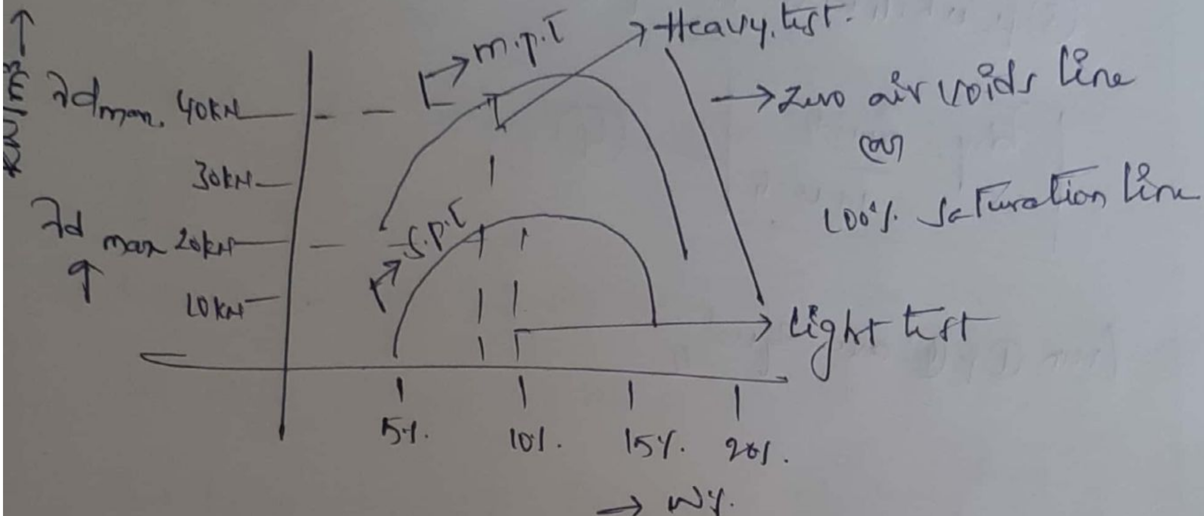
↳ for cohesive soil. - (clays).

e) Vibratory compactors:-

↳ for granular soil (for sand best method).

** Equation for zero air void line (or) 100% saturation line (or) theoritical max. dry density (γ_d theoritical):

↳ The dry density which is achieved practically by the expansion of air from voids of soil.



$$\boxed{\gamma_{dry} = \frac{(1-n_a)\gamma_w \cdot G}{1+w \cdot G}}$$

for zero air voids line $(n_a = 0)$

$$\gamma_{dry} = \frac{\gamma_w \cdot G}{1+w \cdot G} \quad \text{—— (V)}$$

when the soil is fully saturated the entire air replaced by water.

$(S_r = 100\%)$

100% saturation $(S_r = 100\% = 1)$.

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$$\gamma_{dry} = \frac{\gamma_w \cdot G}{1+e} \quad \text{—— (2)}$$

for 100% saturate $e = \frac{w \cdot G}{S_r}$ $\because S_r = 1$

$$e = w \cdot G \quad \text{—— (3)}$$

(3) substitute in eq. (2)

$$\boxed{\gamma_{dry} = \frac{\gamma_w \cdot G}{1+w \cdot G}} \quad \text{—— (4)}$$

from (1) & (4) are equal.

Procedure for find the Dry density :-

↳ Take soil 2.5 kg which is passes through 4.75mm Sieve

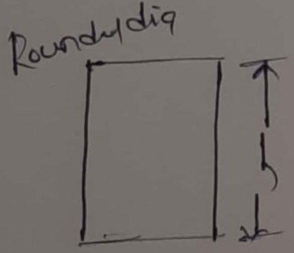
↳ Add some x%. of water

↳ let the weight of the Empty proctor test mould be W_1

↳ the weight of the wet soil + proctor mould be W_2

↳ then weight of the wet soil is $W_2 - W_1$,

↳ Bulk density $(\gamma_{bulk}) = \frac{W_s + W_w}{V} = \frac{W_2 - W_1}{A \times h}$



$$\gamma_{bulk} = \frac{W_2 - W_1}{\frac{\pi}{4} d^2 \times h}$$

Table $\gamma_{dry} = \frac{\gamma_{bulk}}{1+w}$

S.No:	water content in %	γ_{bulk}	γ_{dry}
1	10 %.	$\frac{W_2 - W_1}{\frac{\pi}{4} d^2 \times h}$	$\frac{\gamma_{bulk}}{1+w}$
2	20 %.	"	"
3	30 %.	"	"
4	40 %.	"	"
5	50 %.	"	"

An Earth Embankment is compacted at a water content of 18%. to bulk density of 19.2 kN/m3. If the sp. gravity of sand is 2.7. find the void ratio & degree of saturation of the compacted embankment.

Water content: $w = 18\%$.

Bulk density: $\gamma = 19.2\ kN/m^3$

Sp. Gravity, $G = 2.7$

Dry density: $\gamma_d = \frac{\gamma}{1+w} = \frac{19.2}{(1+0.18)} = 16.27\ kN/m^3$

but: $\gamma_d = \frac{G \cdot \gamma_w}{1+e}$, where $\gamma_w : 9.81\ kN/m^3$

$1+e = \frac{G \cdot \gamma_w}{\gamma_d} = \frac{2.7 \times 9.81}{16.27} = 1.63$

$\therefore 1+e = 1.63$

$e = 1-1.63$

$e = 0.63$

Void ratio: $e = 0.63$.

Also: $wG = Se$

Degree of saturation: $S = \frac{w \cdot G}{e}$

$= \frac{0.18 \times 2.7}{0.63}$

$= 0.7714$

$= 77.14\%$.

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Stress Distribution in Soil:-

Introduction:-

↳ Stress in soil is caused by the soil. first or both of the following:

1) Self weight of soil
2) Structural load applied at or below the surface

↳ many problems in foundation Engineering require a study of the transmition & distribution of stresses in large extensive masses of soil.

↳ Some Examples are wheel loads transmitted through Embankment to Culverts.

↳ foundation pressure transmitted to soil strata below footing.

↳ pressure from isolated footing transmitted to retaing walls, & wheel loads transmitted through stabilised soil pavement to subgrade below.

↳ In such cases the stresses are transmitted in all downward and lateral direction.

↳ The vertical stress in soil owing to its self-weight also called 'Geostatic stress'

$$\boxed{\sigma_z = \gamma . z}$$

Point Load :-

↳ A point load or a concentrated load is strictly speaking, hypothetical in nature, consideration of it serves a useful purpose in arriving at the solution for more loading complex in practice.

↳ The most foundamental of the solution of stress distribution in soil is that for a point load applied at the surface.

↳ Boussinesq & Westergaards have given the solution with the different assumptions regarding the soil medi

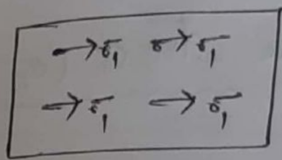
Boussinesq's Equation:-

↳ Boussinesq Equation has given the solution for the stresses caused by the application of point load at the surface of a "Homogeneous, Elastic, isotropic, & semi-infinite medium" weight less & the load is a point load acting on the ground surface.

Homogeneous:-

↳ A material is said to be homogeneous if it is identical properties at the different properties in identical direction.

(or)



↳ At any point the given direction the properties are same Ex: Wood, copper.

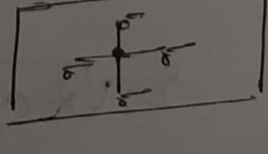
Isotropic:-

↳ At a given point that the any direction the properties are same.

(or)

↳ A material is said to be isotropic when it has identical elastic properties in all direction at a point.

Ex: Steel, Iron, brass.



→ Strength of material should be applied for both Homogenous, Isotrotropic, material.

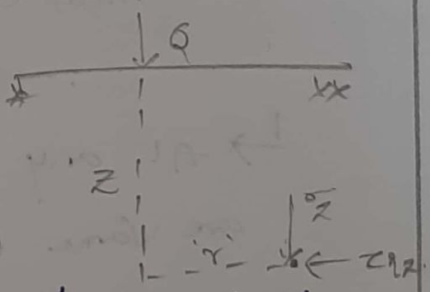
*→ All Homogenious material need not be the isotropic.

*→ All Isotropic materials need to be the Homogenious.

Semi infinite

↳ A material is said to be semi-infinite if it extends infinite in all directions below the Horizontal surface.

* Verticle stress (σ_z) :-



$$\boxed{\sigma_z = \frac{Q}{z^2} \cdot \frac{3}{2\pi} \left[\frac{1}{1+\left(\frac{r}{z}\right)^2} \right]^{5/2}}$$

σ_z = Vertich distance of the point below the load.

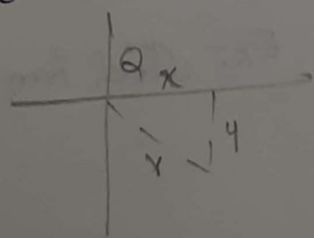
Q = point load

r = Radial distance of the point = $\sqrt{x^2+y^2}$

(or): $$\boxed{\sigma_z = \frac{Q}{z^2} \cdot k_B} \; **$$

To find the settlement criteria.

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



where k_B = Boussinesq's constant.

$$k_B = \frac{3}{2\pi}\left[\frac{1}{1+\left(\frac{r}{z}\right)^2}\right]^{5/2}$$

(or)

$$\boxed{\sigma_z = 0.477\frac{Q}{z^2}.}$$

↳ The verticle stress (σ_z) below the point load. ($\because r=0$)

$$\sigma_z = \frac{Q}{z^2}\cdot\frac{3}{2\pi}$$

$$\boxed{\sigma_z = 0.477\frac{Q}{z^2}} \quad **$$

A point load of 700kN is applied on the surface of clay using boussinesq's Equation estimate the verticle stress at the depth of 2kN & radial distance of 1mts from the point application of load

point load = (Q) = 700kN

depth (z) = 2kN

$r = 1$mt

$$\sigma_z = \frac{Q}{z^2}\cdot\frac{3}{2\pi}\left[\frac{1}{1+\left(\frac{r}{z}\right)^2}\right]^{5/2}$$

$$\sigma_z = \frac{700}{2^2}\cdot\frac{3}{2\times\pi}\left[\frac{1}{1+\left(\frac{1}{2}\right)^2}\right]^{5/2} \Rightarrow 47.83\text{kN/m}^2.$$

A concentrated load of 200kN is applied of ground surface of clay. & find the vertical stress at the point which is 7m directly below the load using the boussinesq's theorem:

Point load: $Q = 200kN$

depth $(z) = 7m$

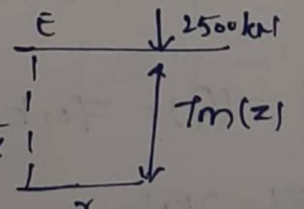
$$\sigma_z = 0.477 \frac{Q}{z^2}$$

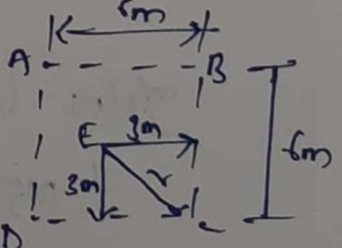
$$\sigma_z = \frac{0.477 \times 200}{7^2}$$

$$\sigma_z = 13.62 \, kN/m$$

An elevated structure with a total weight of 2500kN is supported on a tower with 4 legs. the legs rest on piers located at the corners of a surface 6m on a side. what is the vertical stress increment due to this loading at point 7m beneath the centre of the structure

Sol: $Q = 2500kN$





→ the load can be approximated to a point load acting at the corners of a square of 6m side.

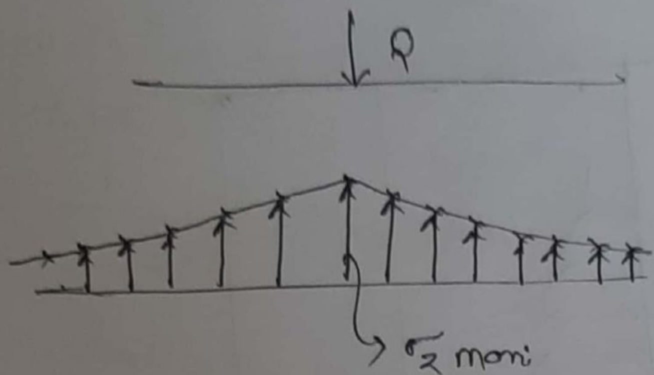
the vertical stress is to be calculated at 7m depth.

