-: SUBJECT :-

Soil Mechanics

Soil Soil is defined as the solid particles formed by disintegration of Rock. These particles contains voids and the voids may be

(i) Air voids

(ii) Water void

(iii) Water and Air voids

Types of Soil on the basis of nature of Voids

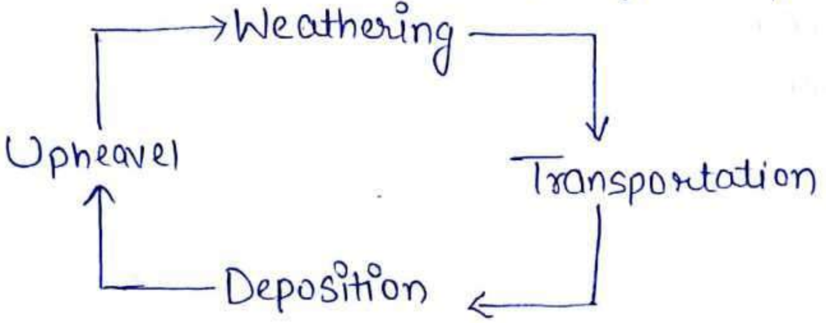
- Solids + Air Voids → Dry soil
  Solid + Water voids → fully Saturated Soil
  Solid + Water Voids + Air Voids → Partially Saturated Soil

Soil Mechanics :- • Dr. Karl Terzaghi is the father of Soil Mechanics

- Soil is the application of laws and Hydraulics to the solid particles in order to study thiir engg.

formation of soil :- The formation of soil is known as Pedogenesis

- In this process, formation of geological cycle takes place..



Weathering | Erosion :- The process of disintegration of Rocks by Physical or Chemical Reason is known as weathering.

1. Physical Weathering / Mechanical Weathering

- Splitting Action of Wind, water, ice
- Siesmic Vibration
- Growth of Plants
- Periodical temp. change [ice ⇌ water]

Eg :- Conesionless Soil (Sand, Gravels)

⟨2⟩ chemical Weathering

(i) $\overset{O_2}{\text{Oxidation}}$ & $\overset{H_2}{\text{Reduction}}$

(ii) Carbonation [Reaction with CO_2]

(iii) Hydration [Reaction with H_2O]

(iv) Hydrolysis [Reaction with OH^-]

(v) Leaching Action [Reaction with Acids]

Types of Soil

⟨1⟩ On the Basis of Particle Size

(i) Coarse Soil - Particle size >0.075m Eg- Sand, Gravels

(ii) fine Soil - Particle size <0.075m Eg- Silt, clay

⟨2⟩ On the basis of Transportation of Particles

(i) Residual Soil

(ii) Transported Soil

(i) Residual Soil :- The soil which Remains over parent Rock directly is known as Residual soil.

(ii) Transported Soil :- The soil which doesnot remain over parent Rock is known as transported soil.

- Wind, Water, ice and glacier are known as transported agen
- On the basis of movement of particles the transported so is divided into no. of Soil Deposits.

Soil Deposits :-

(i) Aeolian Soil Deposit - formed by wind Movement

(ii) Alluvial Soil Deposit - formed by water Movement

(iii) Colluvial Soil Deposit - formed by Gravity

(iv) Glacial Drift - formed by Ice/Glacier

(v) Lacustrine Soil Deposit - formed at bottom of still water b (lake, ponc

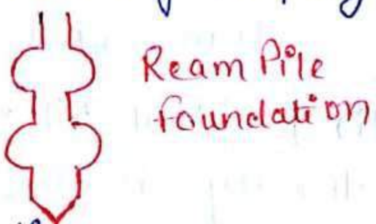
(vi) Marine Soil Deposit - formed at the coastal Regions

Cumulose Soil Deposit :- The soil Deposit which contains Organic matter is known as Cumulose Soil Deposit. This deposit is of 3 three types :-

1. **Muck** :- The cumulose Soil Deposit which contains fully Decompsed organic matter.
2. **Peat** :- The Cumulose Soil Deposit which contains partially decompesed organic matter with considerable thickness.
3. **Humus** :- The culmulose soil deposit which contains partially decomped organic matter of negligible thickness.

4. **Black Cotton Soil** :- It has high shrinkage and swelling property

- found in central part of India.
- Inorganic clay
- Content ~~in~~ of Nitrogen.
- Under Ream Pile foundation, used in construction.



- This soil is black in colour and good for cotton crops.
- It is a type of inorganic clay.
- It has high shrinkage and swelling properties.
- It has lower shear Strength.
- Under Reamed Pile foundation are used for construction in Black Cotton Soil.

IMPORTANT TERMS

1) **Oxidation** :- In this process, oxygens reacts with a substance ~~With~~ in rock and forms oxides which ~~the~~ will result in weathering. The most common example of Chemicacal weathering is rusting of Iron. Water has ~~CO_2~~ O_2. Any rock having iron minerals Undergoes a slow breakdown this way.

(2) **Reduction** :- The process of Removal of oxygen and is th
Riverse of oxydation and is the Reverse of Oxidation an
is equally important in changing soil colour to grey, blue or
green as ferric iron is converted into ferrous iron
compound. Under the ~~action~~ cond^{n} of excess water or water logged
condition (less or no oxygen), Reduction takes place.

(3) **Carbonation** :- is the process of Rock Minerals reactin
with Carbonic acid. Carbonic acid is formed when wat
combines with CO_2. Carbonic acid dissolves or break dow
minerals in Rock.

$$CO_2 + H_2O \rightarrow H_2CO_3 \text{ (Carbonic Acid)}$$

$$H_2CO_3 + CaCO_3 \rightarrow Ca^{2+} + 2HCO^{3+}$$

(carbonic Acid) + calcite → calcium + bicarbonate

(4) **Hydrolysis** :- Hydrolysis is a chemical Reaction caused by
Water. Water changes the chemical composition and size
Minerals in Rock, Making them less Resistant to Weathe

(5) **Hydration** :- is the absorption of Water into the mi
structure., Resulting in formation of Gypsum. Hydration
expands (increase) volume and also Result in Rock Defor
-mation. [Increase in vol. make soil ~~in~~ result in
less stability]

1. **Marl** :- Marine soil which contains some content of $CaCO_3$, is known as Marl.

2. **Loess** :- It is silt deposit which is transported by wind movement.

3. **Sand Dune** :- It is a sand deposit formed by wind movement.

- Stability of loess is less than stability of Sand Dunes.

4. **Collapsible Soil** :- The soil which cannot be converted into any shape and the deposit has less stability is known as collapsible soil.
Eg. loess and Sand Dunes.

5. **Talus** :- The soil particles present in colluvial soil Deposit are known as talus.
(the soil formed by Gravity)

6. **Till** :- It is a mixture of gravels, Sand, silt and clay which is formed by glacier/ Ice movement.

7. **Bentonite clay** : It is a volcanic ash formed by chemical weathering

8. **Tuff** :- The volcanic ash when transported by wind or water, is known as tuff.

9. **Loam** :- [Clay + Mixture 80%] :- Loam is a mixture of soil which contains min^m 80% clay content.

- Loam is the best soil for growth of plants and having max^m porosity.

PROPERTIES OF SOIL

1. **Mass** : Mass represents the content [no. of Molecules] of a body.
Mass is always constant
Units : gm, kg.

Weight : It is the amount of force exerted by the given mass of a body.
Wt. is a variable quantity which depends upon acceleration due to gravity.

$\boxed{W = mg}$ **units**: N, kN

① Basic Properties

i) Mass Density

$$\boxed{\rho = \frac{M}{V}}$$

Units :- g/cm3, kg/m3, etc.

(ii) Bulk Density

$$\rho_{Bulk} = \frac{M_{bulk}}{V}$$ (for Partially saturated soil)

(b) Dry Density

$$\boxed{\rho_d = \frac{M_d}{V} = \frac{M_{solid}}{V}}$$ (Air voids are negligible) (for dry soil) $\boxed{M_d = M_A + M}$

(c) Saturated Density

$$\boxed{\rho_{sat} = \frac{M_{sat}}{V}}$$ (fully saturated Soil)

(d) Submerged Density

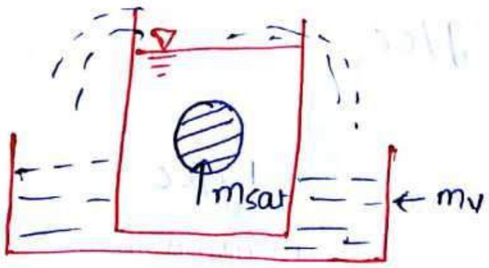
$$\rho' = \frac{M_{sub}}{V}$$

$$\rho' = \frac{M_{sat} - M_w}{V}$$

$$\rho' = \frac{M_{sat}}{V} - \frac{M_w}{V}$$

$$\boxed{\rho' = \rho_{sat} - \rho_w}$$

ρ_w = Density of water
= 1 g/cc (at 4°c tem



$$\boxed{m_{sub} = m_{sat} - m_w}$$

$$\boxed{\rho_{sat} > \rho_{Bulk} > \rho_d > \rho'}$$

$\rho_{sat} > \rho_{Bulk} > \rho_d > \rho'$

Ques. A saturated Soil has mass of 190 gm and vol. of 100 cc after drying of soil its mass gets Reduced to 160 gm. Calculate :-

(i) Saturated Density

(ii) Dry Density

(iii) Submerged Density

Sol

(i) $\rho_{sat} = \frac{M_{sat}}{V} = \frac{190}{100} = 1.9$ g/cc

(ii) $\rho_d = \frac{M_d}{V} = \frac{160}{100} = 1.6$ g/cc

(iii) $\rho' = \rho_{sat} - \rho_w$

$\rho' = 1.9 - 1 = 0.9$ g/cc

Ques 2:- Consider the given observations from a soil sample and calculate Bulk Density, dry Density and Saturated Density of soil.

components	Mass (g)	vol. (cm3)
Air	—	0.2
Water	0.3	0.3
Solids	1.0	0.5

(i) $\rho_{Bulk} = \frac{1+0.3+0}{0.5+0.3+0.2} = 1.3$ g/cc

(ii) $\rho_d = \frac{M_{solid}}{V} = \frac{1}{0.5+0.3+0.2} = 1$ g/cc

(ii) $\rho_{sat} = \frac{M_{sat}}{V} = \frac{1+0.3+\underline{0.2}}{0.5+0.3+0.2} = 1.5$ g/cc

additional water instead of Air voids.

2. Weight Density or Unit Weight

$$\boxed{\gamma = \frac{W}{V}}$$

Units:- N/cm3, kN/m3 etc.

(a) Bulk Unit Wt.

$$\boxed{\gamma_{Bulk} = \frac{W_{bulk}}{V}}$$

(b) Dry Unit Wt.

$$\boxed{\gamma_d = \frac{W_{solids}}{V}}$$

(c) Saturated Unit Wt.

$$\boxed{\gamma_{sat} = \frac{W_{sat}}{V}}$$

(d) Submerged Unit Wt.

$$\gamma' = \frac{W_{sub}}{V} = \frac{W_{sat} - W_w}{V}$$

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

γ_w = Unit wt. of water

= 9.81 kN/m3 ≃ 10 kN/m3

$$\boxed{\gamma = \rho g} \qquad \boxed{\begin{array}{l}\gamma = kN/m3 \\ \rho = g/cc\end{array}}$$

Concept

$$F = ma = mg$$

$$1N = kg \cdot m/s^2$$

$$g \xrightarrow{\times 10^{-3}} kg \xrightarrow{\times \overset{g}{\underline{9.81}}} N \xrightarrow{\times 10^{-3}} kN$$

$$x \frac{g}{cm3} = \frac{x \times \cancel{10^{-3}} \times 9.81 \times \cancel{10^{-3}}}{\cancel{(10^{-2})}^{3}}$$

$$\boxed{x\, g/cm3 = x \times 9.81\ kN/m3}$$

1 cm = 10^{-2} m

1 cm3 = $(10^{-2})^3$.

Note :- **Density of solids** :- It is the Ratio of M_{solids} to V_{solids}

$$\boxed{\rho_{solids} = \frac{M_{solids}}{V_{solids}}}$$

Density of solid is always greater than Dry Density of soil

$$\boxed{\rho_d = \frac{M_{solids}}{V}}$$

$\{ V \rightarrow$ Total vol. $\}$
$\{ V\uparrow \rightarrow \rho\downarrow \}$

$$\# \ \boxed{\rho_{solids} > \rho_{dry}}$$

(3) **Specific Gravity of solids** = :- (Temp 27°C) ⇒ Measured at this temp.

- It is defined as the Ratio of Mass of solids to the Mass of water at constant volume. Both are measured at constant temp. of 27°C.

$$\boxed{G_s = \frac{m_{solids}}{m_{water}}}$$

$$G_s = \frac{m_{solids}}{m_{water}} \times \frac{V}{V}$$

$$\boxed{V_{solids} = V_w = V}$$

$$\boxed{G_s = \frac{\rho_{solids}}{\rho_{water}}}$$

- Cohesionless → 2.65 – 2.68
- Cohesive soil → 2.65 – 2.70
- Average value → 2.67
  (if not given)

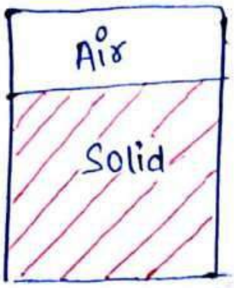
Units :- Unit less.

- **Bulk Specific Gravity** / **Mass Specific Gravity** =

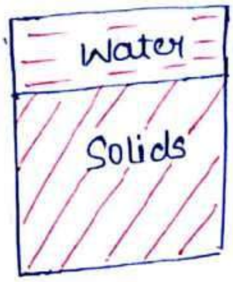
It is Ratio of Bulk Density of soil to Density of water

$$\boxed{G_m = \frac{\rho_{bulk}}{\rho_{water}}}$$

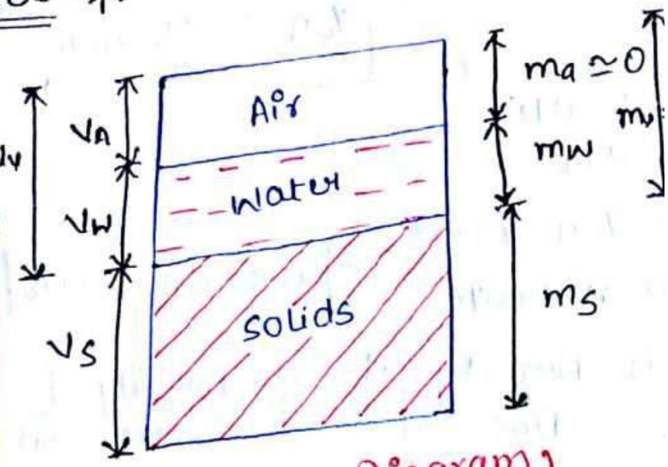
✳ PHASE PROPERTIES ✳



Dry Soil
[2-phase Diagram]



fully saturated Soil
(2-phase Diagram)



(3-phase Diagram)

1. Water Content (W%) :-

$$\boxed{\% W = \frac{m_W}{m_S} \times 100}$$

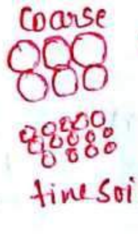
- Unitless
- Values

(i) $\% W = 0$ [for oven-dried Soil]

(ii) $\% W \geq 0$ [It has no upper limit]

(iii) $\% W = 4\%$ [for Air-Dried Soil]

The water content at which soil becomes fully saturated, is known as saturated water Content.



Total no.o vDide in fine so greater of coa

2. Void Ratio (e) :-

$$\boxed{e = \frac{V_V}{V_S}}$$

- Unitless
- Value

$e \neq 0$

$e > 0$ [It has no upper limit]

Void Ratio of fine soil > void Ratio of Coarse Soil

Void Size of fine soil < Void Size of coarse soil

3. Porosity or % voids :-

$$\boxed{\% n = \frac{V_v}{V} \times 100}$$

- Unitless
- Value

(i) $\% n \neq 0$

(ii) $n < 100\%$

$$\boxed{0 < \% n < 100\%}$$

(iii) Porosity of fine soil > Porosity of Coarse soil

- Porosity Represents water storage capacity of soil

4. Relation b/w Void Ratio and Porosity

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

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

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

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

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

***

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

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

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

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

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

***

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

Ques1. If void Ratio of soil is 0.5. calculate porosity of soil

$e = 0.5$

$n = ?$

$n = \frac{e}{1+e} \Rightarrow n = \frac{0.5}{1+0.5}$

$n = \frac{0.5}{1.5} = \frac{1}{3}$ or $.33$

$\boxed{n = 33.3\%}$

Ques2. If porosity of soil is 40%.. calculate void Ratio

$n = 40\%$

$e = ?$

$e = \frac{n}{1-n}$, $e = \frac{0.4}{1-0.4} = \frac{0.4}{0.6} = \frac{2}{3}$

$\boxed{e = 0.67}$

Ques3. If vol. of voids V_v becomes equal to volume of Solids V_s. Calcu
void Ratio or Porosity.

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

$\boxed{e = 1}$

$n = \frac{e}{1+e} = \frac{1}{1+1} = \frac{1}{2}$

$\boxed{n = 0.5}$ $\boxed{n = 50\%}$

Q4 If vol. of Voids (V_v) become equal to twice of V_s . then
calculate Void Ratio and Porosity.

$e = \frac{V_v}{V_s} = \frac{2V_s}{V_s} = 2$

$\boxed{e = 2}$

$n = \frac{e}{1+e} = \frac{2}{3} = .6667$

$\boxed{n = 0.66.67\%}$

Q5. If volume of air is $\frac{1}{6}$th of total volume and volume of water is $\frac{1}{3}$rd of total volume. Calculate void Ratio and Porosity.

Given:- $V_a = \frac{1}{6}^{th}$ of total vol. $\Rightarrow V_a = \frac{V}{6}$

$V_w = \frac{1}{3}^{rd}$ of Total vol. $\Rightarrow V_w = \frac{V}{3}$

$e = ? \quad n = ?$

$$V_v = V_a + V_w$$

$$V_v = \frac{V}{6} + \frac{V}{3} \Rightarrow \frac{2V + V}{6} = \frac{3V}{6}$$

$$\boxed{V_v = \frac{V}{2}}$$

$$V_s = V - \frac{V}{2} \Rightarrow \boxed{V_s = \frac{V}{2}}$$

$$e = \frac{V_v}{V_s} = \frac{\frac{V}{2}}{\frac{V}{2}} = 1$$

$$\boxed{e = 1}$$

$$n = \frac{e}{1+e} = \frac{1}{1+1} = \frac{1}{2} = 50\%$$

$$\boxed{n = 50\%}$$

2nd Method

$$n = \frac{V_v}{V} = \frac{\frac{V}{2}}{V} = \frac{1}{2} \text{ or } 50\%$$

$$\boxed{n = 50\%}$$

$$e = \frac{n}{1-n} = \frac{0.5}{1-0.5} = 1$$

$$\boxed{e = 1}$$

Q6. If W represents total Mass of soil and W_s reprents mass of soil Solids, then what will be correct option for water Content

(a) $\left(\frac{W}{W_s}+1\right)100$

(c) $\% w = \left(\frac{W_s}{W}+1\right)100$

[✓ (b) $\left(\frac{W}{W_s}-1\right)100$]

(d) $\% w = \left(\frac{W_s}{W}-1\right)100$

Sol Mass of water $= W - W_s$

$$W\% = \left(\frac{W-W_s}{W_s}\right)100$$

$$= \left(\frac{W}{W_s}-\frac{W_s}{W_s}\right)100$$

$$\boxed{W\% = \left(\frac{W}{W_s}-1\right)100}$$

4. Degree of Saturation :-

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

- Unitless
- Values

(a) for Dry soil $\boxed{V_w = 0} \Rightarrow \boxed{S = 0}$

(b) for fully saturated soil $\boxed{V_a = 0}$ & $\boxed{V_w = V_v}$

$$\boxed{S = 1}$$

(c) for Partially saturated soil

$$\boxed{0 < S < 1}$$

Note:- Degree of Saturation cannot be determined in case Super saturated or Submerged soil.

Degree of Saturation	Soil Type
0	Dry Soil
0 - 0·25	Humid Soil
0·25 - 0·5	Damp soil
0·5 - 0·75	Moist Soil
0·75 < S < 1	Wet Soil
= 1	fully saturated Soil

Partially saturated Soil

5. Air content :-

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

- Unitless
- Value

(i) for fully saturated soil

$V_a = 0$; $\boxed{a_c = 0}$

(ii) for Dry Soil

$V_w = 0$ & $\boxed{V_v = V_a}$; $\boxed{a_c = 1}$

(iii) Partially Saturated Soil

$\boxed{0 < a_c < 1}$

(iv) Overall value

$\boxed{0 \leq a \leq 1}$

$$S = \frac{V_w}{V_v} \quad , \quad a_c = \frac{V_a}{V_v}$$

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

$$= \frac{V_w + V_a}{V_v} = \frac{V_v}{V_v} = 1$$

***

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

6. % Air voids :-

$$\boxed{\% n_a = \frac{V_a}{V} \times 100}$$

- Unitless
- Values

(i) $\% n_a = 0$ (fully saturated soil)

(ii) $\% n_a \neq 100\%$

$$\boxed{0 \leq \% n_a < 100\%}$$

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

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

Q.7 If air content of a soil is 40%, then identify the types of soil.

(a) Humid (b) Damp [✓ (c) Moist] (d) wet

Sol :-

$a_c + S = 1$

$0.4 + S = 1$

$S = 1 - 0.4$; $\boxed{S = 0.6}$ Range lies b/w $\boxed{.5 - .7}$

So the soil is Moist.

Q8 Porosity of a soil is 75% and %age air voids are 25%. Calculate Degree of Saturation.

$\% n = 75\%$

$\% n_a = 25\%$

$S = ?$

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

$0.25 = a_c \times 0.75$

$a_c = \frac{0.25}{0.75} = \frac{1}{3}$

$\boxed{S + a_c = 1} \Rightarrow S + \frac{1}{3} = 1 \Rightarrow S = 1 - \frac{1}{3} = \frac{2}{3}$

$\boxed{S = 0.67}$ Ans

Q9. In a Soil Sample the vol. of air is $\frac{1}{4}$th of Total vol. anc vol. of water is $\frac{1}{2}$ of total volume. calculate void Ratio & degree of Saturation.

$V_a = \frac{1}{4}V$, $V_W = \frac{1}{2}V$

$V_S = V_s = \frac{1}{4}V - \frac{1}{2}V = \frac{1}{4}V$ $\quad e = \frac{1}{4}$

$V_V = \frac{V}{4} + \frac{V}{2} = \frac{3}{4}V$

$\boxed{e = \frac{V_V}{V_S}} = \frac{3}{4}\cancel{V} \times \frac{\cancel{4}}{\cancel{V}}$

$\boxed{e = 3}$ Ans.

$S = \frac{V_W}{V_V} = \frac{V}{\cancel{2}} \times \frac{\cancel{4}^{2}}{3V} = \frac{2}{3}$

$\boxed{S = 0.67}$ Ans.

Relation b/w Different Properties

(1) $\boxed{e = \frac{n}{1-n}}$; $\boxed{n = \frac{e}{1+e}}$

(2) $\boxed{Se = wG}$ (Sehwag) [Se wGe] Sewage.

(3) $\rho_d = \frac{\rho}{1+w}$; $\gamma_d = \frac{\gamma}{1+w}$

(4) $\rho = \frac{(G+Se)}{1+e}\rho_w$; $\gamma = \frac{(G+Se)\gamma_w}{1+e}$

Dry Soil

$\boxed{S=0}$ $\quad \rho_d = \frac{G\rho_w}{1+e}$

fully Saturated Soil

$\boxed{S=1}$ $\quad \rho_{sat} = \frac{(G+e)}{1+e}\rho_w$

When $\boxed{S=1}$

$$\boxed{e = w_{sat} G}$$

$$\boxed{\rho_d = \frac{\rho_{sat}}{1 + w_{sat}}}$$

$g/cc \rightarrow \rho \longrightarrow \rho_w = 1 g/cc$

$kN/m3 \rightarrow \gamma \longrightarrow \gamma_w = 9.81 \ kN/m3$

$\simeq 10 \ kN/m3$

Q10. A soil has bulk Density of 22 KN/m3. and 10% water Content. Calculate Dry Density of soil.

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

$$= \frac{22}{1+0.1} = \frac{22}{1.1}$$

$$\boxed{\gamma_d = 20 \ kN/m3}$$

Q11 A soil has saturated water content of 10%. Calculate void Ro of soil.

fully saturated soil $\boxed{S = 1}$

$$Se = wG$$

$$1 \times e = 0.10 \times 2.67$$

$$\boxed{e = 0.267}$$

Q12. A soil has water content of 50% and degree of saturation o 75%. calculate void Ratio of soil. If Sp. gravity is 2.7

$$Se = wG; \quad 0.75 \times e = 0.5 \times 2.7$$

$$e = \frac{0.5 \times 2.70}{0.75}; \quad \boxed{e = 1.8}$$

Q13. The porosity of a soil is 40%. calculate
(i) Dry Density of soil (ii) Saturated Density of soil
(iii) Bulk Density of soil if saturation is 50%. [Take G = 2.7)

$$e = \frac{n}{1-n} = \frac{0.4}{1-0.4} = 0.67$$

(i) $$\rho_d = \frac{G\rho_w}{1+e} = \frac{2.670 \times 1}{1+0.67} \quad \boxed{S = 0}$$

$\boxed{\rho_d = 1.616}$ g/cc Ans.

(ii) $$\rho_{sat} = \left(\frac{G+e}{1+e}\right)\rho_w = \left[\frac{2.7 * .67}{1+.67}\right] \times 1 = \boxed{\rho_{sat} = 2.01 \ g/cc}$$

(iii) $S = 05$

$$\rho = \left(\frac{G + Se}{1+e}\right)\rho_w = \left(\frac{2.7 + 0.5 \times 0.67}{1+0.67}\right)1 = 1.81\ g/cc$$

$\boxed{\rho = 1.81 g/cc}$ Ans

Q14 The vol. of soil is 1500 m3. If void Ratio is 0.8, calculate. The vol. of same soil if the void Ratio is increased to 1.4.

Sol:- $\boxed{V_s = \text{constant}}$

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

$$0.8 = \frac{1500 - V_s}{V_s} \Rightarrow 0.8V_s = 1500 - V_s$$

$$\boxed{V_s = \frac{1500}{1.8}}$$

$V = ?$; $e = 1.4$. $\Rightarrow e = \frac{V_v}{V_s} = \frac{V - V_s}{V_s}$

$$1.4 = \frac{V - \frac{1500}{1.8}}{\frac{1500}{1.8}} \Rightarrow 1.4 \times \frac{1500}{1.8} = V - \frac{1500}{1.8}$$

$$V = 1.4 \times \frac{1500}{1.8} + \frac{1500}{1.8}$$

$\boxed{V = 2000 m3}$ Ans.

2nd Alternate Method

$V = 1500 m3$ if $e = 0.8$
$V = ?$ if $e = 1.4$

$$e = \frac{V - V_s}{V_s} = eV_s = V - V_s \Rightarrow eV_s + V_s = V \Rightarrow V_s(1+e) = V$$

$$\boxed{V_s = \frac{V}{1+e}}$$

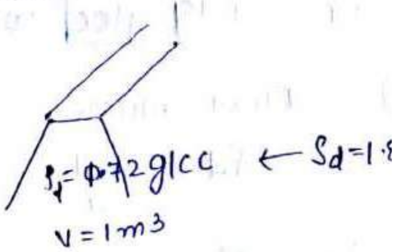
$\boxed{V_s = \text{Constant}}$

$$V_{s_1} = V_{s_2}$$

$$\frac{V_1}{1+e_1} = \frac{V_2}{1+e_2}$$

$$\frac{1500}{1+0.8} = \frac{V_2}{1+1.4} \Rightarrow \boxed{V_2 = 2000 m3}$$ Ans.

Ques 15. An earthen Embankment is compacted to a Dry Density of 1.72 g/cm3. The soil is carried from a borrow pit having dry density of 1.82 g/cc. Calculate the volume of excavation Required in a borrow pit for 1 cu-m earthern embankment.



Sol:-

$$\rho_{d_1} = \frac{G\rho_w}{1+e_1}$$

$$1.72 = \frac{2.67 \times 1}{1+e_1}$$

$$1+e_1 = \frac{2.67}{1.72}$$

$$e_1 = \frac{2.67}{1.72} - 1$$

$$e_1 = 0.55$$

$$\rho_{d_2} = \frac{G\rho_w}{1+e_2}$$

$$1+e_2 = \frac{2.67 \times 1}{1.82}$$

$$e_2 = \frac{2.67}{1.82} - 1$$

$$\boxed{e_2 = 0.46}$$

$$\frac{V_1}{1+e_1} = \frac{V_2}{1+e_2}$$

$$\frac{1}{1+0.55} = \frac{V_2}{1+0.46} \Rightarrow \boxed{V_2 = 0.94m^3} \text{ Ans}$$

Alternate Method

V_s = const. ∵ m_{solids} = Constt.

$$\rho_d V = \text{const}$$

$$\rho_{d_1} V_1 = \rho_{d_2} V_2$$

$$1.72 \times 1 = 1.82 \times V_2 \Rightarrow V_2 = \frac{1.72}{1.82} \quad \boxed{V_2 = 0.94 m^3} \text{ Ans.}$$

Q16. The void Ratio of a soil is 0.7 and soil is 50% saturated Calculate porosity, water content, Dry Density and Bulk Dens Take $\boxed{G = 2.7}$. How much water can be increase such that d density of soil Remains constant.

(i) $$n = \frac{e}{1+e} \Rightarrow \frac{0.7}{1.7} = 0.411 = \boxed{41.17\% = n}$$

(ii) $$Se = wG \Rightarrow 0.5 \times 0.7 = w \times 2.7$$

$$w = \frac{0.5 \times 0.7}{2.7} = 0.129 = \boxed{12.96\% = w}$$

$$\boxed{w = 0.129}$$

(iii) dry Density =

$$\rho_d = \frac{G\rho_w}{1+e} = \frac{2.7 \times 1}{1+0.7} = \boxed{1.59 \text{ g/cc} = \rho_d}$$

(iv) Bulk Density :-

$$\gamma = \frac{(G+Se)\gamma_w}{1+e} = \frac{2.7 + 0.5 \times 0.7}{1+0.7}$$

$\boxed{\gamma = 1.79 \text{ g/cc}}$ Ans. OR

(ii) How much water can be increased , $\boxed{\rho_d = \text{Const}}$

$\rho_d = \frac{M_d}{V}$ $\boxed{S = 100\%}$

$Se = WG$

$1 \times 0.7 = \omega \times 2.7 \Rightarrow \omega = \frac{0.7}{2.7} \Rightarrow \boxed{\omega = 25.96\%}$

$\boxed{\text{Increase} = 25.96 - 12.96 = 13\%}$ Ans.

LABORATORY DETERMINATION OF PROPERTIES :-

<1> Specific Gravity (At 27°C)

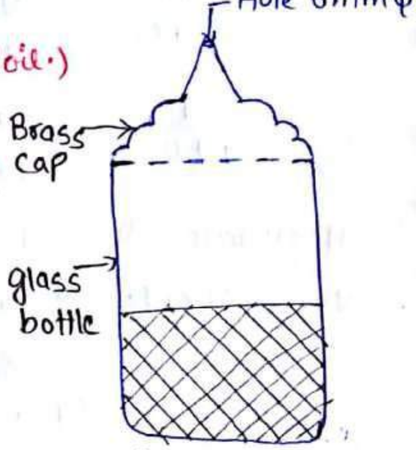
* Pycnometer Bottle Method (only use for coarse soil.)

m_1 = mass of empty Pyc. bottle

m_2 = mass of empty Pyc. bottle + dry soil

m_3 = mass of empty Pyc. bottle + dry soil + water

m_4 = mass of empty Pyc. bottle + water only.



Pycnometer Bottle

$$\boxed{G = \frac{M_2 - M_1}{(M_2 - M_1) - (M_3 - M_4)}}$$

$G_s = \frac{m_s}{m_w}$ (at equal vol.)

$m_s = m_2 - m_1$; $m_w = (m_4 - m_1) - (m_3 - m_2)$.

$m_w = (m_2 - m_1) - (m_3 - m_4)$

(i) Pycnometer Bottle Method is used to Determine Sp. gravity of cohesionless soil. (ii) For fine Soil Sp. gravity is measured by Density Bottle and the Method Remain same.

Q. A 200 gm dry soil was filled in Pycnometer bottle. The mass of Pycnometer bottle + soil + water is 1500gms. The Mass of Pyconeter bottle with water only 1400 gm calculate sp. gravity of soil

Sol :- Wt. of Dry soil = 200 gms

m_3 = 1500gms.

m_4 = 1400 gms.

$M_2 - M_1$ = 200gm (wt of Pyco + dry soil − wt of Pycno = wt. of dry soil)

$$G = \frac{M_2 - M_1}{(M_2 - M_1) - (M_3 - M_4)}$$

$$G = \frac{200}{200 - (1500 - 1400)} = G = \frac{200}{200 - 100} \Rightarrow \frac{20}{10}$$

$$\boxed{G = 2}$$ Ans.

(2) Water content Determination :-

(a) Pycnometer Bottle Method

M_1 = Mass of empty pyc. bottle

M_2 = Mass of empty pyc. bottle + wet soil

M_3 = Mass of empty pyc. bottle + wet soil + water

M_4 = Mass of empty pyc. bottle + water only.

$$\% w = \left[\frac{M_2 - M_1}{M_3 - M_4} \left(\frac{G - 1}{G} \right) - 1 \right] \times 100$$

(B) (i) This method is used for coarse soil only.

(ii) This method can be used only when Specific gravi Of soil particles is known.

(iii)

b) **Oven Drying Method :-** (Any type of soil.

{ IS : 2721, deals with Soil testing }

M_1 = Mass of empty container

M_2 = Mass of empty container + wet soil

Over drying @ Temp 105° - 110°C for 24 HRS

M_3 = Mass of empty container + Dry soil

$$\% w = \frac{m_w}{m_s} \times 100$$

$$\boxed{\% w = \frac{M_2 - M_3}{M_3 - M_1} \times 100} \quad \text{at } 60°C$$

- If there is, gypsum $CaSO_4$ present in soil - 80°C.

(i) This method is the most accurate Method.

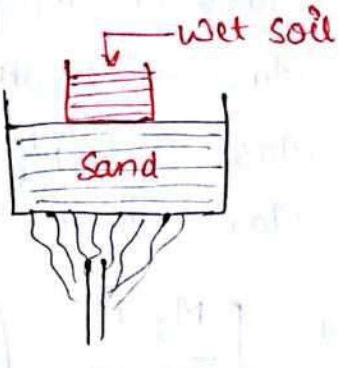
(ii) This method can be used for any type of soil

(iii) If organic soil is considered, the temp. of oven should be 60°C.

(iv) If Soil contains Gypsums, the temp. should be 80°C.

(c) **Sand Bath Method :-**

- field Method



(i) M_1 = Mass of empty container

M_2 = Mass of empty container + wet soil

Heating on Sand Bath

M_3 = Mass of empty container + dry soil

$$\% w = \frac{m_w}{m_s} \times 100$$

$$\boxed{\% w = \frac{M_2 - M_3}{M_3 - M_1} \times 100}$$

(i) This Method is a field Method.

(ii) This Method gives Rough values of Result.

(iii) There is no control over temperature. (heat)

(iv) This method is not used for organic soils and soil containing Gypsum.

(iv) Alcohol Method :-

M_1 = Mass of empty container

M_2 = Mass of empty container + wet soil

Add some Methylated spirit (absorb all moisture)

M_3 = Mass of empty container + dry soil

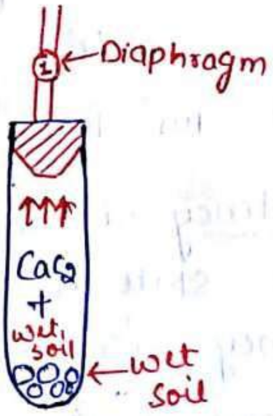
$$\%w = \frac{M_w}{M_s} \times 100$$

$$\boxed{\%w = \frac{M_2 - M_3}{M_3 - M_1} \times 100}$$

(i) This Method gives fairly accurate Results

(ii) This Method is not used for organic soil and soil contai Gypsum.

(iii)



(v) Calcium Carbide Method :-

(i) This method is the Quickest Method (3min - 5min)

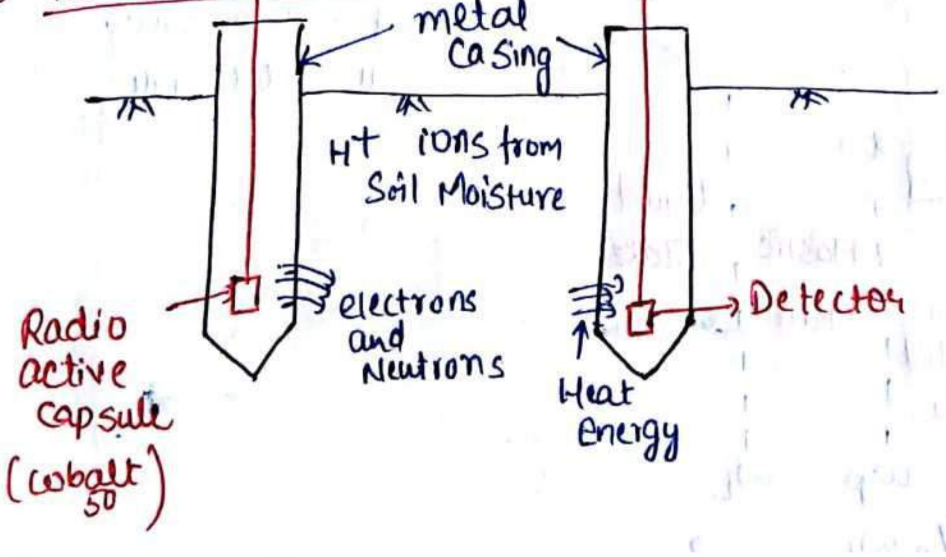
(ii) This Method gives fairly accurate Results.

(iii) This Method is not used for soil containing Gypsum or organic soil.

(iv) The amount of Acetylene gas Produced represents water content present in soil

$$CaC_2 + H_2O \rightarrow CaO + \underbrace{C_2H_2}_{\text{Acetylene Gas.}} \uparrow$$

(vi) Radiation Method :-



(i) This Method is an onsite or insitu Method.

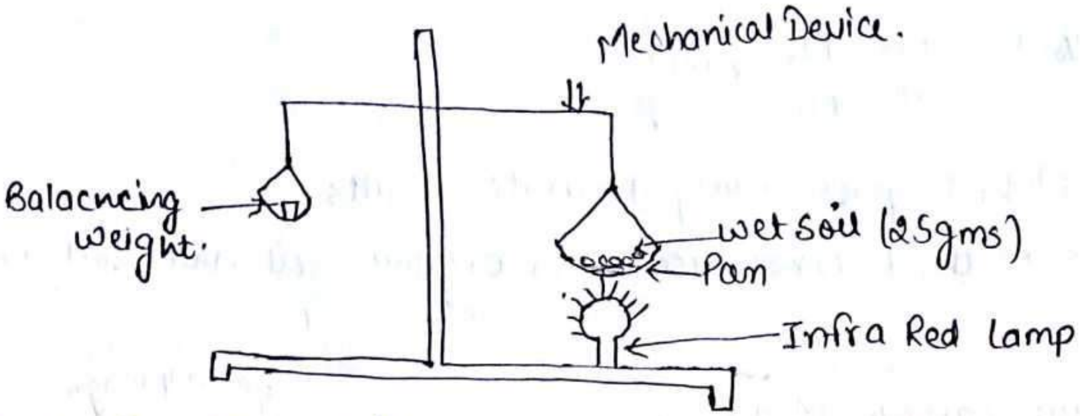
(ii) In this Method, samples are not collected.

(iii) This Method is an uneconomical Method.

(iv) This Method disturbs the soil Moisture.

(v) This Method gives Higher Results as compared to actual values.

(vii) **Torsion Balance Method :-**



- Used for fine soil.

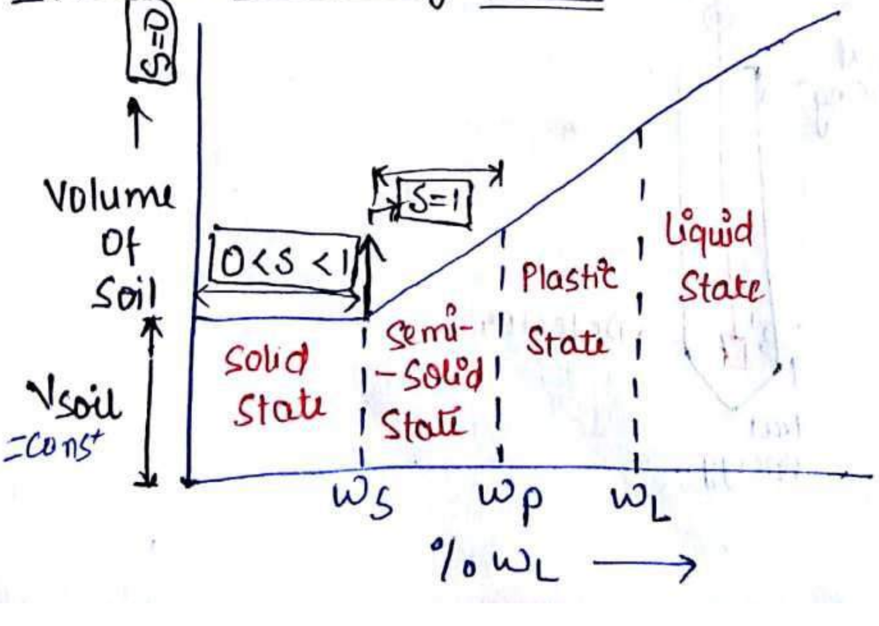
Consistency of Soil :- The property of soil in which soil is converted from one state to another, is known as consistency.

- Consistency is Related to shear strength of soil.

Consistency limit :- The water content at which soil is converted from one state to another is known as consistency limit.

- Consistency limit also known as Atterberg's limit.

Types of Consistency Limit.



Vol. of soil is const. at Solid state

Liquid limit :- The water content at which soil is converted from liquid state to plastic state is known as liquid limit.

- It is the water content at which very small shear strength in soil just develops.

(ii) Plastic Limit :-

- It is the water content at which soil is converted from plastic state to semi-solid state.
- It is the water content at which soil try to gain sum shape

(iii) Shrinkage limit :-

It is the water content at which soil is converted from semi-solid state to solid state.

(iv) - At shrinkage limit, the ~~water~~ degree of saturation is 1 (unity)
- upto shrinkage limit the vol. of soil remains constant.
- Upto shrinkage limit void ratio of soil remains constant.

Ques :- ~~A~~ Shrinkage limit of a soil is 10% calculate porosity of soil.

