15/9/16

1. Origin of Soils

Soil - The loose unconsolidated earth material forming due to weathering or disintegration of rocks.

→ Soil is formed from weathering and decomposition of organic matter.

Soil
- Weathering
  - Physical
  - Chemical.
- decomposition of organic matter

Physical weathering :- Mechanical disintegration Gravels & Sands

1) Temperature change

2) Abrasion

3) Impact & splitting

→ Soil is from Rocks
↳ Every rock particle is a mixture of different rock minerals.

→ Due to presence minerals, Temp changes some minerals expands / contracts then rocks going to break and form soils.

Density of ice - 0.92 g/cc (or) 0.9167 g/cm^3.

→ due to decrease of temp, water enters into voids and forms ice. This will increase the volume and tends to break.

2) Abrasion - The water ~~or~~ flows through rocks and erodes to form soil.

3) Impact - Due to drop of rain water falling from certain height having energy and it will impact the rock and rocks are going to break.

Splitting - Due to roots present under the ground that may cause, increase in volume and length and breaking of rocks takes place.

Chemical weathering:- clays and to some extent silts

1) Hydrolysis
2) Carbonation
3) Oxidation
4) desilication etc leaching by organic acids & water

Rock Cycle:-

Weathering → Transportation → Deposition → Upheaval

settling of one particle over other

Transported Soil :-

soils that have been deposited ~~from~~ by

1) Aeolian soil - wind - Loess (Ex), Tuff

2) Lacustrine Soil - Lakes

3) Alluvial soil - in running water

4) Marine soil - sea water - Marl

5) Glacial soil - Glaciers (ice) Ex. Glacial till

6) Colluvial soil - Gravity - Talus.

Black cotton soil - formed from basalt & contain montmorillonite

Types of soils:-

1) Residual soil / Sedentary soil

2) Transported soil / Sedimentary soil.

Forces

1) Body forces - Gravel & Sand

2) Surface force - clays.

Phase Diagram:

The relative gravimetric and volumetric representation of different ingrediants

Void Ratio :- The ratio of volume of voids to solids

$$e = \frac{V_v}{V_s} \times 100$$

<u>Range</u> :- $V_v > V_s$, $e \geq 100\%$.

$V_v < V_s$, $0 < e < 100$

→ $V_v >>> V_s$, $e >>> 100\%$

The range of void ratio (e) is $e > 0$.

→ $V_s = V_v \Rightarrow e = 100\%$. fine grained soils voids > coarse grained soil void

Porosity:- The ratio of volume of voids to Volume.

$$\eta = \frac{V_v}{V} \times 100$$

Range:- $V_v \neq V \Rightarrow \eta \neq 100\%$

→ $0 < \eta < 100\%$

* Relation b/w void ratio & Porosity.

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

$$= \frac{V_v}{V_s\left[1 + \frac{V_v}{V_s}\right]}$$

$$\boxed{e = \frac{\eta}{1-\eta}} \qquad \boxed{\eta = \frac{e}{1+e}}$$

* The void ratio is more for fine soil compared to coarse grained soils.

Water Content (w%) in a given mass of soil

$$\boxed{w\% = \frac{W_w}{W_s} \times 100\%}$$

For dry state, $W_w = 0 \Rightarrow w\% = 0$.

Range :- $w \geq 0$

Degree of Saturation (S_r) :- The ratio of volume of water to volume of voids.

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

Range :- for dry soil $V_w = 0 \Rightarrow S_r = 0$

for completely saturated $(V_w) = V_v$

$S_r = 100\%$

for partially saturated $= 0 < S_r < 100$

$\rightarrow$ $0 \leq S_r \leq 100\%$.

Air Content (a_c) :- The ratio of volume of air to volume of voids.

$$\boxed{a_c = \frac{V_a}{V_v} \times 100\%}$$

$\rightarrow$ for full saturated soil $V_a = 0 \Rightarrow a_c = 0$

$\rightarrow$ for dry soil $V_a = V_v \Rightarrow a_c = 100\%$

$\rightarrow$ for partially saturated $\Rightarrow 0 < a_c < 100\%$

Range :- $0 \leq a_c \leq 100\%$

Relation b/w air content & Degree of Saturation :-

$$a_c + S_r = \frac{V_a}{V_v} + \frac{V_w}{V_v} = \frac{\cancel{V_v}}{\cancel{V_v}} = 1 / 100\%$$

$a_c + S_r = 1$

$$\boxed{a_c = 1 - S_r}$$

Percentage of air Voids (n_a):-

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

$\boxed{n_a = n a_c}$

$n_a \quad a_c = \frac{V_a}{V_v}, \frac{V_v}{V} \quad \frac{V_a}{V_v} \times \frac{V_v}{V}$