Horizontal distance r from each of the load is equal.

$\sqrt{3^2+3^2} = \sqrt{18} = 4.23m$:

$$= \frac{4 \times 3 \times 2500}{2\pi \times (7)^2} \left[\frac{1}{1+\left(\frac{4.23}{7}\right)^2} \right]^{5/2} = 4 \times 11.143 = 44.57 \, kN/m^2$$

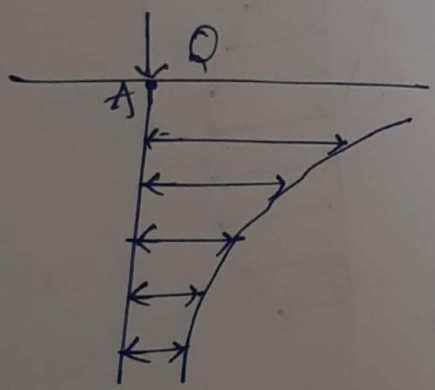
Verticle stress along the Horizontal plane :-



σ_z is max @ $r=0$.

Verticle stress variation on Verticle plane: passing below the load

i.e: $r=0$.

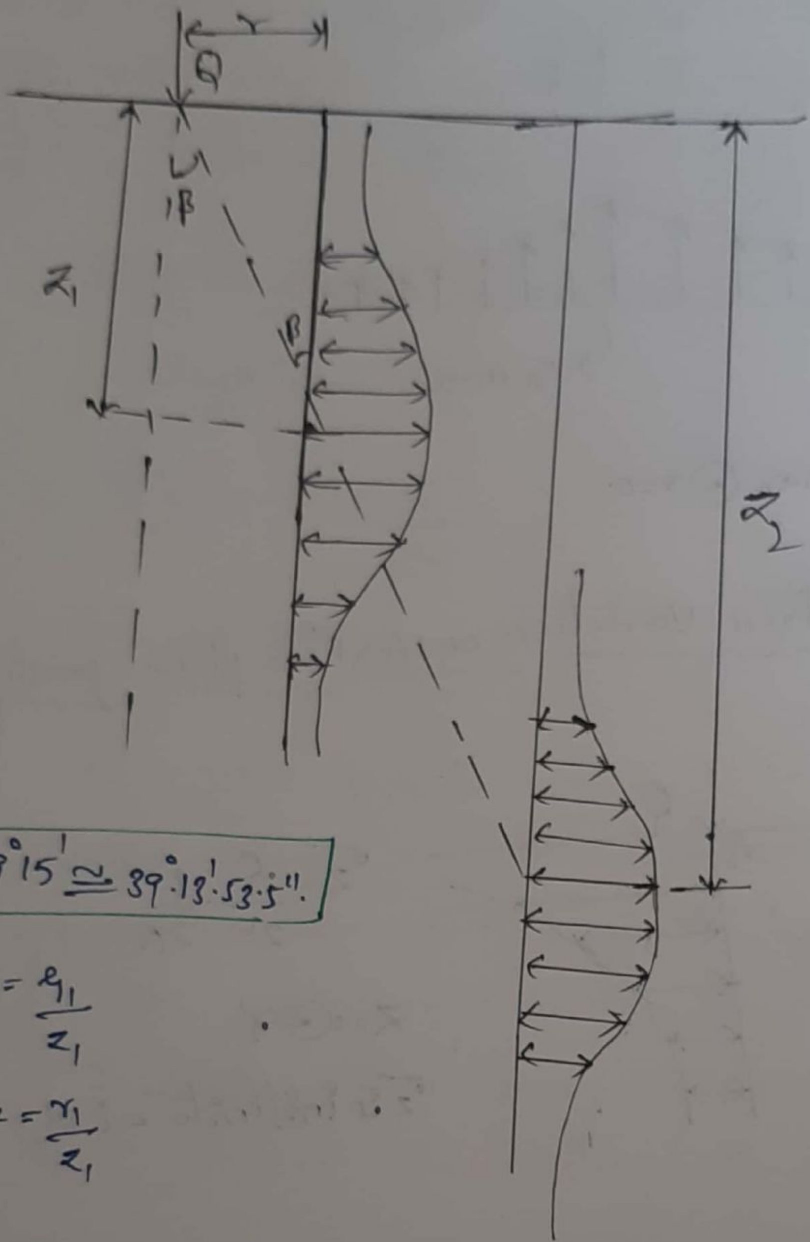


$\sigma_z = \frac{Q}{z^2} \cdot \frac{3}{2\pi}$

$z=0$ @ 'A'

σ_z is infinite at A

2. Verticle stress variation on verticle plane away from the pt. load



$\boxed{\beta = 39^\circ 15' \simeq 39^\circ 13' 53.5''}$

$\tan\beta = \frac{r_1}{z_1}$

$0.617 = \frac{r_1}{z_1}$

$\sigma_{z_1} > \sigma_{z_2}$ $(\because z_1 < z_2)$.

if a verticle plane is drawn at radial distence of '2'm. from a point. Determine the depth at which the max. verticle stress.

$r_1 = 2m.$

$\tan\beta = \frac{r_1}{z_1}$

$\tan(39^\circ 15') = \frac{2}{z_1} \Rightarrow z_1 = 2.44m.$

A concentrated load of 50 acts vertically at a point on the soil surface. if boussinesq's eq is applied for computation of stress then the ratio of vertical stress at depth of 3m & 5m respectively vertically below the point of application of load will be

Vertically below the load:

$$\sigma_z \propto \frac{1}{z^2}$$

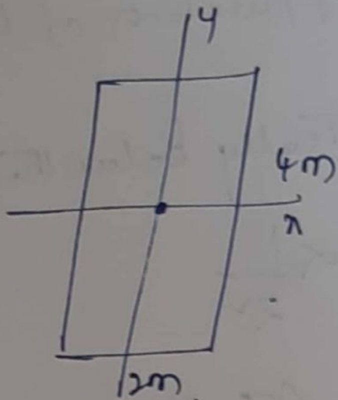
$$\frac{\sigma_{z_1}}{\sigma_{z_2}} = \left(\frac{z_2}{z_1}\right)^2 = \left(\frac{5}{3}\right)^2$$

$$\frac{\sigma_{z_1}}{\sigma_{z_2}} = 2.77$$

σ_z doesn't depend upon young's modulus and possion ratio of the soil.

↳ A point load of 700 kN

→ A rectangle area of 2m×4m carries a load of 8 tons/m² at a ground surface. Find the vertical stress at 5m below the center and corner of loaded area using Boussinesq's theorem.



$Q = 8\,t/m^2$

$z = 5m$

i) σ_z at centre

ii) σ_z at corner

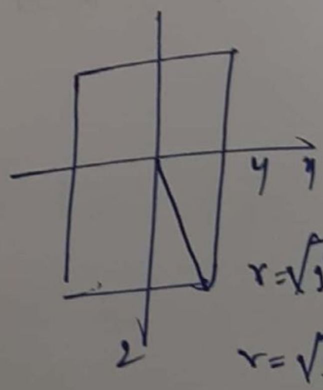
$$\sigma_z = \frac{Q}{z^2}\cdot\frac{3}{2\pi}\left[\frac{1}{1+\left(\frac{r}{z}\right)^2}\right]^{5/2}$$

At Center: (r=0):

$$\sigma_z = \frac{8}{5^2}\cdot\frac{3}{2\pi}\left[\frac{1}{1+\left(\frac{0}{5}\right)^2}\right]^{5/2}$$

$\sigma_z = 0.152\ t/m^2.$

σ_z at corner:



$$\sigma_z = \frac{8}{5^2}\cdot\frac{3}{2\pi}\times\left[\frac{1}{1+\left(\frac{\sqrt{5}}{5}\right)^2}\right]^{5/2}$$

$r = \sqrt{2^2+1^2}$

$r = \sqrt{5}$

$\sigma_z = 0.095\ t/m^2.$

Westergaards Equation :-

Assumption :-

- The soil is Homogeneous, Non isotropic & semi infini
- This theory is suitable for stratified soil.
- The soil is only vertical deformation not a Horizontal deformation.
- The theory is suitable for only sedimentary soil. Not suitable for residual soil.

$$\boxed{\sigma_z = \frac{Q}{z^2} \cdot \frac{1}{\pi} \left[\frac{1}{1+2\left(\frac{r}{z}\right)^2} \right]^{3/2}}$$

σ_z = Vertical stress in westergaards Equation.

Q = point load.

z = depth

r = radial distance

$$\sigma_z = \frac{Q}{z^2} \cdot K_w.$$

K_w = Westergaards constant.

$$\boxed{K_w = \frac{1}{\pi} \left[\frac{1}{1+2\left(\frac{r}{z}\right)^2} \right]^{3/2}}$$

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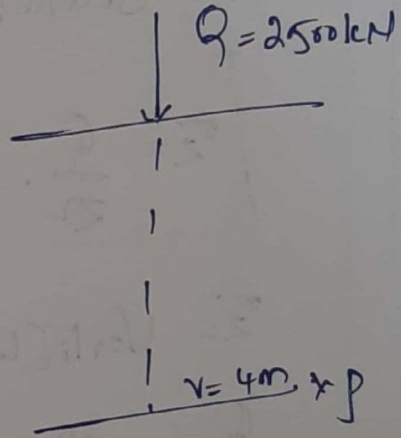
$$\boxed{K_w = \frac{1}{\pi} \left[\frac{1}{1+2\left(\frac{r}{z}\right)^2}\right]^{3/2}}$$

the westergaards Verticle stress directly below the point load (r=0).

$$\boxed{\sigma_z = 0.318 \frac{Q}{z^2}}$$

↳ A concentrated load of 2500kN is applied ground surface determine the verticle stress at a point 'p' which is act at depth of 5m but a horizontal distance of 4m from the axis of the load:



$Q = 2500kN$

$z = 5m$

$$\sigma_z = \frac{Q}{z^2} \cdot \frac{1}{\pi} \left[\frac{1}{1+2\left(\frac{r}{z}\right)^2}\right]^{3/2}$$

$$\frac{2500}{5^2} \cdot \frac{1}{\pi} \left[\frac{1}{1+2\left(\frac{4}{5}\right)^2}\right]^{3/2}$$

$\sigma_z = 9.245 \ kN/m^2.$

Comparision b/w Boussinesq's & Westergaards theorem:

$\frac{r}{z}$	Boussinesq's Vertical stress $\sigma_z = \frac{Q}{z^2} \cdot \frac{3}{2\pi} \left(\frac{1}{1+(\frac{r}{z})^2} \right)^{5/2}$	Westergaards Vertical stress $\sigma_z = \frac{Q}{z^2} \cdot \frac{1}{\pi} \left(\frac{1}{1+2(\frac{r}{z})^2} \right)^{3/2}$.
0	$\sigma_z = 0.477 \frac{Q}{z^2}$	$\sigma_z = 0.318 \frac{Q}{z^2}$.
1	$\sigma_z = 0.084 \frac{Q}{z^2}$	$\sigma_z = 0.061 Q/z^2$
1.5	$\sigma_z = 0.025 \frac{Q}{z^2}$	$\sigma_z = 0.024 Q/z^2$
1.52	$\sigma_z = 0.023 Q/z^2$	$\sigma_z = 0.023 Q/z^2$
2.0	$\sigma_z = 0.0085 Q/z^2$	$\sigma_z = 0.11 Q/z^2$