Shrinkage limit = 10% (means)

means $w = 10\%$ @ $S = 100\%$

$n = ?$

$Se = wG$

$1 \times e = 0.10 \times 2.67$

$\boxed{e = 0.267}$

$n = \frac{e}{1+e} = \frac{0.267}{1+0.267}$

$\boxed{n = 0.21 \simeq 21\%}$ Ans

HOMEWORK

Q A dry soil has mass sp. gravity of 1.35 and sp. gravi of soil solids. is 2.7. Calculate void ratio of soil.

dry soil, $G_m = 1.35$

$G = 2.7$

$e = ?$

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

$G = \frac{M_s}{M_w} = \frac{\gamma_{solids}}{\gamma_w}$

$\boxed{G_m = \frac{\gamma}{\gamma_w}}$

for dry soil

$$\rho = \rho_d$$

$$G_m = \frac{\rho_d}{\rho_w} \Rightarrow 1.35 = \frac{\rho_d}{1} =$$

$$\rho_d = 1.35\ g/cc$$

Lidquid state

$$\rho_d = \frac{G\rho_w}{1+e}$$

$$1.35 = \frac{2.7 \times 1}{1+e}$$

$$1+e = \frac{2.7}{1.35} \Rightarrow 2 \quad ; \quad \boxed{e = 1}$$

$$\boxed{e = 2-1 \Rightarrow 1}$$

#(1)Plasticity Index :-

- It represents the range of water content in which soil contains plastic properties.

$$I_P = W_L - W_P$$

- $W_L > W_P \Rightarrow \boxed{I_P = +ve}$ (Plastic soil)

$W_L = W_P \Rightarrow \boxed{I_P = 0}$ (Non-plastic soil) (e.g. sand)

$W_L < W_P \Rightarrow I_P \neq -ve$

$\boxed{I_P = 0}$

I_P (%)	Soil Type	Examples
= 0	Non-Plastic	Sand
$0 < I_P < 7$	low Plastic	Silt
7 - 17	Medium - Plastic	Silty clay
> 17	Highly - Plastic	clay

(2) Consistency Index / Relative consistency :-

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

W = Natural water content

(i) $I_C = 0 \Rightarrow W = W_L$ [Soil is at liquid limit]

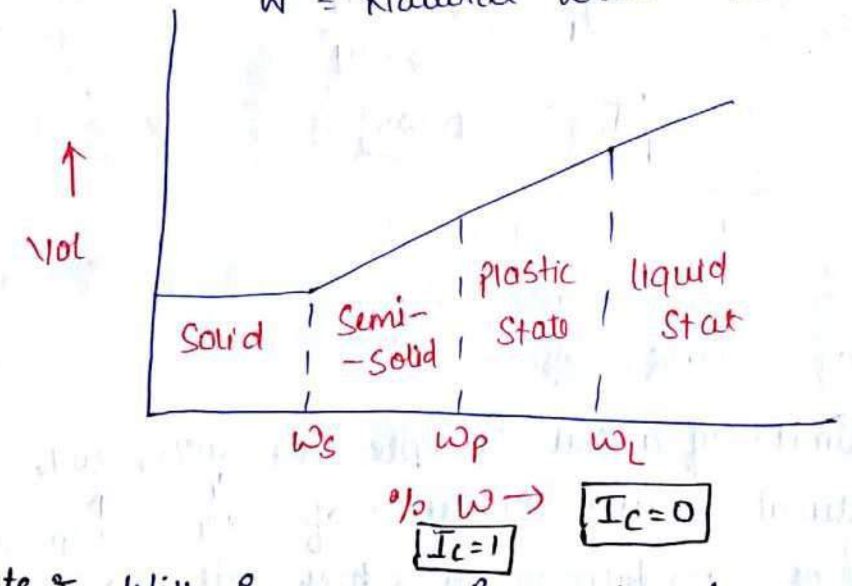
(ii) $I_C = 1 \Rightarrow W = W_P$ [Soil is at plastic limit]

(iii) $0 < I_C \leq 1 \Rightarrow (W_P < W < W_L)$ [Soil is at plastic state]

(iv) $I_C > 1 \Rightarrow (W < W_P)$ [Soil may be at Semi-solid, solid-state or Shrinkage limit]

(v) $I_C = -ve \Rightarrow W > W_L$ [soil is at liquid limit]

W = Natural water content



Note :- With increase in natural water content of soil Relat consistency decreases.

(3) Liquidity Index / Water Plastic Ratio :-

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

$$\boxed{I_C + I_L = 1}$$

W = Natural water content

$$I_C + I_L = \frac{W_L - W}{I_P} + \frac{W - W_P}{I_P}$$

$(W_L - W_P = I_P$

$$= \frac{W_L - W + W - W_P}{I_P} \Rightarrow \frac{W_L - W_P}{I_P} = \frac{I_P}{I_P} = 1$$

Q. The plastic limit and $\overset{\text{liquid}}{\text{solid}}$ limit of soil are 25% , 20% respectively. calculate Ip. (Plasticity Index)

$$I_p = w_L - w_p$$

$$I_p = 20 - 25 = -5\% \; [I_p \neq -ve]$$

$$\boxed{I_p = 0}$$

Q. If liquid limit and plastic limit of soil are 40% and 22% respectively Identify the type of soil.

$$I_P = w_L - w_P$$

$$I_p = 40 - 22$$

$$\boxed{I_p = 18} \; [I_p > 17 \text{ then Highly plastic}] \; (\text{clay})$$

Q Liquid limit and Plastic limit of soil are 32% and 24% . if natural water content of soil is 25%.. calculate consistency and liquidity Index.

$$I_c = \frac{\overset{w_L - w}{32-25}}{\underset{w_L - w_p}{32-24}} = \qquad I_L = \frac{25-24}{32-24}$$

$$\boxed{I_c = \cdot 875} \qquad \boxed{I_L = 0.125}$$

I.

OR

$$\boxed{I_c + I_L = 1}$$

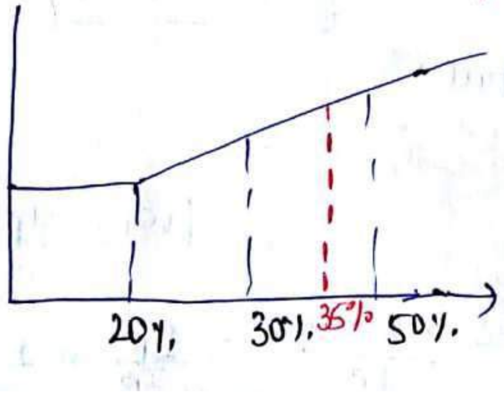
$$I_L = 1 - 0.875 = 0.125 \text{ Ans}$$

Ques: The consistency limits of a soil sample are $\underset{\downarrow w_L}{50\%}$, $\underset{\uparrow w_p}{30\%}$ and $\underset{\uparrow w_s}{20\%}$ Respectively. If natural water content of soil is 35%. Identify the condition in which soil lies.

$$I_c = \frac{50-35}{50-30} = \frac{15}{20} \; ; \; \boxed{I_c = 0.75}$$

$$\boxed{I_c > 1}$$

if soil is 0 < Ic > 1 then soil is at plastic limit



(a) plastic limit

(b) Plastic limit state

(c) Semi solid state

(d) Liquid limit

Q. Which of the following statement is correct :-

(a) I_P can have -ve values

(b) I_C can not have -ve values.

~~(c)~~ I_L can have -ve

(d) N.O.T

Concept :-

$I_C + I_L = 1$

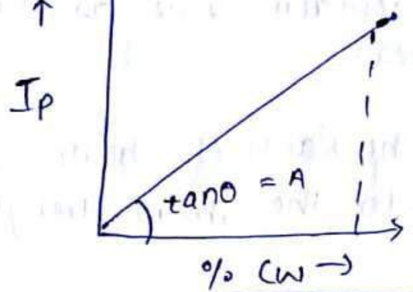
$\boxed{\text{Take } I_C = 2}$

$2 + I_C = 1$

$\boxed{I_L = -1}$

Activity of Clay :- This term is defined in clay particles only.

- Size of clay particles < 0.002 mm
- Activity of soil Represents Shrinkage and Swelling expension properties in soil into clay particles. due to the moisture variation.
- It is the slope of curve b/w plasticity Index and %age clay particles.
- Activity measures water holding capacity of soil



Activity (A)	Type
A < 0.75	Inactive soil
0.75 - 1.25	Normal soil
> 1.25	Active soil

$$\boxed{\tan\theta = A = \frac{I_P}{\%C_W}}$$

- Activity gives information about type of Mineral and its effect in soil.

$\left\{ \begin{array}{l} \%C_W = \%\text{clay particles} \\ [\text{having size} < 0.002\text{mm}] \end{array} \right\}$ present in soil

Q. Properties consider the following properties of two clay specimens

Properties	Clay A	Clay B
W_L	50%	40%
W_P	20%	20%
W	25%	10%
$\%C_W$	10%	20%

(a) Which clay is More plastic Clay A

(b) Which clay is more acti

Clay A

(c) Which sou clay is soft in consistency Clay A

i) Clay A = 50 − 20
= 30

Clay B = 20

(ii)
(i) Clay A Ans

Clay A Clay B

(iii) $\frac{30}{10}$ 3 = $\frac{20}{20}$ 1

Clay A Ans

(iii) Clay A

$\frac{50-25}{50-20} = \frac{25}{30}$

Clay B

$\frac{40-10}{40-20} = \frac{30}{20}$

(iv) ClayB have more consistency
So Clay A is mor softer in consistency

Sensitivity of clay :-

(1) Undisturbed Sample :- The sample in which molecular structures between particles Remains same as that of field.

- These samples are collected Using sampling tubes.

(2) Remoulded Sample / Disturbed Sample :- The sample in which molecular structures b/w particles doesnot remain same as that of field is known as Disturbed Samples.

(3) Sensitivity :- Sensitivity is Defined as the Ratio of shear Shear strength of soil in undisturbed state to the shear strength of soil in remoulded state.

$$\boxed{S_r = \frac{q_{undisturbed}}{q_{remoulded}}}$$

q = compressive strength.

If $S_r = 1 \Rightarrow$ In sensitive soil

$S_r > 1 \Rightarrow$ Sensitive soil

Q. The dry density of soil is 20KN/m3 and Sp. gravity of soil solids is 2.5. Calculate Shrinkage limit of soil.

$\gamma_d = 2.5$

$G = 2.5$

Shrinkage limit = ?

$\omega = ?$ @ $S=1$

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

$20 = \frac{2.5 \times 10}{1+e}$

$e = 0.25$

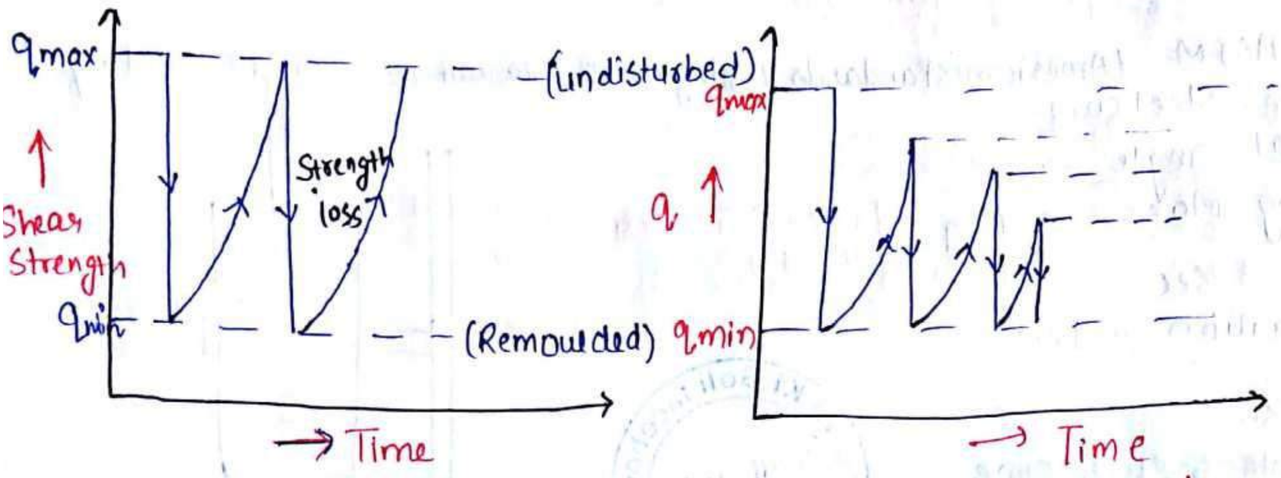
$Se = \omega . G$

$1 \times 2.5 = \omega \times 25$

$\boxed{\omega = 10}$

Thixotropy-

Thix – to touch



(Ideal case)

(Practical)

- Thixotropy is defined as strength loss – strength gain proces without any change of moisture content.
- The gain in strength is due to reformation of molecular str ures b/w soil particles.

Laboratory Determination of Liquid Limit and Plastic limit

(i) Plastic Limit:- (Laboratory Determination)

(i) 30 gms oven dried soil having particle size less 425 μ is co -dered. The soil is mix with some water and a soil bal is prepared.

(ii) Soil ball is rolled into cylinderical thread.

(iii) During crumbling of cylinderical thread if

(a) Cyllindorical Dia > 3mm (Add some water)

(b) Cylinderical Dia = 3mm (Water content = W_P)

(c) Cylinderical Dia < 3mm (Remove some water) by kneadi

(ii) Liquid limit :-

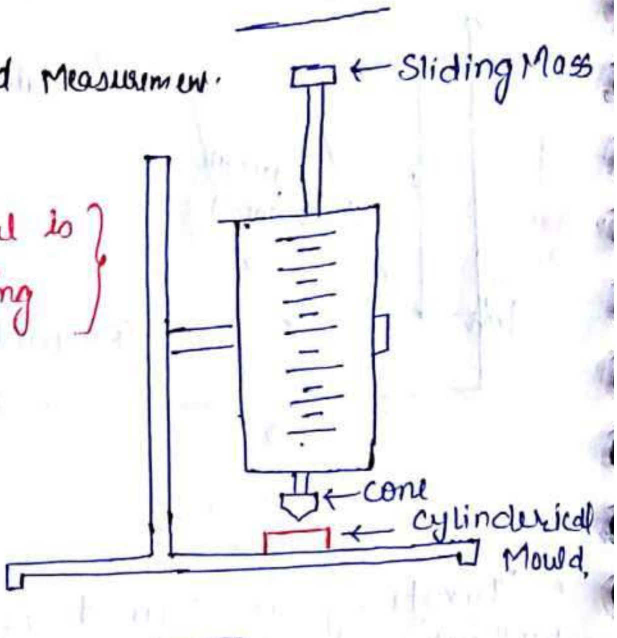
laboratory Test for liquid Test

(1) Static cone Penetration Test

ASTM (American standards Testing and Measurement.

- Stainless steel cone
- Central angle → 30°
- Sliding Mass → 75 g
- first 5 sec
  Penetration - 1cm

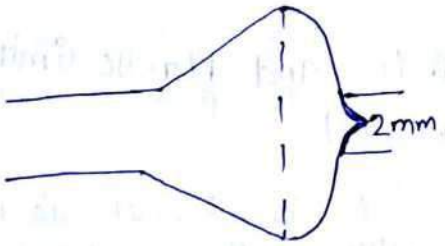
[oven Dried soil is used for Testing]



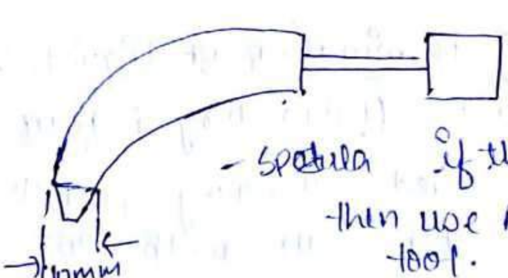
IS

- Stainless Steel cone
- central angle - 31°
- Sliding Mass - 80g
- Penetration → 20mm to 25mm

(2) casagrande's Test

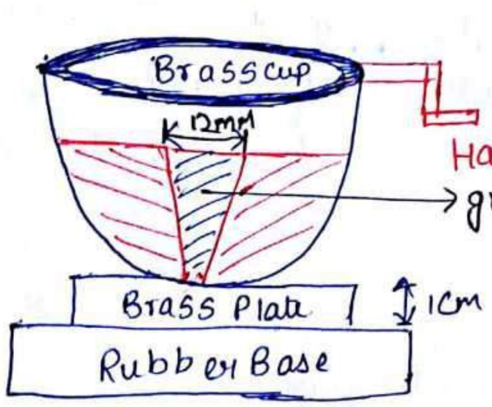


Casagrande's Tool
Grooving tool

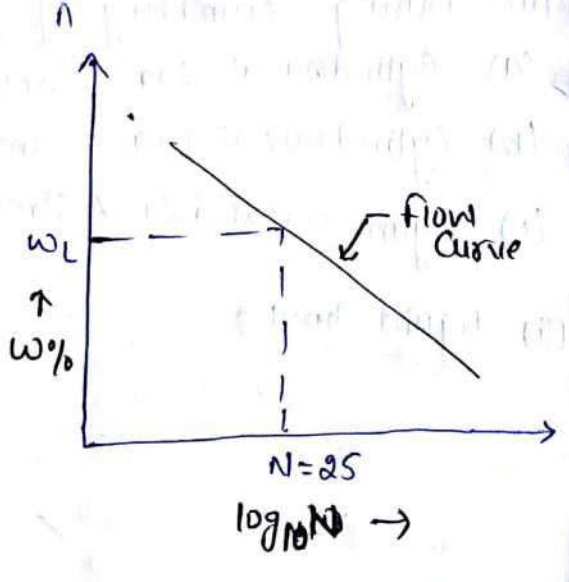


- spatula if the soil is sandy then use ASTM grooving tool.

ASTM Tool



Cassangrande's Apparatus

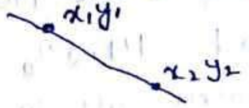


- 120gms of sample is taken.
- Then add Some water. Then place a portion of the sample in cup.

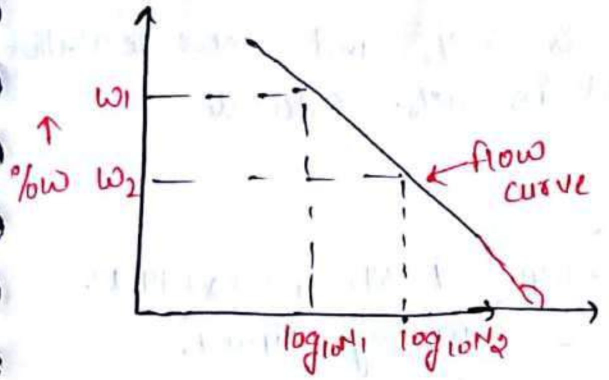
Acc. to this Method at which parts of groove come in contact with each other over $\overset{15-35}{25}$ no. of Revolutions.

Flow Index :- It is the slope of flow curve drawn b/w water content and no. of Revolutions on semi-logarithmic scale.

- S flow Index Represent the Rate of loss of shear strength with increase in water content of soil



$$I_f = \frac{y_2 - y_1}{x_2 - x_1}$$



$$I_f = \frac{-(w_2 - w_1)}{\log_{10} N_2 - \log_{10} N_1}$$ {-ve sign water content in soil is decreasing}

$$\boxed{I_f = \frac{(W_2 - w_1)}{\log_{10} \frac{N_2}{N_1}}}$$

Ques. If the Relation b/w water content and no. of Revolutions is given as $\boxed{w = 20 - \log_{10} N}$. Calculate liquid limit of soil.

$w = 20 - \log_{10}$

$w = w_L$ @ $N = 25$

$W_L = 20 - \log_{10} 25$

$\boxed{W_L = 18.6\%}$

Ques :- The no. of Revolutions on a soil sample are 100. when wate content in soil is 20%. If the water content is increased t 50%, the no. of Revolutions are Reduced to 10. Calculate

(i) flow Index (ii) liquid limit of soil.

$N_1 = 100$ @ $W_1 = 20\%$ ↓ $= .20$

$N_2 = 10$ $\quad w_2 = 50\%$ $= .50$

$$I_f = \frac{W_1 - W_2}{\log\left(\frac{N_2}{N_1}\right)} \Rightarrow \frac{0.2 - 0.5}{\log_{10}\left(\frac{10}{100}\right)} \quad \frac{-.3}{\log(0.1)} = 0.3.$$

OR

$$I_f = \frac{W_2 - W_1}{\log\left(\frac{N_1}{N_2}\right)} = \frac{0.5 - 0.2}{\log\left(\frac{100}{10}\right)} = 0.3$$

(ii) $w = w_L \Rightarrow N = 25$

$$I_f = \frac{w_1 - w_L}{\log\left(\frac{25}{N_1}\right)} = 0.3 = \frac{0.2 - w_L}{\log_{10}\left(\frac{25}{100}\right)} \rightarrow w_L$$

$$\boxed{w_L = 0.38} \boxed{= 38\%}$$

Ques The plastic Limit of two Specimens of clay are 13.8% and 14.2% respectively. The no. of Revolutions in the clay Specimen are 10, when water content is 60% and these revolutions gets increased to 100 when water content is 20%. Calculate
(i) Plastic limit of soil
(ii) Plasticity Index of soil

Sol: $\boxed{I_P = W_L - W_P}$

$w_{P1} = 13.8\%$, $w_{P2} = 14.2\%$
w_P avg $= 14.0\%$

$$\frac{.20 - .60}{\log\left(\frac{10}{100}\right)}$$

$I_f = 40\%$
W_L $I_P = W_L - w_P$
$I_f = .40 =$

$I_f = W_P = 13.8\%$ $W_{P2} = 14.2\%$

W_P avg. $= 14\%$

$$I_f = \frac{w_1 - w_2}{\log\left(\frac{N_2}{N_1}\right)} = \frac{0.6 - 0.2}{\log\left(\frac{100}{10}\right)} = 0.4$$

$w = w_L$ @ $N = 25$

$$I_f = \frac{W_L - W_2}{\log_{10}\frac{N}{25}} \Rightarrow 0.4 = \frac{W_L - 0.2}{\log_{10}\left(\frac{100}{25}\right)}$$

$$\boxed{W_L = 0.44 \quad (\text{or}) \quad W_L = 44\%}$$

$I_P = W_L - W_P$

$I_P = 44 - 14$

$\boxed{I_P = 30\%}$

Toughness Index :- It is the Ratio of Plasticity Index of soil to its flow Index.

$$I_T = \frac{I_P}{I_f} \quad (\text{If } I_P = \text{constant})$$

$$\boxed{I_T \propto \frac{1}{I_f}}$$

- Toughness Index Represents Rate of gain of Shear strength with decrease in water content in soil

Relative Density/ Density Index :- (Cohesionless soil) :- Relative Density represents compactness of cohesionless soil Deposit

$$\boxed{I_D = \frac{e_{max} - e}{e_{max} - e_{min}}}$$

e_{max} = void Ratio of soil in loosest state

e_{min} = void Ratio of soil in Densest state

e = Void Ratio of soil in loosest state

$e_{max} = 0.91$, $e_{min} = 0.35$ } when spherical sand particles.

Value

$I_D = 0 \Rightarrow e = e_{max}$ (soil is at loosest state)

$I_D = 1 \Rightarrow e = e_{min}$ (soil is at Densest state)

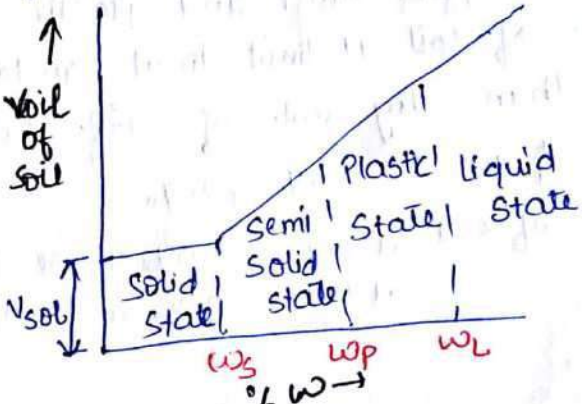
$0 < I_D < 1 \Rightarrow e_{min} < e < e_{max}$

6 Sep 2018

Shrinkage Index :- It represents the Range of water content in which soil contains semi-solid properties.

$$\boxed{I_S = W_P - W_S}$$

- $W_P > W_S$ $I_S = +ve$
- $W_P = W_S$ $I_S = 0$
- $W_P < W_S$ $I_S \neq -ve$ $\boxed{I_S = 0}$



Volumetric shrinkage :-

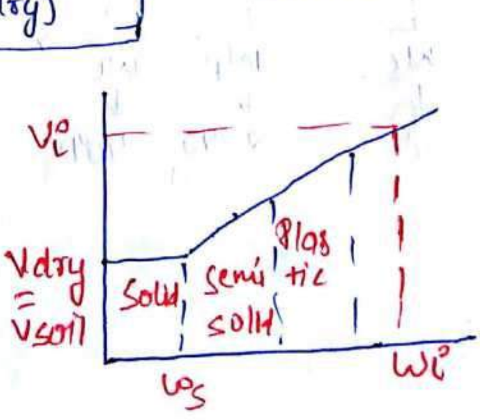
- It represents the loss of volume of soil due to decrease in wat

Content of soil. $\boxed{VS = \frac{V_i - V_{soil(dry)}}{V_{soil(dry)}} \times 100}$ $\frac{\text{final - intial}}{\text{initial}}$.

Shrinkage Ratio :-

$$SR = \frac{VS}{\text{Change in water Content}}$$

$$SR = \frac{VS}{W_i - W_S} = \frac{\left[\frac{V_i - V_{soil(dry)}}{V_{soil(dry)}}\right] \times 100}{W_i - W_S}$$



$$\boxed{SR = \frac{\dfrac{V_i - V_{soil\,(dry)}}{V_{soil\,(dry)}} \times 100}{W_i - W_s}}$$

- It represents the rate of loss of vol. of soil w.r.t decrease in water content

Ques: The vol. of soil gets reduced to 60% of its initial vol. when water content of soil is Reduced from 40% to 20%. Calculate volumetric Shrinkage.

Sol let $V_i = V$; $V_{soil} = 0.6V$

$$V_S = \frac{V_i - V_{soil}}{V_{soil}} \times 100 \Rightarrow \frac{V - 0.6V}{0.6V} \times 100$$

$$V_S = \frac{0.4V}{0.6V} \times 100 = 66.6\% \qquad \boxed{V_S = 66.6\%} \text{ Ans.}$$

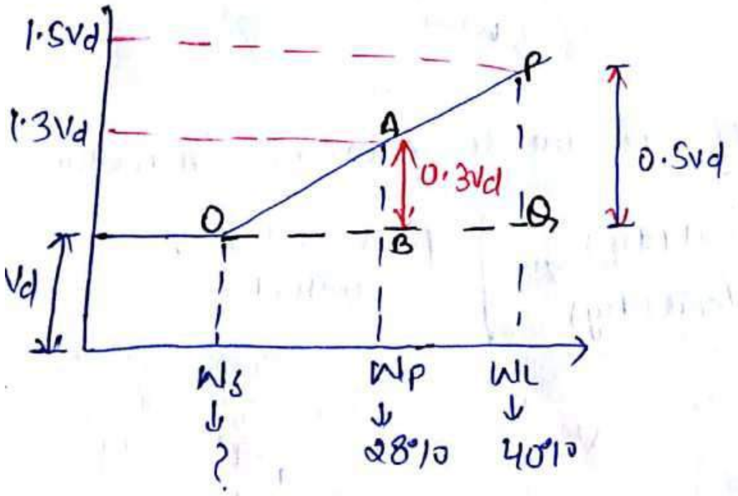
$$SR = \frac{66.6}{40\% - 20\%} = 3.33 \qquad \boxed{SR = 3.33}$$

Ques The liquid limit and Plastic limit of soil are 40% and 20%. The vol. of soil at limit limit and at plastic limit are 50% and 30% more than dry vol. of soil. calculate shrinkage limit of soil.

W_L & W_P = 40% & 20%

Vol. of soil at W_L = 50% more than dry vol. (V_d)

at W_P = 30% more than dry vol. (V_d)



from similar Δ OAB ≅ OPQ

$$\frac{AB}{PQ} = \frac{OB}{OQ}$$

$$\frac{0.3\cancel{V_d}}{0.5\cancel{V_d}} = \frac{0.28 - W_s}{0.40 - W_s}$$

$$0.12 - 0.3W_s = 0.14 - 0.5W_s$$

$$0.2W_s = 0.02 \quad 0.02 \quad 0.02$$

$$W_s = \frac{.02}{.2} \times 100$$

$$W_s = 10\%$$

2nd Method

$$\frac{\frac{V_i - V_d}{V_d} \times 100}{W_i - W_s} = \frac{\frac{1.5V_d - V_d}{V_d} \times 100}{40 - W_s}$$

$$\frac{\frac{1.5V_d - V_d}{V_d} \times 100}{40 - W_s} = \frac{\frac{1.3V_d - V_d}{V_d} \times 100}{28 - W_s}$$

Ques A saturated Sample of clay was immerged in mercury and th
displaced vol. of mercury is 21.8 cm3 and the Mass of saturated
clay sample is 32.2 gms. After oven drying the wt and vol of
soil is gets reduced to 22.2 gms and 11.6 cm3 · Calculate shrink
limit.

Sol: Clay was immersed in mercury displaced = 21.8CC

Mass of Saturated clay = 32.2g

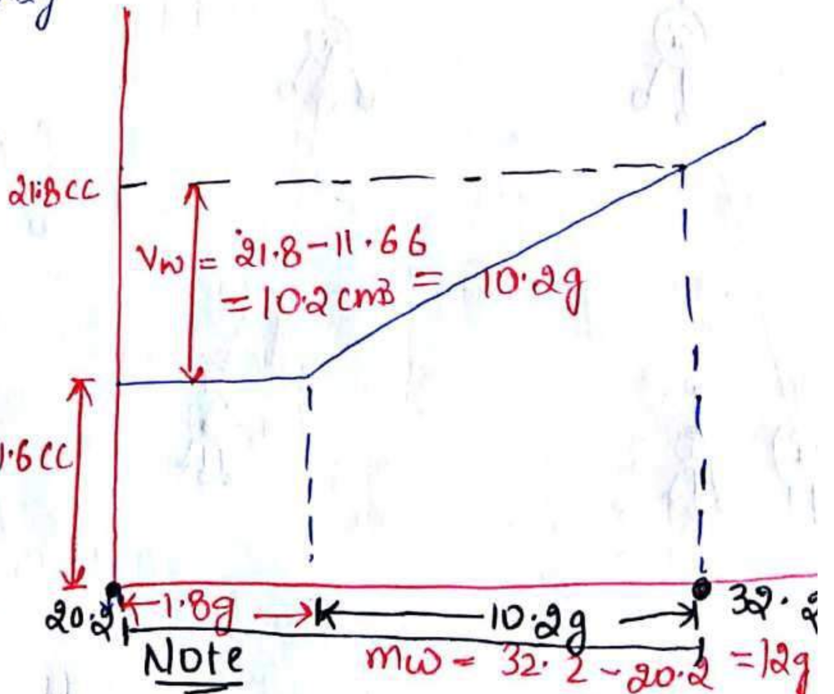
Wt. & volume

$\downarrow$ 20.2g $\downarrow$ 11.6 CC

$$\% w_s = \frac{m_w}{m_s} \times 100$$

$$\% = \frac{12 - 10.2}{20.2} \times 100$$

$$\boxed{W_s = 8.91\%}$$ Ans



Note

$V(\text{cm}^3) = m(g) \rightarrow$ (i)

$V = m$

Ques The diff. b/w void Ratio of soil in loosest and Densest state is 0.5. The Relative Density of soil deposit is 80%. calculate the void Ratio of soil in loosest state. If natural void Ratio is 0.3.

$I_D = 80\%$ $e_{max} = ?$ $e = 0.3$

$I_D = \frac{e_{max} - e}{e_{max} - e_{min}}$ [loosest and Densest state diff b/w $= 0.5$; $e_{max} - e_{min} = 0.5$]

$.8 = \frac{e_{max} - 0.3}{.0.5} \Rightarrow .4 = e_{max} - 0.3$

$\boxed{e_{max} = 0.7}$

Ques :- The Max^m value of Relative Density for spherical sand particles can be (i) 0.91 (ii) 0.6 (iii) 1.2 (iv) 0.95 ✓

Note Select nearest value to 1. $\boxed{e_{max} = 0.91}$ ⇒ Not Relative Density.

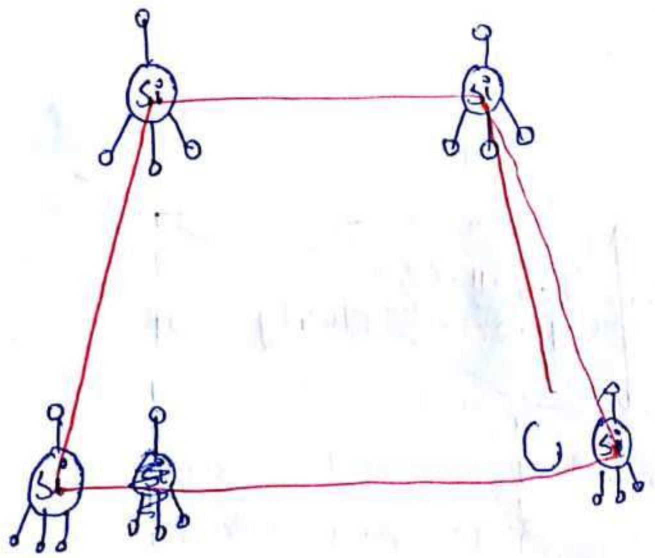
Clay Minerals :-

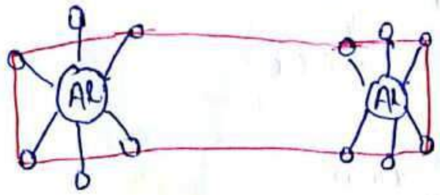
(I) Mineral Unit :- These are basic units which connect with each others to form clay Minerals.

- Types of Mineral Units

Tetrahedral Units | silica → $\boxed{4O^{2-}}$ $\boxed{S}$
oxygen

Octahedral Units | gibbsite → Al
ferrisite → fe
or → $6O^{2-}$ $\boxed{G}$
oxygen



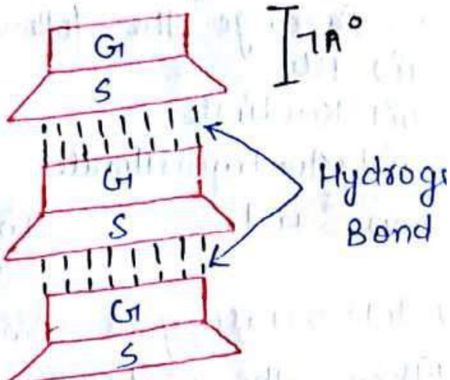


1. Kaolinite Minerals :-

- Unit Ratio → 1 : 1 Ⓢ Ⓖ
- Maxm Strength and Stability
- Minm Shrinkage & Minm Swelling
- least active Mineral [A ⇒ 0.3 to 0.5]
- least Plastic Mineral
- Electrically charged Mineral
- ~~Plastic~~ Particle Size ⇒ 500 A° to 1000A°

$\boxed{1 \text{ Anstrom} = 10 \times 10^{-10} m}$

Eg :- China clay.



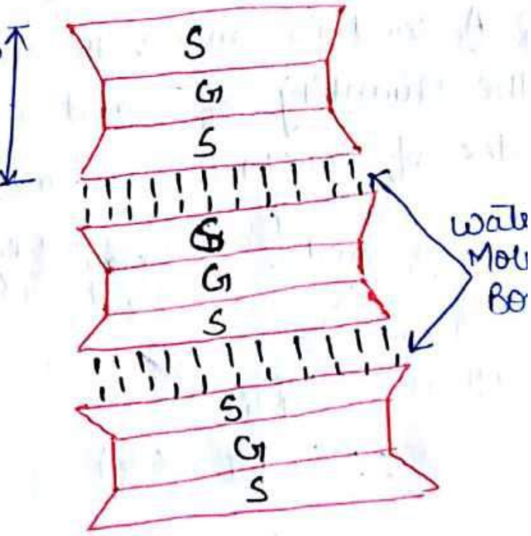
{Hydrogen Bond - Most strong Bond.

2. Montmorillonite Minerals

- Unit Ratio → 2 : 1 Ⓢ Ⓖ = 9.6A° ≃ 10A°
- Minm Strength and stability
- Maxm Shrinkage and Maxm Swelling
- Highly Active Mineral [A ⇒ 1 to 7]
- Highly Plastic Mineral
- ~~Plastic~~ Particle Size = 30A°

Imp:-
Eg :- Black cotton soil and

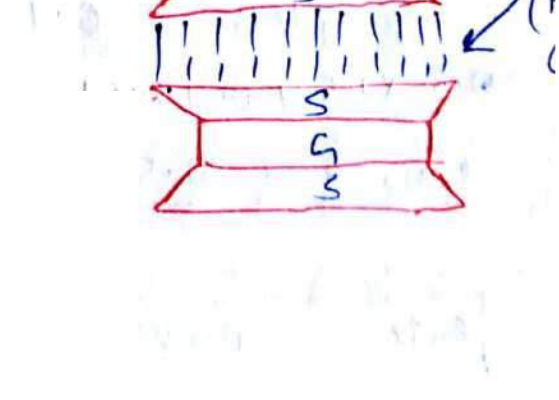
1 Thickness = 10A° (1 particle size)
3 particle makes a total single particle of clay =
total thicke 3×10A° = 30A°



3. Illite Mineral

- Unit Ratio - 2 : 1
- Intermediate strength and ~~activity~~ Stability
- Intermediate Shrinkage and intermediate Swelling.
- Active Mineral [A = 0.5 to 1.0]
- Particle Size = 300A°

eg Talcum Powder



7 Sep 2018

Q. Arrange the following order of their Particle size
(i) Illite
(ii) Kaolinite
(iii) Montmorrilonite
(iv) Sand

Silica < montmorillonite. < Illite < Kaolinite

Increase order of their plasticity basis

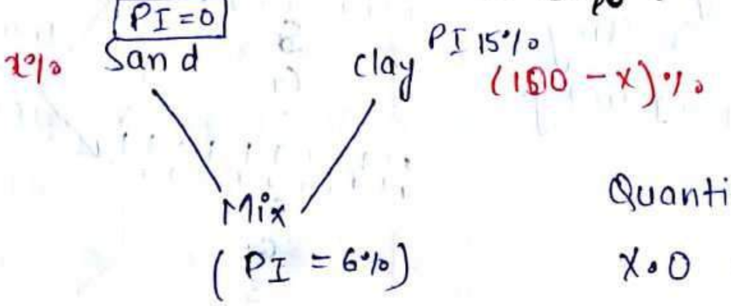
Silica < Kaolinite < Illite < mont.

Note :- Halloysite Mineral

When the water molecules are also present with Hydrogen Bond in Kaolinite Mineral, the Mineral is k/n as Halloysite Mineral.

- It can be converted into Kaolinite Mineral by heating.

Q A sand is mix with clay having plasticity Index 15%. Calculate the Quantity of sand to be added such that the resultant plasticity Index of mixture becomes 6%.



Total Quantity = 100%

Quantity x Property = constt.

$x \cdot 0 + (100 - x)15 = 100 \times 6$

$0 + 1500 - 15x = 600$

$x = \frac{1500 - 600}{15}$

Sand. $\boxed{x = 60\%}$

Q. The void Ratio of two samples A & B are 0.5 and 0.7. The volume of Both samples are 1.5 cu-m and 1.7 cu-m respectively. Calculate the Porosity of Mixture of Both soil.

Sample A = $\overset{e}{\downarrow}$ 0.5 , $\overset{V}{\downarrow}$ 1.5 m3

Sample B = 0.7 , 1.7 m3

$0.5 \times 1.5 + 0.7 \times 1.7 = e_{mix} \times (1.5 + 1.7)$

$e_{mix} = \frac{0.5 \times 1.5 + 0.7 \times 1.7}{3.2}$

$\boxed{e_{mix} = 0.606}$

$$n = \frac{e_{mix}}{1+e_{mix}} \Rightarrow n = \frac{0.606}{1+0.606}$$

$$\boxed{n = 37.7\%}$$

SOIL STRUCTURE

- The Mode of arrangement of soil particles w.r.t each other is Known as soil Structure
- There are two types of forces which define the type of structures in soil particles.

① Gravitation forces ⇐ coarse soil

② Surface forces ⇐ clay

 # Specific Surface :- sp. Surface represents the total area covered by soil particels of 1gm mass.

- It is also defined as Ratio of Surface area of particle to its mass.

Specific surface, $\boxed{S_s = \frac{A}{m}} \boxed{\simeq \frac{A}{V} = \frac{6}{D}}$ $A = 4\pi r^2$, $V = \frac{4}{3}\pi r^3$

$$\boxed{S_s = \frac{6}{D}}$$

$S_s = 4$

#Concept :-

$$S_s = \frac{4\pi r^2}{\underset{\text{vol. of spherical particle}}{\frac{4}{3}\pi r^3}} = \frac{3}{r} = \left\{\frac{6}{D}\right\}$$

- The soil particles having low specific surface contain High Gravitational forces.

Ex :- Sand and Gravels

- The soil particle having to high specific surface high surf. coarses. forces

Ex :- clay.

- Silt particals contain gravitational forces and Surfa forces both

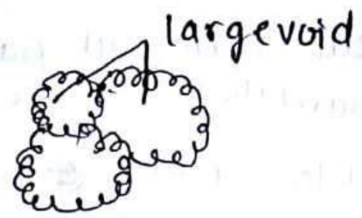
i) Single grained Structure :-

Ex Sand, Gravel

- Contains High Gravitation forces
- Very low surface forces
- Contain large void size

(ii) Honey Comb Structure

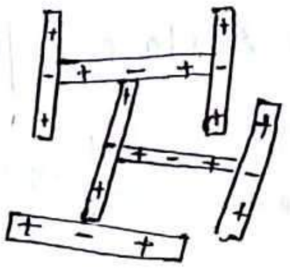
- Silt Particles
- gravitational forces + surface forces
- large Void Ratio
- High Strength & Stability
- Piles are derived to break this structure.



(iii) Flocculated Structure

- Clay Particles
- Surface attractive forces
- High void Ratio
- High Permeability

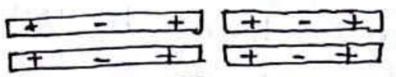
edge-to-face / face to edge orientation



(iv) Dispersed Structure :

- Clay Particles
- Surface Repulsive forces [+ve to +ve ion / -ve to -ve charge]
- low void Ratio
- low Permeability

* Parallel Orientation or face-to face.

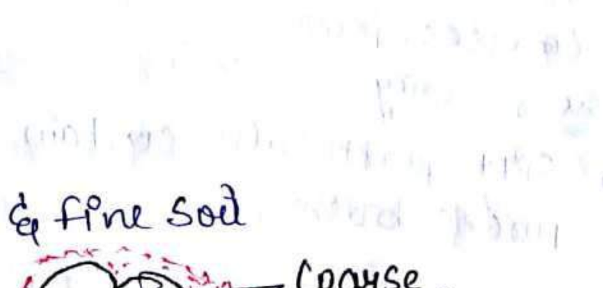


(v) Composite Structure :-

exist in mixture of coarse soil & fine soil

(a) coarse skeleton Structure

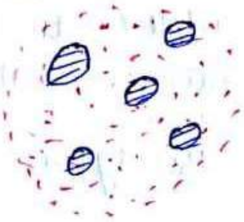
- Quantity of coarse soil is more than that of fine soil
- low void Ratio [Becoz of greater Quantity of coarse soil]



(b) Clay Matrix Structure

- Quantity of fine soil is more than that of coarse soil.
- High void Ratio

(Becoz of High Qty of fine soils)



Particle Size Distribution

* Mechanical ~~Irrigation~~ Analysis

- → Sieve Analysis (Application in coarse soil) (Particle size >0.07
- → Sedimentation Analysis (Applicable in fine soil) (Particle size <0.07 m

<1> Sieve Analysis

Sieve no:- It represents the no. of openings in a given 1 inc length of sieve.

- It is also defined as the square Root of no. of openings in 1 inch square Area over sieve.