Range: $\underline{0 \le n_a \le n}$

for Saturated soil $V_a = 0 \Rightarrow n_a = 0$

for Dry soil $= V_a = V_v \Rightarrow n_a = \frac{V_v}{V} \times 100 \Rightarrow \boxed{n_a = n}$

for Partially saturated soil :- $0 < n_a < n$

Unit weight (γ):-

$$\gamma = \frac{W}{V} = \frac{W_{solids} + W_w}{V_{solids} + V_v}$$

$$= \frac{W_s\left[1 + \frac{W_w}{W_s}\right]}{V_s\left[1 + \frac{V_v}{V_s}\right]}$$

$\frac{W_s\left[1 + \frac{W_w}{W_s}\right]}{V_s\left[1 + \frac{V_v}{V_s}\right]} \quad \frac{\gamma_s(1+w)}{1+e}$

$\gamma_{solids} = \frac{W_s}{V_s}$

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

$a_c + S_r$

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

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

$\frac{V_v}{V_v}$

$a_c = 1 - S_r$

→ Unit wt of material ($\gamma_{material}$) = $G_{mat} \times \gamma_w$

→ Unit wt of Solid (γ_s) = $G_s \times \gamma_w$

$$\boxed{\gamma = \frac{G_s \gamma_w (1+w)}{1+e}}$$

ation b/w S_r, e, w,

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

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

$$e = \frac{1}{S_r} \times \frac{V_w}{V_s}$$

$$e S_r = \frac{W_w/\gamma_w}{W_s/\gamma_s}$$

$$e S_r = \frac{W_w}{W_s} \times \frac{\gamma_s}{\gamma_w}$$

$$e S_r = w \times \frac{G_s \times \cancel{\gamma_w}}{\cancel{\gamma_w}}$$

$$\boxed{e S_r = w \times G_{solid}} \Rightarrow w = \frac{e S_r}{G_s}$$

Relation b/w W_s, W & w

$$W = W_s + W_w$$

$$W = W_s\left[1 + W_w/W_s\right]$$

$$W = W_s[1 + w]$$

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

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

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

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

$$V = W/\gamma$$

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

From Unit wt $(\gamma) = \dfrac{\gamma_s(1+w)}{1+e}$

$$\gamma = \frac{\gamma_s\left[1 + \frac{e S_r}{G_s}\right]}{1+e}$$

$$\Rightarrow \gamma = \frac{G_s \gamma_w [1+w]}{1+e}$$

$$= \frac{G_s \gamma_w \left[1 + \frac{e S_r}{G_s}\right]}{1+e}$$

$$\gamma = \frac{G_s \gamma_w + e S_r \gamma_w}{1+e}$$

γ_s/γ_w

$$\frac{\gamma_s + e \gamma_w S_r}{1+e}$$

$$\frac{G_s \gamma_w \left[\frac{G_s + e S_r}{G_s}\right]}{1+e}$$

$$\gamma = \frac{\gamma_w (G_s + e S_r)}{1+e}$$

$$\boxed{\gamma_{bulk/total} = \frac{\gamma_w (G_s + e S_r)}{1+e}}$$

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

→ $$\gamma_{dry} = \frac{G_s \gamma_w}{1+e}$$

→ $$\gamma_d = \frac{\gamma_t}{1+w}$$ → dry density $\frac{\text{bulk density}}{1+w}$

Submerged unit weight :- Buoyant unit wt

When a soil mass is submerged below the ground water table, a buoyant force acts on the solids which is equal in magnitude to the wt of water displaced by solids. The net wt of solids is reduced, the reduced wt — buoyant / submerged unit wt

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

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

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

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

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

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

volume of soil (V_{soil}) = $V_{solids} + V_v$

$$= V_s\left[1 + \frac{V_v}{V_s}\right]$$

$$= V_s[1+e)$$

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

$$V_{soil} \propto (1+e)$$

V Vs Vv

$V = V_s + V_v$

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

$= V_s(1+e)$

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

A soil sample at different e values

$V_1 \propto 1+e_1$

$V_2 \propto 1+e_2$

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

1) The void ratio of a soil sample is 1. The void ratio of the soil sample is reduced 0.6. Find the Percentage volume lose.