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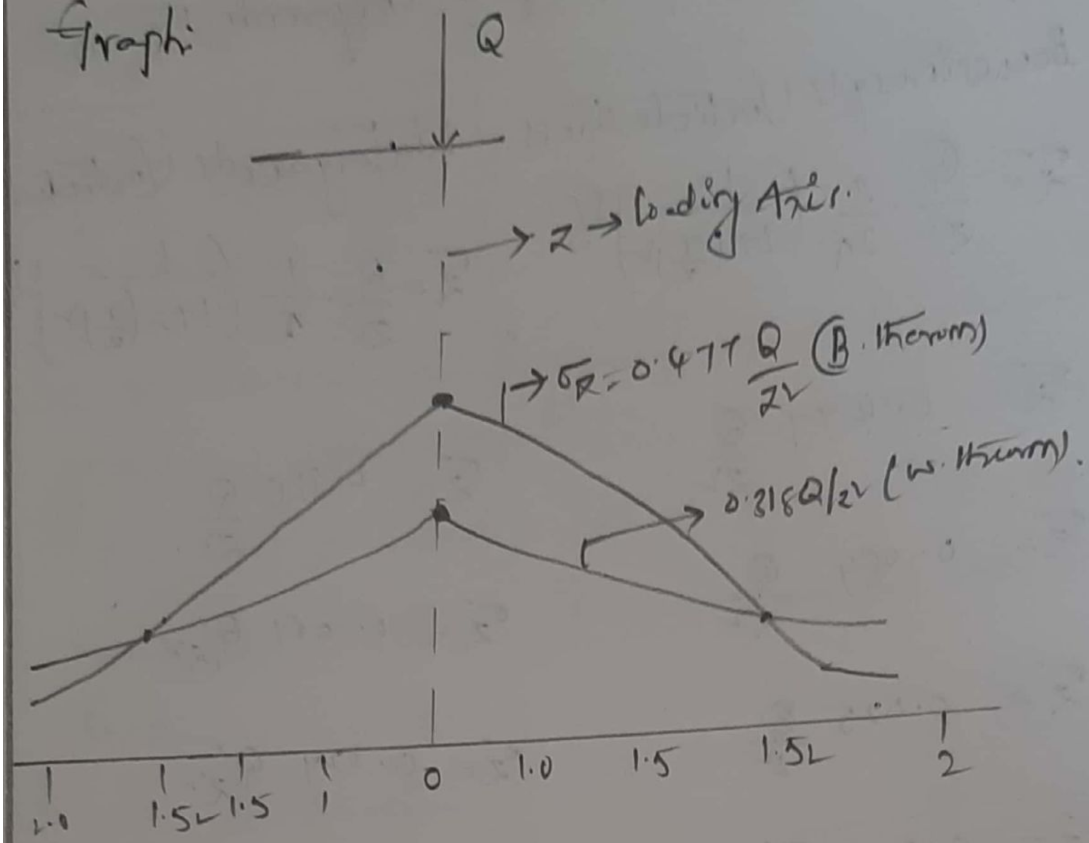
Graph:



Comparision: (Kw & kB):

if $= \frac{r}{z} = 1.52$ $K_w = K_B$

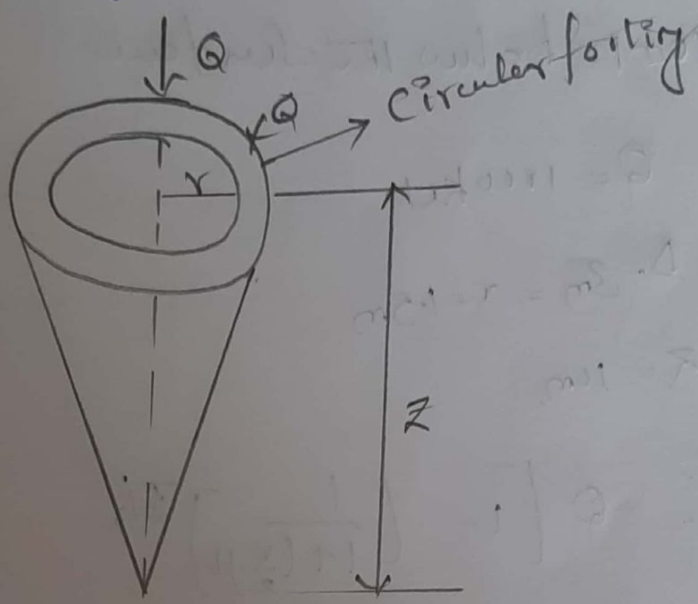
if $\frac{r}{z} > 1.52$ $K_w > K_B$

if $\frac{r}{z} < 1.52$ $K_w < K_B$.

The value is $\boxed{1.50 - 1.52}$

Verticle stress distribution below uniformly Circular loaded areas:-

↳ Generally Circular foundation is adopted beneath the Circular water tank, oil storage tank, well foundation.



$$\boxed{\sigma_z = Q\left[1 - \left(\frac{1}{1+\left(\frac{r}{z}\right)^2}\right)^{3/2}\right]}$$

σ_z = Verticle stress due to Circular load area

Q = Vertical load

r = radius Circular area

z = depth.

A circular foundation it rest on horizontal Surface of soil the load of a foundation 1000kN. that diameter of the foundation is 3m. determine the vertical stress on horizontal plane along the central axis of the foundation to a depth of const. below the surface

$Q = 1000 kN$

$D = 3m = r = 1.5m$

$z = 10m.$

$$\sigma_z = Q\left[1 - \left(\frac{1}{1+(\frac{r}{z})^2}\right)^{3/2}\right]$$

$$1000\left[1 - \left(\frac{1}{1+(\frac{1.5}{10})^2}\right)^{3/2}\right]$$

$\sigma_z = 32.825 \, kN/m^2.$

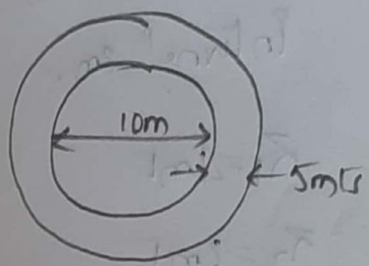
A loading of 50 kN/m2 is acting on circular foundation of width of 5m & inside dia of 10m find the vertical stress intensity at a depth of 10m below the centre of foundation:

load: $Q = 50\ kN/m^2$

width (w) = 5mts

Inside dia (d) = 10mts

Depth (z) = 10mts.



$D_i = 10m \quad r_i = 5m$

$D_o = 20m \quad r_o = 10m$

$\sigma_z = \sigma_z\ \text{outlet} - \sigma_z\ \text{inlet}$.

$$\sigma_z = Q\left[1 - \left(\frac{1}{1+\left(\frac{r_o}{z}\right)^2}\right)^{3/2}\right] - Q\left(1 - \left[\frac{1}{1+\left(\frac{r_i}{z}\right)^2}\right)^{3/2}\right.$$

$$50\left[1 - \left[\frac{1}{1+\left(\frac{10}{10}\right)^2}\right)^{3/2}\right] - Q\left[1 - \left[\frac{1}{1+\left(\frac{5}{10}\right)^2}\right)^{3/2}\right.$$

$\sigma_z = 18.1\ kN/m^2$.

A angular ring footing of External & internal diameter of 6m & 3m resp. Transmit a pressure of 200 N/mm². Calculate vertical stress at a depth of 1m & 3m

Load $Q = 200\ N/mm^2$

External 6m

Internal 3m.

$z_1 = 1mt$

$z_2 = 3mt$

$D_i = 3m \quad r_i = 1.5$

$D_o = 6m \quad r_o = 3m$

$\sigma_z = \sigma_z \text{ outlet} - \sigma_z \text{ inlet}$

$$\sigma_z = Q\left[1 - \left[\frac{1}{1+\left(\frac{r_o}{z}\right)^2}\right]^{3/2}\right] - Q\left[1 - \left[\frac{1}{1+\left(\frac{r_i}{z}\right)^2}\right]^{3/2}\right]$$

$$200\left[1 - \frac{1}{1+\left(\frac{3}{1}\right)^2}\right]^{3/2} - 200\left(1 - \frac{1}{1+\left(\frac{1.5}{1}\right)^2}\right]^{3/2}$$

$$\boxed{\sigma_z = 55.85\ N/m^2}$$

Case: II $\quad z = 3mt$

$$200\left[1 - \left[\frac{1}{1+\left(3/3\right)^2}\right]^{3/2}\right] - 200\left[1 - \left(\frac{1}{1+\left(\frac{1.5}{3}\right)^2}\right]^{3/2} \quad \text{kN/m}^2$$

$$\boxed{\sigma_z = 52.39\ N/m^2}$$

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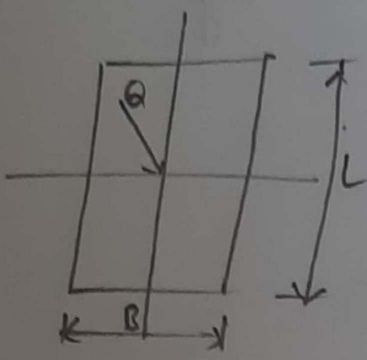
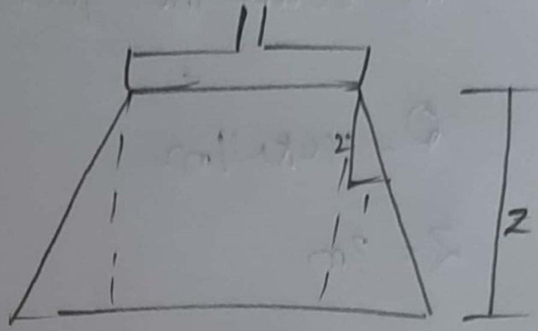
To find Verticle stress by Approximate method

i.e. 2 Verticle 1 Horizontal method (2V:1H):





formula:

$$\boxed{\sigma_z = \frac{Q}{(L+z)(B+z)}}$$

→ Rectangle

→ for 2V:1H

σ_z = Verticle stress Approximate.

Q = load

L = length

B = Breadth

z = depth.

A rectangle area of 2m & 4m carries a load of 180 kN/m² at the ground surface find the vertical stress 3m below the foundation using 2V:1H method.

$Q = 180 kN/m^2$

$z = 3m$

$$\sigma_z = \frac{Q}{(L+z)(B+z)}$$

$$= \frac{180}{(4+3)(2+3)}$$

σ_z = 1428 kN/m².

UNIT – 4:

Consolidation of soil:-

↳ Compressibility:- Gradual decrease in the volume of soil due to application of load on soil mass.

*→ the volume change take place may be due to.

1) Compression & Expulsion of pore air.

2) Explusion of pore water.

3) plastic rearrangement of soil particle.

Compression:

↓ Compaction (or) initial consolidation (or) initial compression (Escaping of air in partially saturated soil)

↓ Consolidation. (Escaping of water from void fully saturated soil). (slow gradual process

↓ (primary Consolidation) (Secondery Consolidatio

i) Initial Consolidation (or) Compaction :-

↳ Escaping of air (or) Compression of the air in voids immediately after loading is initial consolidation

ii) Primary Consolidation :-

↳ After completion of compaction the further decrease in volume due to escaping of water from the voids in primary consolidation.

1) It is slow & gradual process & it occurs under long term static loading.

2) Consolidation is only possible in clays.

3) It does not occurs in sand, due to high permeability.

iii) Secondary Consolidation :-

↳ It is possible only after the completion of primary consolidation.

↳ Decrease in volume occurs due to the plastic rearrengement of solid particle is known as secondary consolidation (or) creep.

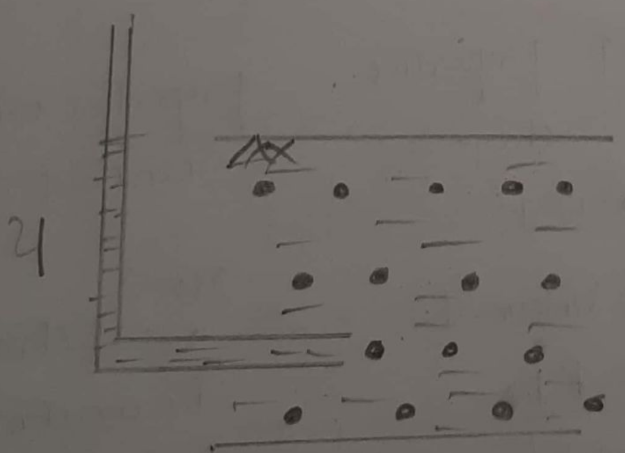
↳ Generally 10-20% of primary consolidation is possible.