(ii) Sieve Size :-

- It is the size of one opening in a sieve.
- Acc. to Indian Standard (IS), Sieves are designed by sieve size :-

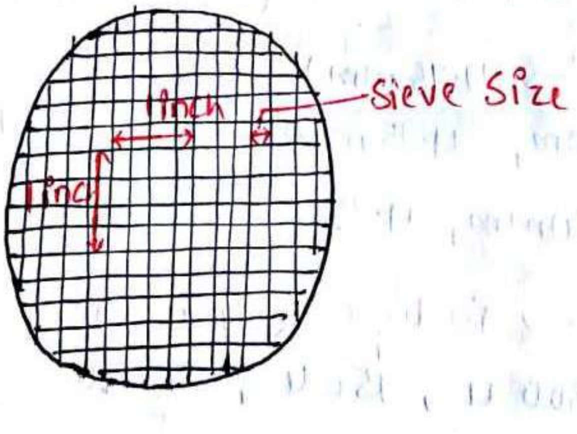
(i) Min^m 5 sieves are used for Sieve Analysis at a given ti

(ii) Min^m size of sieve available in market is 45 μ.

(iii) Min^m size of sieve used in sieve analysis is 75 μ.

(iv) Max^m size of sieve used in sieve analysis is 100 mm.

$\boxed{\text{IS - 0.075 mm Sieve}}$ ⟶ Sieve NO→200.



Q Mass of a soil sample is 0.18 kg and vol. is 10^{-4} m3. The Dry Density of soil is 1600 kg/m3. calculate water content of soil If 0.02 kg more water is added to the soil then what will be the new water content. $\boxed{\text{Ans } 25\%}$

$m = 0.18$ kg $\qquad \rho_d = 1600$ kg/m3

$v = 10^{-4}$ m3

More water added = 0.2kg

$m_d = \rho_d V = 1600 \times 10^{-4} = 0.16$ kg

$m = 0.18$ kg

Initial $m_w = 0.18 - 0.16 = 0.02$ kg

Added $m_w = 0.02$ kg

Total $m_w = 0.04$ kg

$$\% w = \frac{m_w}{m_s} \times 100 = \frac{0.04}{0.16} \times 100$$

$$\boxed{\% w = 25\%}$$

Alternate Method:

$\rho = \frac{0.18}{10^{-4}}$ $\qquad \rho = \frac{m}{v}$ $\quad m = 0.18$ kg, $v = 10^{-4}$ m3

$\rho = 1800$ kg/m3

$\rho_d = 1600$ kg/m3

More water = 0.02kg

$\rho_d = \frac{\rho}{1+w}$ $\qquad m_d = 0.16$ kg

% w added $= \frac{0.02}{0.16} \times 100 = 12.5\%$

10 Sep/2018

Sieve Analysis (Used for coarse soil) (Size > 0.075mm)

(i) gravel fraction (Particle size > 4.75 mm)

100mm, 63mm, 20mm, 10mm, 4.75mm (Practically)

Acc to IS → 80mm, 20mm, 10mm, 4.75

(ii) fine fraction : (0.075mm < Particle size < 4.75 mm)

2mm, 1mm, 600 μ, 425 μ, 300 μ, 150μ, 75 μ

212μ

Size	Mass Retained	% mass Retained	% cumulative mass retained	% finer (N = 100-C)
100	m_1	$\% p_1 = \frac{m_1}{m} \times 100$	$C_1 = p_1$	$100 - C_1$
63	m_2	$= \frac{m_2}{m} \times 100$	$C_2 = p_1 + p_2$	$100 - C_2$
20	m_3	$= \frac{m_3}{m} \times 100$	$C_3 = p_1 + p_2 + p_3$	$100 - C_3$
10	m_4	$= \frac{m_4}{m} \times 100$	$C_4 = p_1 + p_2 + p_3 + p_4$	$100 - C_4$
4.75	m_5	$= \frac{m_5}{m} \times 100$	$C_5 = p_1 + p_2 + p_3 + p_4 + p_5$	$100 - C_5$

$$\boxed{\text{fineness Modulus} = \frac{\Sigma N}{100}}$$

Q. If the % fineness over sieve size IS - 2mm, IS - 600 μ and IS 425 μ are 800%, 50% and 40% Respectively. Calculat %age finer. finess Mudulus.

$$f.M = \frac{\Sigma N}{100} = \frac{80+50+40}{100}$$

$$\boxed{f.M = 1.7}$$

Sedimentation Analysis :- (Used for fine soil)

⇒ Stoke's Law

Assumptions (i) All particles are spherical in shape

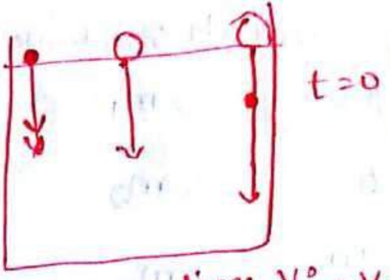
(ii) Density of solids or sp. gravity of solids Remain constan

Definition :-

Acc. to this Law, particles settle down with a constan velocity at which acceleration of particles becomes zero.

- This velocity is known as terminal velocity / settling ve
- This velocity increases with increase in particle size and density of Particles
- This velocity Decreases with viscosity of water.

$$\boxed{V_s = \frac{g}{18} \frac{(\rho_s - \rho_w) D^2}{\mu}}$$



after some time $V_i = V_f$

$\therefore \boxed{a=0}$

$$G = \frac{\rho_s}{\rho_w} \Rightarrow \rho_s = G\rho_w$$

$$V_s = \frac{g}{18} \frac{(G\rho_w - \rho_w) D^2}{\mu} \Rightarrow \frac{g \rho_w (G-1) D^2}{18 \quad \mu}$$

$$\boxed{V_s = \frac{\gamma_w (G-1) D^2}{18 \mu}}$$

$$\boxed{g\rho_w = \gamma_w = \text{unit wt. of water}}$$

Limitations

(i) All particles are not completely spherical

(ii) Density of solids or Sp. gravity Doesnot Remain constant.

(iii) This law is valid for size range $\boxed{0.2m - 0.0002mm\ \mu}$

(iv) Walls of the container offer some frictional Resistance during settlement of the particles.

Sedimentation Analysis (Use for fine soil)

Preparation of soil suspension

(a) Oven Dried fine soil (12g - 30g) + 1 l water $\longrightarrow$ Soil Solution

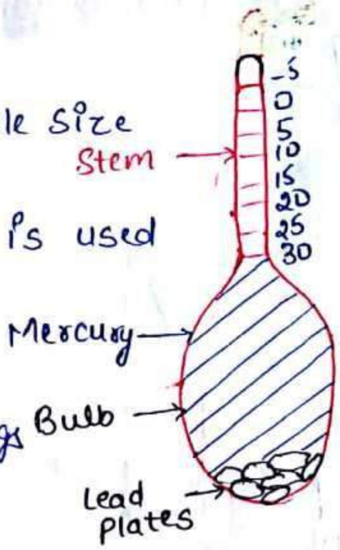
(b) (33g) Sodium Hexa metaphosphate + (7g) Sodium Carbonate + 1 l water $\xrightarrow{\text{Heat}}$ Dispers Agent

(c) Soil Solution + 25ml Dispersing Agent $\xrightarrow[\text{15min cooling}]{\text{10min} \quad \text{10min Heating}}$ Soil Suspens

- $\boxed{\text{Dispersing Agent}}$ is used to remove interparticles forces b/w soil particles.

Hydrometer Method

- This method is used for particle size Distribution of fine soil
- Hydrometer is a Device which is used to calculate sp. gravity or Density of fluid.
- In Hydrometer Methods, 3 readings are taken at top, Middle and bottom Respectively.



$$\rho = 1 + \frac{R_h}{1000}$$

R_h = Reading at Hydrometer

$\boxed{e.g = R_h = 25}$

$$\rho = 1 + \frac{25}{1000} \Rightarrow \boxed{\rho = 1.025\ g/cc}$$

Corrections of Hydrometer

(1) **Correction due to Temperature** :- Hydrometers are Designed / calibrated at 27°C temperature.

(i) If temp is $\boxed{T > 27°C}$ more than 27°C, the viscosity of Mercury will be less and Rise of Mercury will be more. Hence correction will be positive.

(ii) If temp is less than 27°C, the viscosity of Mercury will be more and correction will be negative.

(2) **Dispersing Agent :-(Corrections)** :-

- Addition of Dispersing Agent increases density of soil Suspension and correction will be negative

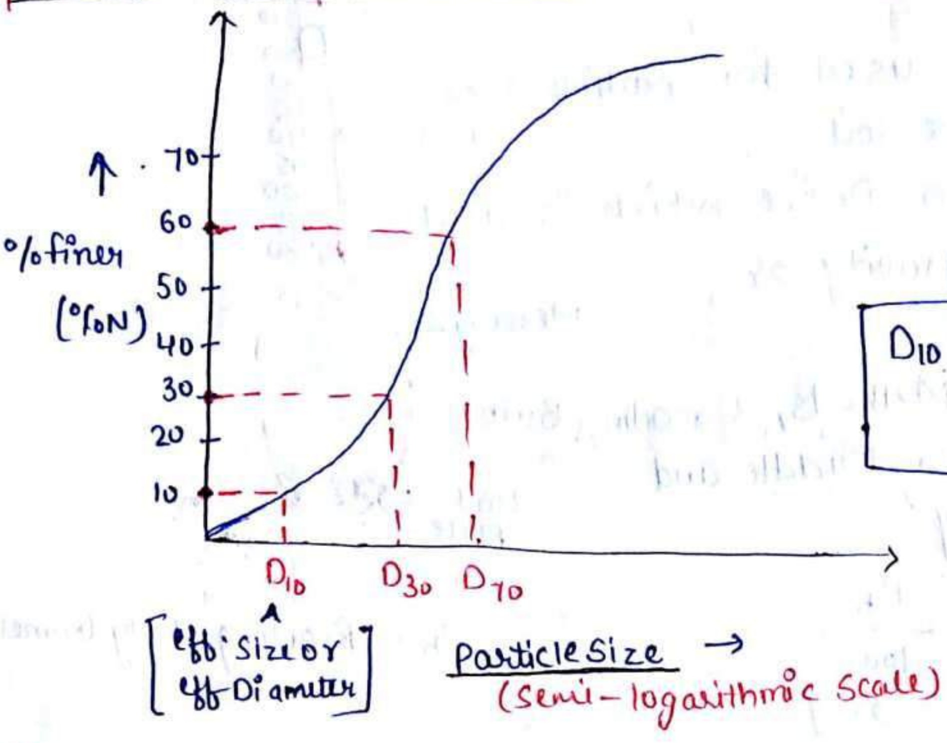
(3) **Correction due to Maniscus**

- Readings are always taken at Lower maniscus but in ca of Mercury lower maniscus is not visible. Hence Redding to be taken at upper Maniscus.
- Hence the correction will be positive.

$$\boxed{R_{hc} = R_h \pm C_T - C_D + C_m}$$

R_{hc} = Corrected Reading of Hydrometer.

Grain Particle size Distribution curve :



$\boxed{D_{10} = \text{eff size OR eff } D_{10}}$

(i) <u>D10</u> : It represents the particle size having 10% particles finer than this size.

- It is also known as effective size or effective Diameter

11 Sep 2018

Uniformity co-efficient : It represents the size Range of particle on particle size distribution curve.

Uniformity co-efficient

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

(i)	for well graded gravels	$C_u > 4$
(ii)	for well graded sand	$C_u > 6$
(iii)	for uniformly graded soil	$C_u \simeq 1$
(iv)	for poorly graded soil	$C_u > 10$

Co-efficient of Curvature

It represents the shape of Particle size distribution curve.

$$C_c = \frac{D_{30}}{10}$$

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

- for well graded soil $\boxed{1 < C_c < 3}$

Ex

Gravel	250g	400 g	900g	500g
Sand	248g	300 g	30 g	—
Silt	246g	200g	30 g	300 g
Clay	256g	100g	40g	200 g
Type of soil.	Well graded Soil	Poorly graded Soil	Uniformly graded	gap graded

The soil

(1) Well graded Soil :- The soil mix which contains a goo Representation of all the particle size, is known as well grac soil.

(2) Poorly graded Soil :- The soil mixture in which some particles are in excess and some particles are in deficier

(3) Uniformly graded soil :- It is a type of poorly grade soil in which almost particles are of same size

(4) Gap Graded Soil :- It is a type of Poorly graded s in which some particle sizes are missing.

Note :- The strength and stability of well graded soil is more than strength and stability of poorly graded soil

Ques A mixture contains $D_{60} = 900 \mu$ and $D_{10} = 300 \mu$ calculate uniformity co-efficient.

$$C_u = \frac{D_{60}}{D_{10}} \Rightarrow \frac{900 \mu}{300 \mu} = 3$$

$$\boxed{C_u = 3}$$

Ques If uniformity co-efficient of a mixture is 4 and co-efficient of curvature is unity. Calculate $\frac{D_{30}}{D_{10}} = ?$

$Cu = 4$

$Cc = 1$

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

$\boxed{D_{60} = 4D_{10}} \Leftarrow \boxed{4 = \frac{D_{60}}{D_{10}}}$

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

$$1 = \frac{D_{30}^2}{D_{10} \times D_{60}} \Rightarrow 1 = \frac{D_{30}^2}{D_{10} \times 4D_{10}} \Rightarrow 4 = \frac{D_{30}^2}{D_{10}^2}$$

$$\sqrt{4} = \sqrt{\frac{D_{30}^2}{D_{10}^2}} \Rightarrow \boxed{\frac{D_{30}}{D_{10}} = 2}$$ Ans.

Ques :- A mixture contains 40% gravels, 50% sand and 10% silt. Calculate uniformity coefficient. (silty gravelly Sand)

Given :- 40% gravels, 50% Sand, 10% Silt

$Cu = ?$

Sol :- Sieve Analysis (> 0.075mm)

- gravel fraction - $\boxed{> 4.75 mm}$
- fine fraction - $\boxed{0.075 - 4.75 mm}$
  Sand

} Concept

40% Particles = size = 4.75mm

60% Particles = Size < 4.75 m $\boxed{D_{60} = 4.75 mm}$

10% Particles = size < 0.075mm $\boxed{D_{10} = 0.075 mm}$

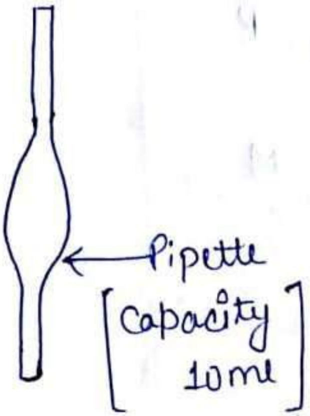
$$\boxed{Cu = \frac{4.75}{0.075} = 63.3}$$ Ans

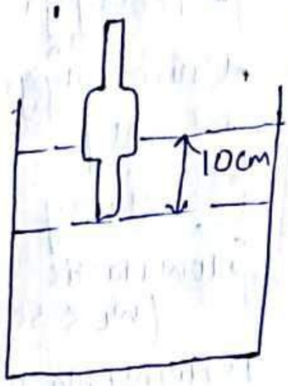
Pipette Method :-

In this Method, a pipettee is used having 10ml capacity.

(ii) Sample is converted into soil suspension and pipettee is inserted at 10cm depth.

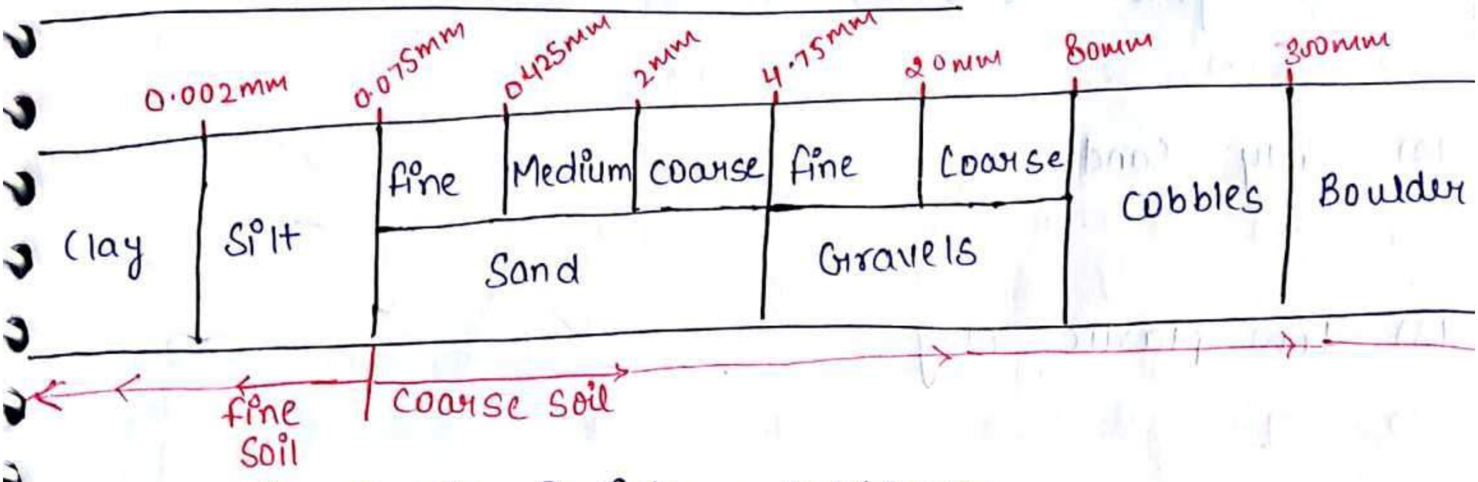
(iii) At 10 cm depth different Samples of suspension are collected at intervals of 30 sec, 1 min, 2 min, 4 min, 8 min, 15 min, 30 min etc. soon





Classification of Soil :- (IS : 1498)

(1) Particle Size classification System :-



- Avg. Size of Silt Particles = 0.062 mm

Q. Which of the following particles are minm in size

a) clay

b) Silts

(c) gravels

✓(d) Colloids (Size < 1 μ)

Soil Particles doesnt Mention otherwise Clay will be answer.

(2) Unified soil classification System :- (USCS)

* Given by casagrande :-

GROUPS		SUB GROUPS	
Gravel	G	• Well graded	W
Sand	S	• Poorly graded OR Uniformly graded	P
Silt	M	• Silty	M
Clay	C	• Clayey	C
Organics	O	• low Plastic ($w_L < 50\%$)	L
		• Highly plastic ($w_L > 50\%$)	H

Examples

(1) Well graded Gravels

⇒ GW

(2) Silty Sand

⇒ SM

(3) low plastic clay

⇒ CL

(4) OH

Highly plastic organics

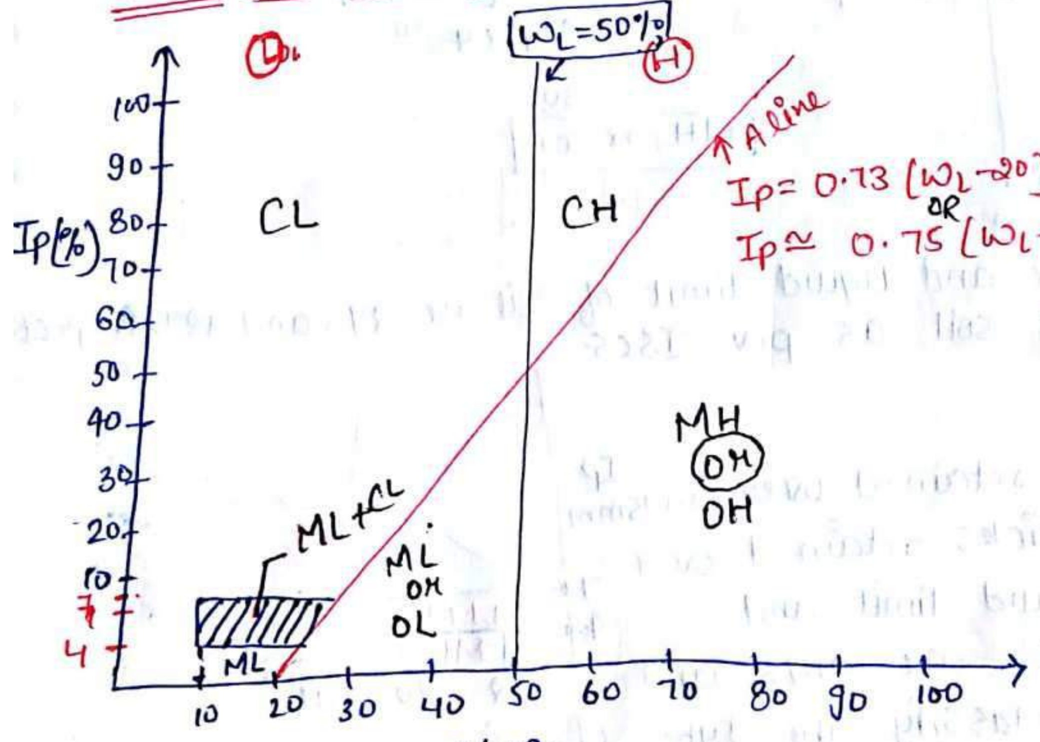
Coarse Soil Classification

Coarse soil More than 50% Particles Retained over 0.075 mm Sieve	**Gravels** More than 50% Particles Retained Over 4.75 mm Sieve	GW
		GP
		GM
		GC
	Sand More than 50% Particles Passing through 4.75 mm Sieve	SW
		SP
		SM
		SC

12 Fine soil classification

L - Low Plasti

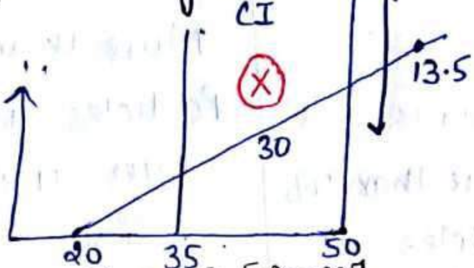
H - Highly Pl



Q. A soil has liquid limit of 70% and lies below A line. classify the soil as per ISCS.

$\boxed{\text{MH or OH}}$

Ques What does X represents from the given Diagram



$\boxed{X = CI}$

Ques A soil has $\boxed{W_L = 40\%}$ and plasticity Index of 28%. Classify soil group as per ISCS

Ans CI

$I_p = 0.75 \, [W_L - 20]$

$I_p = 0.75 \, [40 - 20]$

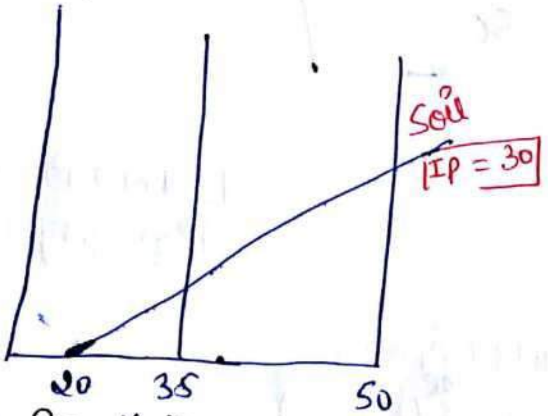
$= 0 \frac{3}{4} \times 20$

$\boxed{I_p = 15}$

***

Ip given > Ip calculated → Clay.
Ip given < Ip calculated → Silt / organics
Ip given = Ip calculated → Boundary soil

Ques If a liquid limit and plastic limit of soil are 60% and 40% respectively classify the soil as per Indian standards



$I_p = 60 - 40 = ⑳$

$I_p = 0.75 \, [60 - 20]$

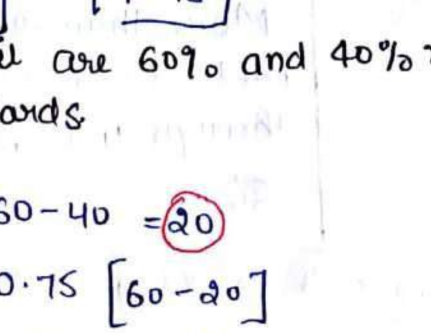
$3/4 \cdot \times 40^{10}$

$㉚$

$\boxed{\text{MH or OH}}$

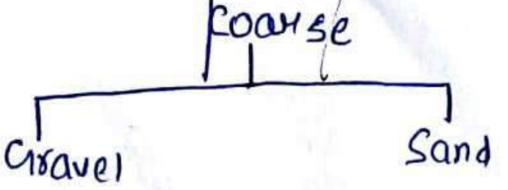
Ques Plasticity Index and liquid limit of soil are 2% and 12% Respectively. classify the soil as per ISCS.

(ML)



Ques 92% particles retained over 0.075mm sieve and 68% particles retained over 4.75 mm sieve, liquid limit and Plastic limit of soil are 40% and 20% Respectively. Classify the type of soil

Coarse soil	fine soil
> 50% → 0.075mm	> 50% Passing.



Coarse

↓ Gravel ↓ Sand

More than 50% Retained On 4.75 mm IS Sieve

- → GW $\boxed{Cu > 4}$
- → GP $\boxed{Cu > 6}$
- → GM $Ip > 4$
- → GC $Ip > 7$

$Ip = 40 - 20$

$\boxed{Ip = 20}$

Ip	
0	Sand
0 - 7	Silt
7 - 17	Silty clay
>17	Clay

Q. w_L and w_P are 60% and 20% respectively. calculate Basis on ISCS

$WL > 50\%$

$\boxed{WL = 50} = H$

$WP = 20\%$

Ip given $w_L - w_P = 60 - 20 = 40$

$Ip_{cal} = 0.75\,(60 - 20)$

$Ip_{cal} = 30\%$

clay $\boxed{CH}$

Indian Standard classification System (IS CS)

★ Coarse soil

Boulder	—	Above 300mm
Cobbles	—	80-300mm
Gravels	G	Coarse (20-80mm)
		fine (4.75-20mm)
Sand.	S	coarse (2-4.75mm)
		Medium (0.425-2mm)
		fine 0.075-0.425

fine soil

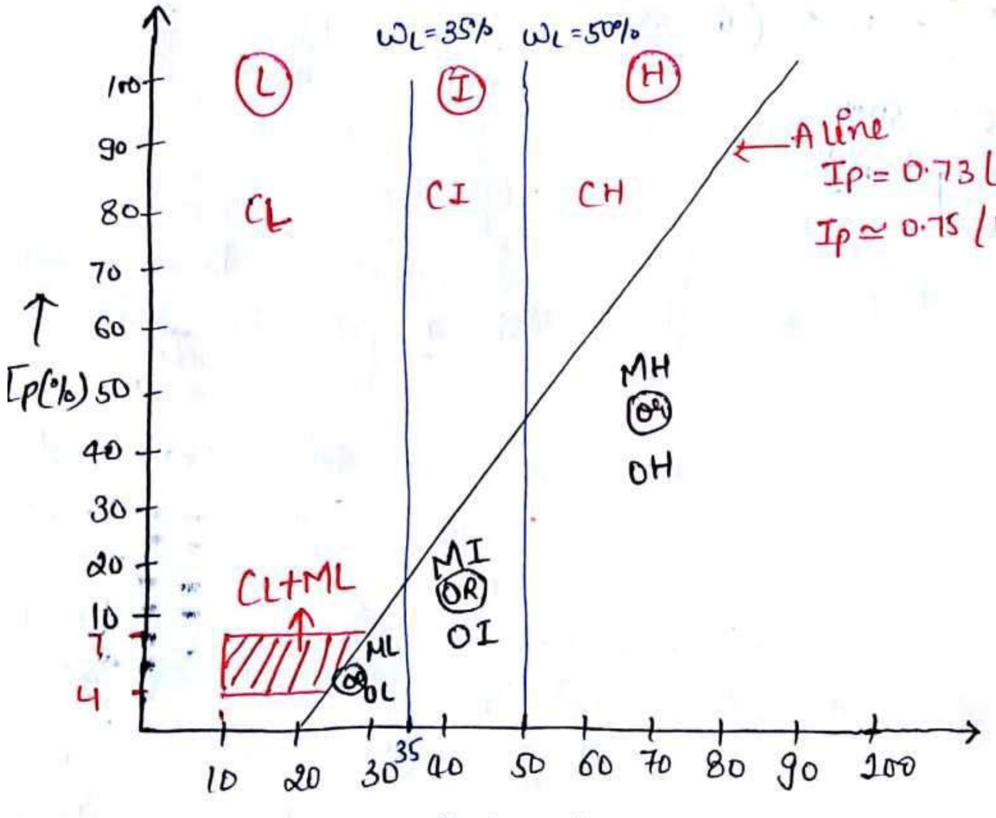
Groups

Silt	M
Clay	C
Organics Muck	O
Peat	Pt

→ Manual Identification (Sponge Nature)

Sub-groups

- Low plastic $(w_L < 35\%)$ L
- Intermediate Plastic $(35 < w_L < 50)$ I
- Highly Plastic $(w_L > 50\%)$ H



- This classification system divides the soil into total 18 groups, 7 Groups for coarse soil, 10 Groups for fine soil from plasticity chart and one group for fine soil from manual identification

Boundary Soil The soil which contains properties of more than 1 group is known as Boundary soil

Which of the following Represent the Symbol for low plastic soil

(a) GW (b) SL [✓ (c) ML] (d) CH

* Low plastic Sand doesnot exist.

(% fineness)
(% Particles < 0.075mm)

Sub - Division of Coarse soil.

% fineness <5%	5% < % fineness < 12%	% fineness > 12%
(a) Gravels More than 50% particles Retained over 4.75mm) i) GW ⇒ $C_u > 4$; $1 < C_c < 3$ ii) GP ⇒ otherwise	a) Gravels More than 50% Particles Retained over 4.75mm) • GW - GC • GW - GM • GP - GC • GP - GM	a) Gravels • GM ($I_p < 4\%$) • GC ($I_p > 7\%$)
(b) Sand (More than 50% particles finer than 4.75 mm) (i) SW ($C_u > 6$; $1 < C_c < 3$) (ii) SP ⇒ otherwise	b) Sand (More than 50% particles finer than 4.75 mm) SW - SC SW - SM SP - SC SP - SM	b) Sand • SM ($I_p < 4\%$) • SC ($I_p > 7\%$)

Ques If 60% particles Retained over 4.75mm sieve and 4% particles are finer than 0.075 m. classify the type of soil if co-efficient of curvature, $C_c = 1.4$, $D_{60} = 90$
$D_{10} = 100\,\mu$.

Sol :- 60% Particles Retained over 4.75 mm sieve

∴ More than 50% particles > 0.075 mm

⇓

coarse soil

60% particle $>$ 4.75mm (gravel)

fineness = 4%

$C_c = 1.4$

Well graded Gravel

$C_u = \frac{D_{60}}{D_{10}} = \frac{0.9}{0.1} \simeq 9$

$C_u > 4$

Q 13% Particles passing through 0.075 mm → coarse soil

70% Particles passing through 4.75mm

$W_L = 18\%$

$W_P = 10\%$

classify type of soil

coarse soil

fineness = 13% , 70% particles $<$ 4.75 mm — sand

$I_p = 18 - 10 = 8$

Se> SC → $I_p > 7\%$.

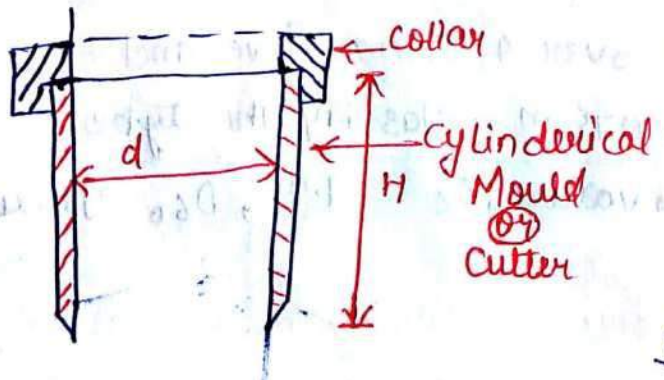
SM → $I_p < 4\%$.

$\boxed{SC}$ Ans

Laboratory Determination of void Ratio

1. Core Cutter Method :-

- Used in soft cohesive soil



M_1 = Mass of empty Mould

M_2 = Mass of empty Mould + soil

V = volume of Mould $= \frac{\pi}{4} d^2 H$

$$\rho = \frac{M_2 - M_1}{V} \; ; \; \rho_d = \frac{\rho}{1+w}$$

$$\rho_d = \frac{G\rho_w}{1+e} \Rightarrow \boxed{e = \frac{G\rho_w}{\rho_d} - 1}$$

Ques consider the Given observation During core cutter Method

Mass of empty Mould = 1200g

" " " " +Soil = 3200g

vol. of Mould = 1000cm3

water content = 15%

Specific Gravity = 2.7

Calculate

i) Bulk Density of soil

ii) Degree of saturation

$$\rho = \frac{M_2 - M_1}{V} = \frac{3200 - 1200}{1000} = \frac{2000}{1000} = \boxed{2\ g/cc}$$

$$\rho_d = \frac{2}{1 + 0.15} = 1.73\ g/cc$$

$$e = 1.73 \times \qquad e = \frac{2.7 \times 1}{1.73} - 1$$

$$e = 0.56$$

$$eS = WG$$

$$S = \frac{2.7 \times 0.15}{0.56} = 0.723$$

$$\boxed{S = 0.723}$$

Alternative Method

$$Se = WG$$

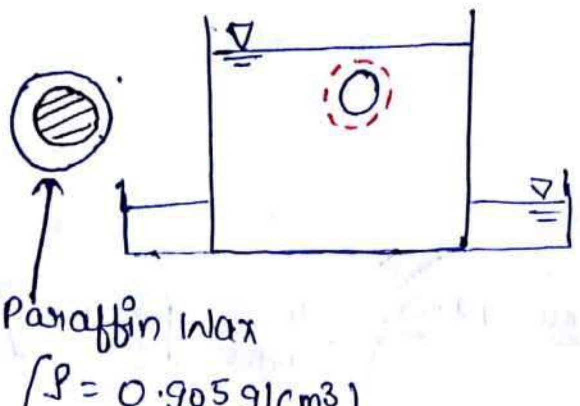
$$\rho = \frac{(G + Se)\rho_w}{1 + e} \Rightarrow \frac{(G + WG)\rho_w}{1 + \frac{WG}{S}}$$

$$2 = \frac{(2.7 + 0.15 \times 2.7) \times 1}{1 + \frac{0.15 \times 2.7}{S}}$$

$$\boxed{S = 0.73}$$

② Water Displacement Method :-

- Used for cohesive soil
  (any shape of soil sample) (Regular or Irregular shape)



M_{soil} = Mass of soil

M = Mass of soil + wax

V_w = vol. of water Displaced = vol. of soil + vol. of wax

$V_{soil} = V_w - V_{wax}$

$= V_w - \frac{m_{wax}}{\rho_{wax}}$

$$\boxed{V_{soil} = V_w - \frac{M - M_{soil}}{\rho_{wax}}}$$

$\rho = \frac{M_{soil}}{V_{soil}}$; $\rho_d = \frac{\rho}{1+w}$ $\boxed{e = \frac{G\rho_w}{\rho_d} - 1}$

Q Mass of soil sample before and after coating of wax are 645 gms and 650 gms Resp. The vol of water Displaced is 370 ml. Calculate (i) ρ_d, (ii) void Ratio (iii) w_{sat}. if the water content of sample is 15%. $G_{solid} = 2.7$; $G_{wax} = 0.9$

$M_{soil} = 645\,gm$ $M = 650\,gm$

$V_w = 370\,ml = 370\,cm^3$

$V_{soil} = V_w - V_{wax}$

V_{soil} $370 - \frac{m_{wax}}{\rho_{wax}} \Rightarrow 370 - \frac{10}{0.9}$

$$\boxed{V_{soil} = 358.8\,cm^3}$$

$\rho_{soil} = \frac{645}{358.8} = 1.797\,g/cc$

$$\rho_d = \frac{\rho_{soil}}{1+w} \Rightarrow \frac{1.797}{1+0.15} = 1.56 \text{ g/cc}$$

$$\rho_d = \frac{G\rho_w}{1+e} \Rightarrow 1.56 = \frac{2.7\times1}{1+e}$$

$$\boxed{e = 0.7}$$

- $Se = wG$

$$1\times0.7 = W\times2.7$$

$$\boxed{W = 0.27 = 27\%}$$

Concept

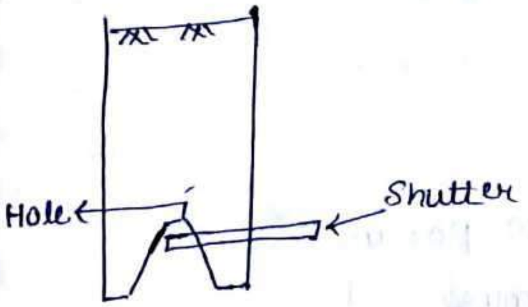
$$G = \frac{\rho}{\rho_w} =$$

$$0.9 = \frac{\rho_{wax}}{1}$$

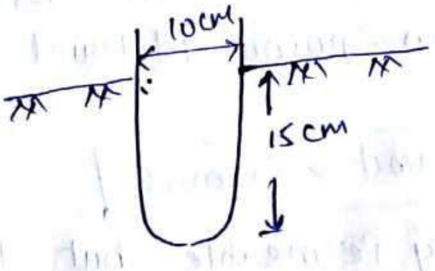
$$\rho_{wax} = 0.9$$

③ Sand Replacement Method :

- It is a field Method
- It is used for Hard soil, Rock
- It is used to calculate field Density of soil
- Enore sand is used



Pouring Cylinder



Msoil = Mass of soil

Vsoil = Vol. of soil = Vol. of Test Pit

Vol. Pit = Vol. of sand filled

$$\boxed{\text{Vol. of Pit} = \frac{M_{sand\ filled}}{\rho_{d\,sand}}}$$

$$\rho = \frac{M_{soil}}{V_{soil}} \; ; \quad \rho_d = \frac{\rho}{1+w}$$

$$\boxed{e = \frac{G\rho_w}{\rho_d} - 1}$$

Unit - II

Permeability of soil

Defn :- The property of soil which allows the movement of water from its interconnecting voids is known as permeability of soil.

- The process of Movement of water from soid voids is known as seepage.
- The Material which allows the Movement of water from it, is known as Permeable Material.
- The Material which doesnot allow Movement of water, from it, is known as impermeable material.
- Permeability of soil increases with void size.

Permeability order

$$\boxed{\text{gravel} > \text{sand} > \text{silt} > \text{clay}}$$

- The material which can store water in its voids is known as Porous Material
- The material which can not store water in its voids is known as non - porous Material

Porosity Order

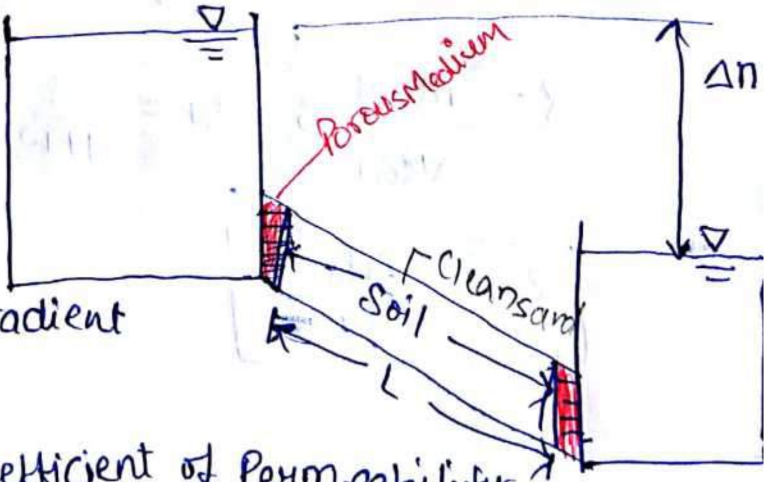
$$\boxed{\text{clay} > \text{sift} > \text{sand} > \text{Gravels}}$$

- Gravels are highly Permeable but less porous.
- Clay is less permeable but highly porous.

Darcy's Law :-

Acc. to this law velocity of flowing water from soil c/s is directly proportional to Hydraulic gradient

$$\left.\begin{aligned} q &\propto A \\ q &\propto \Delta h \\ q &\propto \frac{1}{L} \end{aligned}\right\} \Rightarrow q \propto \frac{A\Delta h}{L}$$



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

$q \propto = iA$ ~~kiA~~

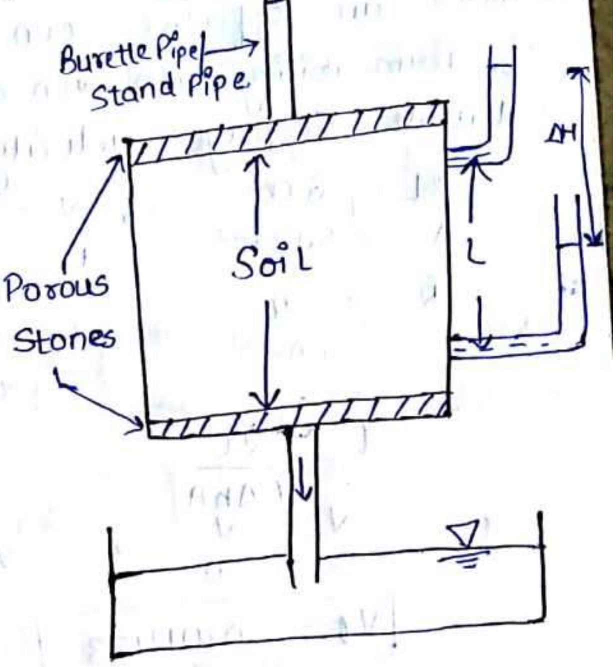
$\boxed{q = kiA}$ k = co-efficient of Permeability

i = head lost due to friction per unit length of tube = h

17 Sep 2018 (V refered to superficial velocity of flow, superficial or fictious. because water is passing through pores not the entire Cross-section)

Laboratory determination of Permeability =

1. Constant Head Permeability Test



* Used for coarse soil / fine sand / silts

$$q = kiA$$

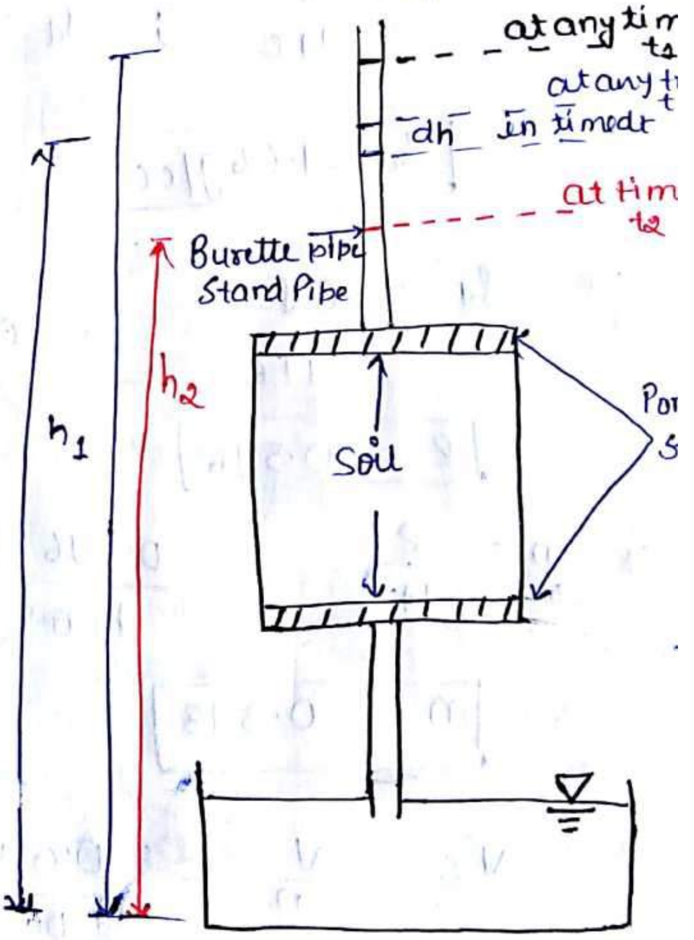
$$q = k\frac{\Delta h}{L}A$$

$$\boxed{k = \frac{qL}{\Delta h A}} \; ; \; q = \frac{V}{t}$$

$$\boxed{k = \frac{VL}{t\,\Delta h A}} \Rightarrow k = \frac{QL}{Aht}$$

(2) falling variable Head Permeability Test

* Used for fine soil



$$q = a\left(-\frac{dh}{dt}\right) = kiA$$

-ve sign indicates Head of water is decreasing.