Sol) $e_1 = 1$

$e_2 = 0.6$

$$\frac{V_1}{V_2} = \frac{1+1}{1+0.6} = \frac{2}{1.6}$$

$$\frac{V_1}{V_2} =$$

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

$$\frac{V_2}{V_1} = \frac{1+0.6}{1+1} = \frac{1.6}{2} = 0.8$$

$$\boxed{V_2 = 0.8\,V_1}$$

$$\% V_{lost} = \frac{V_1 - V_2}{V_1} \times 100 = \frac{V_1 - 0.8V_1}{V_1} \times 100 = 0.2 \times 100$$

$$= 20\%$$

$V_1 - 0.8V_1$

$V_1(1-0.2)$

$V_1(0.2)$

$V_1 = 0.2 \times 100$

20%

dry Unit weight (γ_d) :-

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

$$\gamma_d \propto \frac{1}{1+e}$$

$$\boxed{\frac{\gamma_{d_2}}{\gamma_{d_1}} = \frac{1+e_1}{1+e_2} = \frac{V_1}{V_2}}$$

Mass Specific Gravity (G_m):-

$$G_m = \frac{W}{V\gamma_w}$$

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

Unit weight (γ):- In place of water take Kerosene

$$\gamma = \frac{W}{V} = \frac{W_s + W_K}{V_s + V_v}$$

$$= \frac{W_s\left[1+\frac{W_K}{W_s}\right]}{V_s\left[1+\frac{V_v}{V_s}\right]}$$

$$= \frac{\gamma_s\left[1+\frac{W_K}{W_s}\right]}{1+e}$$

$$= \frac{G_s\gamma_w\left[1+\frac{\gamma_K V_K}{\gamma_s V_s}\right]}{1+e}$$

$$= \frac{G_s\gamma_w\left[1+\frac{G_K S_r e}{G_s}\right]}{1+e}$$

$$= \frac{G_s\gamma_w + G_K S_r e\,\gamma_w}{(1+e)}$$

$$\gamma = \frac{\left[G_s + G_K S_r e\right]\gamma_w}{(1+e)}$$

$\gamma_s = G_s\gamma_w$

$\gamma_K = \frac{W_K}{V_K}$

$\gamma_K V_K = W_K$

$\gamma_s V_s = W_s$

$$\frac{V_K}{V_s} = \frac{V_K}{V_v}\cdot\frac{V_v}{V_s}$$

$$= S_r e$$

$$\frac{\gamma_K}{\gamma_s} = \frac{G_K\cancel{\gamma_w}}{G_s\cancel{\gamma_w}} = \frac{G_K}{G_s}$$

2) A soil sample was saturated with kerosene oil. The saturated unit wt of kerosene is 2.4 g/cc. The specific gravity is solids & kerosene oil are 2.75 & 0.89 respectively then dry unit wt of soil sample was:

Sol) Given $\gamma_{sat} = 2.4$ $\gamma_w = 1 g/cc$

$G_K = 0.89$, $G_S = 2.75$

$$\gamma_{dry} = \frac{G_s \gamma_w}{1+e} \Rightarrow S_r = 0$$

$$\Rightarrow \gamma_{sat} = \frac{(G_s + G_K e)\gamma_w}{1+e} \quad S_r = 1$$

$$2.4 = \frac{[2.75 + (0.89) \times e] \times 1}{1+e}$$

$$2.4 + 2.4e = 2.75 + 0.89e$$

$$[\cancel{2.75 - 2.4 - 0.89e = 2.4}e]$$

$$2.75 - 2.4 = 2.4e - 0.89e$$

$$0.85 = 1.51e$$

$$\boxed{e = 0.231}$$

$$\gamma_{dry} = \frac{G_s \gamma_w}{1+e} = \frac{2.75 \times 1}{1+0.231} = \frac{2.75}{1.23} = 2.233$$

2.75
2.4
0.35
2.40
0.89
1.51

3) A saturated clay has water content of 39.3% and bulk specific gravity of 1.84.

1) Specific gravity of solids is

2) The void ratio of soil is

[γ_t] Partially sat

sol) Bulk specific gravity = $\frac{\gamma_{bulk(t)}}{\gamma_w} = \frac{\gamma_{sat}}{\gamma_w} = \frac{\gamma_{dry}}{\gamma_w}$

water content (w) = $\frac{W_w}{W_s} \times 100\% = 39.3\%$

Bulk specific gravity (G_s) γ_m = $\frac{W_s + W_w}{V_s + V_v} = 1.84$

$G_s = \frac{\gamma_s}{\gamma_w}$ $\Rightarrow G_m = \frac{\gamma_{sat}}{\gamma_w}$

$G_m = \frac{\gamma_{sat}}{\gamma_w}$

1) $1.84 = \frac{\frac{(G_s + e)\cancel{\gamma_w}}{1+e}}{\cancel{\gamma_w}} = \frac{G_s + e}{1+e}$