Consolidation:-

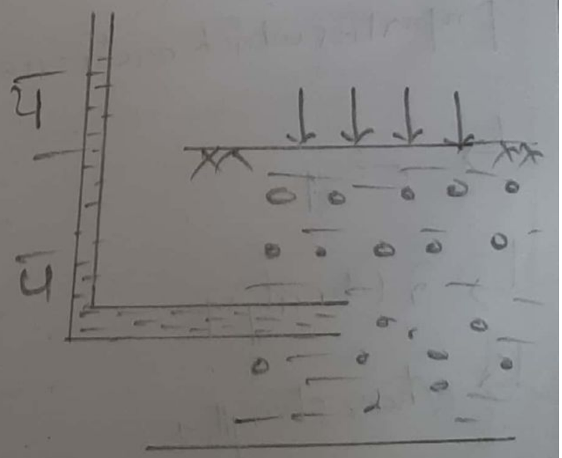
↳ it is the compression of soil mass due to Expulsion of Removes water from the voids. under steady, static long term loading.

Compaction:	Consolidation	Compressibility
↳ Escape of Air voids from voids	↳ Escape of water from voids.	↳ Escape of water.
↳ short term loading	↳ long term or static or steady } loading.	
↳ Quick process. Take less time.	↳ slow process Take more time	

During Consolidation process:-



u → pore water pressure or Hydrostatic pressure



ū → Excess of pore water Hydrodynamic pressure

At Begining of consolidation	At during of consolidation.	At the end of consolidation.
$\sigma = \bar{u}$	$\bar{u} < \sigma$	$\bar{u} = 0$
$\sigma' = \sigma - u$	$\sigma' = \sigma - u$	$\sigma' = \sigma - u$
$\sigma' = 0$	$\sigma' > 0$	$\sigma' = \sigma$
σ. constant	σ constant	σ constant.

During consolidation:-

↳ excess pore water pressure is $\bar{u}$ decrease

↳ Effective stress (σ') is increase

↳ Total stress is σ is constant.

During consolidation:-

properties which are increase ↑	properties decrease	properties which constant.
↳ σ' ↑	→ u ↓,	→ (σ) Total stress is. constant
↳ γ (density) ↑	→ Volume ↓,	
↳ shear strength ↑	→ K ↓	
	→ Water content	
	→ e ↓	

Compressibility :-

↳ Reduction in volume of the soil due to escape of air and water from the voids.

Co-efficient of Compressibility (a_v)

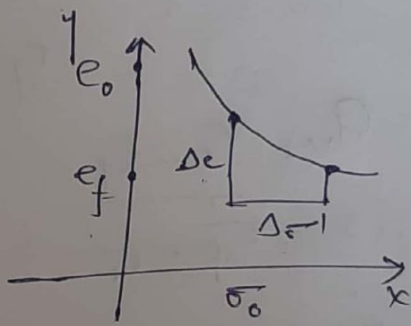
(or)

(e-p Curve (or) e-σ' Curves)

(void ratio - pressure) (void ratio - effective stress):-

↳ it is the ratio of decrease in void ratio per unit increase of effective stress.

$$a_v = \frac{\Delta e}{\Delta \sigma'}$$



$$\boxed{a_v = \frac{e_0 - e_f}{\sigma_f' - \sigma_0'}}$$

units: m^2/kN.

$a_v \rightarrow$ Co-efficient of compressibility.

$e_0 \rightarrow$ initial void ratio.

$e_f \rightarrow$ final void ratio.

$\sigma_f' \rightarrow$ final effective stress

$\sigma_0 \rightarrow$ initial effective stress.

↳ the consolidation test on clay sample of co-efficient of compressibility is $50\,m^2/kN$. the initial void ratio is 0.6. the stress intensity increases from 100 to 300 kN/m². then find the final void ratio:

Sol: Co-efficient of compressibility $(a_v) = 50\,m^2/kN$.

Initial void ratio: $e_0 = 0.6$

the stress intensity $= \sigma_0' = 300$ kN/m²

$\sigma_f' = 300$ kN/m²

$e_f = ?$

$$a_v = \frac{e_0 - e_f}{\sigma_f' - \sigma_0'}$$

$$50 = \frac{0.6 - e_f}{300 - 100}$$

$$50 \times 200 = 0.6 - e_f$$

$$-10 \times 10^3 + 0.6 = e_f$$

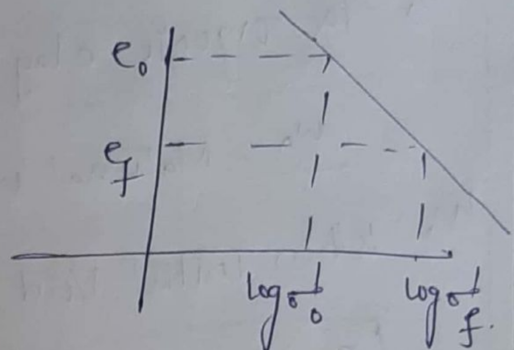
$$e_f = 9.994 \times 10^3.$$

→ e – log p or e – log σ' curve:

Co-efficient of compression index (C_c):–

↳ The slope linear portion of e – log σ' curve is called co-efficient of compressibility.

$$\boxed{C_c = \frac{e_0 - e_f}{\log \sigma'_f - \log \sigma'_0}}$$



$$C_c = \frac{\Delta e}{\log\left(\frac{\sigma'_f}{\sigma'_0}\right)}$$

C_c = co-efficient of compressibility index.

e_0 → initial void ratio.

e_f → final void ratio.

σ'_0 → initial effective stress

σ'_f → final effective stress

for most of the clays C_c lies in between, 0.1 to 0.8.

Empirical relation for determination of C_c :-

i) for undistrubed clay $\boxed{C_c = 0.009(w_L - 10)}$

w_L = Liquid Limit in %.

ii) for remoulded clay $\boxed{C_c = 0.007(w_L - 10)}$

iii) for organic clay $\boxed{C_c = 0.0115\ w_n}$

w_n = Natural water content in %.

iv) When initial void ratio is known as

$\boxed{C_c = 1.15(e_0 - 0.35)}$ e_0 = initial Voidratio:

→ the consolidation test remoulded clay sample of liquid limit 50%. the initial voidratio is 0.6 the stress increasing intensity from 100 k.pa. 800 k.pa then find the final void ratio.

Sol: $C_c = (0.007)(w_L - 10)$ for remoulded clay.

initial void ratio: $e_0 = 0.6$

stress intensity $(\bar{\sigma}_0') = 100$ k.pa.

$\bar{\sigma}_f' = 800$ k.pa

$e_f = ?$

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$$C_c = \frac{e_0 - e_f}{\log \sigma_f' - \log \sigma_0'}$$

$$C_c = \frac{e_0 - e_f}{\log\left(\frac{\sigma_f'}{\sigma_0'}\right)}$$

$$= 0.007(40\%) = \frac{0.6 - e_f}{\log\left(\frac{300}{100}\right)}$$

$$= 0.007\left(\frac{40}{100}\right) \times \log\left(\frac{300}{100}\right) = 0.6 - e_f$$

$$\boxed{e_f = 0.467}$$

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Co-efficient of Volume Compressibility (m_v):-

↳ it is change in volume of the soil per unit initial volume due to increase in stress.

$$m_v = \frac{\Delta V}{V_0 \cdot \Delta\sigma} \quad \text{--- (1)} \quad \text{Units} \rightarrow m^2/kN.$$

$m_v \rightarrow$ Co-efficient of volume compressibility (m_v).

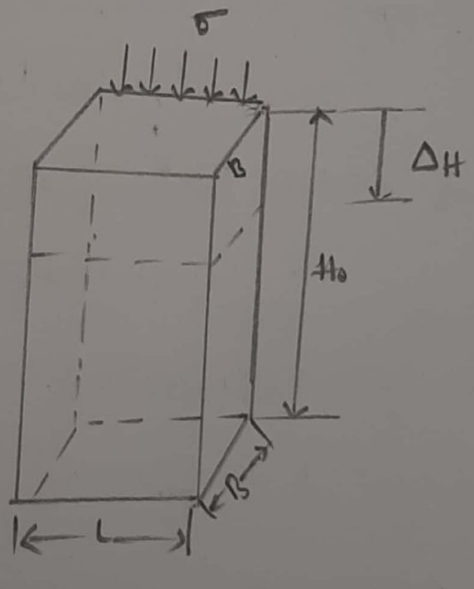
$\Delta V \rightarrow$ change in volume

$V_0 \rightarrow$ initial volume

$\Delta\sigma \rightarrow$ change in stress.

$$\frac{\Delta V}{V_0} = \frac{A \times \Delta H}{A \times H_0} \quad \text{--- (2)} \quad \text{('A' is constant for confined soil).}$$

$$\frac{\Delta V}{V_0} = \frac{\Delta H}{H_0} \quad \text{--- (3)}$$



We know that:

$$\frac{\Delta V}{V_0} = \frac{\Delta e}{1+e_0}$$

$$\frac{\Delta V}{V_0} = \frac{\Delta H}{H_0} = \frac{\Delta e}{1+e_0}$$

$$m_v = \frac{\Delta V}{V_0 \cdot \Delta\sigma} = \frac{\Delta H}{H_0 \cdot \Delta\sigma} = \frac{\Delta e}{(1+e_0)\Delta\sigma}$$

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The following data is Given in laboratory $\sigma_0 = 175$ k.pa $e_0 = 1.1$. $\sigma_0 + \Delta\sigma = 300$ k.pa. $e_f = 0.9$ if the thickness of clay specimen. is 25 mm. then find volume of co-eff. of volume compressibility. (2002 - Gate).

Sol:

$\sigma_0 = 175$ k.pa.

$e_0 = 1.1$

$e_f = 0.9$

$\sigma_0 + \Delta\sigma = 300$ k.pa.

Thickness of clay specimen. = 25 mm.

Co-efficient of compressibility = $(m_v) = \frac{\Delta e}{(1+e_0)\Delta\sigma}$

$\sigma_0 + \Delta e = 300$

$\Delta e = 300 - 175$

$\Delta\sigma = 125$ k.pa.

$\Delta e = e_0 - e_f$

$= 1.1 - 0.9$

$\Delta e = 0.2$

$$m_v = \frac{\Delta e}{(1+e_0).\Delta\sigma}$$

$$= \frac{0.2}{(1+1.1)(125)}$$

$m_v = 7.61 \times 10^{-4}$ m²/kN

→ In a Consolidation test. the void ratio of specimen is 1.052 under the effective pressure is 207. kN/m². The void ratio changed to 0.932 when pressure was increase to. 430 kN/m². then find the.

i) Co-efficient of compressibility.
ii) Co-efficient of volume - compressibility.
iii) Co-efficient of compression index

Void ratio of specimen. $e_0 = 1.052$

effective pressure $(\sigma_0') = 207$ kN/m².

Void ratio changed to : $e_f = 0.932$.

pressure was increased $= (\sigma_f') = 430$ kN/m²

i) Co-efficient of compressibility index.

$$C_c = \frac{e_0 - e_f}{\log \sigma_f' - \log \sigma_0'}$$

$$= \frac{\Delta e}{\log\left(\frac{\sigma_f'}{\sigma_0'}\right)} =$$

$$= \frac{1.052 - 0.932}{\log\left(\frac{430}{207}\right)} = 0.377$$

a) Co-efficient of compressibility –

$$a_v = \frac{e_0 - e_f}{\sigma_f' - \sigma_0'}$$

$$= \frac{1.052 - 0.932}{427 - 207}.$$

$a_v = 5.470 \times 10^{-4}$ m²/kN.

b) Co-efficient of volume compressibility:

$$m_v = \frac{a_v}{1 + e_0}$$

$$= \frac{5.470 \times 10^{-4}}{1 + 1.052}$$

$= 2.66 \times 10^{-4}$,

→ Ultimate settlement (or) final settlement (S.f) (or) change in thickness. (ΔH)

a) When change in void ratio. S_f (or) ΔH :-

from. Eq. (8).

$$\frac{\Delta H}{H_0} = \frac{\Delta e}{1+e_0}$$

$$\therefore \boxed{\Delta H = H_0\left(\frac{\Delta e}{1+e_0}\right)}$$

ΔH (or) S_f = final settlement.

H_0 = initial thickness (or) initial height.

Δe = change in void ratio.

$= e_0 - e_f$.

e_0 → initial void ratio

e_f → final void ratio.