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

$$-\int_{h_1}^{h_2} \frac{-dh}{h} = \frac{Ak}{aL}\int_{t_1}^{t_2} dt$$

$$-\left[\ln h\right]_{h_1}^{h_2} = \frac{Ak}{aL}\left[t\right]_{t_1}^{t_2}$$

$$-\left[\ln h_2 - \ln h_1\right] = \frac{Ak}{aL}\left[t_2 - t_1\right]$$

$$\ln h_1 - \ln h_2 = \frac{Ak}{at}\left[t_2 - t_1\right]$$

$$\ln h_1 - \ln h_2 = \frac{Ak}{aL}\Delta t$$

$$\ln\frac{h_1}{h_2} = \frac{Ak}{aL}\Delta t \Rightarrow k = \frac{aL}{A\Delta t}\ln\left[\frac{h_1}{h_2}\right]$$

$$\boxed{k = 2.303\,\frac{aL}{A\Delta t}\log_{10}\left(\frac{h_1}{h_2}\right)}$$

a = c/s area of Stand Pipe

A = c/s area of So
Sample

Ques A Sample is 6cm in Height and 50 cm² in C/S area. The Qty of water 430ml is passing down in 10 min from soil Sample under an effective constant Head of 40 cm. calculate co-efficient of Permeability. On oven drying the mass of sample is 498gms Calculate Seepage velocity of soil. Take $\boxed{G = 2.65}$

$L = 6\text{ cm}$; $V = 430\text{ml} = 430\text{ cm}^3$; $\Delta h = 40\text{ cm}$

$A = 50\text{ cm}^2$; $t = 10\text{ min} = 600\text{ sec}$

$$k = \frac{VL}{t\Delta h A} = \frac{430\times6}{600\times40\times50} = \boxed{k = 215\times10^{-3}\ \text{cm/sec}}$$ Ans

* $v_s = \frac{v}{n}$; $v = ki = k\frac{\Delta h}{L} = 2.15\times10^{-3}\times\frac{40}{6}$

$$\boxed{v = 0.01433}$$

* $n = \frac{e}{1+e}$; $\rho_d = \frac{m_d}{V_{\text{soil sample}}} = \frac{498}{50\times6} = \frac{498}{300}$

$$\boxed{\rho_d = 1.66\ \text{g/cc}}$$

* $\rho_d = \frac{G\rho_w}{1+e} \Rightarrow e = \frac{2.65\times1}{1.66} - 1$

$$\boxed{e = 0.596}$$

* $n = \frac{e}{1+e} = \frac{0.596}{1+0.596}$

$$\boxed{n = 0.373}$$

* $v_s = \frac{v}{n} = \frac{0.014}{0.37}$

$$\boxed{v_s = 0.037\ \text{cm/sec}}$$ Ans

Ques During constant Head Permeability Test if effective Head get Reduced to 50%. Then Permeability of soil will be

(i) increase by 50%

✓(ii) increase by 100%

(iii) increase by 200%

(iv) No increase

$$k = \frac{VL}{t\Delta h A} = \frac{VL}{t\left(\frac{\Delta h}{2}\right)A} = \frac{2VL}{t\Delta h A}$$

$$\boxed{k = 2k}$$

$\boxed{\text{increase by } 100\%}$

$\boxed{\text{increased to } 200\%}$

Ques Calculate co-efficient of Permeability of soil sample 6 cm Ht. and 50 cm² in c/s. If the Qty of water 450 ml is passing Down in 15 min under constant effective Head of 45 cm. If the void Ratio of sample gets increased from 0.3 to 0.6 calculate the exact value of new coefficient of Permeability.

Sol:- $l = 6cm$ $\quad V = 450ml = 450\,cm^3$

$A = 50\,cm^2$ $\quad t = 15\,min = 15 \times 60 = 900\,sec$

$\Delta h = 45\,cm$ $\quad e_1 = 0.3 \quad e_2 = 0.6$

$$k = \frac{VL}{t\Delta h A} = \frac{450 \times 6}{900 \times 45 \times 50}$$

$$\boxed{k = 1.33 \times 10^{-3}\ cm/sec}$$

$$\frac{k_1}{k_2} = \frac{\frac{e_1{}^3}{1+e_1}}{\frac{e_2{}^3}{1+e_2}} \Rightarrow \frac{1.33 \times 10^{-3} \times \frac{0.6^3}{1+0.6}}{\frac{0.3^3}{1+0.3}} = k_2$$

$$\boxed{k_2 = 8.645 \times 10^{-3}\ cm/sec}$$ Ans

Ques In a falling Head Permeability Test the Head of water g reduced to 35cm from initial Head of 40cms in 10 mins. Cal -ate the co-efficient of Permeability of soil sample 6cm in Ht and 50cm² in c/s. Take c/s area of stand pipe as 0.2 cm²

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

$$= \frac{2.303 \times 0.2 \times 6}{50 \times 10 \times 60}\log_{10}\left[\frac{40}{35}\right]$$

[illegible]

$$\boxed{k = 5.34 \times 10^{-6}\ cm/sec}$$

Ques Equal time intervals were noted down in a falling Head Permeability Test when head of water gets reduced from h_1 to h_2 and from h_2 to h_3 Calculate the Relation b/w h_1, h_2, h_3 ,?

$$k_1 = 2\cdot303 \frac{al}{A\Delta t} \log_{10}\left(\frac{h_1}{h_2}\right) \quad — ①$$

$$k_2 = 2\cdot303 \frac{al}{A\Delta t} \log_{10}\left(\frac{h_2}{h_3}\right) \quad — (2)$$

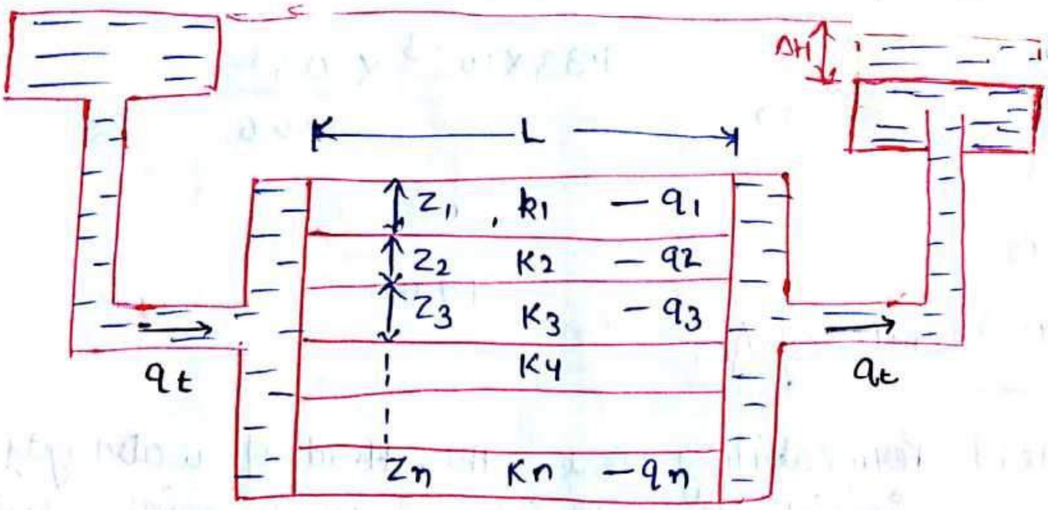
$$\cancel{2.303 \frac{al}{A\Delta t}} \log_{10}\left(\frac{h_1}{h_2}\right) = \cancel{2\cdot303 \frac{al}{A\Delta t}} \log_{10}\left(\frac{h_2}{h_3}\right)$$

$$\cancel{\log_{10}}\left(\frac{h_1}{h_2}\right) = \cancel{\log_{10}}\left(\frac{h_2}{h_3}\right)$$

$$\frac{h_1}{h_2} = \frac{h_2}{h_3} \Rightarrow \boxed{h_2^2 = h_1 h_3}$$

Permeability of Stratified Soil Deposits

(i) When flow is parallel to Bedding Plane



$$q = kiA$$

$$q = k\frac{\Delta h}{L}A$$

$\Delta h, L =$ const

$i =$ const

$$\boxed{q_t = K_H i b z_t}$$

$q_1 = k_1 i b z_1$

$q_2 = k_2 i b z_2$

$q_3 = k_3 i b z_3$

$\vdots$

$q_n = k_n i b z_n$

$$q_t = q_1 + q_2 + q_3 + - - - - + q_n$$

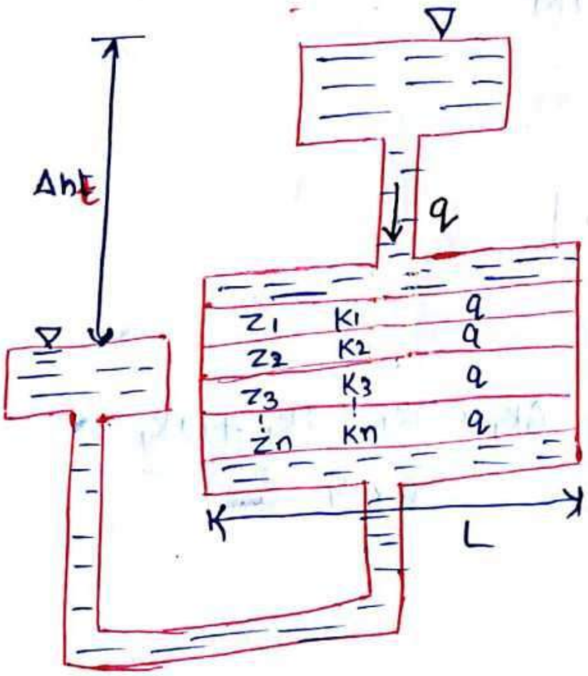
$$k_H i b z_t = k_1 i b z_1 + k_2 i b z_2 + k_3 i b z_3 + - - - - + k_n i b z_n$$

$$k_H z_t = k_1 z_1 + k_2 z_2 + k_3 z_3 + - - - - k_n z_n$$

$$\boxed{K_H = \frac{k_1 Z_1 + k_2 Z_2 + k_3 Z_3 + - - - k_n Z_n}{Z_t}}$$

$$Z_t = z_1 + z_2 + z_3 + - - - - + z_n$$

(ii) When flow is normal to bedding plane



$$A = \overset{\overset{\text{Const}}{\downarrow}}{L} \times b \leftarrow \text{const}$$

$$A = \text{cost.}$$

Assumption

$$\boxed{\text{Unit time}, \ q = \text{const}}$$

$$V = \frac{q}{A} = \text{const.}$$

$$v = ki = k\frac{\Delta h}{Z}$$

$$\Delta h = \frac{VZ}{k}$$

$$\Delta h_1 = \frac{VZ_1}{k_1}$$

$$\Delta h_2 = \frac{VZ_2}{k_2} \ - - - \ \Delta h_n = \frac{VZ_n}{K_n}$$

$$\Delta h_t = \frac{VZ_t}{K_V}$$

$$\Delta h_t = \Delta h_1 + \Delta h_2 + \Delta h_3 + - - - \Delta h_n$$

$$\frac{V Z_t}{k_v} = V\frac{Z_1}{k_1} + V\frac{Z_2}{k_2} + V\frac{Z_3}{k_3} + - - - V\frac{Z_n}{k_n}$$

$$\frac{Z_t}{k_v} = \frac{Z_1}{k_1} + \frac{Z_2}{k_2} + \frac{Z_3}{k_3} + - - - - \frac{Z_n}{k_n}$$

$$\boxed{K_V = \frac{Z_t}{\frac{Z_1}{k_1} + \frac{Z_2}{k_2} + \frac{Z_3}{k_3} + - - - \frac{Z_n}{k_n}}}$$

$$Z_t = Z_1 + Z_2 + Z_3 + - - - - + Z_n$$

Q A Deposit contains four layers of Soil having equal thickness. The co-efficient of permeability of 2nd layer, 3rd layer and 4th layer are $\frac{1}{2}$, $\frac{1}{3}$rd and 2 times permeability of Ist layer Respectively. Calculate Co-efficient of Permeability of complete Deposit along horizontal and vertical Directions.

$$K_2 = \frac{1}{2}K_1 \quad , \quad K_3 = \frac{1}{3}K_1 \qquad K_4 = 2K_1$$

$Z_1 = Z_2 = Z_3 = Z_4$ (Equal thickness)

$$K_H = \frac{Z_1K_1 + Z_2K_2 + Z_3K_3 + Z_4K_4}{Z_1 + Z_2 + Z_3 + Z_4}$$

$$K_H = \frac{Z_1K_1 + Z_1\frac{K}{2} + Z_1\frac{K_1}{3} + Z_1 2K_1}{Z_1 + Z_1 + Z_1 + Z_1}$$

$$K_H = \frac{\cancel{Z_1}\left[K_1 + \frac{K_1}{2} + \frac{K_1}{3} + 2K_1\right]}{4\cancel{Z_1}}$$

$$K_H = \frac{K_1 + \frac{K_1}{2} + \frac{K_1}{3} + 2K_1}{4} = \frac{6K_1 + 3K_1 + 2K_1 + 12K_1}{6 \times 4}$$

$$\boxed{K_H = \frac{23}{24}K_1}$$

$$K_V = \frac{Z_1 + Z_2 + Z_3 + Z_4}{\frac{Z_1}{K_1} + \frac{Z_2}{K_2} + \frac{Z_3}{K_3} + \frac{Z_4}{K_4}}$$

$$K_V = \frac{Z_1 + Z_2 + Z_3 + Z_4}{\frac{Z_1}{K_1} + \frac{Z_2}{\frac{K_1}{2}} + \frac{Z_3}{\frac{K_1}{3}} + \frac{Z_4}{2K_1}} = \frac{4}{\frac{1}{K_1} + \frac{2}{K_1} + \frac{3}{K_1} + \frac{1}{2K_1}}$$

$$\boxed{K_V = \frac{8}{13}K_1}$$

$\boxed{K_H > K_V}$ Should always be greater than

18 Sep 2018

Q. 1m3 embankment has $\gamma_d = 16\ KN/m3$ & Contains sand If 5.5 KN silt is added to embankment, vol. of embankment increases by 20%. Calculate the change on porosity of soil. Assume $G_{sand} = G_{silt} = 2.67$

Sol 1m3 embankment + 5.5 KN silt (vol. increase by 20%.)

$$\boxed{\gamma_d = 16\ KN/m3}$$

Sol

$$\gamma_d = \frac{G\gamma_w}{1+e_1}$$

$$e_1 = \frac{2.67\times9.81}{16} - 1$$

$$e_1 = 0.64$$

$$n_1 = \frac{e_1}{1+e_1} = \frac{0.64}{1+0.64}$$

$$0.39$$

$V = 1.2\ m3$

initial weight = 16KN

Silt = 5.5KN

Total wt = 16 + 5.5 = 21.5 KN

$$\gamma_d = \frac{21.5}{1.2} = 17.9\ KN/m3$$

$$e_2 = \frac{2.67\times9.81}{17.9}$$

$$e_2 = 1.46$$

$$n_2 = \frac{1.46}{2.46} = 0.59$$

$$n_2 - n_1 = 0.59 - 0.39$$
$$= 0.20$$

Q. Three layer of soil having equal thickness. Coefficient of Permeability of 1st and 3rd layer is 0.01 cm each. coefficient of Permeability of 2nd layer is 0.1 cm/sec. The Ratio of co-efficient of Permeability Horizontal Direction to vertical Direction

$k_1 = 0.01$ cm/sec

$k_3 = 0.01$ cm/sec

$k_2 = 0.1$ cm/sec

$$\frac{k_H}{k_V} = ?$$

$$K_H = \frac{Z_1K_1 + Z_2K_2 + Z_3K_3}{Z_1+Z_2+Z_3}$$

$$K_H = \frac{\cancel{Z_1}[0.12]}{3\cancel{Z_1}} = 0.04$$

$$K_V = \frac{Z_1+Z_2+Z_3}{\frac{Z_1}{K_1} + \frac{Z_2}{K_2} + \frac{Z_3}{K_3}} = \frac{3Z_1}{\frac{10Z_1 + 10Z_2 + Z_1}{0.1}}$$

$$\frac{3\cancel{Z_1}\times0.1}{21Z_1} = 0.014$$

$$\boxed{\frac{K_H}{K_V} = \frac{0.04}{0.014} = 0.2.85:1}$$ Ans

Q A Deposit contains 3 layers of soil. The thickness and permeability of 2nd layer is twice of that of 1st layer. The thickness and Permeability of third layer is twice of that of 2nd layer. calculate $\frac{k_H}{k_V}$ Ratio.

$Z_2 = 2Z_1$

$K_2 = 2K_1$

$Z_3 = 2 \times 2Z_1 = 4Z_1$

$K_3 = 2 \times 2K_1 = 4K_1$

$$K_H = \frac{Z_1K_1 + 2Z_1 \cdot 2K_1 + 4Z_1 \cdot 4K_1}{Z_1 + 2Z_1 + 4Z_1}$$

$$= \frac{Z_1K_1 + 4Z_1K_1 + 16Z_1K_1}{7Z_1}$$

$$= \frac{\overset{3}{\cancel{21}}Z_1K_1}{\cancel{7Z_1}}$$

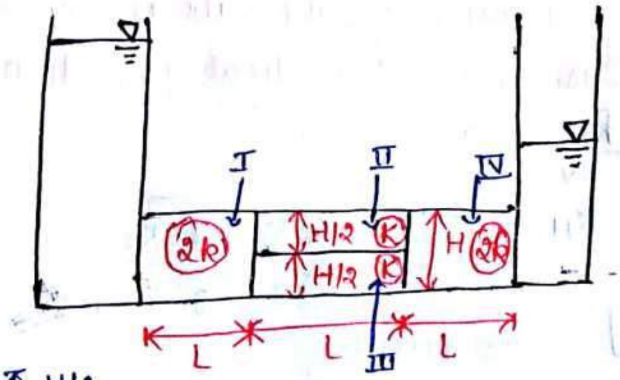
$$\boxed{K_H = 3K_1}$$

$$K_V = \frac{Z_1 + 2Z_1 + 4Z_1}{\frac{Z_1}{K_1} + \frac{2Z_1}{2K_1} + \frac{4Z_1}{4K_1}} = \frac{7Z_1}{\frac{4Z_1 + 4Z_1 + 4Z_1}{4K_1}} = \frac{7\cancel{Z_1} \times \overset{1}{\cancel{4}}K_1}{\underset{3}{\cancel{12}}\cancel{Z_1}}$$

$$\frac{K_H}{K_V} = \frac{3\cancel{K_1}}{\frac{7\cancel{K_1}}{3}} = \frac{9}{7}$$

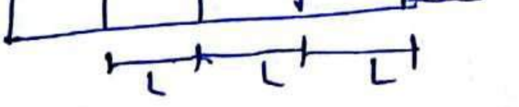
$$\boxed{K_H : K_V = 9:7}$$ Ans

Ques



Calculate equivalent Permeability ?

$$k_1 = \frac{k_1z_1 + k_2z_2}{z_1 + z_2} = \frac{k \cdot \frac{H}{2} + k \cdot \frac{H}{2}}{H/2 + H/2} = \text{Ⓚ}$$



$$k_{eq} = \frac{z_1 + z_2 + z_3}{\frac{z_1}{k_1} + \frac{z_2}{k_2} + \frac{z_3}{k_3}}$$

$$k_{eq} = \frac{L + L + L}{\frac{L}{2K} + \frac{L}{k} + \frac{L}{2K}}$$

$$k_{eq} = \frac{3L}{\frac{4L}{2K}} = \frac{6LK}{4L} = \frac{3}{2}K$$

$$\boxed{K_{eq} = \frac{3}{2}K}$$

Field Determination of Permeability (Pumping out Test)

Pumpin· Well Hydraulics :- It is the branch of Hydrology which deals with geological formations and their Related Movement of water.

Types of Geological formation

1. Aquifer :- (Porous + Permeable)
The geological formation which can store water (porous) and which can yield / discharge water (Permeable) is known as aquifer.

2. Aquicludes (Highly Porous + Non Permeable)
It is the geological formation which is highly porous but non permeable.
eg :- Clay.

3. Aquitard :- It is the geological formation which is porous but very less permeable
eg :- Sandy clay.

4. Aquifuge :- The geological formation which is neither porous norpermeab
eg :- Rocks (without fracture), Granite etc.

Types of Aquifer

1. Unconfined Aquifer
2. Confined Aquifer

1. Unconfined Aquifer :- The Aquifer which Rest over impermeable layer is known as unconfined Aquifer.

- This Aquifer is subject to atmospheric pressure and it is also knov as free aquifer / Non Artesian aquifer.

2. Confined Aquifer :-

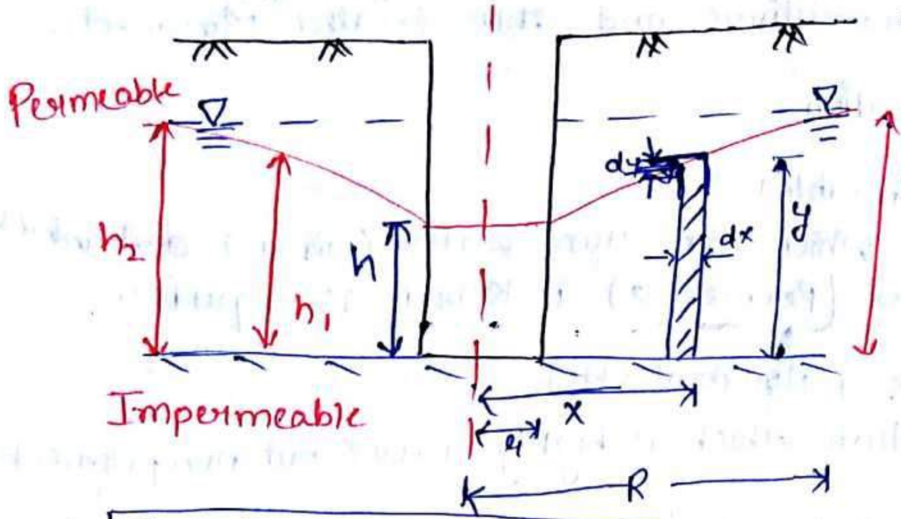
- The Aquifer which lies b/w two impermeable layers and it is subj to over burden pressure of top imperable layer.
- This Aquifer is also known as artesian Aquifer.

1. Specific Yield :- The amount of water that can be yielded discharged from geological formation into well is known as Specific Yield.

2. Specific Retention :- The amount of water that cannot be yie but gets Retained in the soil voids, is known as specific Reten

$$\boxed{S_y + S_R = n}$$

1. Unconfined Aquifer :-



$$\boxed{S = (H-h) = \text{Draw Down}}$$

R = Radius of influence well

$$q = kiA$$

$$q = k\frac{dy}{dx}\left[2\pi x y\right] \Rightarrow q\int_r^R \frac{dx}{x} = 2\pi k\int_h^H y\,dy$$

$$q = \left[\ln x\right]_r^R = 2\pi k\left[\frac{y^2}{2}\right]_h^H$$

$$q = \left[\ln R - \ln r\right] = 2\pi k\left[\frac{H^2-h^2}{2}\right]$$

$$\boxed{\ln m_1 - \ln m_2 = \ln\frac{m_1}{m_2}}$$

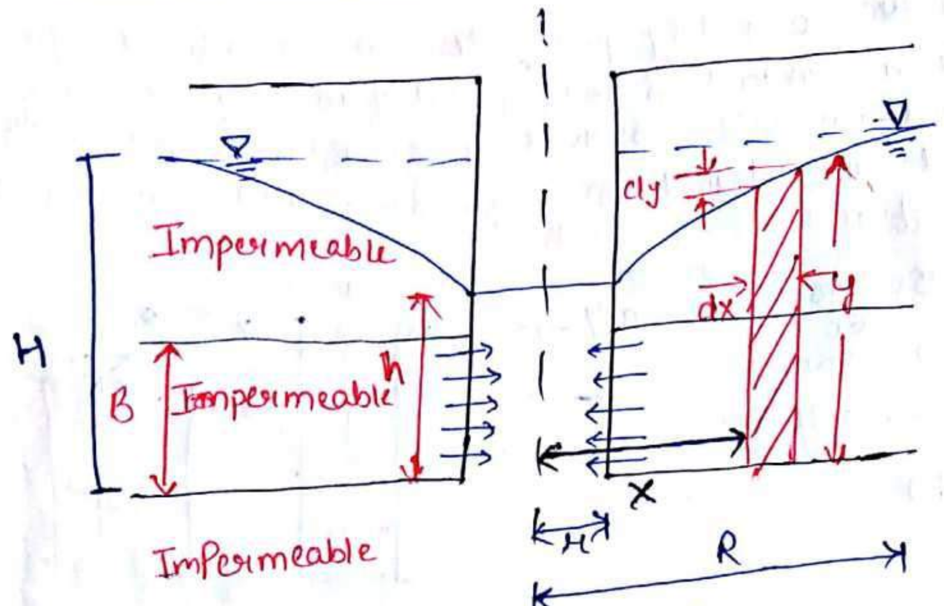
$$q \ln\left[\frac{R}{r}\right] = \pi k\left(H^2-h^2\right)$$

$$\boxed{q = \frac{\pi k\left(H^2-h^2\right)}{2.303\log_{10}\left(\frac{R}{r}\right)}}$$

$$\boxed{q = \frac{1.36\,k\,\left(H^2-h^2\right)}{\log_{10}\left(\frac{R}{r}\right)}} \Rightarrow \text{Dupit's Theory}$$

$$\boxed{q = \frac{1.36\,k\left(h_2^2-h_1^2\right)}{\log\left(\frac{r_2}{r_1}\right)}} \Rightarrow \text{Theim's Theory}$$

2. Confined Aquifer



$$q = kiA$$

$$q = k\frac{dy}{dx}\,2\pi x B \Rightarrow q\int_{r}^{R}\frac{dx}{x} = 2\pi k B\int_{h}^{H}dy$$

$$q\left[\ln x\right]_{r}^{R} = 2\pi k B\left[y\right]_{h}^{H}$$

$$q\left[\ln R - \ln r\right] = 2\pi k B\left[H-h\right].$$

$$q\ \ln\frac{R}{r} = 2\pi R B\ (H-h]$$

$$\boxed{q = \frac{2\pi k B\left[H-h\right]}{2.303\ \log_{10}\left(\frac{R}{r}\right)}}$$

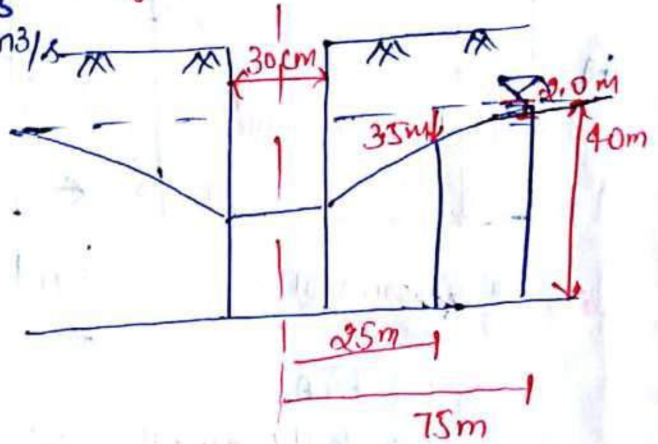
$$\boxed{q = \frac{\overset{2.72}{\cancel{2\pi}}\,k B S}{\log_{10}\left(\frac{R}{r}\right)}} \Longrightarrow$$ Dupit's Theory

$$\boxed{q = \frac{2.72\ kB\ (h_2-h_1)}{\log_{10}\frac{r_2}{r_1}}} \Rightarrow$$ Theim's Theory.

19 Sep 2018 lpm = ~~lp~~ litre per minute AE-JE 85 5:7

Q A 30cm diameter valve completely penetrates an unconfined aquifer of depth 40m. After a constant & long period of Pumping at rate of 1500lpm. Two drawndown were observed at a Distance of 25m & 75m from centre of well. The drawdowns are 3.5m and 2m respectively. Calculate co-efficient of Permeability.

Sol:- $q = 1500\,lpm = \frac{1500\times10^{-3}}{60} = \cancel{0.25}\ 0.025\ m^3/s$



$h_2 = H - s_2 = 40 - 2 = 38m$

$h_1 = H - s_1 = 40 - 3.5 = 36.5m$

$$q = \frac{1.36k(h_2^2 - h_1^2)}{\log_{10}\left(\frac{r_2}{r_1}\right)}$$

$$0.025 = \frac{1.36k\left[(38)^2 - (36.5)^2\right]}{\log_{10}\left(\frac{75}{25}\right)}$$

$$\boxed{k = 7.84\times10^{-5}\ m/sec}$$

Ques A layer of sand 10m thick overlies an impervious layer. The water table lies at 0 1.5m from ground surface. after constant pumping of water @ 100 lps rate. The drawdown of 3m & .5m were observed at a distance of 3.0m and 25m from centre of well. calculate Co-efficient of Permeability.

$q = 100\,lps = \frac{100\times10^{-3}}{60}\ m^3/sec$

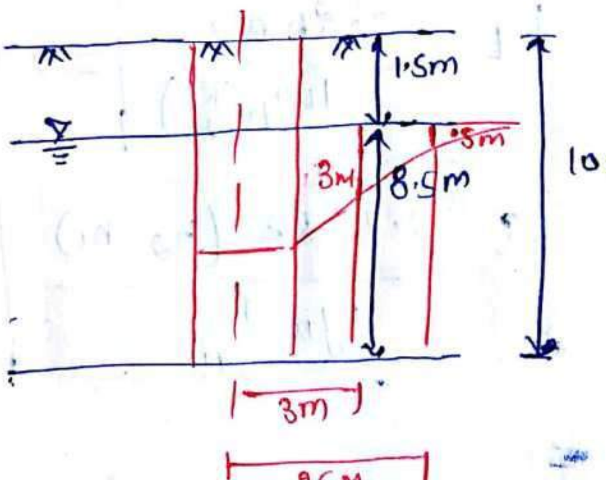
$S_1 = 0.5m \quad S_2 = 3m$

$H = 8.5$

$h_2 = 8.5 - 0.5 = 8m$

$h_1 = 8.5 - 3 = 5.5m$

$r_2 = 25m$; $r_1 = 3m$

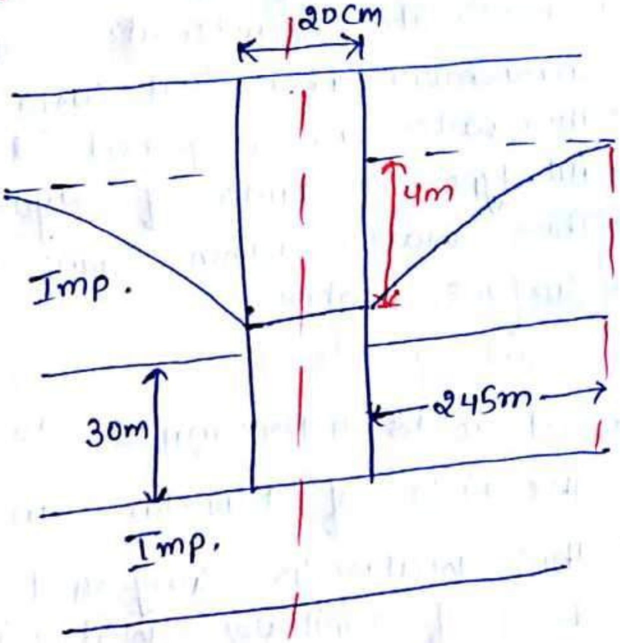


$$q = \frac{1.36k\,(h_2^2 - h_1^2)}{\log_{10}\left(\frac{r_2}{r_1}\right)}$$

$$\frac{\frac{100\times10^{-3}}{60}\times\log_{10}\left(\frac{25}{3}\right)}{1.36\,(8^2 - 5.5^2)} = k$$

$$\boxed{k = 2.00\times10^{-3}\ m/sec}$$ Ans

Ques A 30m thick permeable layer lies b/w two impermeable layers. The draw-down at centre of 20 cm diameter well is 4m. Calculate the Co-efficient of Permeability if Discharge is 40l/sec. Radius of influence well is 245 m.



$$q = 40 lps = 40 \times 10^{-3}$$
$$= 0.04\ m^3/s$$

$$r = \frac{20}{2} = 10cm = 0.1\ m$$

$$q = \frac{2.72\ KB\,(H-h)}{\log_{10}\left(\frac{R}{r}\right)}$$

$$q = \frac{2.72\ KB\,s}{\log_{10}\left(\frac{R}{r}\right)}$$

$$0.04 = \frac{2.72\,K \times 30 \times 4}{\log_{10}\left[\frac{245}{0.1}\right]}$$

$$\boxed{k = 4.15 \times 10^{-4}\ m/s}$$ Ans

Transmissibility :- Transmissibility Represents flow of water to the well at a given unit Radius or unit length.

$$\boxed{T = \text{Co-efficient of Permeability} \times \text{Thickness of Aquifer}}$$

for confined Aquifer $\boxed{T = KB}$

for unconfined Aquifer $\boxed{T = KH}$

$\boxed{\text{Units} = m^2/sec}$

★ Sichardt's Eqn

$$\boxed{R = 3000\ s\sqrt{k}}$$

R = Radius of influence well (m)
s = Drawdown at centre of well (m).
k = Co-efficient of Permeability (m/sec)

Soil water The moisture present in soil voids is known as Soil Moisture.

Types of soil water

1. **free water | Gravity water** This water continiously remains in movement from soil surface to water table.

- This water is subjected to gravitational forces only.
- All types of laws of Hydraulics are applicable on this water.
- This water when Joins the water table is known as sub-Surface water..

2. **Capillary Water :-**

- Soil voids interconnect to form capillary tubes like formation

The Rise of Moisture in these tubes is known as capillary water

- This water is subjected to Surface Tension.
- Rise of capillary water is inversly proptional to void size.
- **Capillary Rise order**

Clay > Silt > Sand > Gravel

- This water is the only water which is responsible for Plant growth.
- **Soil suction :-** It is the decrease in water pressure below water Table due to capillary Rise.

3. **Adsorb Water :-** This water is present on the surface of solid particles due to strong surface forces.

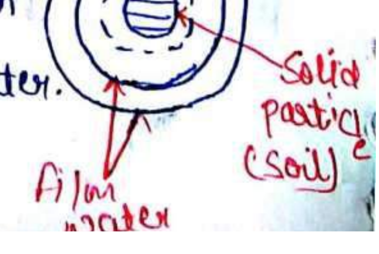
- The soil particles having high specific surface contains high amount of adsorbed water.
- **Qty of adsorb water | Hygroscopicity**

Clay >	Silt >	Sand >	Gravels
16%	6%	1-2%	0%

- This water is divided into two parts :-

(i) **Hygroscopic water :-** The first layer of water which is formed on surface of solid particles.

ii) **film Water :-** It is the indirect water which is present in no. of layer over Hygroscopic water.



Structural Water:- This water is present in the form chemically Combined crystalls. -

- It is the only water which cannot be remove after oven drying.
- This water is Remove by Breaking down the crystall structure of water at temp. of 350 - 420°c

Effective Stress and Seepage

Effective Stress Concept

1. **Total Stress** :- It is the amount of stress exerted by solid particles and water present in voids at a given level.

$\sigma = \frac{W}{A} = \frac{\gamma A h}{A}$

γ = unit wt. of Material / Soil

h = ht. of Material

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

2. **Pore Water Pressure** :- It is the pressure exerted by water Molecules present below water table at given level.

$\boxed{u = \gamma_w h_w}$ or $\left[u = \frac{\gamma_w A h_w}{A} \right]$

γ_w = unit wt. of water

h_w = ht. of water.

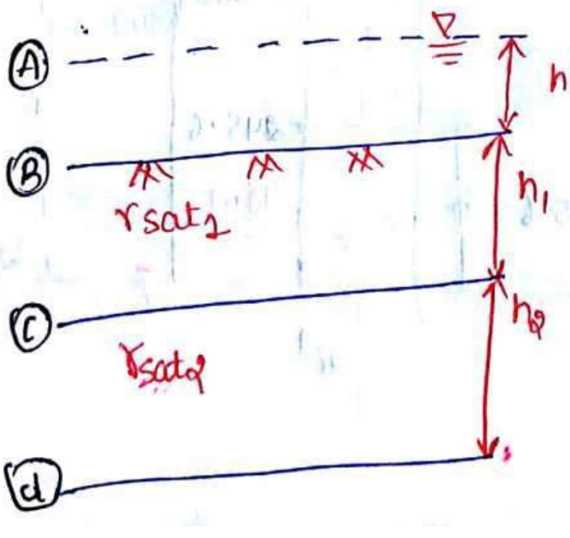
- This pressure doesnot produce any effect on ~~water~~ void Ratio an it is also known as neutral stress.

3. **Effective Stress** :-

It is the total stress exerted by solid Particles only at a given level.

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

- This stress decreases void Ratio of soil and increases eng Properties of soil



★ Level A

$\sigma = 0$

$u = 0$

$\bar{\sigma} = 0$

★ Level B

$\sigma = \gamma_w h'$

$u = \gamma_w h'$

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

$\bar{\sigma} = 0$

level C

$\sigma = \gamma_w h' + \gamma_{sat_1} h_1$

$u = \cancel{\omega} \cdot \gamma_w (h' + h_1)$

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

$= \cancel{\gamma_w h'} + \gamma_{sat_1} h_1 - \cancel{\gamma_w h'} - \gamma_w h_1$

$$\boxed{\bar{\sigma} = \gamma_1' h_1}$$

Level D

$\sigma = \gamma_w h' + \gamma_{sat_1} h_1 + \gamma_{sat_2} h_2$

$u = \gamma_w (h' + h_1 + h_2)$

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

$= \cancel{\gamma_w h'} + \gamma_{sat_1} h_1 + \gamma_{sat_2} h_2 - \cancel{\gamma_w h'} - \gamma_w h_1 - \gamma_w h_2$

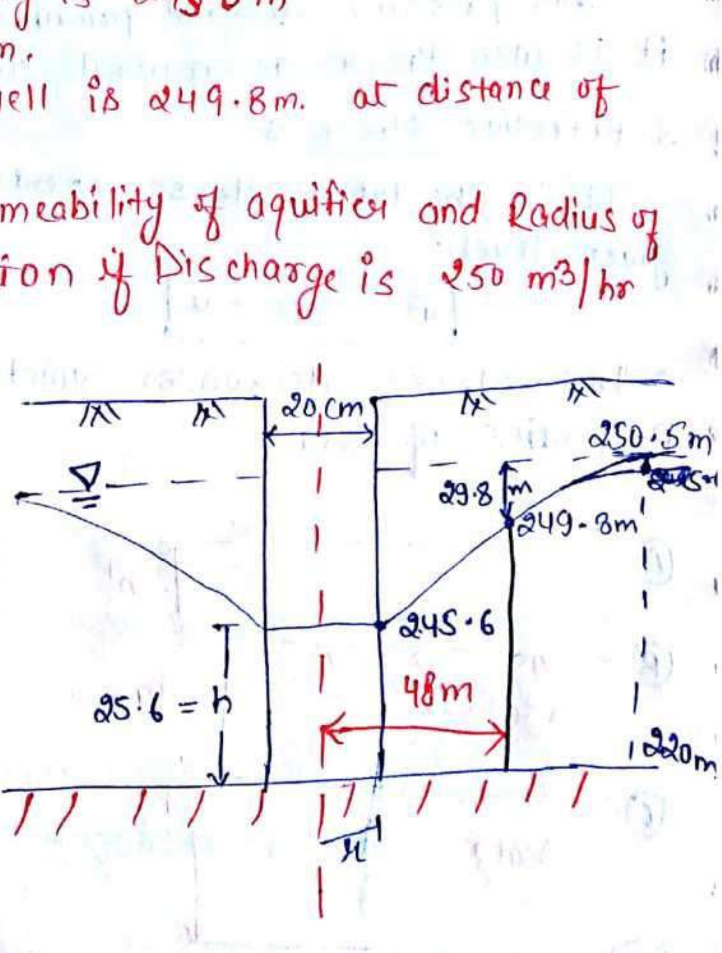
$$\boxed{\bar{\sigma} = \gamma' h_1 + \gamma_2' h_2}$$

20 sep 2018

Q A pumping out test was performed to determine the permeability of unconfined aquifer with following given observation

- R.L of origional water table before Pumping = 250.5 m
- R.L of water table after Pumping is 245.6 m
- R.L of impervious rock is 220 m.
- R.L of water in observation well is 249.8 m. at distance of 48 m from centre of well.

★ Calculate the co-efficient of Permeability of aquifer and Radius of influence using Sichardt's Equation if Discharge is 250 m3/hr Diameter of well is 20 cm.