Co-efficient of volume compressibility. (m_v) :-

if know S_f (or) ΔH.

from Eq. (4)

m_v → Co-efficient of volume compressibility

H_0 → initial thickness (or) height.

Δσ → change in stress.

$$m_v = \frac{\Delta H}{H_0 \cdot \Delta\sigma}$$

$$\boxed{\Delta H \text{ (or) } S_f = m_v \cdot H_0 \cdot \Delta\sigma}$$

Co-efficient of compression Index (C_c):-

$$C_c = \frac{\Delta e}{\log\left(\frac{\sigma_f'}{\sigma_0'}\right)}$$

$$\Delta e = C_c \cdot \log\left(\frac{\sigma_f'}{\sigma_0'}\right).$$

$$\Delta H = H_0 \cdot \frac{\Delta e}{1+e_0}$$

$$\Delta H = \frac{H_0 \cdot C_c \cdot \log\left(\frac{\sigma_f'}{\sigma_0'}\right)}{1+e_0}.$$

$$\boxed{\sigma_f' = \sigma_0' + \Delta\sigma'.}$$

$\sigma_0 \rightarrow$ Initial effective stress. at the middle of the c layer's

$\Delta\sigma_f \Rightarrow$ Increase in stress due to applied loads at midd clay. layer.

$C_c \rightarrow$ Co-efficient of compressibility Index.

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↳ The change in void ratio due to increase in effective pressure is 0.1 the initial void ratio is 0.4 the thickness of the soil strata is 7m. then find consolidation settlement in cm?

Sol: change in void ratio $(\Delta e) = 0.1$

initial void ratio $(e_0) = 0.4$

the thickness of soil strata $= H = 7m$,

$$\Delta H = H_0 \left[\frac{\Delta e}{1+e_0} \right]$$

or

Sf

$$= 7 \left(\frac{0.1}{1+0.4} \right)$$

$$= 0.5 \times 100$$

$$= \frac{5}{10} \times 100$$

$$= 50 \, cm.$$

↳ The clay layer of thickness 10cm & initial void ratio is 0.5 & final void ratio is 0.2 find the ultimate settlement of layer.

Sol: layer of thickness $H_0 = 10\,cm$

Initial void ratio $(e_0) = 0.5$

final " " $(e_f) = 0.2$.

ultimate settlement layer?

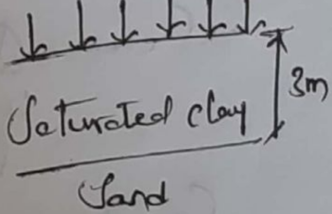
$$\Delta H \text{ (or) } S_f = H_0 \left(\frac{\Delta e}{1+e_0} \right)$$

$$= 10 \cdot \left(\frac{0.5 - 0.2}{1 + 0.5} \right)$$

$$= 10 \times \left(\frac{0.3}{1.5} \right)$$

$\Delta H = 2\,cm$
(or)
S_f.

↳ A clay of layer of thickness 3m, (m_v) Co-efficient of Volume compressibility $5 \times 10^{-4}\, m^2/kN$. If an sand is fill of unit weight $20\, kN/m^3$. & 2m depth is dumped on clay layer then find the ultimate settlement of clay layer is — mm



Sol: $\gamma = 20\, kN/m^3$

$H_0 = 3$

$m_v = 5 \times 10^{-4}$

ΔH or $S_f = H_0 \cdot m_v \cdot \Delta\sigma \Rightarrow 3 \times 5 \times 10^{-4} \times 40$

$\Rightarrow 0.06\,m. = 60\,mm$

$\Delta\sigma' = \gamma h$

$= 20 \times 2.0$

$\Delta\sigma = 40.$

A saturated soil specimen as compression index of 0.28 thickness 6m, if void ratio at stress of 12 kN/m². is 2.05 then compute.

a) change in void ratio. if stress increased to 21.6 kN/m²

b) final settlement.

Sol: Compression index $(C_c) = 0.28$

thickness $(H) = 6m$.

Void ratio at stress of $= 12\,kN/m^2$. is 2.05

a) change in void ratio of stress increased 21.6 kN/m².

$$C_c = \frac{\Delta e}{\log\left(\frac{\sigma'_f}{\sigma'_0}\right)}$$

$$C_c \cdot \log\left(\frac{\sigma'_f}{\sigma'_0}\right) = \Delta e$$

$$0.28 \times \log\left(\frac{21}{12}\right) = \Delta e$$

$$\Delta e = 0.07.$$

$$\Delta e = e_0 - e_f \Rightarrow 0.07 = 0.28 - e_f \Rightarrow e_f = 0.28 - 0.07.$$

$$\boxed{e_f = 0.21}$$

b) final settlement

$$\Delta H = \frac{H_0 \cdot C_c \log\left(\frac{\sigma'_f}{\sigma'_0}\right)}{1 + e_0}$$

$$= \frac{6 \times 0.28 \times \log\left(\frac{21.6}{12}\right)}{1 + 0.28} \Rightarrow 0.15m.$$

Co-efficient of consolidation: (C_v):-

↳ function of permeability & volume compressibi[lity]

$$\boxed{C_v = \frac{k}{m_v \cdot \gamma_w}}$$

units:- m^2/sec

$K \rightarrow$ permeability m/sec

$m_v \rightarrow$ Co-efficient of volume compressibility.

$$m_v = \frac{\Delta v}{V_0 \, \Delta\sigma} \; ; \; \frac{\Delta H}{H_0 \, \Delta\sigma} \; ; \; \frac{a}{(1+e) \cdot \Delta\sigma}$$

$\gamma_w \rightarrow$ unit weight of water.

$\gamma_w = 10 \, kN/m^3$

$$= \frac{k}{m_v \cdot \gamma_w} = \frac{m/sec}{\frac{\cancel{m^2}}{\cancel{kN}} \times \frac{\cancel{kN}}{\cancel{m^3}}} = \frac{m^2}{sec}$$

↳ Co-efficient of permeability in m/sec if co-efficient of consolidation & if volume compressibility are 4.8 mm²/min & 1.02×10^{-3} m²/kN.

Co-efficient of consolidation $C_v = 4.8\ mm^2/min$.

Volume of compressibility $(m_v) = 1.02 \times 10^{-3}\ m^2/kN$

unit weight $(\gamma_w) = 10\ kN/m^3$

permeability $(k) = ?$ m/sec

$$C_v = \frac{k}{m_v \cdot \gamma_w}$$

$$k = C_v \cdot m_v \cdot \gamma_w$$

$$= \frac{4.8}{60} \times 1.02 \times 10^{-3} \times 10$$

$$\frac{mm^2}{sec} \times \frac{m^2}{kN} \times kN/m^3$$

$$\frac{mm^2}{sec} \times \frac{1}{m} \times \frac{1}{1000(mm)}$$

$$\frac{4.8}{60} \times 1.02 \times 10^{-3} \times 10 \times \frac{1}{10^3} \Rightarrow \frac{4.8}{60} \times 1.02 \times 10^{-3} \times 10 \times 10^{-3}$$

$$\Rightarrow 8.16 \times 10^{-7}\ mm/sec$$

$$8.16 \times 10^{-7} \times 10^{-3}\ m/sec.$$

↳ Consolidation of undistrubed Specimen :-

Based on history of loading, soil deposits are divided into '3' stages:

a) Pre consolidated (or) pre compressed (or) over Consolidated :-

↳ if a soil has ever been subjected to a pressure in excess of its present over burden, it is said to be a over consolidated.

$$\boxed{\sigma'_{present} < \sigma'_{past}}$$

(or):

↳ if the applied present effective stress is less then past applied effective stress is called pre consolidation (or) over consolidated clays.

b) Normally consolidated clay. → (N.C.C).

↳ if the present applied effective stress is equal to past applied effective stress is called Normally consolidated clay.

c) under Consolidated clays. (U.C.C) :-

↳ if the present applied effective stress is grater then past applied effective stress is called under Consolidated cla

↳ Over Consolidated ratio. (OCR):-

$$OCR = \frac{\sigma'_c}{\sigma'} = \frac{\text{maximum applied } \sigma' \text{ in past}}{\text{Maximum applied } \sigma' \text{ at present}}$$

σ'_c = The maximum pressure to which an occ had been subjected (consolidated) in the past pre consolidation stress

σ' = The present existing pressure on the soil.

for. OCC → OCR > 1;

N.CC → OCR = 1;

U.C.C → OCR < 1;

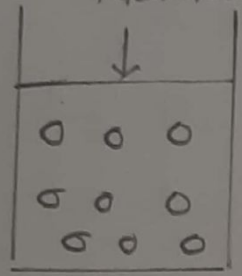
Stages in consolidation.

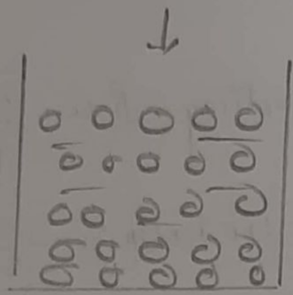
Initial Consolidation (or) Immediate Settlement Elastic - Settlement.

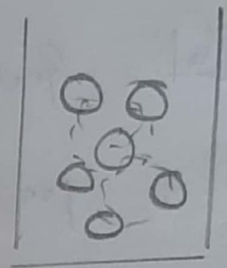
→ Primary Consolidation. occurs due to. Escape of water

Secondary consolidation. occures due to plastic rearrange -ment of soil particle

Nature of the soil.







Initial/Immediate Settlement :-

→ which is occurs due to elastic nature of the soil.

$$S_i = \frac{q}{E} \cdot B \cdot I \, (1-\mu^2) \quad \text{units: m.}$$

→ All highly permeable soil are undergoes Immediate Settle

Ex: Sand

→ q → Contact pressure per unit area kN/m^2

E → young's modulus; $\frac{kN}{m^2}$.

B → least dimension of area (m).

μ → possion's ratio.

I → Influence factor.

I of rigid footing $= 1.8 \times I$ of flexible footing.

↳ A rectangular footing of size 2m×3m exterts a pressure of 100kN/m². Determine the immediate settlement Assuming -

a) footing is flexible -

b) footing is rigid take the influence factor of flexible footing is 1.3-6 & young's modulus $E = 5\times10^4$ kN/m² & $\mu = 0.5$

Sol: Rectangular footing size = 2m×3m.
B. L.

pressure(q) = 100 kN/m²

$S_I = ?$

$E = 5\times10^4$ kN/m² $\quad \mu = 0.5$

a) footing flexible.

$$S_I = \frac{q}{E} \times I \times B \times (1-\mu^2)$$

$$= \frac{100}{5\times10^4} \times 2 \times 1.36 (1-(0.5)^2)$$

$S_I = 4.08 \times 10^{-3}$ m

footing rigid:

$$S_I = \frac{100}{5\times10^4} \times 2 \times 1.36 \times 1.8 (1-(0.5)^2)$$

$S_I = 7.34 \times 10^{-3}$ m.

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Terzaghi - 1-Dimensional consolidation theory:

Assumption: ÷

↳ The soil is Homogeneous & isotropic.

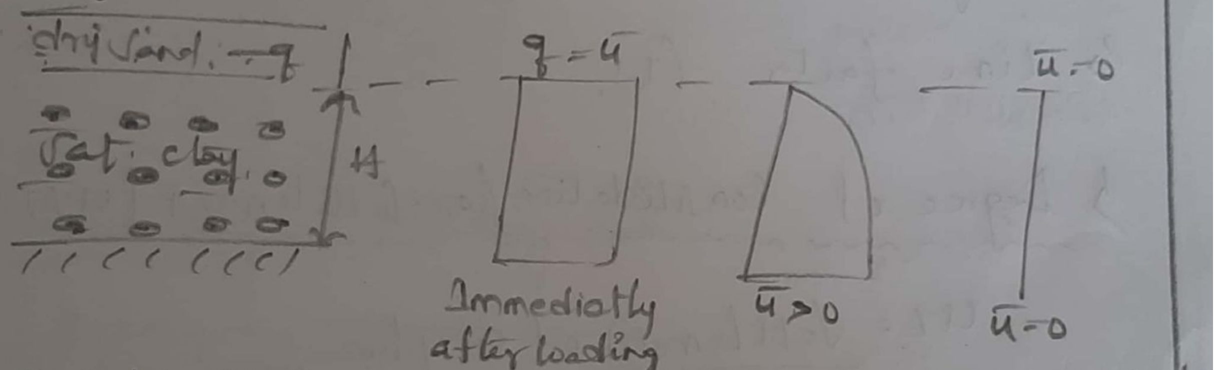
↳ The consolidation only in 1-Dimension it means the Escape of the water only in 1-Dimension (Verticle).