Sol $r = \frac{20}{2} = 10\text{ cm} = 0.1\text{ m}$

$r_2 = 48\text{ m}$

Note :- H is Replaced by h_2, because R is not given

$h = 245.6 - 220 = 25.6\text{ m}$

$h_2 = 249.8 - 220 = 29.8\text{ m}$

$$q = \frac{1.36K\,(H^2-h^2)}{\log_{10}\left(\frac{R}{r}\right)}$$

This formula is Replaced by

$$q = \frac{1.36K\,(h_2^2-h^2)}{\log_{10}\left(\frac{r_2}{r}\right)} \Rightarrow \boxed{\begin{matrix} k = ? \\ R = ? \end{matrix}}$$

$$q = \frac{250}{60\times60} = 0.069\ m^3/sec$$

$$0.069 = \frac{1.36K\,[29.8^2-(25.6)^2]}{\log_{10}\left(\frac{48}{0.1}\right)}$$

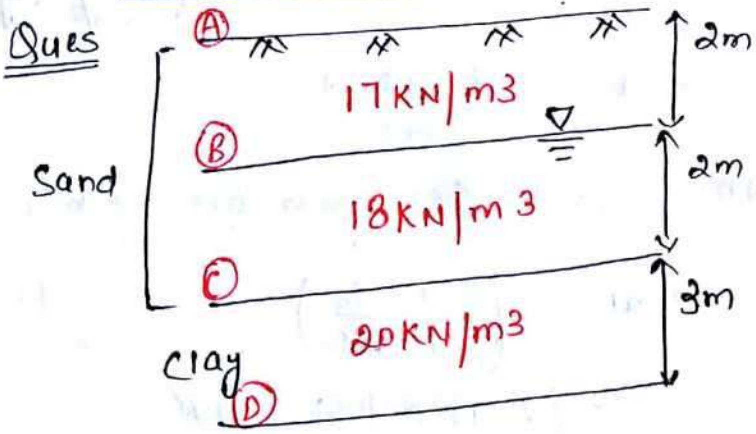
$$\boxed{k = 5.88\times10^{-4}\ m/sec}$$

$R = 3000\,s\sqrt{k}$ $\qquad$ $s = 250.5-245.6 = 4.9m$

$= 3000\times4.9\sqrt{5.88\times10^{-4}}$

$\boxed{R = 356.45\ m}$ Ans

Ques



γ_d sand = 17KN/m3

γ_{sat} sand = 18 KN/m3

γ_{sat} clay = 20 KN/m3

At level A $\quad \sigma = u = \bar{\sigma} = 0$

level B $\quad \sigma = 17\times2 = 34$

$u = 0$

$\bar{\sigma} = 34-0 = 34\ KN/m^2$

level C $\quad \sigma = 17\times2+18\times2 = 70\ KN/m^2$

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

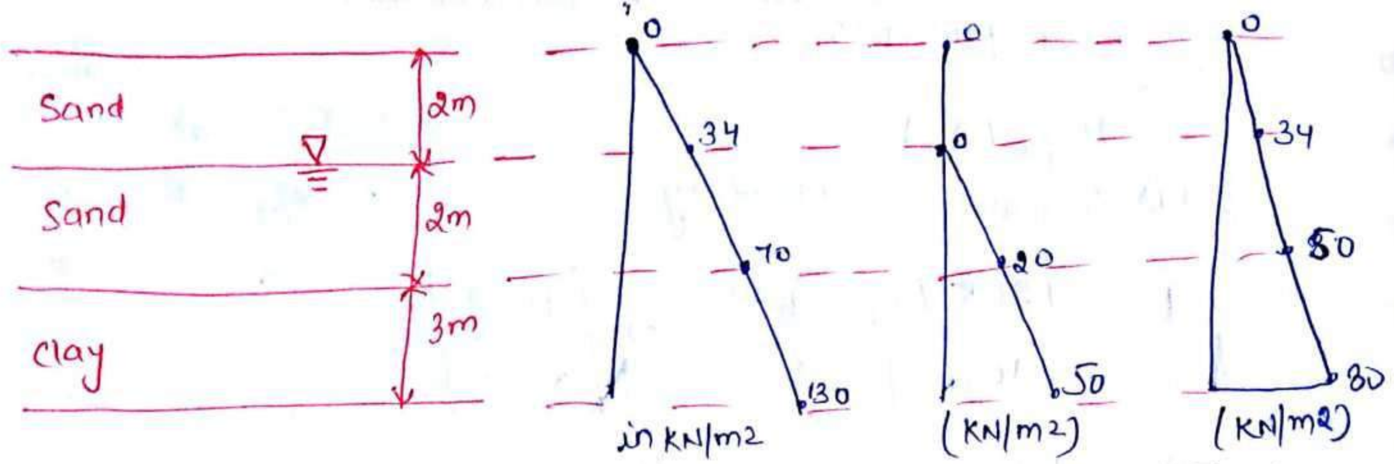
$\bar{\sigma} = 70-20 = 50\ KN/m^2$

level D $\quad \sigma = 17\times2+18\times2+20\times3$

$= 130\ KN/m^2$

$u = 10\,[2+3] = 50\ KN/m$

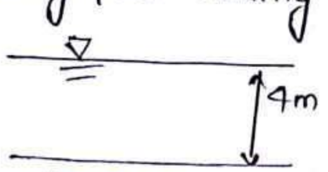
$\bar{\sigma} = 130-50 = 80\ KN/m$



σ-Diagram u-Diagram $\bar{\sigma}$-Diagram

<u>Ques</u> Calculate the value of Effective Stress at the bottom of Swimming pool having water Depth of 4m.

<u>Sol:</u>



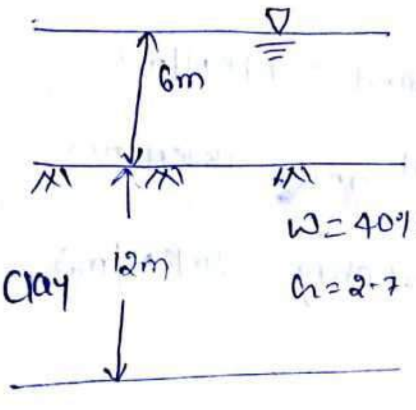
$\sigma = 10 \times 4 = 40\ KN/m^2$

$u = 40\ KN/m^2$

$\bar{\sigma} = 40 - 40 = 0$

No solid present so eff. stress is / always be zero.

<u>Ques</u> A soft clay 12m thick lies below 6m depth of water. The water content of clay is 40% and specific gravity of clay particles is 2.7. Calculate total stress and eff. stress at the bottom of clay layer.



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

When $S = 1$ $Se = wG$ (or) $e = wG$

$$\gamma_{sat} = \left(\frac{G+wG}{1+wG}\right)$$

$$= \frac{(2.7+0.4 \times 2.7)10}{1+0.4 \times 2.7}$$

$$\boxed{\gamma_{sat} = 18.17\ KN/m^2}$$

$\sigma = 10 \times 6 + 18.17 \times 12$

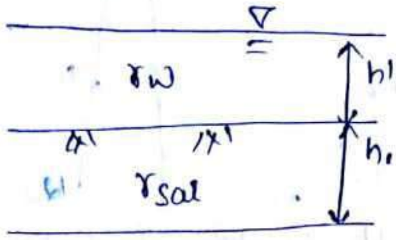
$\sigma = 278.04\ KN/m^2$

$u = 10(12+6) = 180\ KN/m^2$

$\bar{\sigma} = 278.04 - 180$

$\boxed{\bar{\sigma} = 98.04\ KN/m^2}$ Ans.

Note :- (i) With increase in water Table above ground surface total Stress and pore water pressure increases but eff. stress Remains Constant.



$$\sigma = \gamma_w h' + \gamma_{sat} h_1$$

$$u = \gamma_w (h' + h_1)$$

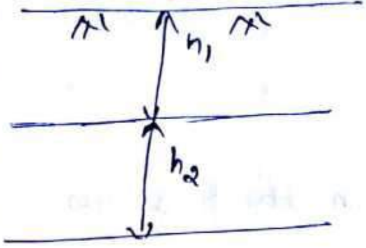
$$\bar{\sigma} = \gamma_w h' + \gamma_{sat} h_1 - \gamma_w (h' + h_1)$$

$$= \cancel{\gamma_w h'} + \gamma_{sat} h_1 - \cancel{\gamma_w h'} - \gamma_w h_1$$

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

$$\boxed{\bar{\sigma} = \gamma' h_1}$$

(ii) With Rise of Water table upto ground surface total stress remains almost constant (very small increase), Pore water pressure increased and effective stress decreases.



$\sigma = \gamma h_1 + \gamma_{sat} h_2$ almost constant (iv)

$u = \gamma_w h_2 \uparrow$

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

(iii) Effective stress in soil depends upon unit wt. of soil above Water Table and submerged unit wt. of soil below water table

(iv) With increase in depth of water water table total stress Remains almost constant. Pore water pressure decreases and eff.-stress increases.

Ques A Deposit contains sand layer of 5m thickness which lies above clay layer of 3m thickness. The water table lies at inter-section of sand and clay layer. Calculate total stress, neutral stress and eff. stress along with their variation diag

for Sand

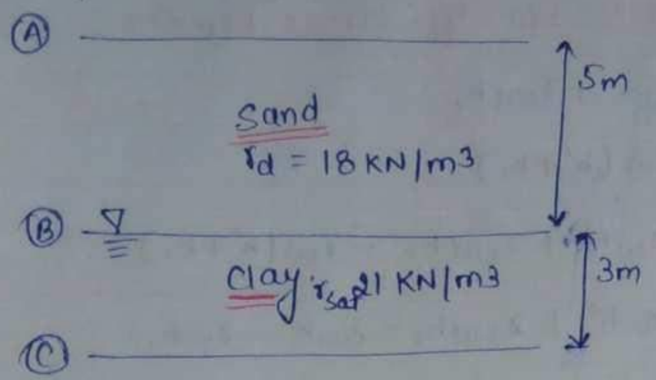
$\gamma_d = 18\ kN/m^3$

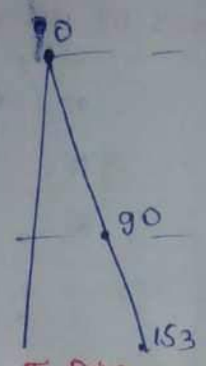
$\gamma_{sat} = 20\ kN/m^3$

for clay

$\gamma_d = 19\ kN/m^3$

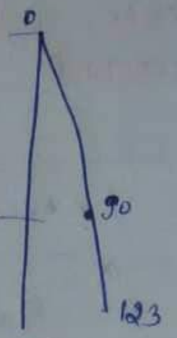
$\gamma_{sat} = 21\ kN/m^3$





σ - Diagram

u - Diagram



$\bar{\sigma}$ - Diagram

At A - A σ = 0 , u = 0, $\boxed{\bar{\sigma} = 0}$

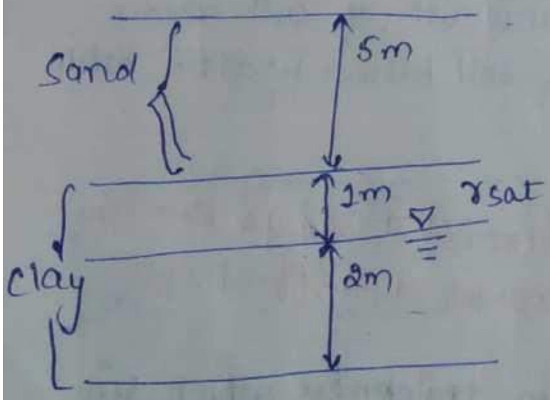
At B-B σ = 18×5 , = 90 , u = 0 $\boxed{\bar{\sigma} = 90 KN/m^2}$

At C-C σ = 18×5 + 21×3 = 153 KN/m2

u = 10×3 = 30

$\boxed{\bar{\sigma} = 153 - 30 = 123 KN/m^2}$

Ques what will be the effect on effective stress in the previous ques if water Table Decreases by 1m.



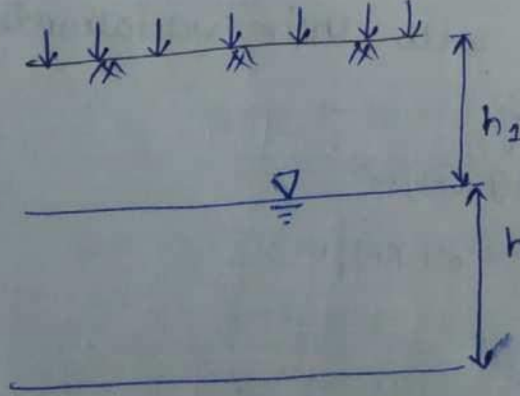
σ = 18×5 + 21×1 + 21×2 = 153 KN/m2

u = 10×2 = 20 KN/m2

$\bar{\sigma}$ = 153 - 20 = 133 KN/m3

$\boxed{\text{Change} = 133 - 123 = 10 KN/m^2 \text{ increase}}$

4 Effective of Surcharge load



(i) Due to surcharge load total stress always increases.

(ii) If surcharge is applied suddenly, the surcharge is carried by water and pore water pressure increases, eff. stress remain constant, total stress decreases.

(iii) When surcharge is applied Gradually, water pressure does not contain surcharge load and remain constant, eff. stress increases.

$\sigma = [\gamma h_1 + \gamma_{sat} h_2] + q_0$ q_0 = Surcharge load.

Case I : When surcharge is applied suddenly.

$$\boxed{u = \gamma_w h_2 + q_0}$$

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

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

Case II Surcharge is applied gradually

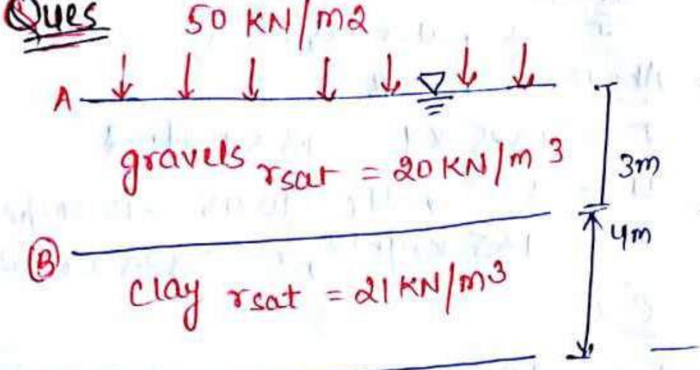
$u = \gamma_w h_2$ $\sigma = (\gamma h_1 + \gamma_{sat} h_2) + q_0$

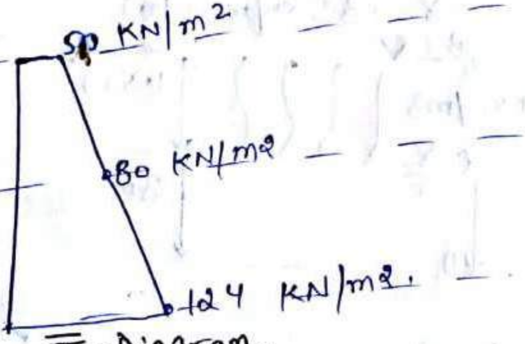
$$\bar{\sigma} = \sigma - u$$

$$\bar{\sigma} = \gamma h_1 + \gamma_{sat} h_2 + q_0 - \gamma_w h_2$$

$$\boxed{\bar{\sigma} = \gamma h_1 + \gamma' h_2 + q_0}$$

Ques





$\bar{\sigma}$-Diagram.

Ⓒ Draw eff- stress variation diagram if surcharge is applied gradu

At A-A $\sigma = 50$, $u = 0$, $\bar{\sigma} = \boxed{50 \text{ KN/m}^2}$

At B-B $\sigma = 50 + 20 \times 3 = 110$ KN/m2

$u = 10 \times 3 = 30$ KN/mm2

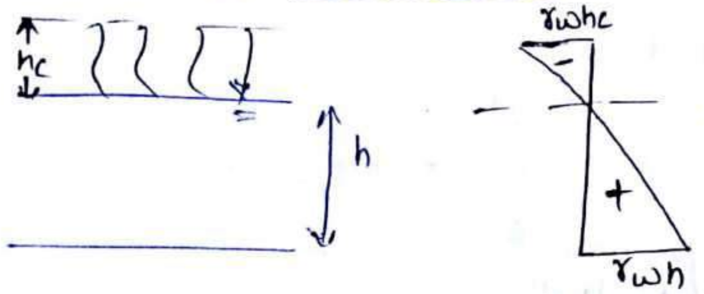
$$\boxed{\bar{\sigma} = 80 \text{ KN/m}^2}$$

At C-C $\sigma = 50 + 20 \times 3 + 21 \times 4 = 194$ KN/m2

$u = 10 \times [3+4] = 70$ KN/m2

$$\boxed{\bar{\sigma} = 194 - 70 = 124 \text{ KN/m}^2}$$

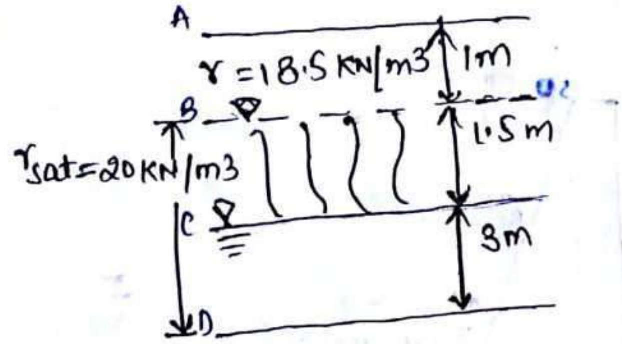
Effect of capillary :- Due to capillary Rise of water the pore water pressure exists in upward direction. This pressure is known as soil suction.



Below W.T $\boxed{u = \gamma_w \cdot h}$

Above W.T $\boxed{u_c = -\gamma_w \cdot h_c}$

Ques ~~with~~ · Draw σ, u, σ̄ variation Diagram



At A−A

$\sigma = 0$, $u = 0$, $\bar{\sigma} = 0$

Above W.T B−B

$\sigma = 18.5 \times 1 = 18.5 \text{ KN/m}^2$

$u = 0$, $u_c = 10 \times 1.5 = -15 \text{ KN/m}^2$

$\bar{\sigma} = 18.5 \text{ KN/m}^2$, $\bar{\sigma} = 33.5 \text{ KN/m}^2$

C−C

$\sigma = 20 \times 1.5 + 18.5 \times 1 = 48.5 \text{ KN/m}^2$

$u = 0$; $u_c =$ [illegible]

$\bar{\sigma} = 48.5 - 0 = 48.5 \text{ KN/m}^2$

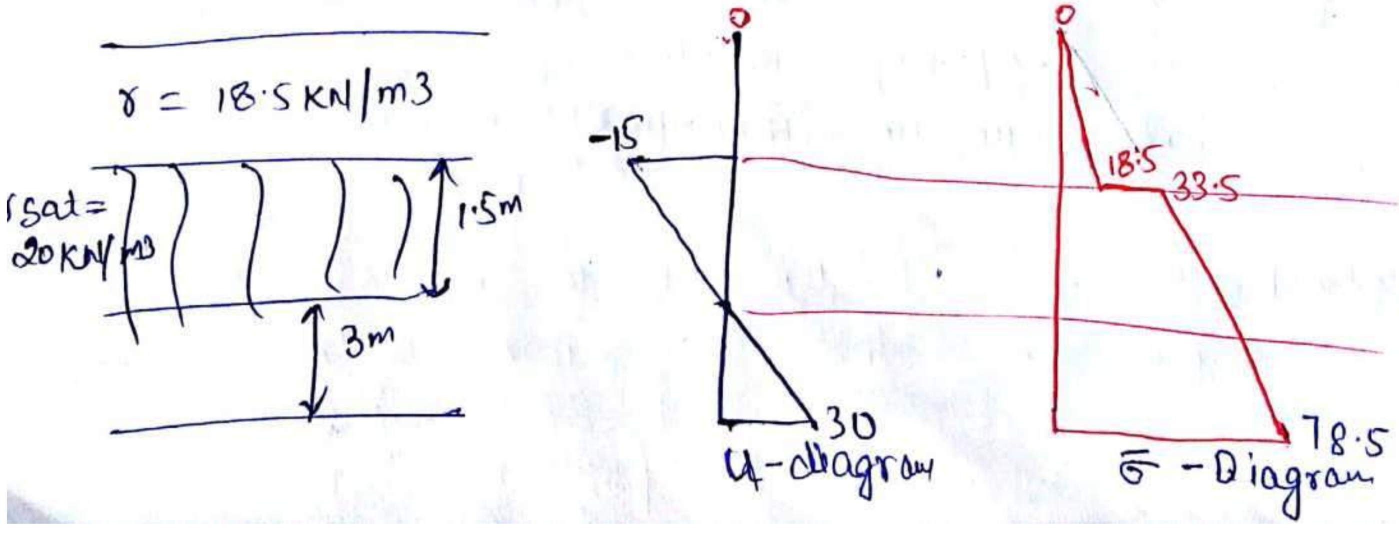
= [illegible]

D−D

$\sigma = 20 \times [4.5] + 18.5 \times 1$
$= 108.5 \text{ KN/m}^2$

$u = 10 \times 3 = 30 \text{ KN/m}^2$

$\bar{\sigma} = 108.5 - 30 = 78.5 \text{ KN/m}^2$



21/09/18

Seepage (concept)

Seepage Pressure :- The total transfor of energy that take place due to movement of water present in soil voids, is known as seepage pressure.

$$\frac{\gamma_w h_w Z}{Z}$$

$$i = \frac{h_w}{z} = \text{hydraulic Pressure}$$

$$\boxed{j = \gamma_w i z}$$

Z = thickness of soil

Seepage force :- The total force exerted by transfer of energy during movement of water is known as seepage force.

$$J = j \times A = \gamma_w i Z A \qquad \boxed{ZA = V}$$

$$\boxed{J = \gamma_w i V}$$

V = vol. of soil

Effect of seepage on effective stress

$$\boxed{\bar{\sigma}_0 = \bar{\sigma} \pm \gamma_w i z}$$

$\bar{\sigma}_0$ = overall eff. stress.

+ve sign ⇒ when flow is downward → $\bar{\sigma}_0 \uparrow$

-ve sign ⇒ when flow is upward → $\bar{\sigma}_0 \downarrow$

Quick condition (दलदल) / Quick Sand / Boiling Condition / Boiling Sand / Liquefaction of soil

(i) Due to upward Movement of water in upward direction seepage pressure also exerts in upward direction.

(ii) When the overall effective stress becomes zero $\boxed{\bar{\sigma}_0 = 0}$ due to upward seepage pressure, the condition is known as Quick condition. It is a condition of zero eff. stress in cohesionl soil

(iii) This condition acts in ~~for~~ sand and highly sensitive clay

$\boxed{\text{Sensitivity} > 8}$

(iv) At this condition the shear strength of soil becomes zer and soil behaves like liquid. This condition is known as liquefaction of soil

(v) At this condition soil particles try to move in upward direction and this condition is similar to boiling.

$$\overline{\sigma_0} = \bar{\sigma} \pm \gamma_w i z$$

When flow is in upward direction

$$\bar{\sigma}_0 = \bar{\sigma} - \gamma_w i z$$

At quick condition :- $\bar{\sigma}_0 = 0$; Let $\boxed{i = i_c}$

$$0 = \bar{\sigma} - \gamma_w i_c z$$

$$0 = \gamma' z - \gamma_w i_c z$$

$$\gamma' z = \gamma_w i_c z$$

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

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

$$\gamma' = \left[\frac{G+e}{1+e}\right]\gamma_w - \gamma_w$$

$$\gamma' = \gamma_w\left[\frac{G+e}{1+e} - 1\right]$$

$$\gamma' = \gamma_w\left[\frac{G+\cancel{e}-1-\cancel{e}}{1+e}\right]$$

$$\boxed{\gamma' = \gamma_w\left[\frac{G-1}{1+e}\right]}$$

$$i_c = \frac{\cancel{\gamma_w}\left[\frac{G-1}{1+e}\right]}{\cancel{\gamma_w}}$$

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

i_c = Critical Hydraulic Gradient

Ques calculate the critical Hydraulic Gradient of a cohesionless soil having void Ratio 0.67

$$i_c = \frac{G-1}{1+e} = \frac{2.67-1}{1+0.67} = 1$$

$$\boxed{i_c = 1 \longleftrightarrow e = 0.67}$$

Ques If Porosity of a soil is 50% and sp gravity b/w particles is 2.7. calculate critical hydraulic gradient

$$n = e = \frac{\cdot 50}{1-\cdot 50} = 1 \quad \boxed{e = 1}$$

$$i_c = \frac{G-1}{1+e} = \frac{2.7-1}{1+1} = \frac{1.7}{2}$$

$$\boxed{i_c = \cdot 85}$$

Ques calculate i_c if saturated unit wt. of a soil is 18 KN/m

$$i_c = \frac{\gamma'}{\gamma_w} = \frac{\gamma_{sat}-\gamma_w}{\gamma_w} = \frac{18-10}{10} = 8 \cdot 8 \text{ KN/m3}$$

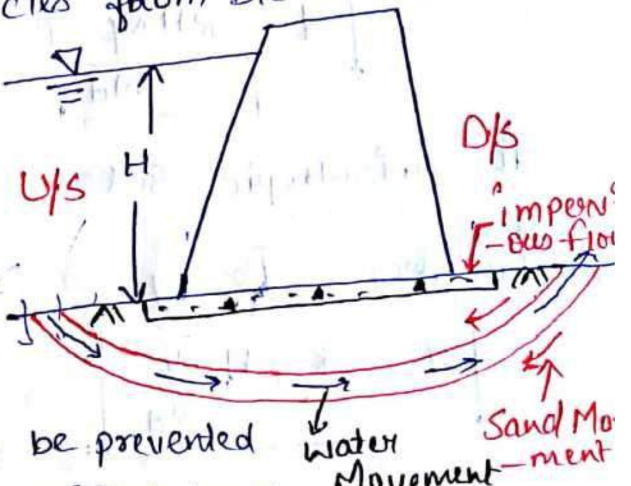
$$\boxed{i_c = \cdot 8}$$

Ques calculate the head of water Required at which boiling b/w sand particles exists. Thickness of sand deposit is 2m and void Ratio = 0.68.

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

$\frac{h_w}{2} = \frac{2.67-1}{1+0.68}$ $\boxed{h_w = 1.98m}$

Piping Failure in Hydraulic Structure

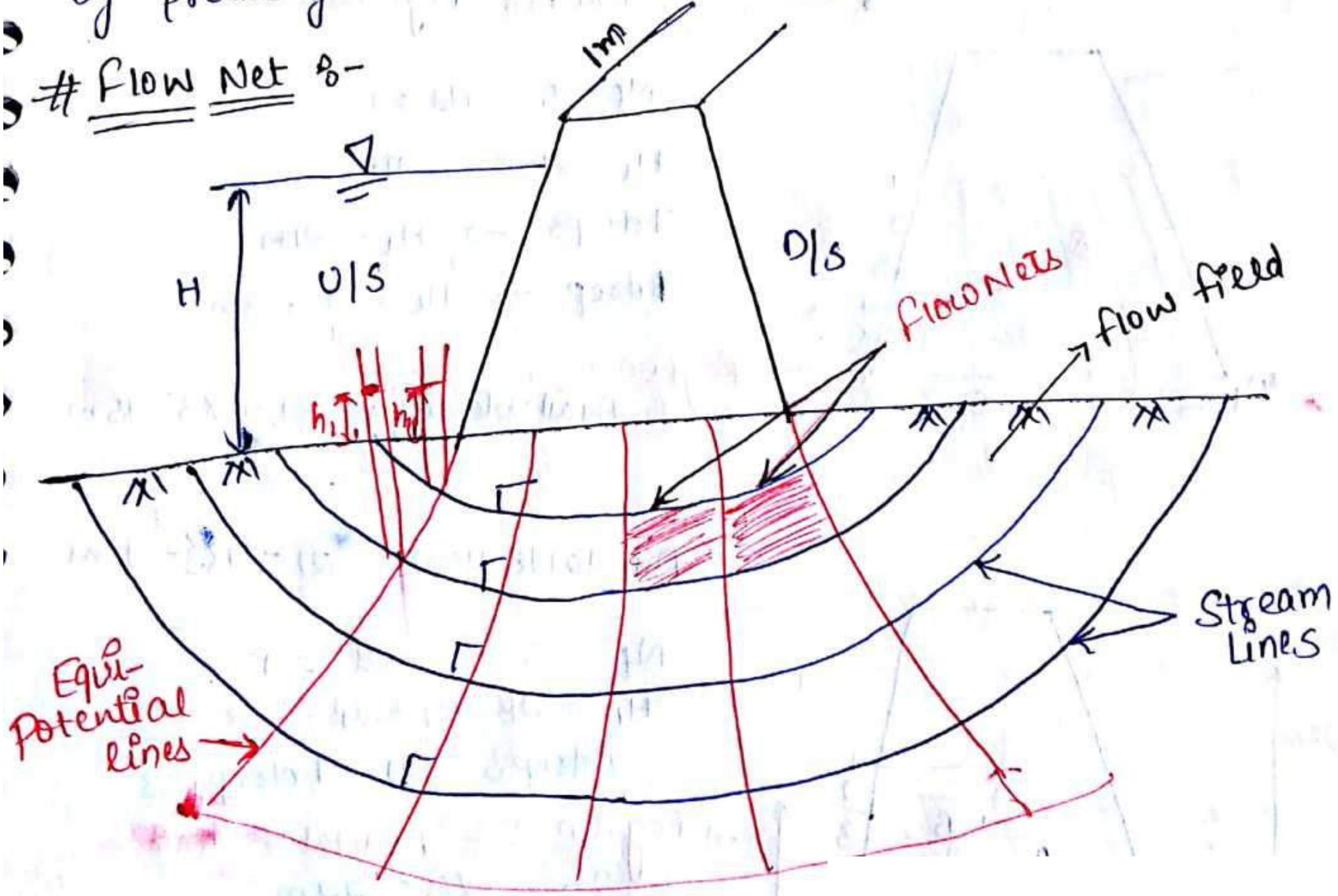


* The Gradual Removal of soil particles from D/s to U/s side due to Movement of water from U/s to D/s side below Hydraulic structure is known as Piping failure.

* The formation of piping take place due to formation of Hydraulic Gradient (Head loss / unit length.

* The piping failure in structures can be prevented by providing an Impervious floor of sufficient length.

Flow Net :-



(i) Stream lines are the lines which Represent path of Moving water

(ii) Equipotential lines are the lines which join voids of equal water head.

(iii) Both lines intersect each other at 90° angle.

(iv) Each field of flow Net is almost square and curvilinear.

(v) Each field of flow Net has variable area but constant Discharge.

(vi) The disharge from each field is given by

for isotropic Soil

$$\boxed{q = kH\frac{N_f}{N_d}}$$

N_f = No. of flow channel [Vertical Channel]

N_d = No. of drop channel [Horizontal Channel]

H = Diff. of water Head b/w U/S to D/S side

for anisotropic Soil

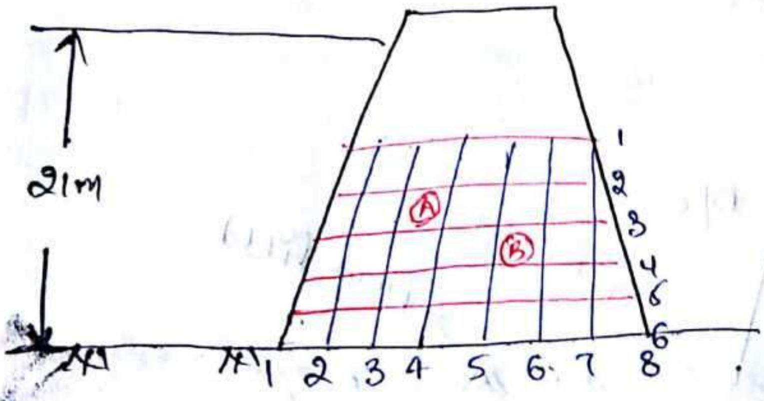
equivalent Permeability, $k_{eq} = \sqrt{k_x k_y}$

$$\boxed{q = k_{eq} H \frac{N_f}{N_d}}$$

k_x = co-efficient of Permeability in x-direction

k_y = co-efficient of Permeability in y-direction.

Ques Det. Available Head of water in Marked Regions = ?



$N_f = 5 \quad N_d = 7$

$H_L = 21 - 0 = 21m$

7 drops ⇒ $H_L = 21m$

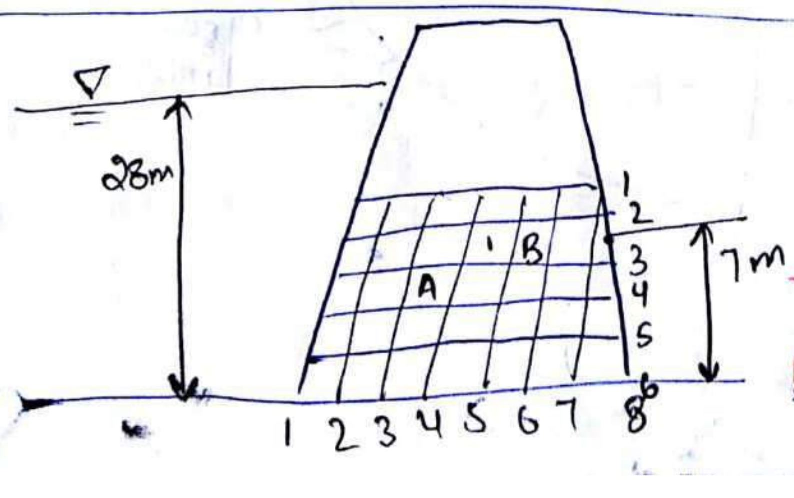
1 drop ⇒ $H_L = \frac{21}{7} = 3m$

Region A

Available Head = 21 − 2×3 = 15m

Region B

Available Head = 21 − 4×3 = 9cm



$N_f = 5, \quad N_d = 7$

$H_L = 28 - 7 = 21$

7 drops = 21, 1 drop = 3

Region A : Available Head

= 28 − 2×3 = 22m

Region B

= 28 − 4×3 = 16m

the co-efficient of Permeability of soil in x and y direction is 4 x 10^{-3} cm/sec and 1×10^{-3} cm/sec. The available head of water in U/s and D/s is 15m and 5m Respectively. Calculate the Discharge per unit length if flow channels & drop channels are 5 & 30.

$N_f = 5$ $\quad K_x = 4 \times 10^{-3}$ cm/sec

$N_d = 30$ $\quad K_y = 1 \times 10^{-3}$ cm/sec

U/S available head = 15m

D/s available head = 5m

$$q = \sqrt{K_x K_y}\; H \frac{N_f}{N_d}$$

$$H = 15 - 5 = 10m = 1000\,cm$$

$$q = \sqrt{(4 \times 10^{-3})(1 \times 10^{-3})} \times 1000 \times \frac{5}{30}$$

$$q = 2 \times 10^{-3} \times 1000 \times \frac{1}{6}$$

$$q = \frac{2}{6} = 1/3 \ cm^3/sec$$ / metre length of struc

Unit - III

Compressibility & strength

Compression :- The process of decrease in volume of soil due to external applied load.

The property of decrease in volume of soil is known as **Compressibility.** On the basis of nature of voids compress is divided into

(1) **Consolidation** :- The process of removing of water voids from the soil due to external applied load is known as consolida-ti

- This process is a natural process. which take place under natural loads of structure.
- It is a time taken process.
- Consolidation is related to settlement of soil.

(2) **Compaction** :- The process of Removal of air voids from soil due to external mechanical loads. is known as Compaction.

Stages of Consolidation

(a) **Initial Consolidation** :- Due to sudden application of load the decrease in the volume of soil is known as initial conso-lidation

- This decrease in volume may be into impact of the load or removable of air from voids etc.

(b) **Primary Consolidation** :- With increase in load the removabl of water from voids takes place

- This decrease in volume due to external movement of water is known as Primary consolidation.

(c) **Secondary Consolidation** :- After removable of water from voids rearrangement of solid particles take place.

- The decrease in vol. due to re-arrangement is known a Secondary Consolidation.
- Creep is an example of secondary consolidation

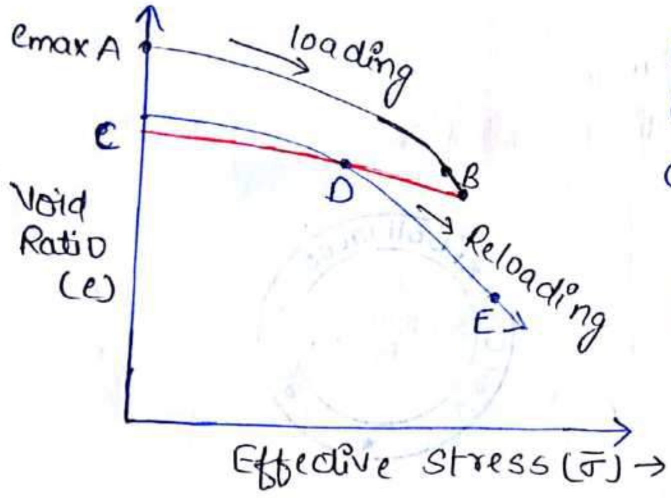
Hydro-dynamic log

The time interval b/w application of load and removab of water voids is known as hydro-dynamic log.

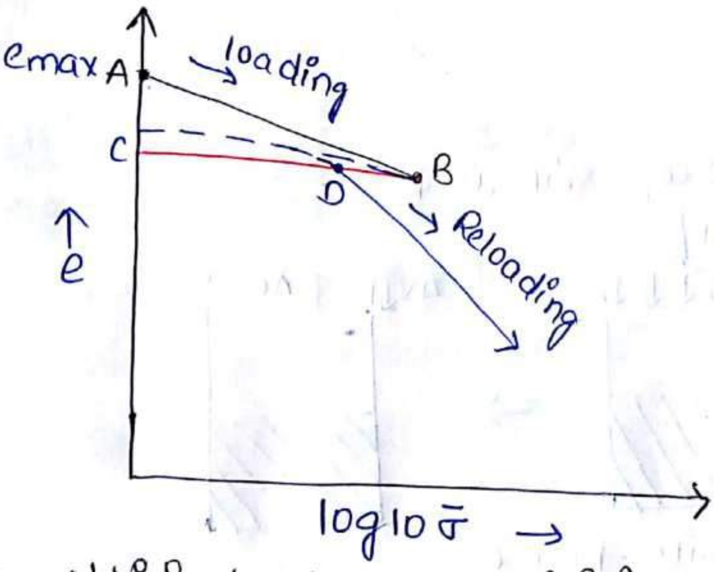
Consolidation

Hydro dynamic log represents time Required for initial Consolida

Consolidation Curve :-



AB = Compressive Curve
BC = Expansion Curve
CDE = Re-Compression Curve



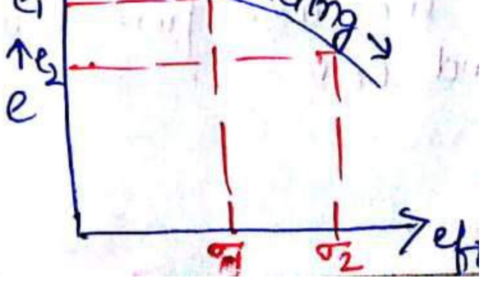
Co-efficient of Compressibility :- It is the slope of co -ssion curve draw b/w void Ratio of soil to change effective stress.

$$\boxed{a_v = -\left[\frac{e_2-e_1}{\bar{\sigma}_2-\bar{\sigma}_1}\right]}$$

-ve sign ⇒ void Ratio of soil decreasing ↓

Most Jmp

Unit ⇒ m^2/kN



Compression Index :- It is the slope of compression curve drawn b/w void Ratio & logarithmic of effective stress.

$$C_c = \frac{-(e_2-e_1)}{\log_{10}\bar{\sigma}_2 - \log_{10}\bar{\sigma}_1}$$

-ve sign ⇒ void Ratio of soil is ↓

$$\boxed{C_c = \frac{-(e_2-e_1)}{\log_{10}\left[\frac{\bar{\sigma}_2}{\bar{\sigma}_1}\right]}}$$

*** Units → Unitless

Co-efficient of volume change

$$m_v = \frac{\text{volumetric strain}}{\text{change in eff. stress}} = \frac{-\frac{\Delta V}{V}}{\Delta\bar{\sigma}} = \frac{-\frac{A\,\Delta H}{H}}{\Delta\bar{\sigma}} = \frac{-\frac{\Delta H}{H}}{\Delta\bar{\sigma}}$$

$$\boxed{m_v = \frac{-\frac{\Delta H}{H}}{\Delta\bar{\sigma}}}$$

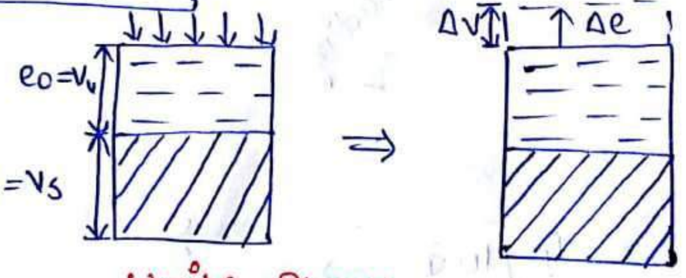
-ve sign → volume of soil is ↓

Units ⇒ m2/kN

$e_0 = \frac{V_v}{V_s}$

Let $V_s = 1$

$V_v = e_0$



Units-Phase Diagram

$$m_v = \frac{-\frac{\Delta H}{H}}{\Delta\bar{\sigma}} = \frac{-\frac{\Delta e}{1+e_0}}{\Delta\bar{\sigma}} = \frac{a_v}{1+e_0}$$

$$\boxed{m_v = \frac{a_v}{1+e_0}} \qquad \boxed{a_v = \frac{-\Delta e}{\Delta\bar{\sigma}}}$$

ues The void ratio of soil changes from 0.55 to 0.35 wh
= is increase from 100 kN/m2 to 200 kN/m2. Calculate
av and Cc !

$e_1 = 0.55 \qquad \bar{\sigma}_1 = 100 \ \text{kN/m}^2$

$e_2 = 0.35 \qquad \bar{\sigma}_2 = 200 \ \text{kN/m}^2$

$$a_v = \frac{(e_2 - e_1)}{(\bar{\sigma}_2 - \bar{\sigma}_1)} = \frac{0.35 - 0.55}{200 - 100}$$

$$a_v = \frac{0.2}{100} = 0.002 \ \text{m}^2/\text{kN}$$

$$C_c = \frac{-(e_2 - e_1)}{\log_{10}\left[\frac{\bar{\sigma}_2}{\bar{\sigma}_1}\right]} = \frac{-(0.35 - 0.55)}{\log_{10}\left[\frac{200}{100}\right]}$$

$$C_c = \frac{0.2}{\log_{10} 2} = \frac{0.2}{0.3} = \frac{2}{3}$$

$$\boxed{C_c = 0.66}$$

Ques The void Ratio of soil decrease from 1·068 t 0·994 then eff. stress is increase from 60 kN/m² t 120 kN/m². Calculate C_c, a_v and m_v.
Calculate settlement in soil $(\Delta H) = ?$ if initial thickn of soil is 8m.

$$\Rightarrow a_v = \frac{e_2 - e_1}{\bar{\sigma}_2 - \bar{\sigma}_1} = \frac{0.994 - 1.068}{120 - 60} = \frac{-0.014}{60}$$

$$\boxed{a_v = 1.23 \times 10^{-3} \ \text{m}^2/\text{kN}}$$

$$\Rightarrow C_c = \frac{-(e_2 - e_1)}{\log_{10}\left[\frac{\bar{\sigma}_2}{\bar{\sigma}_1}\right]} = \frac{-(0.994 - 1.068)}{\log_{10}\left(\frac{120}{60}\right)} = 0.24$$

$$\boxed{C_c = 0.246}$$

Note :-

Skempton's formula :- To calculate compression Index :-

$$C_c = 0.009 \left[W_L - 10\%\right] \quad \left[\text{Undisturbed or Natural Deposit}\right]$$

[exact values] $C_c = 0.007 \left[W_L - 7\%\right)$ [Remoulded]

[approx values] $C_c = 0.007 \left(W_L - 10\%\right]$ [Remoulded Soil Deposit]

Settlement in soil

NCC $\bar{\sigma}_1 \to \bar{\sigma}_2$ $[\bar{\sigma}_2 > \bar{\sigma}_1]$

$$\frac{\Delta H}{H} = \frac{\Delta e}{1+e_0}$$

$$\Delta H = \frac{\Delta e H}{1+e_0} \quad \text{--- (i)}$$

$$C_c = \frac{-(e_2 - e_1)}{\log_{10}\left(\frac{\bar{\sigma}_2}{\bar{\sigma}_1}\right)} = \frac{\Delta e}{\log_{10}\left[\frac{\bar{\sigma}_2}{\bar{\sigma}_1}\right]}$$

$$\boxed{\Delta e = C_c \log\left[\frac{\bar{\sigma}_2}{\bar{\sigma}_1}\right]} \quad \text{--- (ii)}$$

Put (ii) in (i)

$$\boxed{\Delta H = \frac{C_c H}{1+e_0} \log_{10}\left[\frac{\bar{\sigma}_2}{\bar{\sigma}_1}\right]}$$

Ques A NCC settles down by 10cm, when eff. stress is increase from 100 KN/m² to 200 KN/m². calculate the settleme in same soil if effective stress is increased from 200 KN/ to 400 KN/m².

ns

$$\Delta H = \frac{C_c H}{1+e_0} \log_{10}\left[\frac{\bar{\sigma}_2}{\bar{\sigma}_1}\right]$$

$$10 = \frac{C_c H}{1+e_0} \log_{10}\left[\frac{200}{100}\right]$$

$$\frac{C_c H}{1+e_0} = \frac{10}{\log_{10} 2}$$

2nd Case

$$\Delta H = \frac{C_c H}{1+e_0} \log_{10}\left[\frac{400}{200}\right]$$

$$\Delta H = \frac{10}{\cancel{\log_{10} 2}} \times \cancel{\log_{10} 2}$$

$$\boxed{\Delta H = 10\,\text{CM}}$$ - Ans

Settlement in OCC

:- If σ_f (initial stress) is the initial over consolidation Pressure at which soil is over consolidated.