↳ There is no change in Area. Volume change due to only depth only.

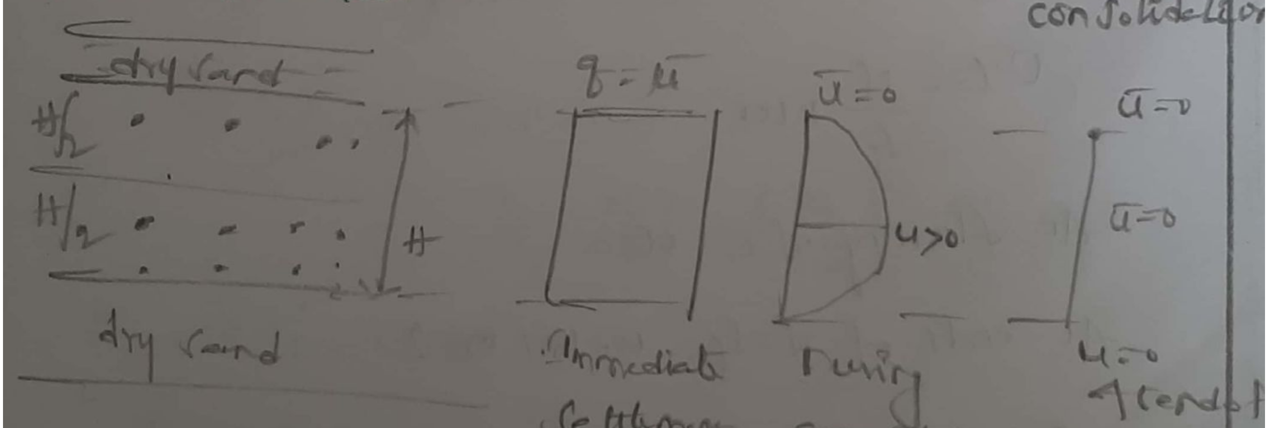
↳ The soil is saturated.

↳ Darcy's law equation. $\boxed{KiA = Q}$

Single - drained: ÷



Double drained: ÷



d → drainage path.

d → single drained.

d → H/2 for double drained.

Terzaghi - 1-Dimensional Consolidation Eq^n:-

$$\boxed{\frac{\partial u}{\partial t} = C_v \cdot \frac{\partial^2 u}{\partial^2 z}}$$

$\frac{\partial v}{\partial t}$ → Rate of change of pore water pressure with respect to time

$\frac{\partial^2 u}{\partial^2 z}$ → Rate of change of pore water pressure with respect to depth.

C_v → Co-efficient of consolidation, $C_v = \frac{k}{m_v \cdot \gamma_w}$.

the solution of for above eq^n give in time of

1) Degree of consolidation (or) settlement (u%)

2) Time factor (T_v).

1) Degree of Consolidation (or) Settlement (u%) :-

$$u\% = \frac{\text{Settlement of any time}}{\text{final settlement } (S_f)} \times 100$$

$$U\% = \frac{S}{S_f} \times 100$$

At starting $S=0$, $u=0$.

At ends of $S=S_f$. $u=100\%$ (or) 1.

Range. $0 \leq U \leq 100\%$ (or) 1.

↳ It is the function of ultimate consolidation, which is completed at any stage of time during consolidation

↳ It is defined only for 1° consolidation process.

↳ At the begining ($t=0$) $\%U$ is 0'

↳ At $t=\infty$ at the end of the primary consolidation. $\%U$ is 100%

2, Time factor (T_v):-

↳ It is the parameter which relates to the degree of consolidation & time required for that consolidation

$$\boxed{T_v = C_v \cdot \frac{t}{d^2}}$$

$$\boxed{C_v = \frac{k}{m_v \cdot \gamma_w}}$$

$d \to$ Length of drainage path

2-way drainage path. $d_2 = \frac{H_0}{2}$

1-way. drainage path $d_1 = H_0$.

$$T_v = \frac{C_v \cdot t}{d^2} \Rightarrow T_v \cdot d^2 = C_v \cdot t$$

$$\frac{t_2}{t_1} = \left(\frac{d_2}{d_1}\right)^2 \Rightarrow \left(\frac{H/2}{H}\right)^2 \Rightarrow \frac{t_2}{t_1} = 1/4$$

$$\Rightarrow t_2 = t_1 \times 1/4.$$

Time taken under double drained $= \frac{1}{4} \times$ Time taken under single drained,

→ The time required for consolidation with double drainage estimate as '8' yr. All other remaining conditions are same. Estimate the time required for single drained.

Sol: Time req. for double drained $t_2 = 8$.

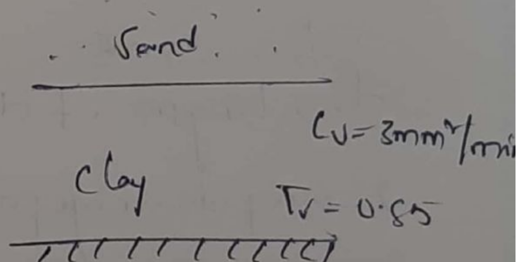
$$t_2 = \frac{1}{4} \times t_1$$

$$t_1 = t_2 \times 4$$

$$t_1 = 8 \times 4$$

$$T_1 = 32\ yr.$$

→ A 3m thick clay layer is subjected to an initial pore water pressure of $145\ kN/m^2$ as shown in fig given ground condition find the time req. for 90% consolidation.



$C_v = 3mm^2/min$

$T_v = 0.85$

for. $H = 3m \Rightarrow 3 \times 10^3 mm$

$$T_v = \frac{C_v . t}{d^2} \Rightarrow t = \frac{T_v . d^2}{C_v}$$

$$t = \frac{0.85 \times (3 \times 10^3)^2}{3}$$

$= 2.55 \times 10^6$ min

$$\left\{ \begin{array}{l} 1 \text{ day} = 24 \times 60 \text{ min} \\ \frac{1}{24 \times 60 \text{ min}} \end{array} \right.$$

$$= \frac{2.55 \times 10^6}{24 \times 60}$$

$t = 1771$ days

Relation ship between Time factor. (T_V) & degree of consolidation ($U\%$).

$$\boxed{T_V = \frac{\pi}{4}\left(\frac{U\%}{100}\right)^2}$$

↳ A saturated clay layer 5m thickness takes 1.5yr for 50% consolidation, when drained both sides if co-eff of volume compressibility is $1.5\times10^{-3}\ m^2/kN$. then find the co-eff of permeability.

Thickness of layer $H = 5m = 5/2 = 2.5m$.

$T = 1.5$ yr

The co-eff. of volume compressibility. $m_v = 1.5\times10^{-3}\ m^2/kN$.

$U\% = 50\%$.

find co-eff of permeability k.

$$C_v = \frac{k}{m_v.\gamma_w} \quad \text{—— (1)}$$

but

$$T_v = \frac{C_v . t}{d^2} \quad \text{—— (2).}$$

We know that:

$$T_v = \frac{\pi}{4}\left(\frac{U\%}{100}\right)^2 \text{—— (3)}$$

$$= \frac{\pi}{4}\left(\frac{U\%}{100}\right)^2 \Rightarrow \frac{\pi}{4}\left(\frac{50}{100}\right)^2$$

$\Rightarrow 0.196.$

$$T_v = \frac{C_v \cdot t}{d^2} \Rightarrow C_v = \frac{T_v \cdot d^2}{t}$$

$$= \frac{0.196 \times (2.5)^2}{1.5}$$

$C_v = 0.816$

from: 1.

$k = C_v \times m_v \times \gamma_w$.

$= 0.816 \times 1.5 \times 10^{-3} \times 10$

$k = 0.0122\ m/yr$

→ Isochrone :-

↳ the graph which represents the variation of Hydrodynamic pore water pressure with respt to depth is called 'Isochrone'

Shear Strength:-

: UNIT-5:

↳ The shear strength of soil is constituted by.

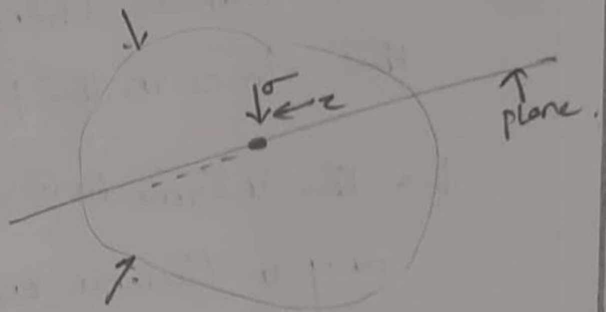
a) Structural resistance due to Interlocking

b) Frictional resistance

c) Cohesion.

σ = Normal stress

τ = shear stress



3 → principal planes on which ($\tau = 0$).

3 → principal stress ($\sigma_1 : \sigma_2 : \sigma_3$)

σ_1 = Major principal stress.

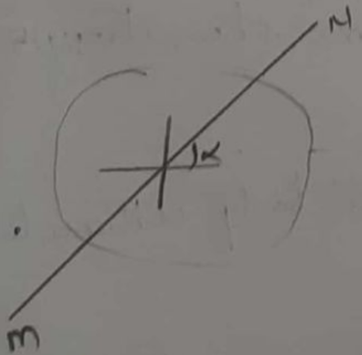
σ_2 = Intermediate " "

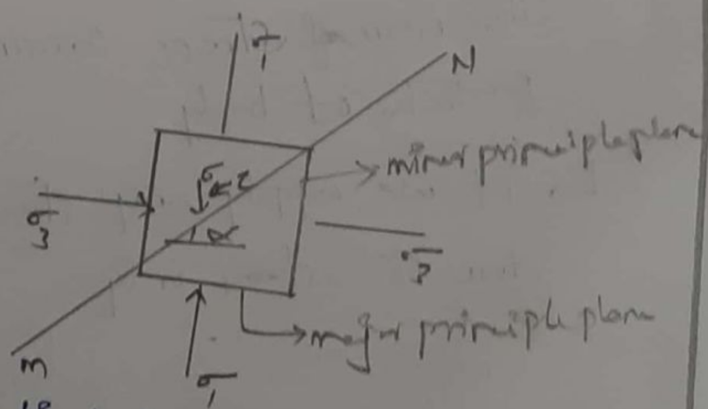
σ_3 = Minor " "

Generally ($\sigma_1 > \sigma_2 > \sigma_3$)

(but always not correct)

→





- Analytical method to find: (σ & τ):-

$$\sigma = \frac{\sigma_1 + \sigma_3}{2} + \frac{\sigma_1 - \sigma_3}{2} \cos 2\alpha \; : \; \tau = \frac{\sigma_1 - \sigma_3}{2} \sin 2\alpha .$$

Mohr's theory :-

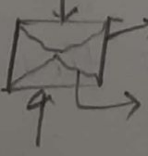
- The material essentially fails due to shear stress.
- The critical shear stress causing failure depends upon the properties of the material as well as normal stress on the failure plane.
- The ultimate strength of the material is determind by the stress in the potential failure plane
- The intermediate (σ_2) principal stress does not have any influence on the strength of the material.