- $\bar{\sigma}_i$ is the existing pressure
- σ_0 is the Pressure on the soil at which settlement is to be calculated.
- when the stress is increased from σ_f to σ_i, behaviour of soil will be OCC. and when stress is increased from σ_i to σ_0, the behaviour of soil will be NCC.

$$\underbrace{\sigma_f \text{ to } \sigma_i}_{\text{OCC}} \quad \boxed{\Delta H = \frac{C_r H}{1+e_0} \log_{10}\left[\frac{\bar{\sigma}_f}{\bar{\sigma}_i}\right] + \frac{C_c H}{1+e_0} \log_{10}\left[\frac{\bar{\sigma}_0}{\bar{\sigma}_f}\right]} \quad \underbrace{\bar{\sigma}_i \text{ to } \bar{\sigma}_0}_{\text{NCC}}$$

$\bar{\sigma}_f$ = initial over consolidation Pressure

$\bar{\sigma}_i$ = Existing Pressure, C_c = compression Index

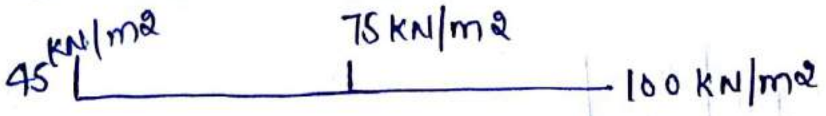
Doubt $\bar{\sigma}_0$ = final Pressure on the Soil, C_r = Recompression Index.

e_0 = Initial Void Ratio   H = initial thickness

Ques An overconsolidated clay has overburden Pressure of 75 KN/m² And Existing Pressure of 45 KN/m². If the stress is increa by 55 KN/m². Calculate settlement in soil. Consider the given da

(i) Initial void Ratio = 1.2, (ii) Initial thickness = 2m

(iii) Compression Index = 0.24 (iv) Re compression Index = 0.08.
C_c C_r



$$\Delta H = \frac{0.08 \times 2}{1+1.2} \log_{10}\left[\frac{75}{45}\right] + \frac{0.24 \times 2}{1+1.2} \cdot \log_{10}\left[\frac{100}{75}\right]$$

$$\Delta H = 0.0433$$

Terzaghi's Theory of Consolidation

Assumptions

(i) Soil is Homogeneous and Isotropic.

(ii) Soil is fully saturated.

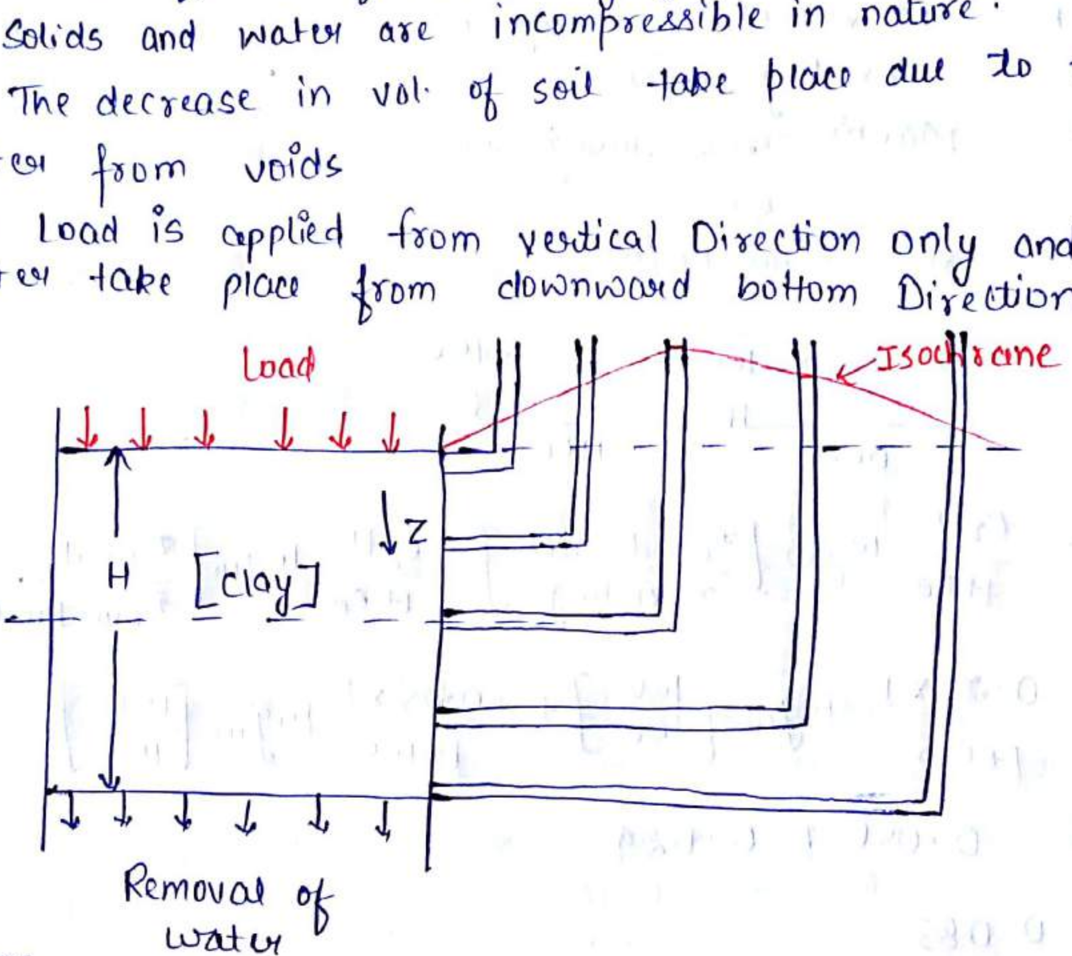
(iii) Darcy's law is valid [$v \propto i$]

(iv) The co-efficient of Permeability of soil remains constant

(v) Solids and water are incompressible in nature.

(vi) The decrease in vol. of soil take place due to Removal of water from voids

(vii) Load is applied from vertical Direction only and Removal of water take place from downward bottom Direction only.



(i) This theory deals with primary consolidation stage.

(ii) The load is applied on soil in the form of small increme
Ratio (Not Sudden change / Dynamic loading] [Only gradual change]

(iii) Acc. to this theory, the Rate of Removal of Pore water is directly proportional to square of change of water Pressure with Distance 'z'.

$$\frac{\partial u}{\partial t} \propto \frac{\partial^2 u}{\partial z^2} \Rightarrow \boxed{\frac{\partial u}{\partial t} = \frac{k}{m_v \gamma_w} \cdot \frac{\partial^2 u}{\partial z^2}}$$

$\frac{k}{m_v \gamma_w}$ = C_v = Co-efficient of Consolidation

m_v = co-efficient of vol. change

$\boxed{\text{Units} \Rightarrow m^2/s}$

26 Sep 2018

what is av

$$m_v = \frac{a_v}{1+e_0}$$

$$C_v = \frac{k}{\left[\frac{a_v}{1+e_0}\right]\gamma_w} \Rightarrow \boxed{C_v = \frac{k(1+e_0)}{a_v \gamma_w}}$$

Degree of Consolidation : It is Ratio of settlement of soi at any time to its ultimate settlement

$$\boxed{\% U = \frac{\Delta H_t}{\Delta H_f} \times 100} \Rightarrow \frac{U_i^o - U_t}{U_i^o - 0} \times 100 \Rightarrow \boxed{\frac{U_i^o - U_t}{U_i^o} \times 100}$$

ΔH_t = Settlement at any time t

ΔH_f = Ultimate Settlement

U_i^o = Initial Pore Pressure

U_t = Pore Pressure at any time t.

Time Required for consolidation :-

$$\left.\begin{array}{l} t \propto \frac{1}{C_v} \\ t \propto d^2 \end{array}\right] \Rightarrow t \propto \frac{d^2}{C_v}$$

$$\boxed{t = \frac{T_v d^2}{C_v}}$$

T_v = Time factor [depends upon degree of consolidation]

d = drainage Path

$$\boxed{T_v = \frac{\pi}{4}\left[\frac{U\%}{100}\right]^2} \quad [U \leq 60\%]$$

$$\boxed{T_v = 1.783 - 0.991 \log_{10}[100 - U\%]} \quad (U > 60\%)$$

Drainage Path

1. Single Drainage $\Rightarrow \boxed{d = H}$
2. Double Drainage $\Rightarrow \boxed{d = H/2}$
3. Triple Drainage $\Rightarrow \boxed{d = H/3}$

Q. A soil is 6m thick with Double drainage having co-efficient of consolidation 5×10^{-4} cm2/sec. Calculate the time Required for 50% consolidation in days.

$$t = T_v \frac{d^2}{C_v}$$

Double drainage , $d = H/2 = \frac{6}{2} = 3m$

$$\boxed{d = 300cm}$$

$$T_v = \frac{\pi}{4}\left[\frac{50}{100}\right]^2 = 0.197$$

$$t = \frac{0.197 \times (300)^2}{5 \times 10^{-4}} \Rightarrow t = 35460000 \text{ sec}$$

$$t = \frac{35460000}{24 \times 60 \times 60} = 410.42 \text{ days.}$$

$$\boxed{t = 410.42 \text{ days}}$$

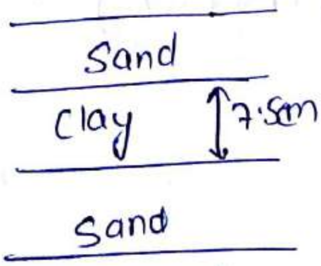
Q The ultimate settlement of soil under Single Drainage is 50cm. If Drainage is doubled what will be ultimate settlement.

Ans Under Single Drainage = 50 cm

" Double " , ΔH_f = 50 cm Ans

ultimate Settlement doesnot change with Drainage Path. Drainage Path can Change Time Required for consolidation only.

Q A 7.5 cm thick clay layer lies b/w two sand layer such that if a 2.5 cm thick same clay was tested under Double Drainage , the time Required for 50% consolidation was 12.5 min. Calculate the time Required for 50% consolidation of origional clay layer.



$U = 50\%$

$t = ?$

$$T_v = \frac{\pi}{4} \times \left[\frac{500}{100}\right]^2$$

$$\boxed{T_v = 0.196}$$

$$t = \frac{T_v d^2}{C_v}$$

$$12.5 = \frac{0.196 \times \left[\frac{2.5}{2}\right]^2}{C_v}$$

$$C_v = 0.0245$$

$$t = \frac{\frac{\pi}{4} \times \left[\frac{50}{100}\right]^2 \times \left[\frac{7.5 \times 100}{2}\right]^2}{C_v} = \frac{\frac{\pi}{4} \times (0.5)^2 \times \left(\frac{750}{2}\right)^2}{0.0245}$$

$$t =$$

Alternate Method

$$\frac{t_1}{t_2} = \frac{\left[\frac{T_v d^2}{C_v}\right]_1}{\left[\frac{T_v d^2}{C_v}\right]_2} \qquad \frac{12.5}{t} = \frac{\frac{\cancel{\frac{\pi}{4}\left[\frac{50}{100}\right]^2}\left[\frac{2.5}{2}\right]^2}{\cancel{C_v}}}{\frac{\cancel{\frac{\pi}{4}\left[\frac{50}{100}\right]^2}\left[\frac{7.5 \times 100}{2}\right]^2}{\cancel{C_v}}}$$

$$\frac{12.5}{t} = \frac{\left[\frac{2.5}{2}\right]^2}{\left[\frac{7.5 \times 100}{2}\right]^2}$$

$$\boxed{t = 1125000 \text{ min}}$$

Q The thickness of a soil sample is 25mm with Double Drainage. The void Ratio of soil is Reduced from 0.92 to When eff. stress is increased from 60 kN/m² to 120 KN calculate co-efficient of compressibility, vol. change, consolidation. If time Required for 50% consolidati is 5min - calculate co-efficient of Permeability also.

$a_v = ?$ $m_v = ?$ $C_v = ?$ $k = ?$

Double Drainage, $\frac{H}{2} = \frac{0.02.5}{2}$ cm

$$a_v = \frac{\Delta e}{\Delta \sigma} = \frac{0.92 - 0.78}{\cancel{1208} - \; 120 - 60} = \boxed{2.33 \times 10^{-3} \ m^2/kN}$$

$$m_v = \frac{a_v}{1+e_0} = \frac{2.33 \times 10^{-3}}{1+0.92} = \boxed{1.21 \times 10^{-3} \ m^2/kN}$$

$$C_v = \frac{k}{m_v \gamma_w}$$

$$1.02 \times 10^{-7} = \frac{k}{1.21 \times 10^{-3} \times 9.81}$$

$$\boxed{k = 1.22 \times 10^{-9} \ m/s}$$

$$\boxed{C_v = 1.02 \times 10^{-7} \ m^2/s}$$

$$\boxed{a_v = 2.33 \times 10^{-3} \ m^2/kN}$$

$$\boxed{m_v = 1.21 \times 10^{-3} \ m^2/kN}$$

for 50 % consolidation

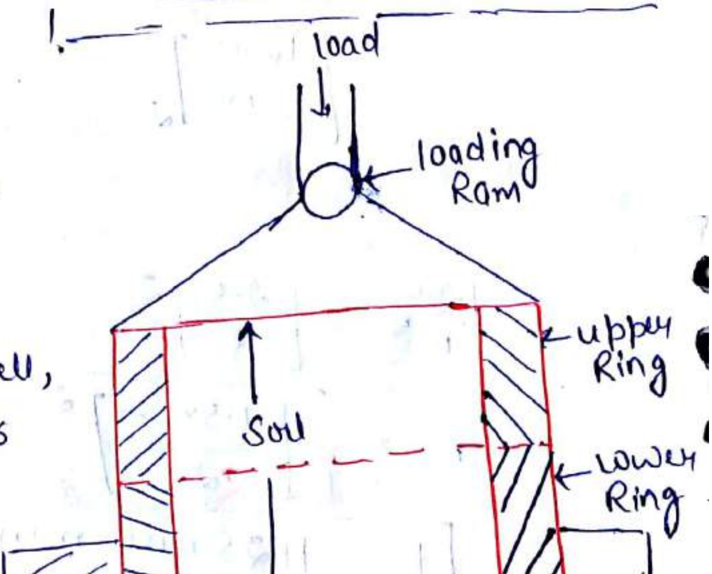
$$\boxed{t = 50 \ min}$$

$k = ?$

$$t = \frac{T_v d^2}{C_v}$$

$$5 \times 60 = \frac{\frac{\pi}{4} \times \left[\frac{50}{100}\right]^2 \times \left[\frac{0.025}{2}\right]^2}{C_v}$$

$$\boxed{C_v = 1.02 \times 10^{-7} \ m^2/s}$$



Measurement of Consolidation

- Major component of consolidometer is Ring cell which has two parts
(a) Upper Ring (b) lower Ring.
- On the basis of Movement of Ring cell, Ring cell is classified into two parts

(a) fixed type Ring cell :-

In this type, lower Ring of Ring cell is fix to the cell base and upper Ring is Removable.

- Measurement of Permeability and Pore pressure is possible

(b) free Type/floating type Ring cell :- In this type both the Rings upper Ring and Lower Ring are Removable from cell base. Measurement of Permeablity and pore pressure is not possible.

Note (i) With increase in temp the consolidation in soil increases. due to increase in permeability.

(ii) Time factor in any soil can be calculated using (a) square Root of time fitting Method.

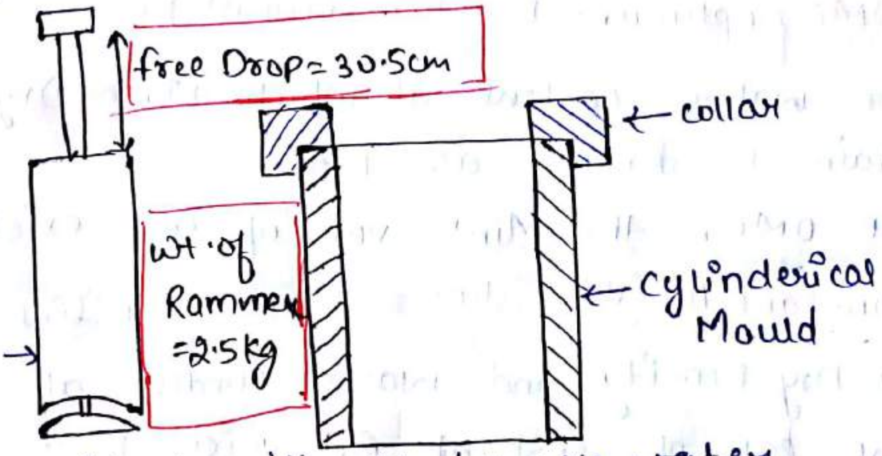
(b) logarithmic of Time fitting Method. $\log_{10} t$

27 Sep 2018

Compaction :- It is the process of Removal of air voids from soil under Mechanical loads.

- Compaction is a short term process which deals with decrease in void Ratio of soil and increase in dry density of soil.

⇒ Standard Proctor Test



(i) 3kg Air Dried sand is Mixed with 4% by wt. water

(ii) Allow the soil b/w 5 min ~~Min~~ to 30 min for maturing

(iii) The soil is field in 3 layers to the cylinderical Mould with 25 no. of blows or drops in each layer

(iv) Dry Density of soil is calculated.

(v) The Quantity of water in increased by 2% and the sam test is repeated in no. of times.

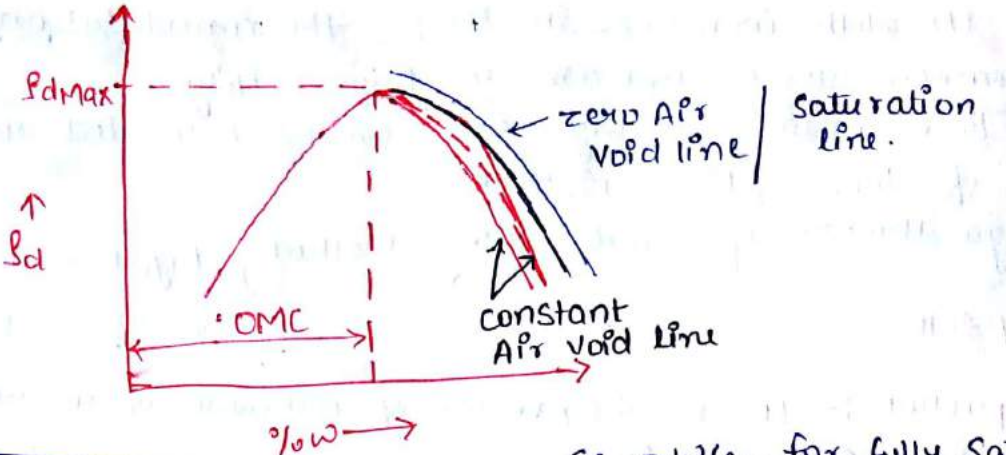
M_1 = Mass of empty Mould

M_2 = Mass of empty Mould + soil

V = Vol. of empty Mould

$$V = \frac{\pi}{4} d^2 H$$

$$\boxed{\rho = \frac{M_2 - M_1}{V}} \; ; \; \boxed{\rho_d = \frac{\rho}{1+w}}$$



$$\boxed{\rho_d = \frac{(1-n_a)G\rho_w}{1+wG}}$$

$Se = wG$ for fully Saturated
$S=1 \Rightarrow e = wG$

$$\boxed{\rho_d = \frac{G\rho_w}{1+wG}}$$

n_a = %age of air voids

1. **OMC (Optimum Moisture Content) :-**

The water content at which Max^m Dry Density of soil can be attain is known as OMC.

- At OMC, the Min^m vol. of soil exist.

2. **Constant Air Void line :-** It is a line which Represents relation b/w Dry Density and water content at constant air voids.

- The eq^n of constant Air void line is given by :-

$$\boxed{\rho_d = \frac{(1-n_a)G\rho_w}{1+wG}}$$

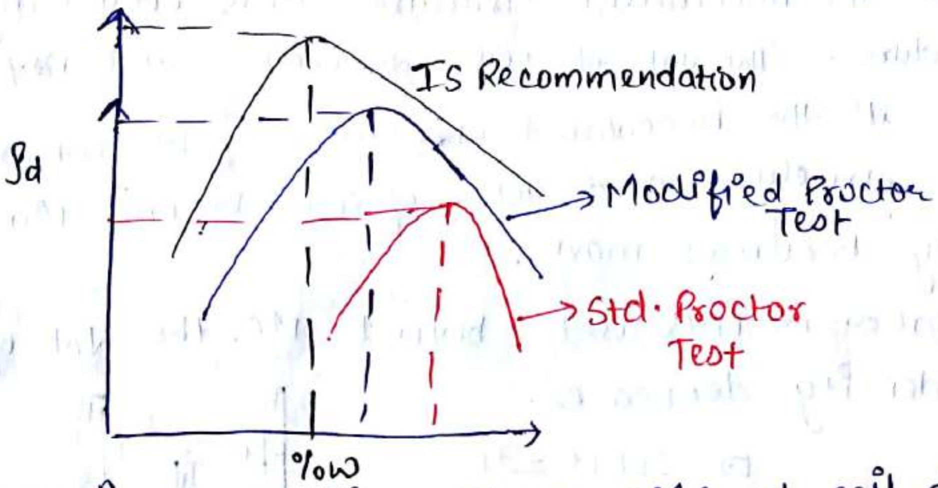
3. **Zero Air Void line / Saturation line :-** It is the line which represents relation b/w dry Density and water content at zero air voids.

Eq^n of zero air void line, $\boxed{n_a = 0}$

$$\boxed{\rho_d = \frac{G\rho_w}{1+wG}}$$

Zero Air Void line is also known as Saturation line.

Standard Proctor Test	Modified Proctor Test	IS Recommendation
Wt. of Rammer → 2.5kg	Wt. of Rammer = 4.5kg	Wt. of Rammer = 4.8kg
free Drop → 30.5cm	free Drop = 45cm	free Drop = 45cm
No. of layers → 3	No. of layers = 5	No. of layers = 5
No. of Drops in each layer = 25	No of Drops in each layer = 25	No. of Drops in each layer = 25



- With increase in compactive efforts, OMC of soil decreases and Maxm Dry Density of soil increases.

Factors Affecting compaction in soil

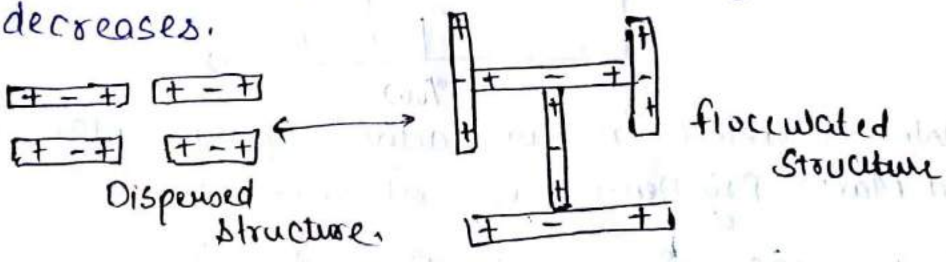
① Water Content

(a) Lubrication Theory :- Dry Sand particles offer Maxm frictiona Resistance which doesnot allow the particles to compact

- As some water is added, this water acts as lubricant and Reduces frictional Resistance.
- Due to which the particles comes in contact with each othe and vol. of soil decreases. Hence dry density increases.
- At OMC, the vol. of soil become Minm because at ~~at~~ the frictional forces b/w Particles become zero and γ_d becomes Maxm.
- As the water is increased beyond OMC, the vol. of soil start increasing and dry density of soil decreases.

(b) Electrical Double Diffuse layer Theory

- Initial clay particle contain flocculated structure with some diffused layer of water. Due to strong attractive forces clay particles cannot subjected to compaction. (Decrease in volume)
- As water is added to clay particles the difuse layer of water tries to expand and their attractive forces decreases.
- During expansion of Diffuse layer external load breaks attractive forces and the flocculated structure starts conversing into dispersed structures. The vol. of soil decreases and Dry Density increases.
- At OMC, all the flocculated structures gets converted into Dispersed structure and vol. of soil become Minm. Hence dry density becomes maxm.
- As the water is increased beyond OMC, the vol. of soil increases and dry density decreases.

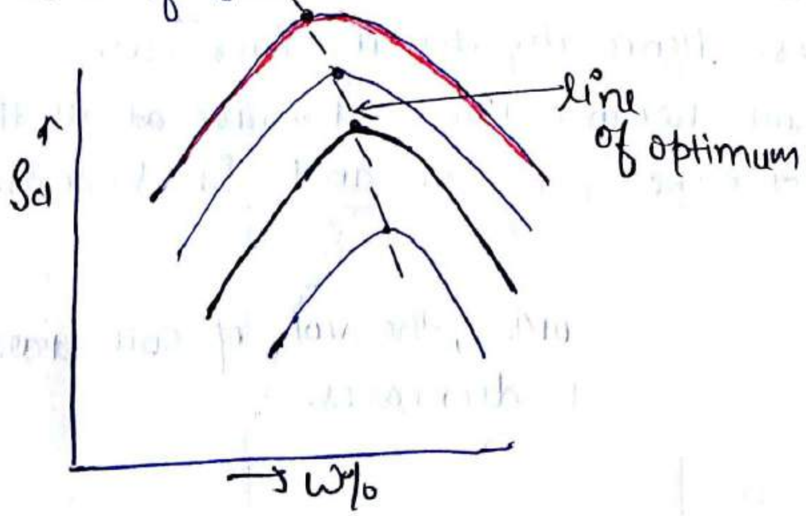


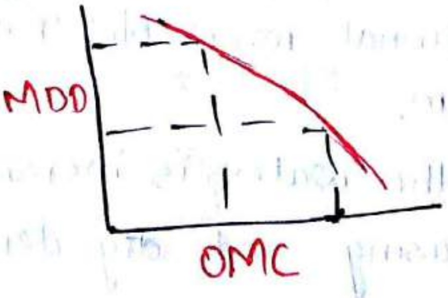
⟨2⟩ Compactive Efforts | Compactive Energy :-

(i) With increase in compactive efforts higher value of Maxm Dry Density can be attained in soil at lower value of OMC.

(ii) The line joining points of Maxm Dry Densities or compaction Curve is known as line of optimum.

*** (iii) Line of optimum Represents Relation b/w Maxm Dry Density (MD and OMC of soils.





3) Type of Soil :- (i) Due to high surface forces in fine soil, OMC of fine soil is more than OMC of coarse soil.

(ii) MDD of fine soil is less than MDD of coarse soil.

'OMC order → gravel < sand < Silt < clay

MDD order → Clay < silt < Sand < Gravel

(iii) With increase in Quantity of fine soil to the available coarse soil, OMC value increase and MDD decreases.

(iv) With increase in Plasticity of soil, OMC of soil increases and Maxm Dry Density decreases.

OMC order ⇒ low plastic < Intermediate plastic < High Plastic

MDD order ⇒ High plastic < Intermediat plastic < Low plastic

low plastic $\boxed{W_L < 350}$

Intermediate Plastic $\boxed{35 < W_L \leq 50}$

High Plastic $\boxed{W_L > 50}$

Q. Arrange following Soil is in increasing order of OMC

CH, GP, ML, CL, SW

fine Soil > coarse soil
OMC OMC

$\boxed{GP < SW < ML < CL < CH}$

Q. Arrange in increasing order of MDD

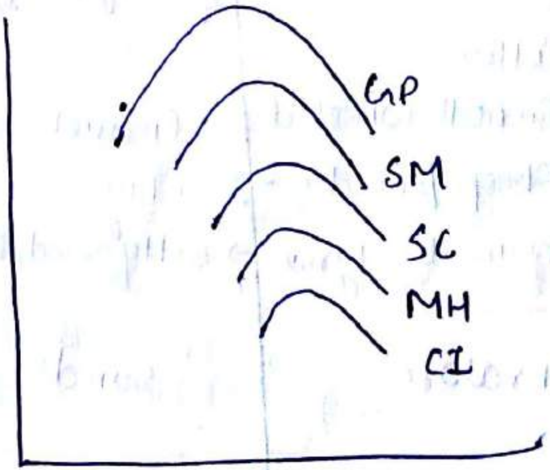
Sand Sand
GC, SM, CI, MH

$\boxed{CI < MH < SM < GC}$

Q. SM, CI, MH, GP, SC
increasing order of MDD

CI < MH < SC < SM < GP

↓ Clay ↓ Sil ↓ Sand ↓ Gravel



(4) **Amount of Compaction :-**

(i) With increase in time of loading such that amount of loading and contact area remains constant. The compaction in soil increases

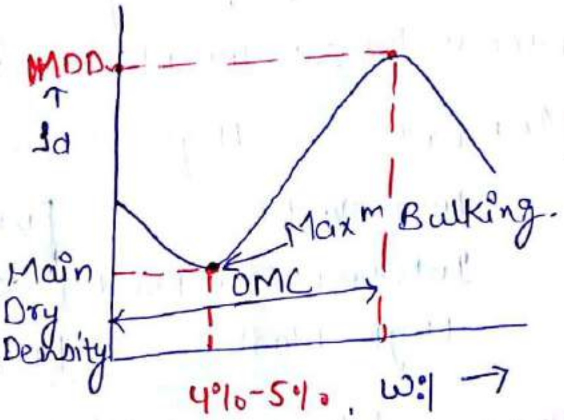
(ii) With increase in amount of loading such that time and contact area remains constant. The compaction in soil increases.

(iii) With increase of contact area such that time and amount of loading remains constant. The compaction in soil decreases.

(5) **Compaction in Cohesionless soil :-**

→ film of water increases in vol.

$$\boxed{\gamma_d \downarrow = \frac{M}{V \uparrow}}$$



- Initially, when some water is added to dry sand the vol. of sand particles increases.
- This is due to formation of water layer over the surface of solid particles and vol. increases. This property is known as Bulking of sand
- This Bulking of sand is due to formation of surface tension or capillary tension over solid particles.
- Maxm Bulking take place at water content 4% - 5%
- Beyond the water content the dry density of soil increases upto OMC and then decreases. (Acc. to lubrication theory)

Compaction in field

Device Name	Type of Soil	Uses
① Rammer	All type of soil	Confined construction Areas
② Roller		
(a) Smooth wheeled →	Gravels	→ Pavements, Subgrade (Base, Sub-base)
(b) Sheep footed →	Clay	→ Earthen Dams
(c) Pneumatic Tyred →	Silty Sand, Silty Gravels →	Base and Sub base of Pavements
③ Vibrators	Sand	Earthen Embankments

Conditions of Compactions :-

(a) **Wet of optimum** :- In this condition, the soil is compacted at water content more than OMC.

- Dispersed structure b/w clay particles exist.
- Value of pore water pressure b/w particles will be more
- Swelling b/w particles will be more.
- less chances of brittle failure exist.

(b) **Dry of optimum** :- In this condition, soil is compacted at water content less than OMC. $\boxed{\%W < OMC}$

- Flocculated structures b/w clay particles exist.
- Pore water pressure b/w particles will be less.
- No swelling b/w particles exist.
- Compressibility of soil particles will be more due to presence of flocculated structures.
- High chances of Brittle failure exist.

Note :- (i) With compaction b/w soil particles, the permeability of soil decreases.

(ii) Sand particles are always compacted at dry of optimum condition

(iii) Clay particles are compacted at wet of optimum condition.

Types of compaction :-

IS Light wt. Compaction	IS Heavy wt. compaction
→ Wt. of Rammer = 2.6 kg	→ Wt. of Rammer = 4.8 kg – 4.9 kg
→ free Drop = 31 cm	→ free Drop = 45 cm
→ No. of layer = 3	→ No. of layers = 5
→ No. of Drop in each layer = 25	→ No. of Drop in each layer = 25

$$\text{Compactive } \cancel{\text{eng}}\text{ energy} = \frac{\text{Wt. of Rammer} \times \text{free Drop} \times \text{No. of layer} \times \text{No. of dro} \text{ in each lay}}{\text{Vol. of soil } (1\,m^3)}$$

$$\boxed{\text{Heavy wt. compactive Energy} = 4.55 \times \text{light wt. Compactive Energy}}$$

Ques Indian Standard light wt. compaction is always perform in 3 layer of soil having vol. 1000 cm3. Calculate the no. of drop required in each layer During same test if vol. of soil is 2250 cc

$$\text{Compactive Energy} = \frac{\text{Wt. of Rammer} \times \text{free Drop} \times \text{no. of layers} \times \text{No. of Drops in each layer}}{\text{vol. of soil}}$$

$$\text{Compactive Energy} = \frac{2.6 \times 31 \times 3 \times 25}{1000} = \frac{2.6 \times 31 \times 3 \times \text{no. of Drops}}{2250}$$

$$\boxed{n = 56.25 \simeq 57 \text{ drops}}$$

Relative Density & Density Index :

$$I_d = \frac{e_{max} - e}{e_{max} - e_{min}}$$

$\boxed{I_d = 0}$ $\boxed{e_{max} = e}$

$I_d = 100\%$ $\boxed{e = e_{min}}$

$$\boxed{0 \le I_D \le 100}$$

Relative compaction

$$RC = \frac{[\gamma_d]_{field}}{[\gamma_d]} \times 100$$

$$\boxed{80\% \le RC \le 100}$$

⇒ **Relative Compaction** :- The Ratio of dry density of soil in its natural State to its maxm Dry Density is known as Relative compaction.

$$\boxed{RC = \frac{[\gamma_d]_{field}}{\gamma_d} \times 100}$$

$\boxed{80\% \le RC \le 100\%}$ — for Practical Case

* Relative compaction in any soil can never be zero but Relative Density can be zero.

* Relative Density and Relative compaction can be 100%.

$$RC = \frac{[\gamma_d]_{field}}{(\gamma_d)_{max}} \times 100$$

$$RC = \frac{\frac{\cancel{G\gamma_w}}{1+e}}{\frac{\cancel{G\gamma_w}}{1+e_{min}}} \times 100$$

Concept

$$\gamma_{d\,field} = \frac{G\gamma_w}{1+e}$$

$$\boxed{\gamma_{d\,max} = \frac{G\gamma_w}{1+e_{min}}}$$

$$\boxed{RC = \left[\frac{1+e_{min}}{1+e}\right] \times 100}$$

e = void Ratio in natural State

e_{min} = Min void Ratio in Densest State

SHEAR STRENGTH OF SOIL

\# **Shear Strength** :- The maxm shear stress that a soil can carry just before its failure is known as shear strength.

* Shear strength is a function of effective stress.

* Shear strength b/w soil particles depends upon three parameters:

(i) **Structural Resistance** :- It represents the Resistance offered by interlocking b/w soil particles.

(ii) **frictional Resistance** :- It is the Resistance offered by frictional forces b/w sand particles.

* This Resistance is expressed in terms of angle of internal fricti

$$\tan\phi = \mu \Rightarrow \boxed{\phi = \tan^{-1}\mu}$$

μ = co-efficient of friction

ϕ = Angle of internal friction

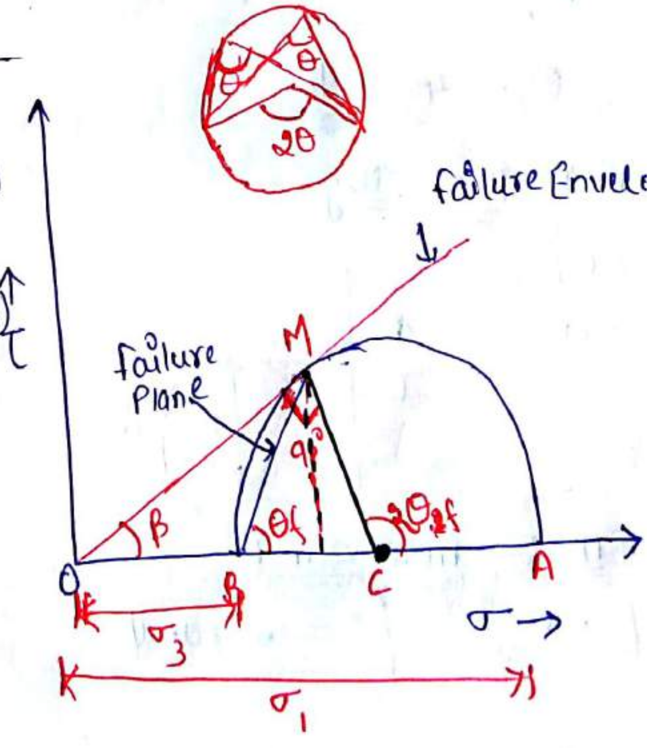
* With increase in compaction b/w sand particles value of ϕ increases.

* for clay particles $\boxed{\phi = 0}$

(iii) **cohesion b/w Particles** :- It is the Resistance offered by surfac cohesive forces b/w clay Particles

* for Pure Sand, cohesion is zero

\# **Mohr's Stress Circle** :-



σ_1 = Major Principal stress (Maxm stress)

σ_2 = Intermediate Principal Stress

σ_3 = Minor Principal Stress (Minm stress)

Dia of circle = $\sigma_1 - \sigma_3$

Radius of circle = $\dfrac{\sigma_1 - \sigma_3}{2}$

In ΔOCM,

$$2\theta_f = 90° + \beta$$

θ_f = failure Angle, $\boxed{\theta_f = 45° + \beta/2}$

Q 0-3m Ht. $\phi = 30°$, $\gamma = 18$ KN/m3 c = 0

3-9m Ht. $\phi = 35°$, $\gamma = 20$ KN/m3

Mohr's coulomb Theory

⇒ **Assumptions** :- 1. All the materials fails by shear only.

(2) Minor principal stress down and Major Principal stress are subjected to failure of material.

(3) Intermediate principal stress donot have any influence on failure of material.

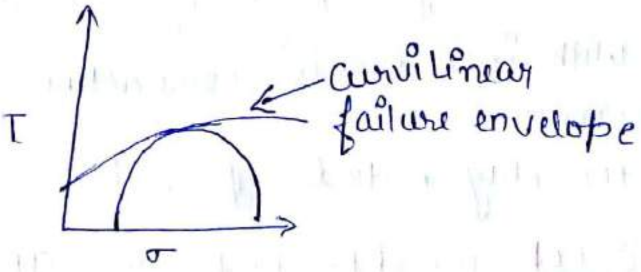
i) Coulomb's Theory

$s = \tau_f = f(\sigma)$ τ_f = Shear stress at failure

↑ function of total stress

(ii) Mohr's Theory

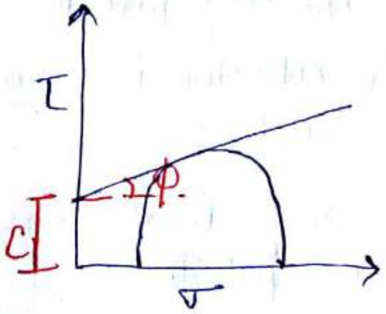
$s = \tau_f = f(\sigma)$



(iii) Mohr-coulomb theory

$$\boxed{s = \sigma \tan\phi + c}$$

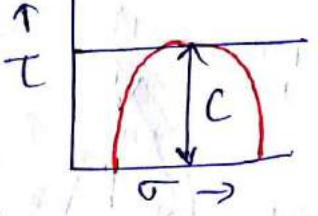
$$\theta_f = 45° + \frac{\phi}{2}$$



Case-I Pure Clay

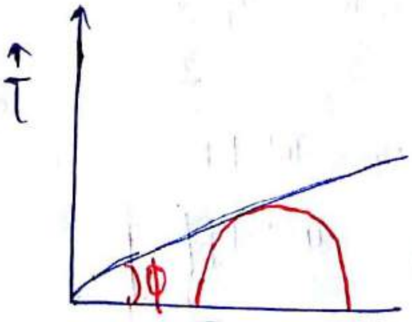
$\boxed{s = C}$

$\phi = 0$



Case II :- Pure Sand

$\boxed{C = 0}$ $\boxed{s = \sigma \tan\phi}$



(iv) Revised coulomb theory :-

$\boxed{s = \bar{\sigma}\tan\phi' + c'}$ $\boxed{\bar{\sigma} = \sigma - u}$

$\uparrow$ eff stress

Ques Calculate shear strength of soil which is subjected to total stress of 100KN/m2. The strength Parameters of soil are 60° and 30 KN/m2.

$s = \sigma \tan\phi + c$

$s = 100\tan 60 + 30 = 100\sqrt{3} + 3$

$\boxed{s = 203 \text{ KN/m2}}$

Ques calculate shear strength of Pure clay which is subjected to external stress of 50 KN/m2. The cohesion b/w particles is 60 KN/m2

Pure clay $\boxed{\phi = 0}$ $\sigma = \cancel{30}\ 50 \text{ KN/m2}$

$C = 60 \text{ KN/m2}$

$s = \sigma \tan\phi + c$

$s = 0 + c \Rightarrow 0 + 60$ $\boxed{s = 60 \text{ KN/m2}}$

Q failure plan makes an angle of 55° with horizontal in pure sand calculate shear strength parameters.

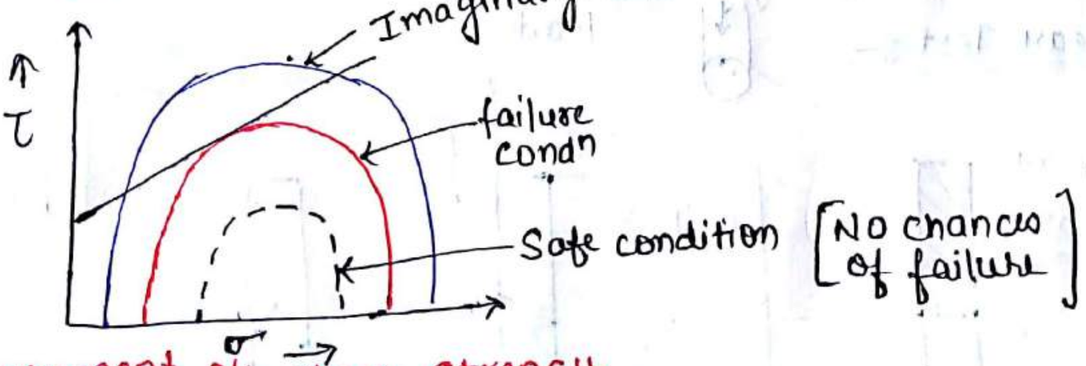
$\theta_f = 55°$

Pure Sand $\Rightarrow \boxed{C = 0}$

$\theta_f = 45° + \frac{\phi}{2} \Rightarrow \frac{\phi}{2} = \cancel{55}° - 45°$

$\boxed{\phi = 20°}$ Ans

Note :-



Measurement of Shear strength

- Stages of Shear strength Test :

1. Consolidation stage : In this stage, the soil is allowed to consolidate for 24 Hours under constant minimum load.

2. Shear Stage :- In this stage, the load is increased at const. strain Rate

- The load at which shear crack in soil take place, is known as shear strength.

Conditions of Shear Test

1. UU Test condition (Unconsolidated Undrained Test)

In this test, the drainage of water is not allowed in any stage.