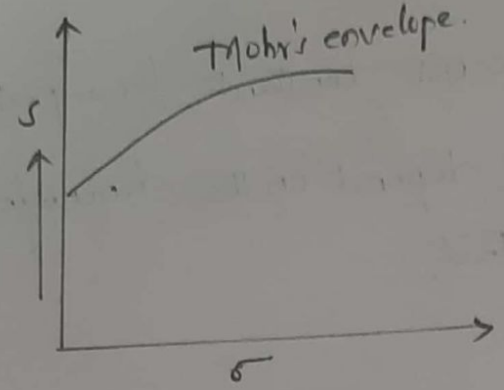
Note:

: in the fig:

- we are applying the normal stress. it fails. but it is not the normal stress causing the any stress failure (flexular, twisting etc)
- it is the shear stress causing the failure i.e, this normal stress causing some shear stress on each particle of body.
- if we apply any stress: the body ultimately fails due to shear only.

if friction occurs at surface





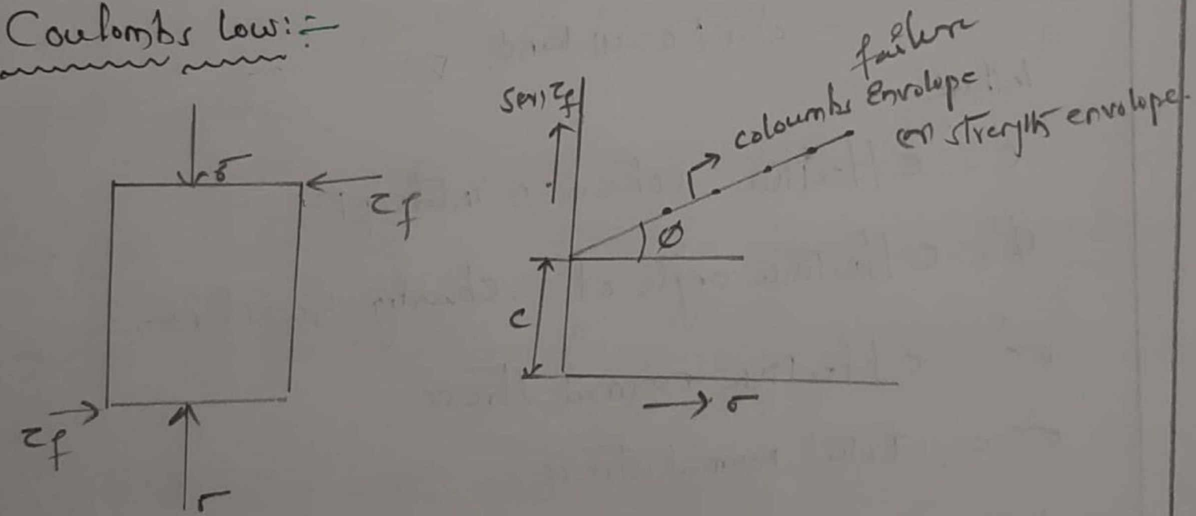
S = f (σ). (→ shear strength is function of Normal stress).

S is a function of Normal stress.

σ_2' have no effect on behaviour of soil.

$\boxed{S = f(\sigma)}$.

Coulombs low:-



$S = c + \sigma \tan\phi$

where: 'c' & φ are empirical constants.

S → shear strength → kN/m2.

C → Cohesion → kN/m².

φ → slope of failure envelope (or) Angle of Internal friction or Angle of shearing resistence

Note:

↳ c' & ϕ are not constant for a particular soil.

↳ In fact they depend on the drainage conditions & test conducted.

Terzaghi's Concept:- (The effective stress principle):

↳ The effective normal stress control the shearing resistance of soil.

Revised Mohr - Coulomb Equation:

$$S = c' + \sigma' \tan\phi$$

$$= c' + (\sigma - u) \tan\phi.$$

Where:

c' = effective cohesion intercept.

ϕ' = effective angle of shearing resistance.

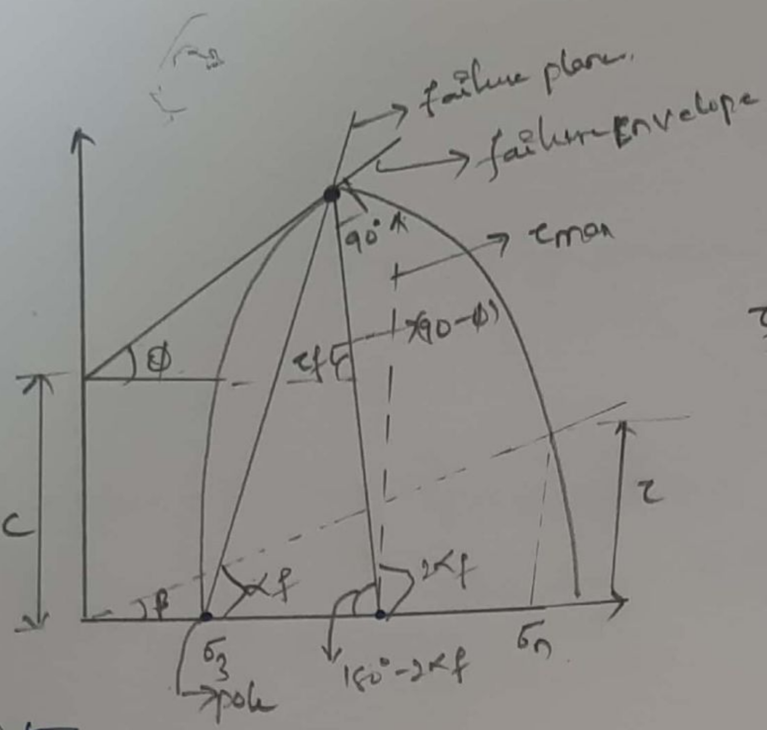
σ' = effective normal stress.

σ = Total normal stress.

for any combination of applied principle effective stress σ_1', σ_3', failure will occure only if the stress circle touches the failure envelope.

β = angle of obliquity.

(pole: origin of planes)



$\tau_f < \tau_{max}$

$(\because \phi > 0)$

Note:

↳ the shear stress at the failure is less than the maximum shear stress.

↳ the failure plane does not carry the maximum shear stress and the plane which is the maximum shear stress is not the failure plane.

(short term shear strength)

↳ for short term condition $S = C_u + \sigma \tan \phi_u$ (undrained condition).

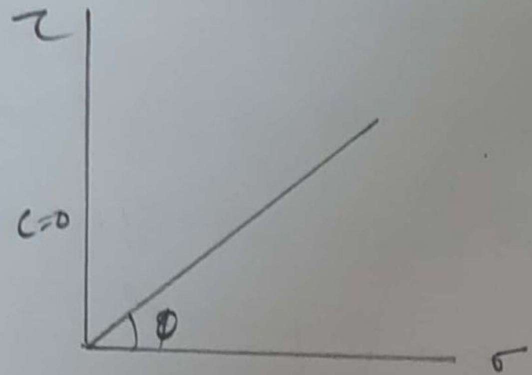
↳ for long term condition $S = c' + \sigma' \tan\phi'$. (for drained condition).

Total pressure the shear strength is

$$S = C_u + \sigma \tan \phi_u$$

C_u = apparent cohesion.

ϕ_u = apparent angle of shearing resistance.

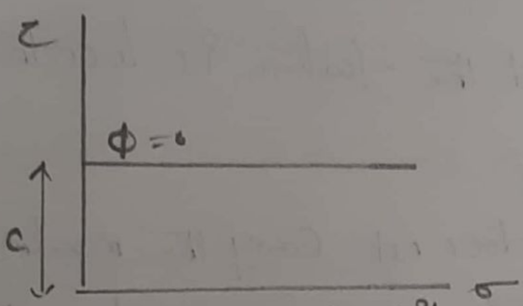


pure frictional soil.

Eg: dry sand - Granular soil

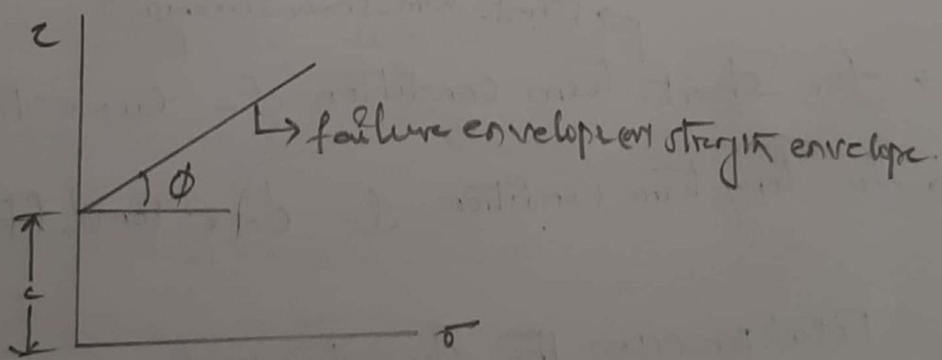
(or)

cohesionless soil.



pure cohesive soil.

Ex: plastic clay



c - φ soil.

ex: 1) clayey sand

2) clayey gravel.

failure envelope:-

↳ locus of point having combination of critical stress

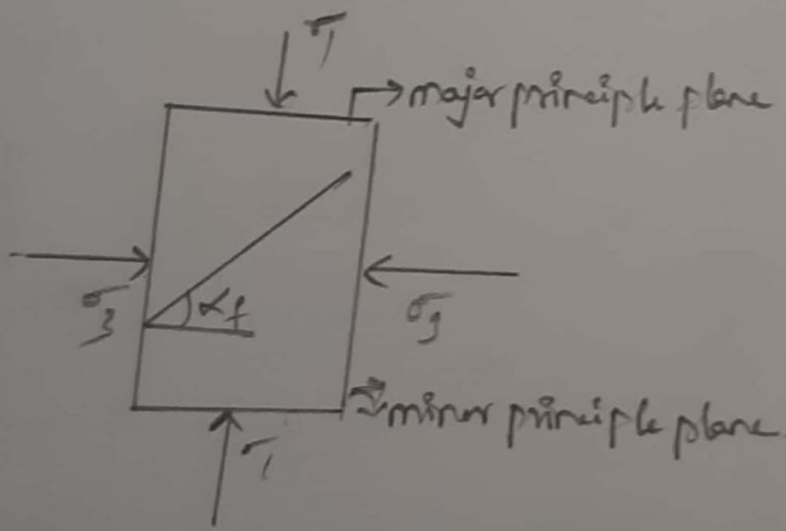
failure plane:-

↳ plane at which the material fails.

Here: $90 - \phi = 180 - 2\alpha_f$

$\Rightarrow \alpha_f = 45 + \phi/2$

** failure plane always makes an angle of $45 + \phi/2$ w.r.to. major principal plane.



$\tau < s \rightarrow$ safe

$\tau \leq s \rightarrow$ critical condition

$\tau > s \rightarrow$ failure

Laboratory test for strength parameters:

↳ Box-shear test / Direct shear test.

↳ Tri-axial shear test

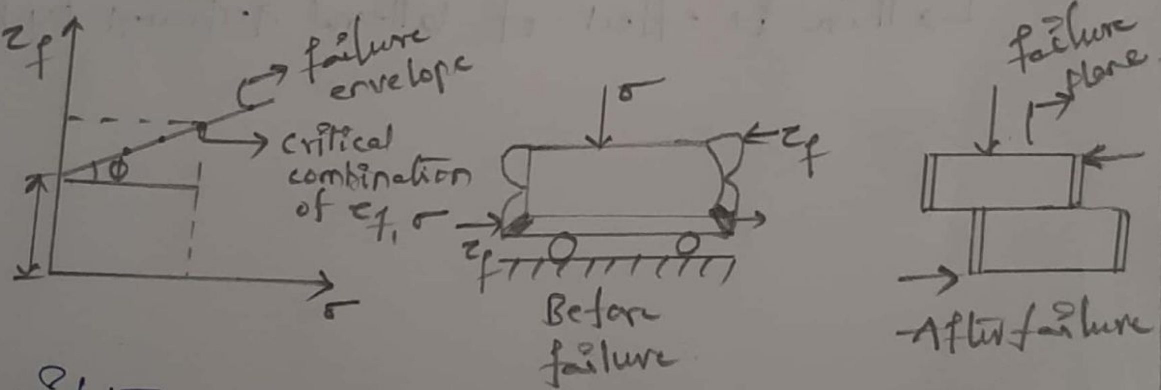
↳ unconfined compression test.

↳ Vane shear test.

* Box-shear test / Direct shear test:-

↳ soil specimen of size 60×60×25mm is used generally.

↳ it is suitable for cohesionless soil as drained test.



↳ if the condition is drained (i.e, drained holes are open) use c', ϕ'

↳ if the condition is undrained (i.e, drained hole is plugged). use. c_u, ϕ_u.

usually $c' \neq c_u$ & $\phi' \neq \phi_u$.