- Measurement of pore water pressure is not done.
- The consolidation stage does not exist. only shear stage exists.
- This test is a quick test (Q-Test) 10 - 15 min time.
- This test is used for low permeable soil [ex - Clay]

2. CU Test (Consolidated Undrained Test)

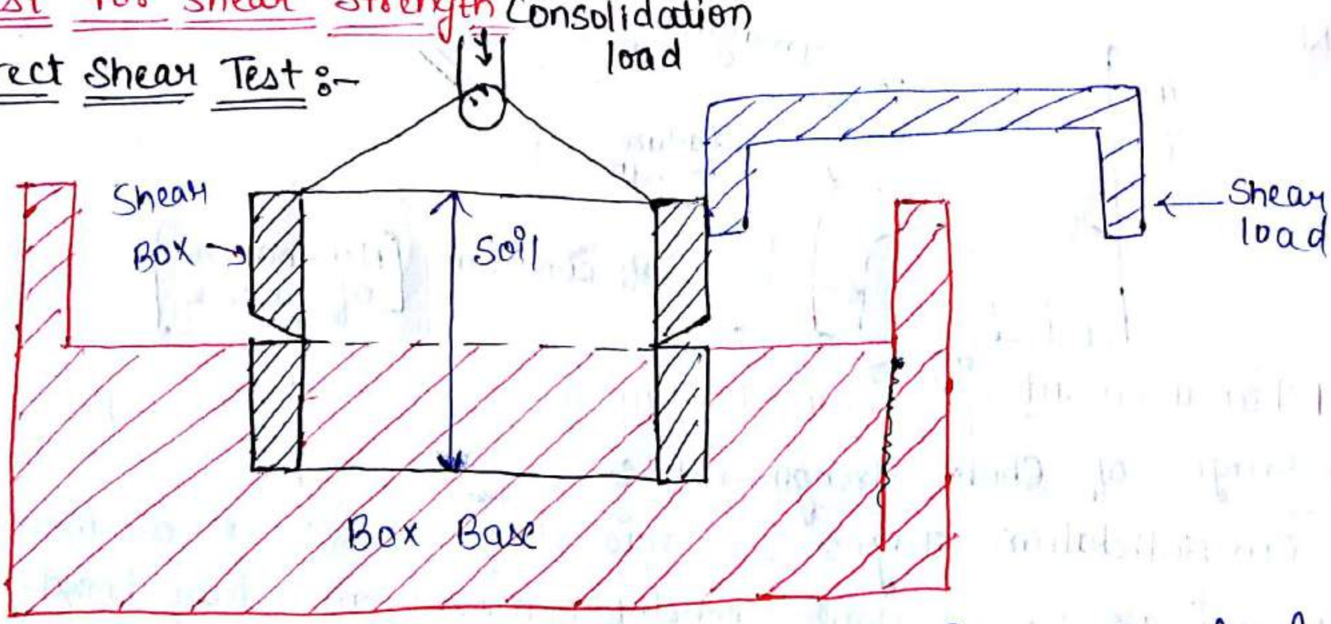
- The drainage of water is allowed only in consolidation stage.
- Measurement of pore water pressure is done in consolidation stage only.
- It is also known as (R - Test)
- This test is used for steady seepage conditions.

3. CD Test (Consolidated Drained Test) :-

- The Drainage of water is allowed in both stages.
- Measurement of Pore pressure is done in both stages
- This test is a slow Test (S - test).
- This test is used for Highly permeable soils. [Ex :- sand]

Test for shear Strength

1 Direct Shear Test :-



- ~~Square~~ Shear Box can be a square / circular Box having size 60-90 mm

(i) This test is used for unconsolidated Undrained test condition for Sand.

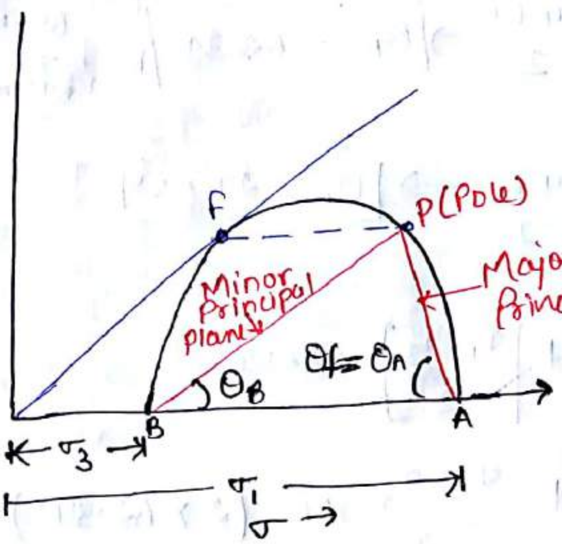
(i) The shear box may be circular or square in shape having size 60 - 90mm.

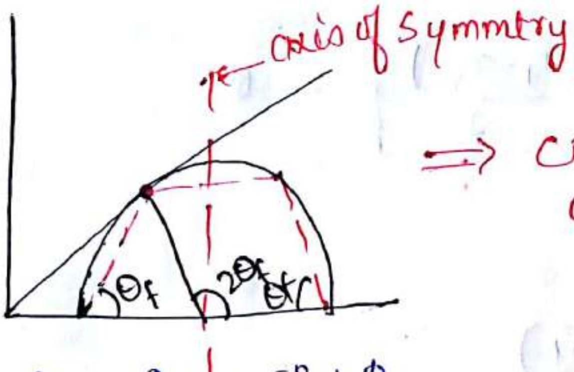
LIMITATIONS

(i) There is no control over Drainage of water
(ii) There is no Device to measure pore water pressure.
(iii) Stress Distribution is non - uniform.
(iv) failure plain is always pre - determined. which may not be weakest plane.

θ_A = Obliquity of Major Principal Plane

θ_B = Obliquity of Minor Principal plane





⟹ Concept of above Dia

$\theta_A = \theta_f = 45° + \frac{\phi}{2}$

In ΔPAB

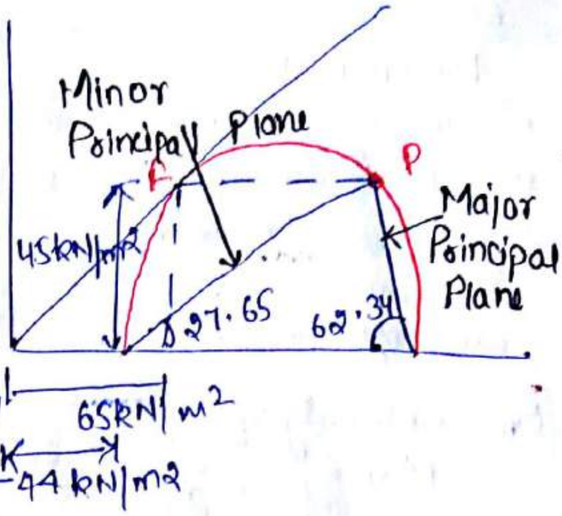
$\theta_A + \theta_B + 90 = 180°$

$45° + \frac{\phi}{2} + \theta_B + 90° = 180°$

$$\boxed{\theta_B = 45° - \frac{\phi}{2}}$$

Q A Direct Shear test was performed on pure sand havi normal stress and shear stress at failure 65 kN/m2 and 45 kN/m2 Respectively. Calculate
(i) Angle of internal friction
(ii) Major and Minor Principal Stress
(iii) Obliquity of Major and Minor Principal stress.

(iv) Draw the Mohr's Stress circle and locate Pole Point.



$$\tan\phi = \frac{\tau}{\sigma} = \frac{FE}{OE} = \frac{45}{65}$$

$$\phi = \tan^{-1}\left[\frac{45}{65}\right]$$

$$\boxed{\phi = 34.69°}$$

(iii) $\theta_A = 45° + \frac{\phi}{2}$

$$\theta_A = 45 + \frac{34.69}{2} \Rightarrow \boxed{\theta_A = 62.34°}$$

$$\theta_B = 45° - \frac{\phi}{2}$$

$$= 45° - \frac{34.69}{2} \Rightarrow \boxed{\theta_B = 27.65°}$$

(ii) $\sigma = \left[\frac{\sigma_1+\sigma_3}{2}\right] + \left[\frac{\sigma_1-\sigma_3}{2}\right]\cos 2\theta_f$

$$65 = \frac{\sigma_1+\sigma_3}{2} + \frac{\sigma_1-\sigma_3}{2}\cos(2 \times 62.34°)$$

$$\boxed{65 = \frac{\sigma_1+\sigma_3}{2} + \cancel{\sigma}(-0.56)\left(\frac{\sigma_1-\sigma_3}{2}\right)} \quad — ①$$

$$\tau = \frac{\sigma_1-\sigma_3}{2}\sin 2\theta_f$$

$$45 = \frac{\sigma_1-\sigma_3}{2}\sin(2 \times 62.34)$$

$$\boxed{45 = 0.82\left[\frac{\sigma_1-\sigma_3}{2}\right]} \quad — ②$$

$$\boxed{\sigma_1 - \sigma_3 = 109.75} \quad — ①$$

$$65 = \frac{\sigma_1+\sigma_3 - 0.56\sigma_1 + 0.56\sigma_3}{2}$$

$$\boxed{65 \times 2 = 0.44\sigma_1 + \cancel{0.44}\,1.56\sigma_3} \quad — ②$$ Equate (i) & (ii)

$$130 = 0.4\cancel{4}\sigma_1 + 1.56\sigma_3$$

$$-48.29 = -0.\cancel{4}4\sigma_1 \mp 0.44\sigma_3$$

σ_1 = ~~40~~ 152 kN/m2

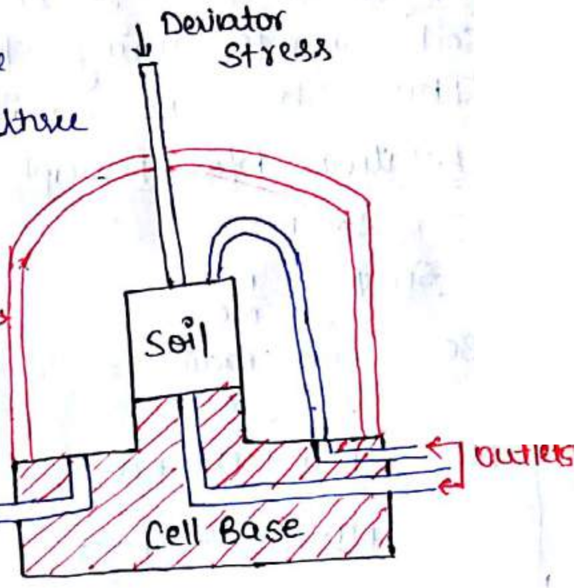
σ_3 = 44 kN/m2

$$2\sigma_3 = 81.71$$

σ_1 = 157.6 kN/m2

σ_3 = 40.855 kN/m2

Triaxial Compression Test



(i) In this test the compressive forces are applied on soil sample from all the three Directions

(ii) This test is used for any type of soil

(iii) This test can be used for any condition of soil test. (UU, CU and CD)

(iv) Minor principal stress and intermediate principal stress are equal which are known as cell pressure or confining pressure.

$$\boxed{\sigma_2 = \sigma_3 = \text{cell pressure} = \text{Confining Pressure}}$$

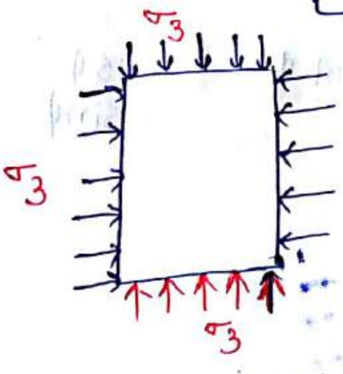
(v) There is control of drainage of water and Measurement of pore pressure is possible.

(vi) Stress Distribution is uniform

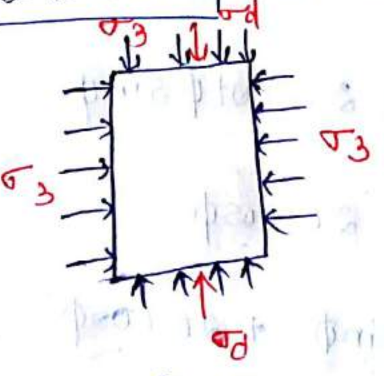
(vii) failure plan is not Predetermined.

(viii) Length of soil Sample is 2 to 2.5 ~~Dia~~ times Dia of soil.

$$\boxed{L = 2 \text{ to } 2.5D}$$



Consolidation Stage



Shear stage

Major Principle stress, $\sigma_1 = \sigma_d + \sigma_3$

$$\boxed{\text{Deviator Stress} = \frac{\text{ext. load applied}}{\text{Area at failure}} = \frac{P}{A_f}}$$

$$\boxed{A_f = \frac{A_0}{1 - E_L}}$$

E_L = linear Strain $= \frac{\Delta L}{L}$

A_0 = Origional c/s Area

* **Deviator stress** :- It is a true stress applied externally on a Soil sample during shear stage

- This Stress is considered at area at failure.

* **Relation b/w Principle stresses at failure:** OR Prove $\sigma_1 = \sigma_3 N\phi + 2c\sqrt{N\phi}$

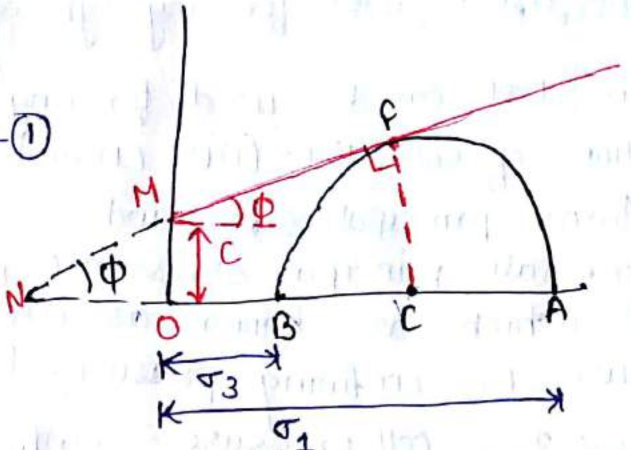
In ANCF

$$\sin\phi = \frac{FC}{NC}$$

$$BC = FC = \text{Radius} = \frac{\sigma_1 - \sigma_3}{2} \quad ①$$

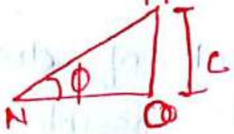
$$NC = NO + OB + \alpha$$

$$= NO + \sigma_3 + \frac{\sigma_1 - \sigma_3}{2}$$



$$NC = c\cot\phi + \sigma_3 + \frac{\sigma_1}{2} - \frac{\sigma_3}{2}$$

$$\boxed{NC = c\cot\phi + \frac{\sigma_1 + \sigma_3}{2}} \quad ②$$



$$\tan\phi = \frac{MO}{ON} = \frac{c}{ON}$$

$$\boxed{ON = c\cot\phi}$$

$$\frac{\sin\phi}{1} = \frac{\frac{\sigma_1 - \sigma_3}{2}}{c\cot\phi + \frac{\sigma_1 + \sigma_3}{2}}$$

$$\frac{\sigma_1 - \sigma_3}{2} = \left[\frac{\sigma_1 + \sigma_3}{2}\right]\sin\phi + c\cot\phi\sin\phi$$

$$\sigma_1 - \sigma_3 = (\sigma_1 + \sigma_3)\sin\phi + 2c\cot\phi\sin\phi \qquad \cot\phi = \frac{\cos\phi}{\sin\phi}$$

$$\sigma_1 - \sigma_3 = (\sigma_1 + \sigma_3)\sin\phi + 2c\cos\phi$$

$$\sigma_1 - \sigma_1\sin\phi = \sigma_3 + \sigma_3\sin\phi + 2c\cos\phi$$

$$\sigma_1[1 - \sin\phi] = \sigma_3[1 + \sin\phi] + 2c\cos\phi$$

$$N\phi = \tan^2\left[45 + \frac{\phi}{2}\right] + \sqrt{N}\left[\frac{\phi}{2} + 45\right]$$

$$\sigma_1 = \sigma_3\left[\frac{1 + \sin\phi}{1 - \sin\phi}\right] + 2c\frac{\cos\phi}{1 - \sin\phi}$$

$$\boxed{\sigma_1 = \sigma_3 \underbrace{\tan^2\left[45 + \frac{\phi}{2}\right]}_{N\phi} + 2c\underbrace{\tan 45 + \frac{\phi}{2}}_{\sqrt{N\phi}}}$$

$$\boxed{\sigma_1 = \sigma_3 N\phi + 2c\sqrt{N\phi}}$$

Ques A pure sand sample is subjected to cell pressure of 100 kN/m2 under triaxial compression test. if the angle of internal friction is 36°. calculate Deviator Stress applied on soil

Ans $\sigma_3 = 100$ kN/m3

$\phi = 36^\circ$

$\sigma_d = ?$

Pure sand $\boxed{C=0}$

$$\sigma_1 = \sigma_3 \tan^2 \left[45 + \frac{\phi}{2}\right]$$

$$\sigma_1 = 100 \tan^2 \left[45 + \frac{36}{2}\right] = 385.183 \text{ kN/m}^2$$

$$\sigma_1 = \sigma_d + \sigma_3 \Rightarrow 385.18 = \sigma_d + 100$$

$$\boxed{\sigma_d = 285.18 \text{ kN/m}^2}$$ Ans.

Q A clay specimen when tested under triaxial comp. test subjected to cell pressure of 150 kN/m2. calculate value of Major Principal stress if cohesion b/w particles is 30 kN/m2

Ans $\sigma_3 = 150$ kN/m2 $\boxed{\phi = 0} \Rightarrow$ Pure clay.

$$\sigma_1 = 150 \times \tan^2[45] + 2 \times 30 \left[\tan 45^\circ\right]$$

$$\boxed{\sigma_1 = 210 \text{ kN/m}^2}$$ Ans

Ques Select the correct option for area at failure

(a) $A_f = \frac{V}{1-\Delta V}$ (b) $A_f = \frac{A_0}{1+\Delta A}$

~~(c)~~ $A_f = \frac{V}{L-\Delta L}$ (d) $A_f = \frac{V}{1-E_L}$

★ **Concept**

$$A_f = \frac{A_0}{1-E_L} = \frac{A_0}{1-\frac{\Delta L}{L}}$$

$$A_f = \frac{A_0}{\frac{L-\Delta L}{L}} \Rightarrow \frac{A_0 L}{L-\Delta L} = \frac{V}{L-\Delta L}$$

★★ $$\boxed{A_f = \frac{V}{L-\Delta L}}$$

Ques Pure clay specimen has c/s area of 50 cm² and subjected to cell pressure of 20 ~~kN/cm²~~ N/cm². The Deviator load at failure is 500N. with 10% strain during failure. Calculate Major Principle stress at failure.

Pure clay = $\boxed{\phi = 0}$

$A_0 = 50$ $\quad \sigma_3 = 20\ N/cm^2$ $\quad$ ~~σ_d~~ $= 500\ N$ $\sigma_d =$ ~~$10\ N/cm^2$~~ $\frac{500}{}$ $\quad \sigma_d = \frac{500}{1-0.1}$

$$\sigma_d = \frac{P}{A_f} = \frac{500}{\left[\frac{50}{1-0.1}\right]} = 9\ N/cm^2$$

$$\sigma_1 = 20\ \sigma_3 + \sigma_d \Rightarrow 20 + 9$$

$$\boxed{\sigma_1 = 29\ N/cm^2}$$

③ Unconfined Compression Test

(i) This test is a special case of tri-axial compression test, in which cell pressure is zero.

(ii) This test is used for U-U test condition.

(iii) This test is done in clayey soil

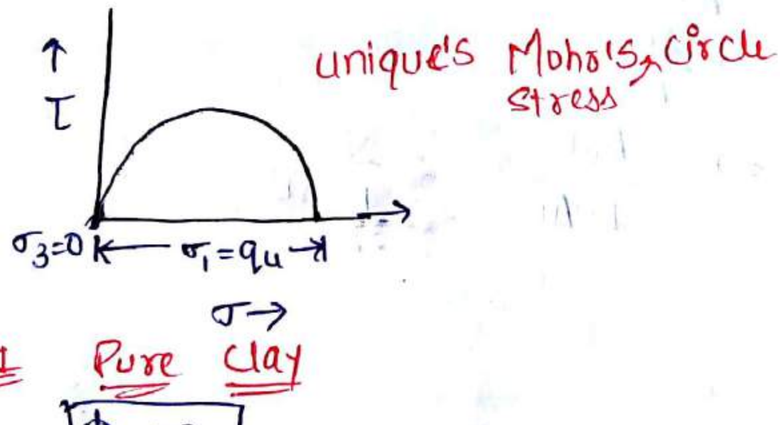
$$\sigma_1 = \sigma_3 \tan^2\left[45 + \frac{\phi}{2}\right] + 2c \tan\left[45 + \frac{\phi}{2}\right]$$

$$\boxed{\sigma_1 = \sigma_3 + \sigma_d}$$

$$\boxed{\sigma_3 = 0}$$

$\sigma_1 = \sigma_d = q_u$ [unconfined compression strength]

$$\boxed{\sigma_1 = q_u = 2c \tan\left[45 + \frac{\phi}{2}\right]}$$



Case 1 Pure Clay

$$\boxed{\phi = 0}$$

$$\Rightarrow q_u = 2c \cdot \tan\left[45 + \frac{0}{2}\right]$$

$\boxed{q_u = 2c}$ (or) $\boxed{c = \frac{q_u}{2}}$

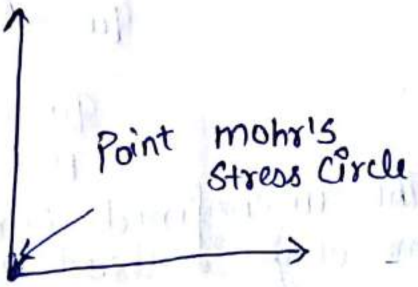
Shear Strength of clay, $\boxed{s = c = \frac{qu}{2}}$

Case II Pure Sand

$\boxed{C = 0}$

$\Rightarrow qu = 2C \tan\left[45 + \frac{\phi}{2}\right]$

$\therefore qu = 0$



Ques 6- calculate shear strength Parameters (c, ϕ) (Triaxial test)

Sample	Cell Pressure (kN/m2	Deviator Stress (kN/m2)	σ_1
I	70	230	300
II	350	550	900

$\sigma_1 = \sigma_3 \tan^2\left(45 + \frac{\phi}{2}\right) + 2c\tan\left(45 + \frac{\phi}{2}\right)$

$300 = 70\tan^2\left(45 + \frac{\phi}{2}\right) + 2c\,\cancel{\tan}\left(45 + \frac{\phi}{2}\right)$ ——①

$-900 = -350\tan^2\left(45 + \frac{\phi}{2}\right) + 2c\,\cancel{\tan}\left(45 + \frac{\phi}{2}\right)$ ——②

$-600 = -280\tan^2\left[45 + \frac{\phi}{2}\right]$

$2.14 = \tan^2\left[45 + \frac{\phi}{2}\right]$ $\boxed{\phi = 21.28^\circ}$ (Pending)

$\phi = 21^\circ$ $c = 51\ kN/m^2$

Q A soil specimen was fail at 200 kN/m² stress when tested under unconfined compression test. The failure plan mak an angle of 55° with Horizontal. calculate the cohesion b/w soil particles.

$\sigma_1 = \sigma_d = qu = 200\ kN/m^2$

$\boxed{\theta_f = \left[55^\circ = 45 + \frac{\phi}{2}\right]}$

$qu = 2C\tan\left[45 + \frac{\phi}{2}\right]$

$200 = 2c\tan 55^\circ$

$\boxed{C = 70.02\ kN/m^2}$

Q Calculate the cohesion b/w clay particles if unconfined compression strength of clay particle is 15 kg/m²

Ans

Clay $\boxed{\phi = 0}$

$$q_u = 2c \tan\left(45^\circ + \frac{\phi}{2}\right)$$

$$q_u = 2c$$

$$15 = 2C \Rightarrow \boxed{C = 7.5 \text{ kg/cm}^2}$$

Q The unconfined comp strength of pure clay is 45 kN/m² if same clay is tested under triaxial compression test having cell pressure 120 kN/m². calculate Major Principal stress.

Ans $q_u = 45$ kN/m² $\sigma_3 = 120$ kN/m2

$\sigma_1 = ?$

Pure clay :- $q_u = 2c = 45$ kN/m²

$$\sigma_1 = \sigma_3 \tan^2\left[45 + \frac{\phi}{2}\right] + 2c \tan\left(45 + \frac{\phi}{2}\right)$$

$\boxed{\phi = 0} \Rightarrow$ Pure clay

$$\sigma_1 = 120 \tan^2(45+0) + 45 \tan(45+0)$$

$$\sigma_1 = 120 + 45 = \boxed{165 \text{ kN/m}^2}$$

Q A pure clay 8cm in Dia and 20cm in length subjected to failure load of 100N under unconfined compression test calculate the shear strength of clay particles if the change in length of sample was observed 2cm.

$\cancel{q_u} = \cancel{100}\ x$

$$\sigma_1 = \sigma_d = q_u$$

$$\sigma_d = \frac{P}{A_f} \Rightarrow \frac{P}{\frac{A_0}{1-\epsilon_L}}$$

$$A_f = \frac{A_0}{1-\epsilon_L} \Rightarrow \frac{\frac{\pi}{4}(8)^2}{1-\frac{2}{20}} = 55.85 \text{ cm}^2$$

$$q_u = \sigma_d = \frac{P}{A_f} = \frac{100}{55.85} = 1.79 \text{ kN/m}^2$$

Shear strength $s = c + \sigma \tan\phi$

$$s = c = \frac{q_u}{2} = \frac{1.79}{2} = \boxed{0.895 \text{ N/cm}^2}$$

Note:- (i) In case of drained test, $\boxed{\text{CU and CD}}$ all types of NCC behaves like purely frictional soil. $\boxed{C=0}$

$$\sigma_1 = \sigma_3 \tan^2\left(45^\circ + \frac{\phi}{2}\right) + 2c \tan\left(45 + \frac{\phi}{2}\right)$$

⇒ Purely frictional soil ⇒ $\boxed{C=0}$

$$\boxed{\sigma_1 = \sigma_3 \tan^2\left[45^\circ + \frac{\phi}{2}\right]}$$

$$\sigma_1 = \sigma_3 \left[\frac{1+\sin\phi}{1-\sin\phi}\right]$$

$$\sigma_1\left[1-\sin\phi\right] = \sigma_3\left[1+\sin\phi\right]$$

$$\sigma_1 - \sigma_1\sin\phi = \sigma_3 + \sigma_3\sin\phi$$

$$\sigma_1 - \sigma_3 = (\sigma_1 + \sigma_3)\sin\phi$$

$$\boxed{\sin\phi = \frac{\sigma_1 - \sigma_3}{\sigma_1 + \sigma_3}}$$

Q A normally consolidated clay subjected to cell pressure and Deviator stress of 450 kN/m2 each under consolidated undrained test. The ~~cell~~ pore pressure was found to be 20 kN/m2. Calculate Angle of internal friction and eff. angle of internal friction.

$\sigma_3 = 50$ kN/m2 $\quad \sigma_d = 50$ kN/m2

$u = 20$ kN/m2

$\sigma_1 = \sigma_3 + \sigma_d = 100$ kN/m2

$$\sin\phi = \frac{100-50}{100+50} = \frac{50}{150}$$

$$\boxed{\phi = 19.47^\circ}$$

$\bar{\sigma}_1 = \sigma_1 - u \Rightarrow 100 - 20 = 80$ kN|

$\bar{\sigma}_3 = \sigma_3 - u \Rightarrow 50 - 20 = 30$ kN|

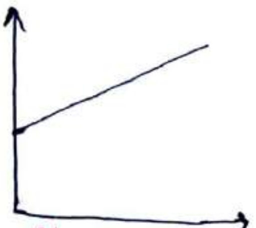
$$\sin\phi' = \frac{\bar{\sigma}_1 - \bar{\sigma}_3}{\bar{\sigma}_1 + \bar{\sigma}_3}$$

$$\sin\phi' = \frac{80-30}{80+30} = 27.03$$

$$\boxed{\phi' = 27.03}$$

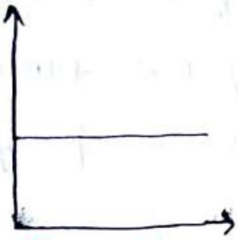
Ques Select the correct statement for shear strength in NCC

✓(a)



Shear strength in NCC

(b)



Pure clay

(c)

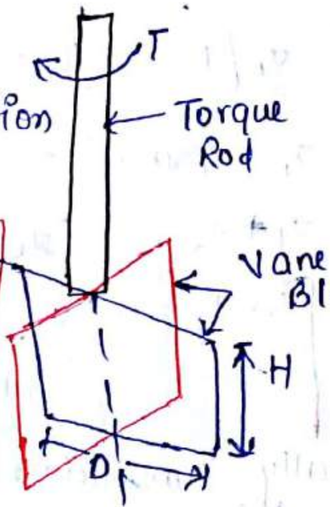
{NCC under drained condition}

(d) None

VANE SHEAR TEST

(i) This test is a field test.

(ii) This test is used for U-U test condition in clayey soil [soft clay]



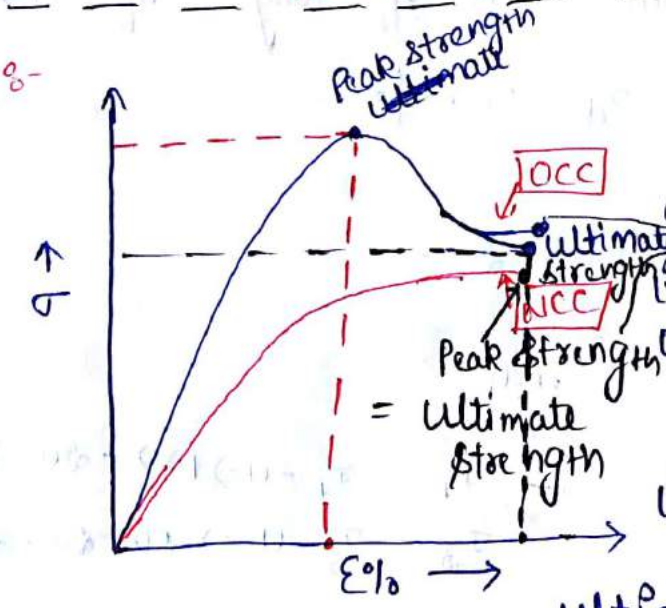
$$\boxed{s = \frac{T}{\pi D^2\left[\frac{H}{2}+\frac{D}{6}\right]}}$$

When (top + Bottom + Sides) of vanes are used to shear the soil

$$\boxed{s = \frac{T}{\pi D^2\left[\frac{H}{2}+\frac{D}{12}\right]}}$$

When (Bottom + side) of vanes are used for shear

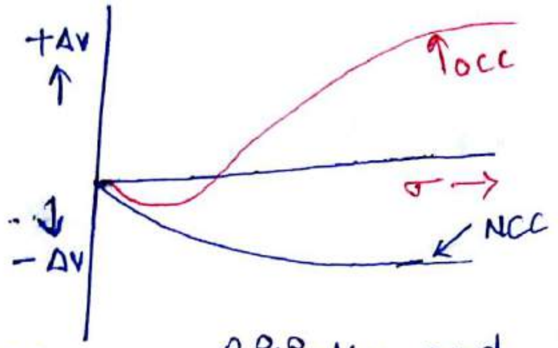
Note :-



(i) The ultimate strength and Peak strength of normally consolidated clay (NCC) are same.

(ii) The ultimate strength of OCC is ~~more~~ less than peak strength of OCC.

(iii) The ultimate strength of OCC is more than ultimate strength of NCC by very small manner. but peak strength of OCC is more than Peak strength of NCC in large Manner.



(i) In case of NCC the les in vol. of soil take place with increase in stress.

(ii) In case of OCC the stress increment reduces the volume initially and then increase. due to pre-consolidation pressure over soil.

Unit - IV

Earth Pressure theory

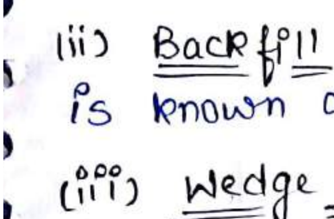
(i) Retaining wall :- The wall which is used to Retain some material is known as Retaining wall.

(ii) Backfill :- The material which is Retained by a Retaing wall is known as backfill.

(iii) Wedge failure :- The part of backfill which takes part in shear failure of wall is known as wedge and the shear failure in wedge is known as wedge failure

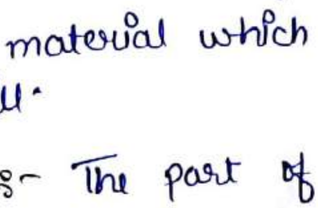
(iv) Theory of Plasticity / condition of Plastic equillibrium Acc. to this condition every point of soil mass is added verge of failure. There are three conditions of plastic equillibrium:

Active state



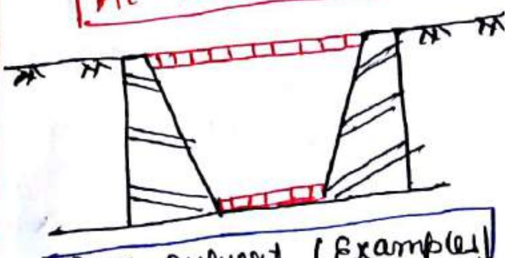
→ Movement of Retaining wall and movement of wedge take place away from backfill
→ pressure is known as active Pressure.
→ Boundary structure are design in this state.

Passive State



→ Movement of Retaining wall & wedge takes place toward Backfill.
→ Pressure is known as Passive Pressure.
→ No specific structure is Designed

At - Rest State



Box Culvert (Example)

→ There should not be any Movement b/w Retaining wall & back fills.
→ Pressure is known as at Rest Pressure.
→ Box Culverts, Bridge Abutments, Culverts are Design on this state.

8 Oct 2018

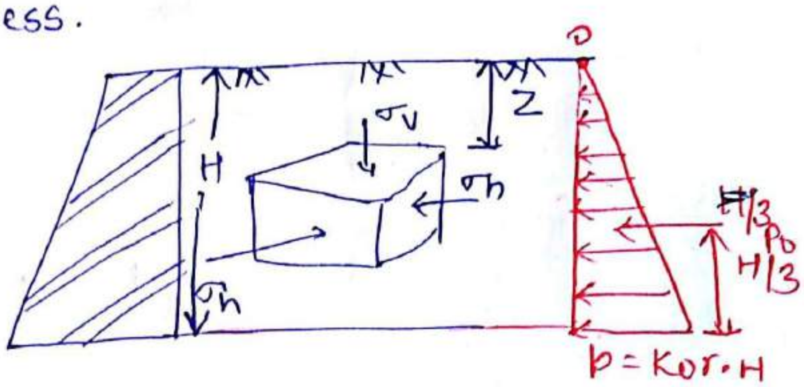
(1) At Rest - State

Assumption

1. Soil is Homogenous, iso-tropic, and semi - infinite.
- Modulus of elasticity of soil Remains constant.
- Soil is dry and cohesionless.

$$\varepsilon_x = \frac{\sigma_x}{E} - \mu\frac{\sigma_y}{E} - \mu\frac{\sigma_z}{E}$$

$$\varepsilon_h = \frac{\sigma_h}{E} - \mu\frac{\sigma_h}{E} - \mu\frac{\sigma_v}{E}$$



$\boxed{\text{At rest condition } \varepsilon_h = 0}$

$$0 = \frac{\sigma_h}{E} - \frac{\mu\sigma_h}{E} - \frac{\mu\sigma_v}{E} \Rightarrow 0 = \sigma_h\left[1-\mu\right] - \mu\sigma_v$$

$$\sigma_h(1-\mu) = \mu\sigma_v$$

$$\boxed{\frac{\sigma_h}{\sigma_v} = K_0 = \frac{\mu}{1-\mu}}$$

$$\frac{\sigma_h}{\sigma_v} = K_0 \Rightarrow \boxed{\sigma_h = K_0\sigma_v}$$

At Rest state $\boxed{\sigma_h = p_0}$

$p_0 = K_0\sigma_v$ K0 = co-efficient of earth Pressure at Rest

$\boxed{p_0 = K_0\gamma . z}$ $\boxed{\sigma = \gamma z}$

At $Z = 0$ $p_0 = 0$

at $Z = H$ $p = K_0\gamma H$

$\boxed{\begin{array}{l}\text{Total Pressure force} = \frac{1}{2}K_0\gamma H^2 \\ \text{Point of Application} = \frac{H}{3} \text{ from Base}\end{array}}$ ← Area of Triangle
← C.G of Triangle.

7) Co-efficient of Earth Pressure

$$k = \frac{\text{Horizontal Pressure}}{\text{vertical Pressure}}$$

At Rest $K_0 = \frac{\mu}{1-\mu} = 1 - \sin\phi$ μ = Poisson's Ratio

Active state $K_a = \frac{1-\sin\phi}{1+\sin\phi} = \tan^2\left(45° - \frac{\phi}{2}\right)$

Passive state

$$K_p = \frac{1+\sin\phi}{1-\sin\phi} = \tan^2\left(45° + \frac{\phi}{2}\right)$$

$\boxed{K_a \cdot K_p = 1}$ $\boxed{K_p > K_0 > K_a}$

Q If angle of internal friction of a soil is 30°. Calculate Rati of co-efficient of earth Pressure in active state to Passive sta

Ans $\frac{K_a}{K_p} = \frac{\tan^2\left[45° - \frac{30}{2}\right]}{\tan^2\left[45° + \frac{30}{2}\right]} = \frac{1}{9}$

$$K_a = \frac{1-\sin 30}{1+\sin 30} = \frac{1-\frac{1}{2}}{1+\frac{1}{2}} = \frac{\frac{1}{2}}{3/2} = \frac{1}{3}$$

$$K_p = \frac{1+\sin 30}{1-\sin 30} = \frac{1+\frac{1}{2}}{1-\frac{1}{2}} = \frac{3/2}{1/2} = 3$$

$$\boxed{\frac{K_a}{K_p} = \frac{1/3}{3} = \frac{1}{9}}$$

Q If co-efficient of earth Pressure at Rest is 0.5. calcula co-efficient of earth Pressure in Passive state.

Ans ~~$K_a \cdot K_p = 1$~~ $K_p = k - \frac{k}{2} = \frac{1}{2}$

$K_0 = 1 - \sin\phi = \frac{1}{2}$ $\sin\phi = \frac{1}{2}$

$$K_p = \frac{1+\sin\phi}{1-\sin\phi} = \frac{1+\frac{1}{2}}{1-\frac{1}{2}} = \frac{3/2}{1/2}$$

$\boxed{K_p = 3}$

Rankine's Theory

Assumption

1. Soil is Isotropic, Homogeneous, and semi- infinite.
2. Soil is Dry and cohesionless.
3. Back of the wall is smooth and vertical.
4. Backfill is Horizontal or inclined at its top surface.
5. Every point of soil Mass is at the verge of failure.

At Rest	Active ← Rankine's theory Cohesionless	→ Passive
1. $K_0 = \frac{\mu}{1-\mu} = 1-\sin\phi$	1. $K_a = \frac{1-\sin\phi}{1+\sin\phi}$	1. $K_p = \frac{1+\sin\phi}{1-\sin\phi}$
2. $p_0 = K_0 \sigma_v$ $\boxed{p_0 = K_0 \gamma z}$	2.) $p_a = K_a \sigma_v$ $\boxed{p_a = K_a \gamma . z}$	2. $p_p = K_p \sigma_v$ $\boxed{p_p = K_p . \gamma . z}$
3) At $\boxed{z=0}$ $\boxed{p_0 = 0}$ $z = H \Rightarrow \boxed{p_0 = K_0 . \gamma . H}$	3) $z=0$ $\boxed{p_a = 0}$ $z=H$ $\boxed{K_a \gamma H}$	3) At $z=0$ $\boxed{p_p = 0}$ $z=H$ $\boxed{K_p \gamma H}$
4). Pressure force $P_0 = \frac{1}{2} K_0 . \gamma . H^2$	4) Pressure force $P_a = \frac{1}{2} K_a . \gamma . H^2$	4) Pressure force $P_p = \frac{1}{2} K_p \gamma . H^2$
Application H/3 from Base	Application H/3 from Base	Application H/3 from Base

Ques Calculate total earth Pressure force at Rest condition. for a Retaining wall having 10m Height. The wall is subjected to Retain backfill having unit wt. 20 KN/m3. Shear Strength Parameter are 30°, 0.

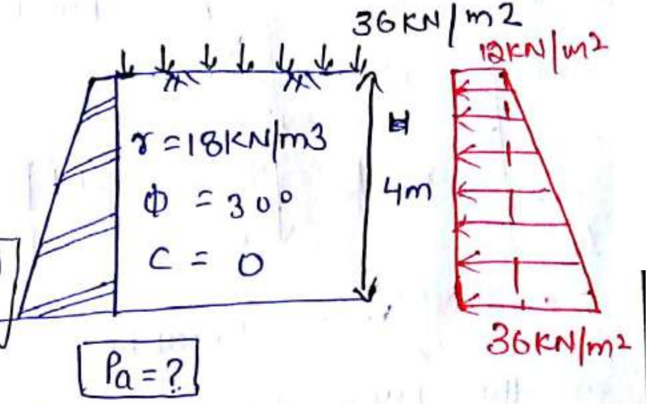
Sol $H = 0 \quad \gamma = 20 KN/m3 \; ; \; \phi = 30°$

$$P_0 = \frac{1}{2} K_0 \gamma H^2 \Rightarrow K_0 = 1 - \sin\phi = 1 - \sin 30 = 1 - \frac{1}{2} = \frac{1}{2}$$

$\boxed{K_0 = 1/2}$ $\quad P_0 = \frac{1}{2} \times \frac{1}{2} \times 20 \times 10^2$

$\boxed{P_0 = 500 \ KN/m}$ Ans

Ques calculate total Rankine active pressure force behind a Retaining wall as given in Diagram. Also Determine point of Application of the force.



Sol $Ka = \frac{1-\sin 30}{1+\sin 30} = \frac{1}{3}$ $\boxed{Ka = \frac{1}{3}}$

$pa = Ka\,\sigma_v$

$pa = ka\,(\gamma z + q_0)$ $\boxed{q_0 = \text{Surcharge load} = 36KN/m^2}$

At $\boxed{Z=0}$ $pa = \frac{1}{3}\left[0+36\right] = 12\ KN/m^2$

At $\boxed{Z=4m}$ $Pa = \frac{1}{3}\left[18\times4+36\right] = 36\ KN/m^2$

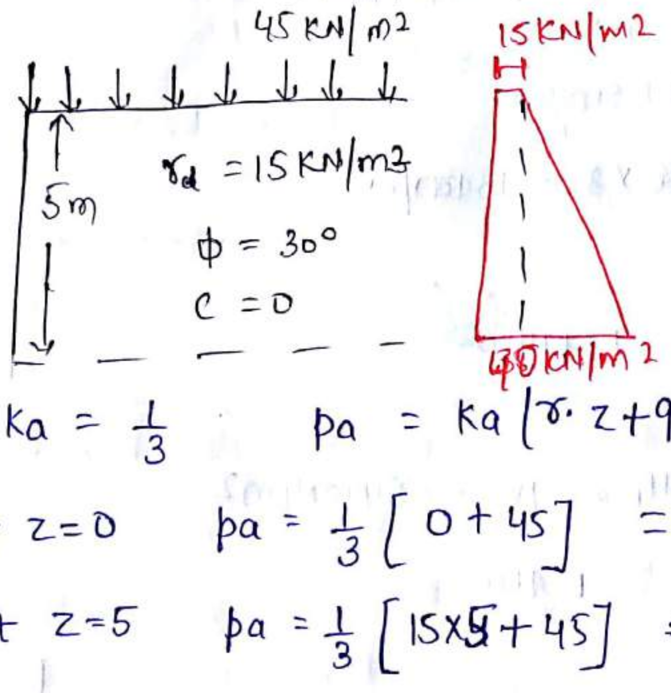
$Pa = 12\times4 + \frac{1}{2}\left[36-12\right]\times4$

$Pa = 48+48 = \boxed{96\ KN/m}$ Ans.

Point of Application

$$\bar{y} = \frac{48\times\frac{4}{2} + 48\times\frac{4}{3}}{48+48} = 1.67m\ \text{[from Base]}\ \text{Ans}$$

Ques



$Ka = \frac{1}{3}$ $pa = Ka\left[\gamma\cdot z + q_0\right]$

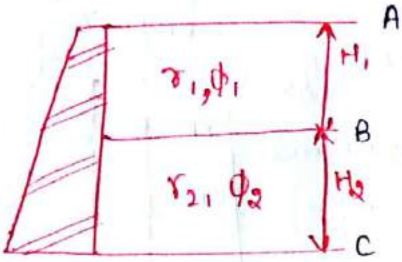
At $z=0$ $pa = \frac{1}{3}\left[0+45\right] = 15\ KN/m^2$

At $z=5$ $pa = \frac{1}{3}\left[15\times5+45\right] = \frac{120}{3} = 40\ KN/m^2$

$Pa = 15\times5 + \frac{1}{2}\times25\times5 = \overset{75+62.5}{75+50} = \overset{137.5}{125}\ KN/m$

$$\bar{y} = \frac{75\times\frac{5}{2} + 62.50\times\frac{5}{3}}{137.5} \Rightarrow \frac{187.5+104.167}{137.5} = \boxed{2.12}$$

Effect of Diff. type of Backfill



layer AB $\quad Ka_1 = \dfrac{1-\sin\phi_1}{1+\sin\phi_1}$

$pa_1 = Ka_1 \sigma_v = Ka_1 \gamma_1 z$

At $\boxed{z=0}$ $\quad pa_1 = 0$

$\boxed{z=H_1}$ $\quad pa_1 = Ka_1 \gamma_1 H_1$

layer BC :- Layer AB will act as a surcharge load on the layer BC

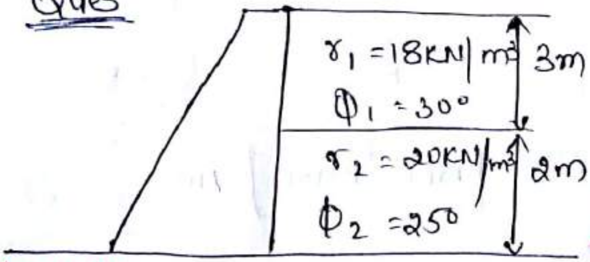
Surcharge due to 1st layer = $\boxed{q_0 = \gamma_1 H_1}$

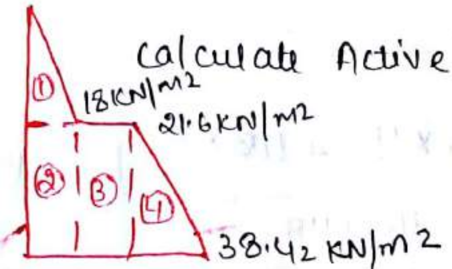
$pa_2 = Ka_2 \sigma_v = Ka_2 [\gamma_2 \cdot z + q_0]$

At $\boxed{z=0}$ $\quad pa_2 = Ka_2\, q_0$

At $\boxed{z=H}$ $\quad pa_2 = Ka_2 [\gamma_2 H_2 + q_0]$

Ques





calculate Active Pressure force.

layer AB $\quad Ka_1 = \dfrac{1-\sin\phi}{\cancel{3}+\sin\phi} = \dfrac{1-\sin 30}{1+\sin 30} = \dfrac{1}{3}$

$pa_1 = \cancel{\frac{1}{3}}\, Ka_1 \overset{(\gamma_1 z)}{\sigma_v} = \frac{1}{3} \times \cancel{18} \times \cancel{3} = 18\, \cancel{KN}/m$

At $z=0$ $\quad \boxed{pa_1 = 0}$

At $z=3$ $\quad pa_1 = \frac{1}{3} \times 18 \times 3 = 18\, KN/m^2$

layer BC

Surcharge, $q_0 = \gamma_1 H_1 = 18 \times 3 = 54\, KN/m^2$

$Ka_2 = \dfrac{1-\sin 25}{1+\sin 25} = 0.406$

$pa_2 = Ka_2 [\gamma_2 z + q_0]$

$\boxed{\text{At } z=0}$ $\quad pa_2 = 0.406\, [0 + 54] = 21.6\, KN/m2$

At $z=2$ $\quad pa_2 = 0.406\, [(20 \times 2) + 54] = 38.42\, KN/m^2$

Rankine's Theory for cohesive Soil

$$\sigma_1 = \sigma_3\left[\frac{1+\sin\phi}{1-\sin\phi}\right] + 2c\frac{\cos\phi}{1-\sin\phi}$$

$$\sigma_1 = \sigma_v \;;\; \sigma_3 = pa$$

$$\sigma_v = pa\left[\frac{1+\sin\phi}{1-\sin\phi}\right] + 2c\left[\frac{\cos\phi}{1-\sin\phi}\right]$$

$$pa\left[\frac{1+\sin\phi}{1-\sin\phi}\right] = \sigma_v - 2c\left[\frac{\cos\phi}{1-\sin\phi}\right]$$

$$pa = \underbrace{\sigma_v\left[\frac{1-\sin\phi}{1+\sin\phi}\right]}_{\tan^2(45-\frac{\phi}{2}) = Ka} - \underbrace{2c\left[\frac{\cos\phi}{1+\sin\phi}\right]}_{\tan[45-\frac{\phi}{2}] = \sqrt{Ka}}$$

$$\boxed{pa = Ka\,\sigma_v - 2c\sqrt{Ka}}$$

$$\boxed{pa = Ka\cdot\gamma\cdot Z = 2c\sqrt{Ka}}$$

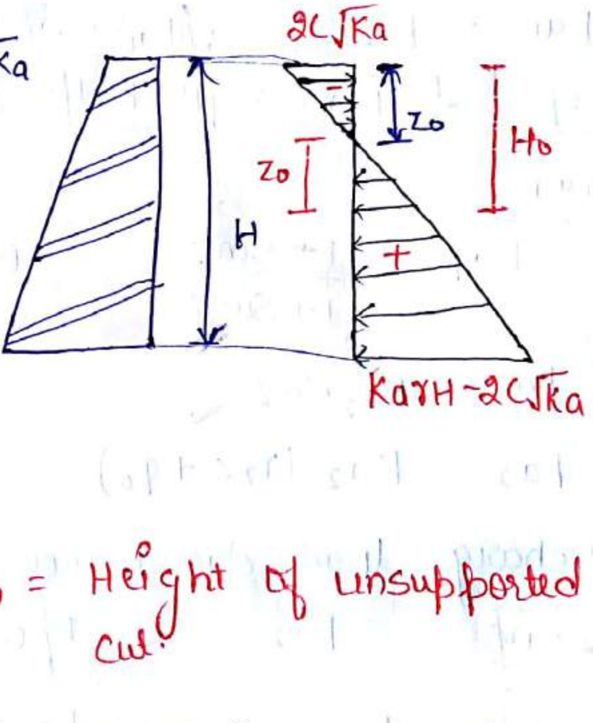
At $Z=0 \Rightarrow pa = -2c\sqrt{Ka}$

At $Z=H \Rightarrow pa = Ka\cdot\gamma\cdot H - 2c\sqrt{Ka}$

when $pa = 0 \;;\; Z = Z_0$

$\rightarrow 0 = Ka\gamma . Z_0 - 2c\sqrt{Ka}$

$Ka\gamma\cdot Z_0 = 2c\sqrt{Ka}$



Depth of Tensile Crack → $\boxed{Z_0 = \frac{2c}{\gamma\sqrt{Ka}}}$

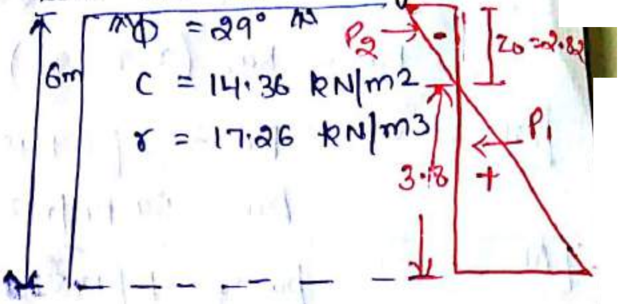
$\boxed{H_0 = 2Z_0 = \frac{4c}{\gamma\cdot\sqrt{Ka}}}$ H_0 = Height of unsupported vertic cut.

Ques 3- Calculate depth of tensile crack for a Retaining wall subjected to backfill having unit wt = γ = 20 KN/m3 and cohesion = 15 KN/m2. $\phi = 0$ (if not given)

$$Ka = \frac{1-\sin 0}{1+\sin 0} = 1$$

$$Z_0 = \frac{2c}{\gamma\sqrt{Ka}} = \frac{2\times 15}{20\times\sqrt{1}} = 1.5\text{ m} \quad \boxed{Z_0 = 1.5\text{m}}$$

Q A 6m Ht. Retaining wall is subjected to backfill having unit wt. 17.26 KN/m3. The strength parameters of soil are 29° and 14.36 kN/m². Calculate depth of tensile crack and the active pressure force exerted by soil on Retaining wall.



$$Ka = \frac{1 - \sin 29}{1 + \sin 29} = 0.34$$

$$z_0 = \frac{2 \times 14.36}{17.26\sqrt{0.34}} = 2.85m.$$

$$pa = Ka \cdot \gamma \cdot z - 2C\sqrt{Ka}$$

$\boxed{\text{At } z=0}$

$$pa = -2C\sqrt{Ka} \Rightarrow -2 \times 14.36\sqrt{0.34}.$$

$$pa = 16.74 \text{ kN/m2}$$

$\boxed{\text{At } z=6}$

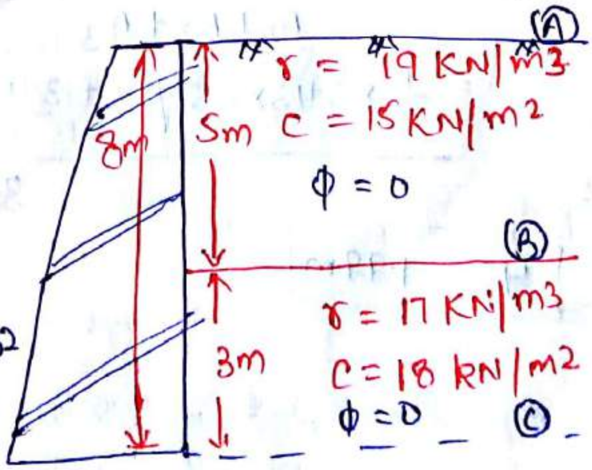
$$pa = 0.34 \times 17.26 \times 6 - 2 \times 14.36\sqrt{0.34}$$

$$pa = 18.92 \text{ kN/m2}$$

$$Pa = \frac{1}{2} \times 18.92 \times 3.18 = 30.11 \text{ kN/m}$$

$$\boxed{Pa = 30.11 \text{ kN/m}}$$

Ques 8m Height Retaining wall is subjected to Retain two backfills upto 5m Height from top Retains backfill having unit wt. 19 KN/m3 and cohesion of 15 kN/m2. Bottom 3m Ht. Retains backfill having unit wt. 17 kN/m3 and cohesion 18 kN/m2. Calculate the total active pressure force exerted on a Retaining wall by its backfill.



Sol: Layer AB

$$Ka_1 = \frac{1 - \sin 0°}{1 + \sin 0°} = 1$$

$$pa = Ka_1 \gamma_1 z - 2C_1\sqrt{Ka_1}$$

At $z=0$ $pa = 0 - 2 \times 15\sqrt{1} = -30$ kN/m2 [level A]

At $= z=5$ $pa = 1 \times 19 \times 5 - 2 \times 15 \times \sqrt{1}$

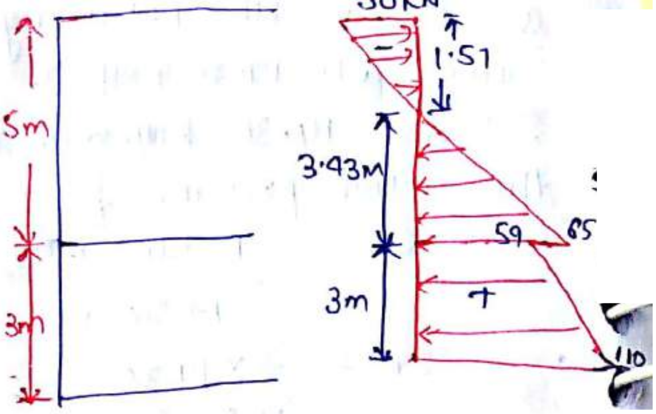
$$\boxed{pa = 65 \text{ kN/m2}}$$

$\boxed{\text{Layer BC}}$ $K_{a_2} = \dfrac{1-\sin 0}{1+\sin 0} = 1$

Surcharge $\cdot q_0 = 19 \times 5 = 95$ kN/m^2

$$p_{a_2} = K_{a_2}\sigma_v - 2C_2\sqrt{K_{a_2}}$$

$$p_a = K_{a_2}(\gamma_2 z + q_0) - 2C_2\sqrt{K_{a_2}}$$



$\boxed{\text{At } z=0}$ $p_a = 1\left[0+45\right] - 2 \times 18\sqrt{1}$

$p_a = 59$ kN/m^2

$\boxed{\text{At } z=3}$ $p_a = 1\left[17\times 3 + 95\right] - 2\times 18\sqrt{1}$

$p_a = 110$ kN/m^2

$$z_0 = \frac{2C}{\gamma\sqrt{K_a}}$$

$$z_0 = \frac{2\times 15}{19\sqrt{1}} = \boxed{1{\cdot}57\text{ m}}$$

$$P_a = \frac{1}{2}\times 3{\cdot}43\times 65 + 59\times 3 + \frac{1}{2}\times 3\times(110-59)$$

$$\boxed{P_a = \overset{364\cdot97}{310}\text{ kN/m}}$$

Resultant force $= 364{\cdot}97 - \frac{1}{2}\times 1{\cdot}57\times 30$

$= 364{\cdot}97 - 23{\cdot}55 = 341{\cdot}42$ kN/m

Point of Application

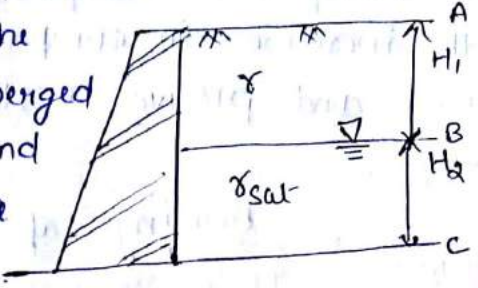
$$\bar{y} = \frac{A_1y_1 + A_2y_2 + A_3y_3}{A_1+A_2+A_3} =$$

$$= \frac{\frac{1}{2}\times 3{\cdot}43\times 65\left[3 + \frac{3{\cdot}43}{3}\right] + (59\times 3)\left(\frac{3}{2}\right) + \frac{1}{2}\left[110-59\right]\times 3\left[\frac{3}{3}\right.}{364{\cdot}97}$$

$$\boxed{\bar{y} = 1{\cdot}99\text{ m}}$$

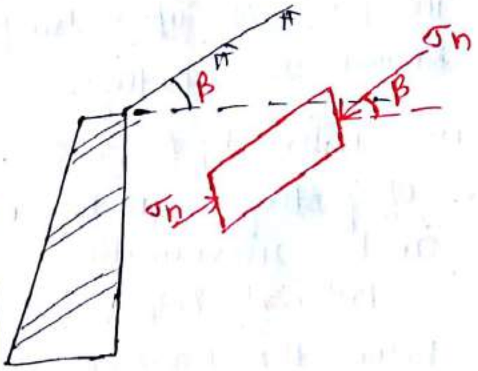
// Effect of water Table

When water Table is considered, the earth pressure depends upon submerged unit wt. of soil below W.T and and the effect of water doesnot relate to co-efficient of earth pressure.



$$\boxed{p_a = K_a \cdot \gamma H_1 + K_a \cdot \gamma' H_2 + \gamma_w H_2}$$

Effect of inclined backfill



Active State

$$\boxed{P_a = \frac{1}{2} K_a \gamma \cdot H^2 \cos\beta}$$

$$\boxed{K_a = \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos 2\beta \mp \cos 2\phi}}}$$

Passive State

$$\boxed{K_p = \frac{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}}$$

$$\boxed{P_p = \frac{1}{2} K_p \cdot \gamma H^2 \cdot \cos\beta}$$

Angle of Repose :- It is the maxm angle upto which a heap of soil is completly stable. There isno lateral Movement o soil particles under gravity from the heap.

Coulomb's theory

Acc. to culomb's theory the active pressure and passive pressure behind a Retaining wall can be considered with given assumptions:-

(i) Back of the Retaining wall is Rough and inclined.

(ii) A co-efficient of friction exist b/w Retaining wall and soil particles (In terms of angle of internal friction)

(iii) Backfill is completly inclined.

(iii) The backfill is completely inclined.

(iv) With increase in compaction in soil active earth pressure decreases and passive earth pressure increases.

10 oct 2018

Bearing capacity of soil

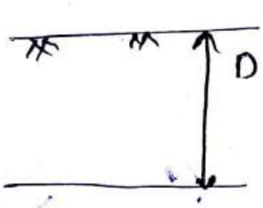
(1) **Foundation** :- It is the part of structure which Remains in direct Contact with soil / Ground Surface.

(2) **Footing** :- The part of Sub-structure which transmit load to the Sub soil is known as footing.

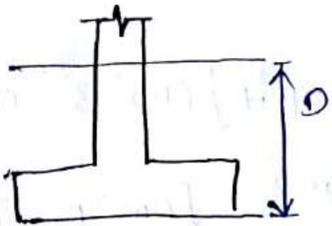
(3) **Gross Pressure Intensity (GPI)** :- It is the pressure intensity generated at the base of footing due to wt. of super-structure, sub-structure and surrounding soil.

(4) **Net Pressure Intensity (NPI)** :- The Net increase in intensity of Pressure below the base of footing after construction.

It is the numerical difference b/w pressure intensities after construction and before construction at a given level.



Before construction



after construction

$$\boxed{q_n = q_0 - \gamma D}$$

q_0 = gross pressure intensity [after Constructi

(5) **Ultimate Bearing Capacity** :- (q_u) The Maxm gross-pressure Intensity that a soil can carry just before shear failure.

(6) **Net Ultimate bearing capacity (q_{nu})** :- The Maxm Net pressure intensity that a soil can carry just before shear failure.

$$\boxed{q_{nu} = q_u - \gamma \cdot D}$$

(7) **Net Safe Bearing capacity (q_{ns})** :- The Maxm Net pressure intensity at which there is no Risk of shear failure in soil is known as Net safe bearing capacity.

$$\boxed{q_{ns} = \frac{q_{nu}}{FOS}} \qquad \boxed{F.O.S = 2 \text{ to } 3}$$

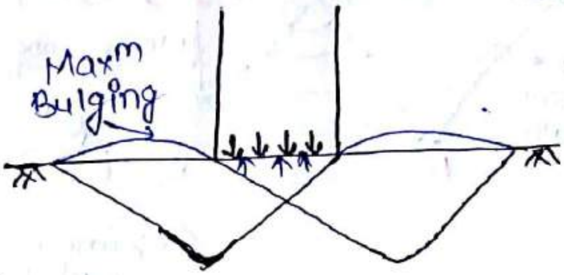
(8) Safe Bearing capacity (q_s):- The max^m gross pressure intensity at which there is no Risk of shear failure in soil.

$$\boxed{q_s = q_{ns} + \gamma D}$$

- All the foundations are design on the basis of safe bearing capacity

★ Types of shear failure

1 General Shear failure



- low compressible
- Dense sand, silt

→ $\phi > 36°$

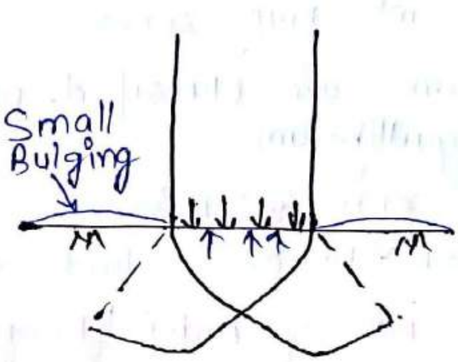
$I_D > 70\%$

→ Max^m tilting of footing exists.

2 Local Shear failure :-

exists in

- High compressible soil
- loose soil Deposits
- $\phi \leq 29°$
- $30\% \leq I_D \leq 70\%$



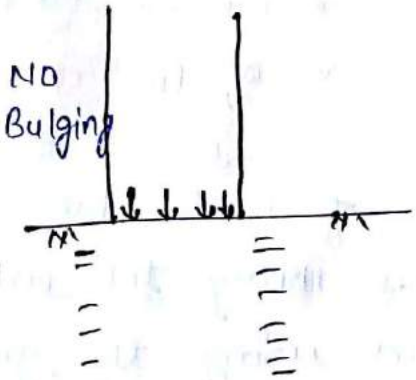
3 Punching shear failure :-

- exist in

→ soil having $I_D \leq 30\%$

→ No tilting of footing exists

→ In this failure the behavoir of structure is considered and to Resist this failure thickness of footing is to be designed.



Terzaghi's Theory of Bearing capacity

Assumption

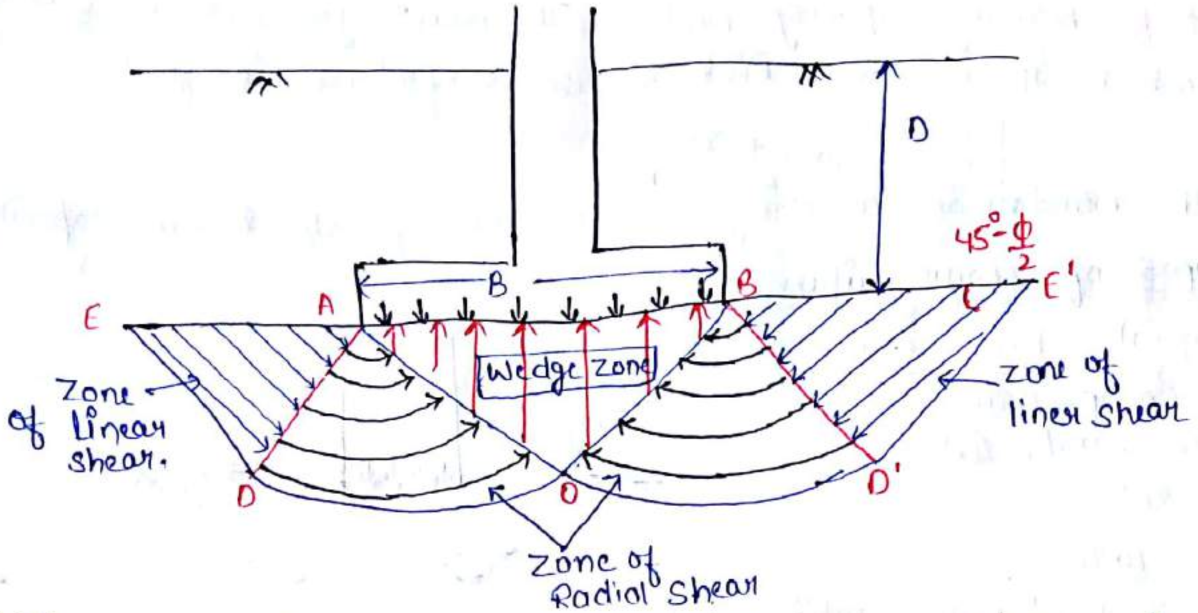
(i) Soil is Homogenous, isotropic and semi-infinite.

(ii) General shear failure in soil take place.

★★(iii) footing is (strip footing).

(iv) Base of the footing is Rough.

(v) Every Point of soil Mass is at the verge of shear failure



(i) In a strip footing due to general shear failure soil is divided into five zones

(ii) These zone are classified on the basis of Passive state of Plastic equillibrium

(a) **Wedge zone** :- This zone lies just below the footing which is subjected to Maxm load and Maxm settlements.

(b) **Pair of zone of Radial shear** :- These zones lies at either sides of zone 1 (wedge zone)

(ii) The load transmission takes place by Radial Direction.

(iii) **Pair of zone of liner shear** :- These zones lies at either sides of Radial shear zone.

- The transfer of load takes place by an inclination angle $\left[45° - \frac{\phi}{2}\right]$

* Acc to this theory the ultimate bearing capacity of soil can be calculated using the given equation :-

$$\boxed{q_u = C\cdot N_c + q\cdot N_q + \frac{1}{2}\gamma\cdot B\cdot N_\gamma}$$

where,

C = cohesion

q = effective stress $[=\bar{\sigma}]$ $\boxed{= \gamma D}$

B = width of footing

γ = unit wt of soil

N_c, N_q, N_γ = capacity factors [Depends upon Angle of internal friction]

~~EDep~~

$$q_{nu} = q_u - \gamma . D$$

$$q_{nu} = C.N_c + \underbrace{q}_{\gamma D}.N_q + \frac{1}{2} B.\gamma.N_\gamma - \gamma D$$

$$q_{nu} = C.N_c + \gamma D\left[N_q - 1\right] + \frac{1}{2} B.\gamma.N_\gamma$$ -(Gamo)

$$\boxed{q_{ns} = \frac{q_{nu}}{FOS} + \gamma D}$$

Pure clay ($\phi = 0$)

Rough Base	Smooth Base
$N_c = 5.7$	$N_c = 5.14$
$N_q = 1$	$N_q = 1$
$N_\gamma = 0$	$N_\gamma = 0$
$\boxed{q_u = 5.7C + q}$	$\boxed{q_u = 5.14C + q}$

Q Calculate Net ultimate Bearing capacity of Pure clay having cohesion 20 KN/m2 and unit wt. 19 KN/m3 with 1m depth.

$$q_{nu} = q_u - \gamma D$$

$$= 5.7C + \cancel{\underbrace{q}_{\gamma D} - \gamma D}$$

$$q_{nu} = 5.7 \times 20 = \boxed{114\ KN/m^2}$$ ~~m2~~ Ans

Q A strip footing having width 2m lies 1.5m below ground surface. Calculate the net ultimate, net safe and safe bearing capacity of soil having unit wt. 18.5 KN/m2. C = 20 KN/m2 $N_c = 60$, $N_q = 42$ and $N_\gamma = 47$. F.O.S = 3

$D = 1.5m$, $B = 2m$, $\gamma = 18.5\ KN/m^2$, $C = 20\ KN/m^2$

α

$q_u = 5.7C + q = 5.7 \times 20 + 18.5 \times 1.5$

$q_u = 141.75\ KN/m^2$

$q_{nu} = 141.75 - \gamma D \Rightarrow 141.75 - 18.5 \times 1.5$

$q_{nu} = 114\ KN/m^2$

$$q_u = C.N_c + \underbrace{q}_{\gamma D}.N_q + \frac{1}{2}\gamma.B.N_\gamma$$

$$= 20 \times 60 + 18.5 \times 1.5 \times 42 + \frac{1}{2} \times 18.5 \times 2 \times 47$$

$$q_u = 3235\ KN/m^2$$

$q_{nu} = q_u - \gamma D$

★ $\boxed{q_{nu}} = 3235 - 18.5 \times 1.5 = \boxed{3207.25 \text{ kN/m2}}$

★ $\boxed{q_{ns}} = \frac{q_{nu}}{fos} = \frac{3207.25}{3} = \boxed{1069.08 \text{ kN/m2}}$

★ $\boxed{q_s} = q_{ns} + \gamma D = 1069 + 18.5 \times 1.5 = \boxed{1096.8 \text{ kN/m2}}$

(1) Strip footing

$$\boxed{q_u = C \cdot N_c + q \cdot N_q + \frac{1}{2} B \cdot \gamma N_\gamma}$$

(2) Square footing

$$\boxed{q_u = 1.3 C \cdot N_c + q \cdot N_q + 0.4 B \cdot \gamma \cdot N_\gamma}$$ B = side of footing

(3) Circular footing

$$\boxed{q_u = 1.3 C N_c + q \cdot N_q + 0.3 B \cdot \gamma \cdot N_\gamma}$$

B = Dia of footing

(4) Rectangular footing

$$\boxed{q_u = C N_c \left[1 + 0.3 \frac{B}{L}\right] + q \cdot N_q + \frac{1}{2} B \cdot \gamma \cdot N_\gamma \left[1 - 0.2 \frac{B}{L}\right]}$$

B = width of footing
L = Length of footin

Ques A square footing 2.2m x 2.2m in size lies at depth of 1.6m below ground surface. Calculate the safe load that a footing can carry without any risk of shear failure. Consider the following properties of soil.

$\gamma = 16.5$ kN/m3 , $C = 7.3$ kN/m2

$N_c = 11.8$, $N_q = 3.8$, $N_\gamma = 1.3$

$q_u = 1.3 C \cdot N_c + \underbrace{q}_{\gamma D} \cdot N_q + 0.4 B \cdot \gamma \cdot N_\gamma$

$q_u = 1.3 \times 7.3 \times 11.8 + 16.5 \times 1.6 \times 3.8 + 0.4 \times 2.2 \times 16.5 \times$

$$\boxed{q_u = 231.78 \text{ kN/m2}}$$

$q_{nu} = q_u - \gamma D$
$= 231 - 16 \times 1.6$

$$\boxed{q_{nu} = 205.4 \text{ kN/m2}}$$

$\boxed{q_{ns}} = \frac{q_{nu}}{f.o.s} = \frac{205.4}{2.5} = \boxed{82.16 \text{ kN/m2}}$

Increase load = 0.07 kg/cm²
upto 1 kg/cm²

(2) Test Pit

- Depth of Test Pit = Depth of foundation
- Width of Test Pit = 5 × widen of Plate

(3) Loading Arrangements

(i) The load is applied to the Base plate using graviometric Method or Reaction truss Method.

(ii) The initial load applied to the plate is 0.07 kg/cm² and the load is increased at constant Rate upto 1 kg/cm² or failure of Plate whichever is earlier.

(iii) The loading point at which failure in plate exists Represents Bearing capacity of Plate.

limitations of Plate load Test

(i) This test is a short duration test

(ii) Effect of size of foundation is not considered

(iii) The properties of surrounding soil are not considered for calculation of Bearing capacity.

(iv) ## Bearing Capacity of foundation

$$\boxed{\frac{q_F}{q_P} = \frac{B_F}{B_P}}$$

Settlement of foundation

★ $$\boxed{\frac{S_F}{S_P} = \left[\frac{B_F\ (B_P+0.3)}{B_P\ (B_F+0.3)}\right]^2}$$ $\boxed{B_F, B_P, \text{ in m}}$

★ $$\boxed{\frac{S_F}{S_P} = \left[\frac{B_F\ (B_P+30)}{B_P\ (B_F+30}\right]^2}$$ $\boxed{B_F, B_P \text{ in cm}}$

(2) Penetration Test :-

(i) Standard Penetration Test → Cohesionless Soil

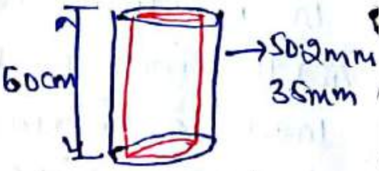
(ii) Static Cone Test → Cohesive soil

Used to :- (i) Bearing capacity
(ii) Relative Density (I_D)
(iii) Consistency
(iv) Shear Strength

1. **Standard Penetration Test** :- This test is used to Measure Bearing capacity of soil, Relative Density of Deposit, consistency and unconfined shear strength of soil

(ii) This test is used for cohesionless soils.

(iii) In this test, split spoon sampler is used. Outer Dia=50.2mm, inner Diameter = 35mm, Minm open length = 60 cm



(iv) The load applied over the sampler is 63.5Kg with free drop of 75cm.

(v) The sample is allowed to Penetrate by 45 cm in 3 equal parts.

(a) first 15 cm Penetration doesnot represent any penetration Resistance.

(b) Next 30cm Penetration are considered to measure total penetration Resistance which can be used to Measure the properties of soil

Correction for Dilatency =

i) Dilatency Represents the Property of soil Related to shear strength due to change of Moisture w.r.t application and Removal of load

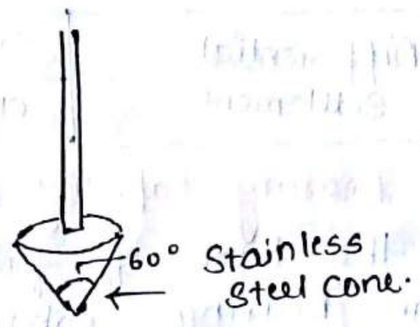
(ii) The correction for Dilatency is applicable when Penetration Resistance is more than 15.

$\boxed{N > 15}$ $\boxed{N_c = 15 + \frac{1}{2}(N-15)}$ Nc = Corrected Penetration Resistance

Corrected Penetration Resistance	Soil Nature	Unconfined Compressive strength (kN/m²)
0-2 →	very soft →	<25
2-4 →	soft →	25-50
4-8 →	Medium →	50-100
8-15 →	Stiff →	100-200
15-30 →	very stiff →	200-400
Above-30 →	Hard →	>400

Corrected Penetration Resistance	φ	I_D (%)	
< 4	→ 25°-30°	→ 0-15	→ very loose
4 - 10	→ 27°-32°	→ 15-65	→ loose
10 - 30	→ 30°-35°	→ 65-75	→ Medium
30 - 50	→ 35° - 40°	→ 75-100	→ Dense
> 50	→ 38°-43°	→ > 100	→ very Dense

4 Static Cone Test :- (SCT)



→ vertex angle = 60°

→ load applied = 63.5 kg

→ free Drop = 75 cm

→ Total Penetration = 20 cm

→ Intial Penetration = 8 cm

→ final Penetration = 12 cm

* Penetration Resistances are not considered

12 Oct 2018

* Requirement of Good foundation

1 Depth Criteria :- The depth of foundation must be such that the external loads foundation must be able to transmit all the external loads to the sub-soil and there shouldnot be any effect on behavio of footing due to seasonal vol. change of soil.

for Pure clay :-

→ The Minm depth of footing should be 0.9 m.

for cohesionless soil :- The Minm depth of footing should be

$$D_{min} = \frac{q}{\gamma}\left[\frac{1-\sin\phi}{1+\sin\phi}\right]^2$$ [Rankine's formula]

q = Bearing capacity of soil

γ = Unit wt. of soil

φ = Angle of int. friction

2. Settlement criteria :- The settlement in any foundation due to external load must be in permissible limits.

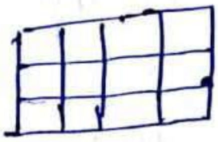
- This criteria is mainly considered in case of cohesionless soil

Permissible limits for settlement

Isolated footings	Sand	65mm
	clay	40mm
Raft footing	sand	65-100mm
	Clay	40-65mm
Differential settlement	Sand	25 mm
	clay	40mm

Differential Settlement :-

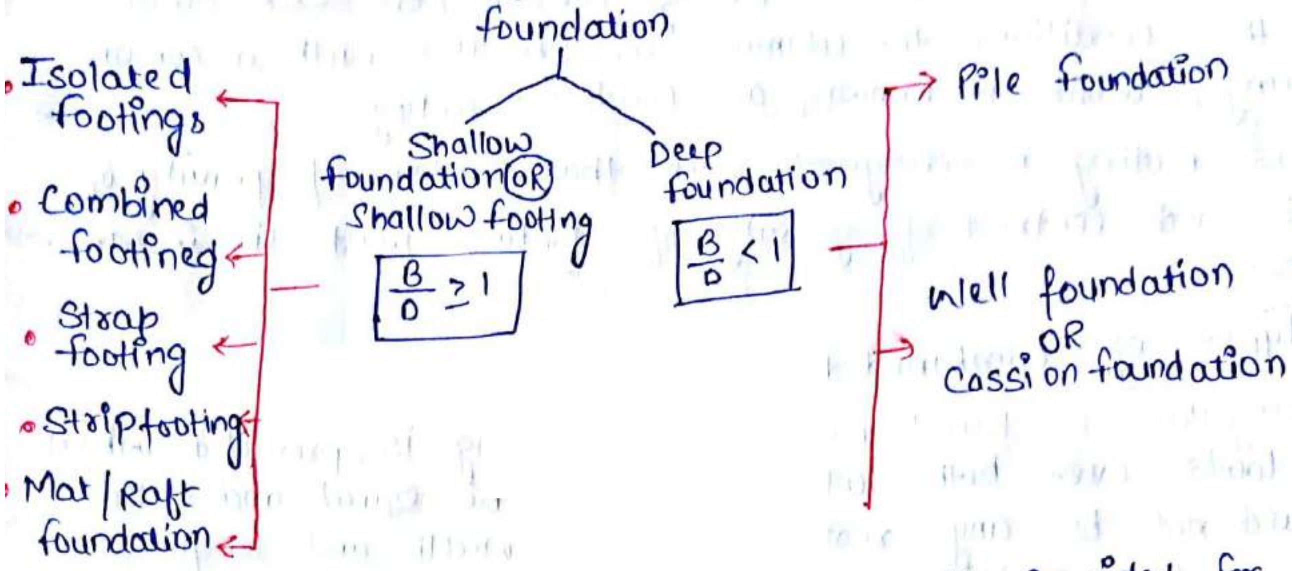
* Unequal amount of Settlement



3. Bearing capacity & shear strength criteria :- The stress develop at the base of footing due to external load must be less than bearing capacity of soil.

- This criteria is considered in case of Pure clay.

Types of foundation

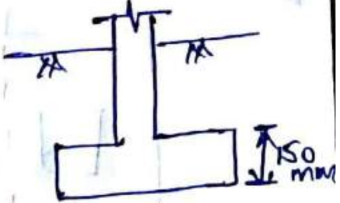


1). **Isolated footings** :- The footings which are provided for individual columns are known as isolated footings.

Types

(a) **Concrete Pedestal footing** :- (i) This footing is provided under columns subjected to very small External load.

(ii) In this footing, concrete is only sufficient to carry External loads but Min^{m} Reinforcement is always provided to convert brittle failure into ductile failure.



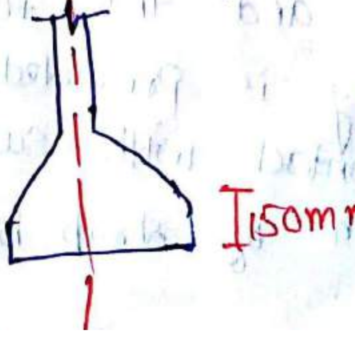
(iii) The min^{m} depth of footing for any case should be 150mm

(b) **Stepped footing** :- It is a type of spread footing in which Max^{m} thickness is provided at centre and min^{m} thickness is provided at outer ends in the form of no. of steps. This footing is ~~less~~ more economical but the formwork of ~~wor~~ footing is difficult.

(c) **Sloped footing** :- This footing is provided in the form of a slope b/w max^{m} depth at centre and Min^{m} depth at outer ends.

- The formwork of this footing is simple and easy.

Stepped footing.



Sloped footing

2 **Combined footing** :- When c/c distance b/w column is so small, such that their isolated footings overlap with each other.

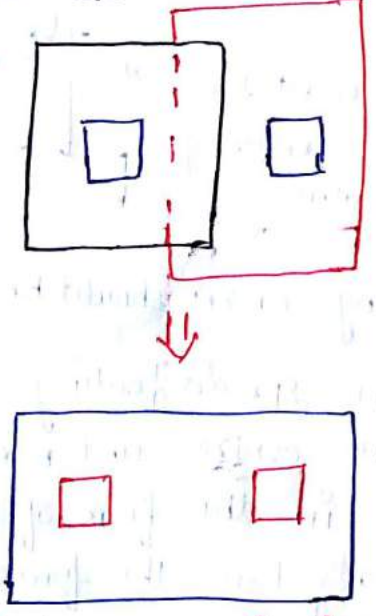
- At this condition, the columns are provided with a comon footing, which is known as combined footing.
- This footing is designed such that centre of gravity of loads and centre of gravity of footing must lie in same plane.

Types of combined footing

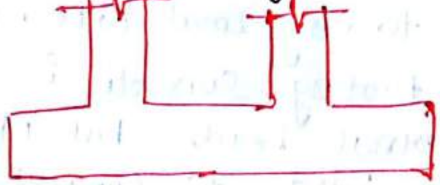
1 **Rectangular combined footing** :- This footing is provided when the loads over both columns should be equal and there should not be any restriction b/w width and length.

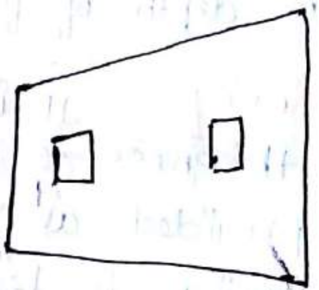
2 **Trapezoidal combined footing** :- This footing is provided when load over columns are unequal or length of footing and one width of footing is Restricted / limited.

★ when c/c distance b/w two foundations/footing is less



Rectangular footing



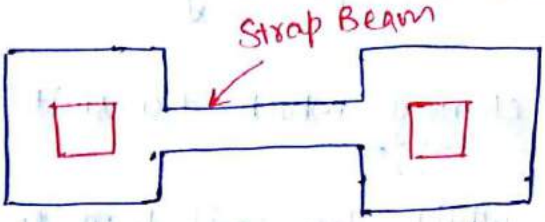


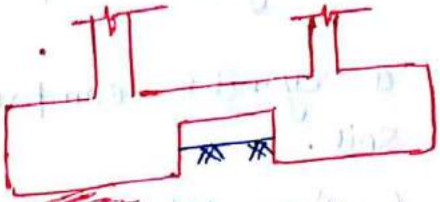
Trapezoidal footing

3 **Strap footing** :- When c/c distance b/w columns is high. but safe bearing capacity of soil be small. The isolated footings of column are joined together by a beam known as strap beam and the footing is known as strap footing

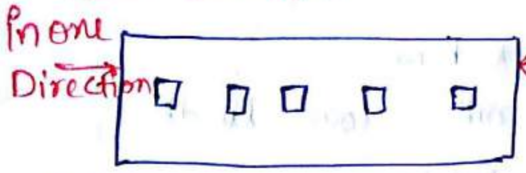
- This footing is provided such that the strap beam must not be in contact with sub- soil.
- The function of strap beam is to Redistributed the load

from heavy column to small loaded column.





4). Strip footing :-

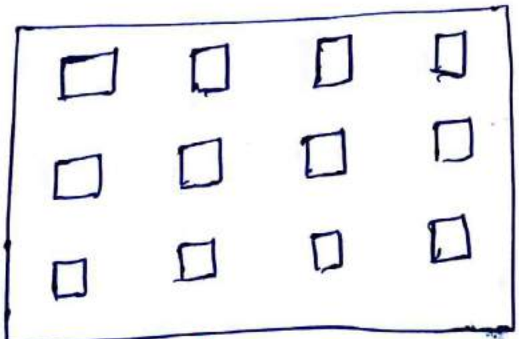


The footing in which length is very high as compared to its width is known as strip footing.

- Strip footing may be provided in the form of continous wall footing / a common footing under no. of column in a series

5 Mat / Raft footing :- when c/c distances b/w no. of columns in a given area are small and the safe bearing capacity of soil is low.

- The soil may be subjected to differential settlements.
- In above condition, a common footing for all the columns is provided which is known as Mat / Raft foundation.



2 Deep foundation :-

1 Pile foundation

(a) On the basis of Material
① Timber Piles ② steel Piles ③ Concrete Piles ④ composite Piles

(b) On the basis of use
① load bearing Piles ② compaction Piles ③ fender Piles.

(c)

On the basis of load transfer

i) End Point bearing Piles (ii) frictional Piles / floating Piles

→ **Pile** is a cylinder compression element which transmits load to the soil.

1. Point Bearing Piles :- The piles which transmits load to the sub soil particle -to- particle or grain -to- grain contact is known as End bearing Piles
- These Piles Rest over a Strong Rock base

2 Frictional Piles :- These piles transmitts load to the surround soil by skin frictional forces
- These piles doesnot Rest over strong base and are also known as floating piles