According colombs theory $\boxed{S_{(\sigma)\tau} = c + \sigma' \tan\phi}$

Demerits: -

↳ The stress distribution across the soil sample is very complex.

↳ As the test progresses the area under shear gradually decrease.

↳ The orientation of failure plane is fixed (horizontal). The plane may not be the weakest one.

Little control on drainage of soil.

↳ consequently only drained test can be conducted on highly permeable soil.

↳ Measurement of pore pressure is not possible

↳ There is effect of latteral restraint by the side

A direct shear stress are conducted on cohesionless soil specimen under a Normal stress of 200 kN/m². The specimen of will failed at shear stress of 100 kN/m². then find the angle of internal friction in Degrees.?

Sol:

Normal stress (σ) = 200 kN/m³

$\tau = 100$

$\phi = ?$

$S = C + \sigma . \tan\phi$

$100 = 0 + 200 \tan\phi$

$\frac{100}{200} = \tan\phi$

$\phi = 26.56°$.

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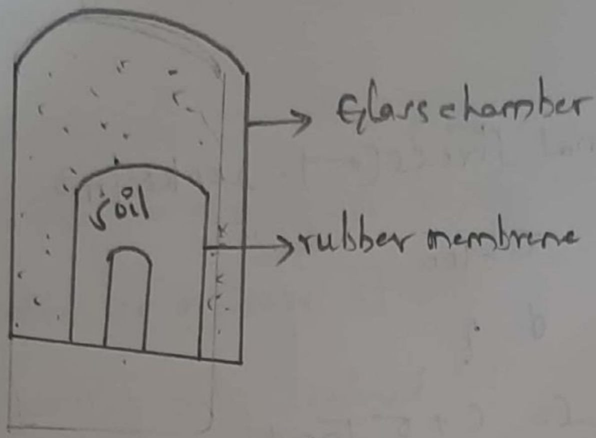
Tri-axial shear strength test:-

↳ Take a cylindrical saturated soil sample of height = 2×dia of the sample.



Stage 1:

↳ All around the soil the confining pressure are Cell pressure (σ_3) is applied using External water pressure.

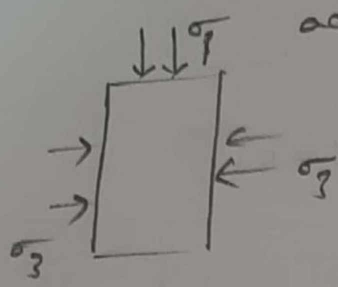
↳ when Expulsion of pore water is stop the first stage is stage completed.

Cell pressure or confining pressure.

Stage: 2:

↳ The confining pressure (σ_3) is kept constant & additional axial stress is applied is called as ~~Divator~~ stress (σ_D)
Deviator.

↳ The ~~deviator~~ deviator stress increase gradually and till soil specimen is failure



$$\boxed{\sigma_1 = \sigma_3 + \sigma_d}$$

σ_d → Deviator stress.

The tri-axial strength is based on mohr's theory

$$\sigma_1 = \sigma_3 \tan^2\alpha + 2c\tan\alpha$$

$$\boxed{\alpha = \phi/_2 + 45}$$

$$\sigma_1 = \sigma_3 \tan^2(45+\phi/2) + 2.c\tan(45+\phi/2).$$

$\sigma_1' = \sigma_3' \tan^2(45+\phi/2) + 2c\tan(45+\phi/2)$ → (According to Terzaghi)

A Triaxial test was conducted on granular soil under drained condition. The soil sample failed when the effective minor principle stress was 150kN/m² and principle effective stress ratio was 4.2 then find ϕ value & Direct stress at failure

↳ effective minor principle stress (σ_3') = 150 kN/m².

principle effective stress $\frac{\sigma_1'}{\sigma_3'} = 4.2$

$\phi = ?$

$\sigma_1' = 4.2 \times 150 = 630$ kN/m²

$$\boxed{\sigma_1 = \sigma_3 + \sigma_d}$$

According to Mohr's theory.

$$\sigma_1' = \sigma_3' \tan^2(45+\phi/2) + 2c\tan(45+\phi/2)$$

$$c = 0$$

$$630 = 150\tan^2(45+\phi/2) + 0$$

$$\frac{630}{150} = \tan^2(45+\phi/2)$$

$$\tan^2(45+\phi/2) = 4.2$$

sq. b.s. side:

$$\tan(45+\phi/2) = \sqrt{4.2}$$

$$\tan^{-1}(\sqrt{4.2}) = 45+\phi/2$$

$$\phi/2 = 63.98-45$$

$$\phi = 37.97.$$

i, $\sigma_1 = \sigma_3 + \sigma_d$

$$\sigma_d = 630-150$$

$$\boxed{\sigma_d = 480\ kN/m^2}$$

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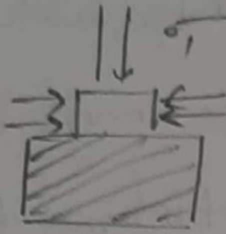
un confined compressive strength test:-



σ_1 Axial stress (or) major principle stress (or) Normal stress

$(\sigma_3 = 0)$

↳ minor principle stress (or) Cell pressure $(\sigma_3 = 0)$

$$\boxed{\sigma_1 = \sigma_3 \tan^2\alpha + 2c\tan\alpha}$$

* → the Cylindrical soil sample is subjected only major principle stress or Normal (or) axial load (σ_1 is acting).

* the cell pressure are minor pressure is $\sigma_3 = 0$.

$$\sigma_1 = \sigma_3 \tan^2\alpha + 2c\tan\alpha$$

$$= 2c\tan\alpha$$

$$= 2c\tan(45 + \phi/2)$$

if soil is clay $(\phi = 0)$

$$\sigma_1 = 2c\tan(45 + \phi/2)$$

$$\boxed{\sigma_1 = 2c}$$

→ it is special form of tri-axial test in which the confining pressure is zero. it can be conducted only on clayey soil. which can with stand without coefficient it is an undrained test.

$\sigma_1 = 2c_u \tan(45° + \phi_u/2)$ here: $\sigma_1 = q_u$.

q_u = unconfined compression strength for pure clays - $\phi_u = 0$, $q_u = 2c_u$

$c_u = (q_u/2)$.

when confined compression strength is conducted on cylindrical soil sample it fail under axial stress of 1.2 kg/cm². The failure plane makes an angle of 50° with horizontal plane the find Angle of internal friction & cohesion c,

- Axial stress $(\sigma_1) = 1.2$ kg/cm²

The failure plane makes angle $\alpha = 50°$

$$\sigma_1 = \sigma_3 \tan^2\alpha + 2c\tan\alpha$$

$$\boxed{\alpha = 45 + \phi/2}$$

$$50 - 45 = \phi/2$$

$$5 = \phi/2$$

$$\boxed{\phi = 10°}$$

$$\sigma_1 = 2c\tan\alpha$$

$$\frac{\sigma_1}{2\tan\alpha} = c$$

$$= \frac{1.2}{2\times\tan(50)} = c$$

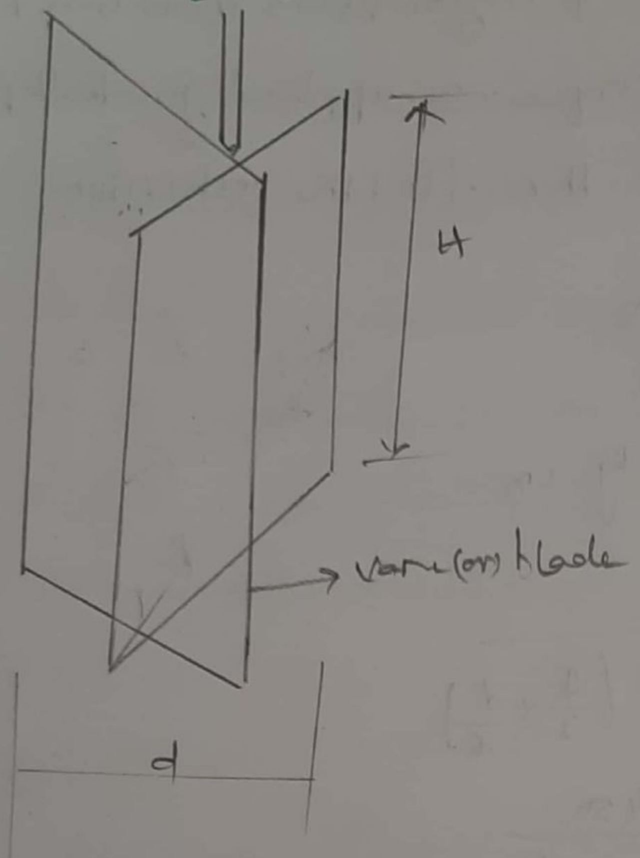
$$\boxed{c = 0.503 \text{ kg/cm}^2}$$

The r/Dlw. soil cohesion c & unconfined compression strength of soil

$$q_u = 2c_u$$

$$c_u = q_u/2$$

Vane shear test:-



↳ To find the shear strength of clay

$$\boxed{C_u = \frac{T}{\pi d^2 \left(\frac{H}{2} + \frac{d}{6}\right)}}$$

↳ Quick test to determine the undrained shear strength of soft clay in the lab or field.

↳ when both ends part take in shearing

C_u → undrained cohesion.

H → Height of the Vane

d → dia of the Vane

T → Torsion

units → kN/m².

The Vane 10cm & height & 8cm India was pressed into the clay. The torque was applied gradually increased. To 450 kg-cm. Then find the cohesion.

$H = 10cm$

$d = 8cm$

$T = 450$ kg-cm

$$C = \frac{T}{\pi d^2 \left(\frac{H}{2} + \frac{d}{6}\right)}$$

$$C = \frac{450}{\pi \times (8)^2 \left(\frac{10}{2} + \frac{8}{6}\right)}$$

$C = 0.353$ kg-cm.

Application:

↳ for sudden draw down causes.

C. Consolidated - Drained Test :-

↳ Drainage is permitted through out the test during the application of both normal and shear test so that full consolidation occurs and excess pore pressure is setup any stage of test. Generally takes 2 to 5 days.

Application :-

↳ cohesionless soils and for soils having high permeability

↳ for ascertaining long term stability of clays.

Sensitivity :-

↳ it can be easily determined using the unconfined compression test.

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

if sensitivity = 1 ⇒ insensitive soil

if sensitivity > 16 ⇒ quick soil.

Depending upon the drainage condition:-

↳ There are three types:

↳ 1, Unconsolidated-Undrained Test (or) Quick Test (Q-test (or) UU-test):

↳ 2, Consolidated undrained test (CU-Test).

↳ 3, Consolidated drained test (or) slow test

* Unconsolidated-undrained test:-

↳ No drainage is permited either during consolidation stage or shear stage. therefore. there is no dissipation of pore pressure during the test & test can be conduct in few minutes (5-10).

- Application:

- Generally done for less permeable soil such as clays for short term stability analysis and also for earth dam during construction.

Consolidated. undrained test:-

↳ Drainage is permitted under the initially applied normal stress only and fully primary consolidation (or) Softning is allowed to take place. No drainage allowed in shear stage.

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Liquefaction:-

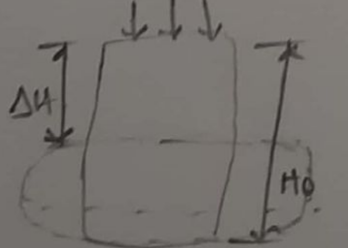
↳ when saturated sandy soil is subjected to earth quakes loads, the pore pressure suddenly increases and thus decreases the shear strength of soil and it may become zero also.

↳ the soil momentarily liquefies and behaves as dense fluid. this phenomena when sand loses it is shear strength due to oscillatory motion is called liquefaction of sand.

↳ the soil most susceptable to liquefaction are saturated fine sand and saturated medium sand.

Dilatency:-

↳ the phenomenon of increase in volume of soil during the shear is called Dilatency.



Critical Void ratio :-

↳ Critical void ratio is defined as the ratio at which there is no change in sand. with an increase in strain.

↳ The properties of change in volume rely upon different factor such as shape, grain size, principle stress

↳ The representation of shear strain versus void ratio are shown below in the graph.