From relation

$S_r e = w G_s$

$e = w G_s$

$1.84 = \frac{G_s + w G_s}{1 + w G_s}$

$1.84 + (1.84 \times 0.393 G_s) = G_s[1+0.393]$

$1.84 + 0.723 G_s = 1.393 G_s$

$1.84 = 0.67 G_s$

$$\boxed{G_s = 2.75}$$

2) $e = w G_s = 0.393 \times 2.75$

$$\boxed{e = 1.08}$$

* Unit weight of soils: $\gamma_s, \gamma_{sat}, \gamma_d, \gamma_t, \gamma_{sub}$

$$\boxed{\gamma_s > \gamma_{sat} > \gamma_t > \gamma_d > \gamma_{sub}}$$

Types of Structures:-

→ The loosest / cubical Packing - Void ratio(e) - 0.91

→ The densest / diagonal Packing - Void ratio(e) - 0.35

• Sphericity :-

$$\text{Sphericity} = \frac{D_e}{L}$$

where,

De - equivalent dia of particle

L - Length of particle.

Volume of sphere $(V) = \frac{\pi D^3}{6} \Rightarrow \frac{\pi}{6} \times [2\gamma]^3 = \frac{\pi}{6} \times 8\gamma^3$

$V = \frac{4}{3}\pi\gamma^3$

$$\boxed{D_e = \left(\frac{6V}{\pi}\right)^{1/3}}$$

$\left(\frac{6V}{\pi}\right)^{1/3}$

$D = 2\gamma$, $\left(\frac{D}{2}\right)^3 \times \frac{4}{3}$, $\frac{D^3}{8} \times \frac{4}{3}\pi$, $\pi\frac{D^3}{6}$, $D^3 = \left(\frac{6V}{\pi}\right)^{1/3}$

→ To increase water content value from w_1 to w_2

The amount of water to be added

$A_w \Rightarrow \gamma_d V_{soil} (w_2 - w_1)$

$\gamma_d (w_2 - w_1) V_{soil}$

$\gamma = \frac{W_s}{V_s}$

4) A soil has a porosity of 40%, specific gravity 2.5. The water content is 12%.

1) Water content at full saturation is

2) The wt of water required to be added to 100 cubic cmt of soil for full saturation is

3) The water content which can fully saturate and increase the volume 5%.

Sol) Given

$n = 0.4$

$G_s = 2.5$

$w = 0.12$

$S_r \cdot e = w G_s$

1) $e = 0.12 \times 2.5$ $\quad S_r = 1$

$e = 0.3$

$\Rightarrow e = \frac{n}{1-n} = \frac{0.4}{1-0.4} = 0.66$

$\Rightarrow S_r \cdot e = w_{sat} G_s$

$w_{sat} = \frac{0.66}{2.5} = 0.26$

Water content at full Sat is 26%

2) $\gamma = \frac{W}{V}$

Given

$V = 100 cm^3$

$\gamma_{sat} =$

$W = ?$

$\gamma_{sat} = \frac{(G_s + e)\gamma_w}{1+e} = \frac{[2.5 + 0.66] \times 1}{1+0.66}$

$= 1.90 \ g/cc$

$1.90 \times 100 = W$

$[W_w + W_s] = 190$

$\gamma_s = G_s \gamma_w$

$\gamma_s = \frac{W_s}{V_s}$

$V_s = V_s + V_w$

3) $\Rightarrow \gamma_d V_{soil} (w_2 - w_1)$

$\gamma_d = \frac{G_s \gamma_w}{1+e} = \frac{2.5 \times 1}{1+0.66} = 1.5 \ g/cc$

$= 1.5 \ g/10^{-6} m^3$

$= 1.5 \times 10^6 \ g/m^3$

$\gamma_d = 1500 \ kg/m^3$

$w_2 = 26\% = 0.26$

$w_1 = 12\% = 0.12$

$A_w \Rightarrow 1500 \times 100 \times [0.26 - 0.12]$

$\Rightarrow 21000$ kg. $\quad$ 1 ton = 1000kg

$A_w \Rightarrow$ 21 tons

3) Given

volume increased - 5% = 0.05

$V_2 = V_1 + 0.05 V_1 = 1.05 V_1$

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

$$\frac{1.05 \cancel{V_1}}{\cancel{V_1}} = \frac{1+e_2}{1+0.66}$$

$1.05 \times 1.66 = 1 + e_2$

$e_2 = 0.75$

From relation

$S_r \cdot e = w G$

$e = w \cdot G$

$$\frac{0.75}{2.5} = w$$

$w = 0.3 \Rightarrow 30\%$

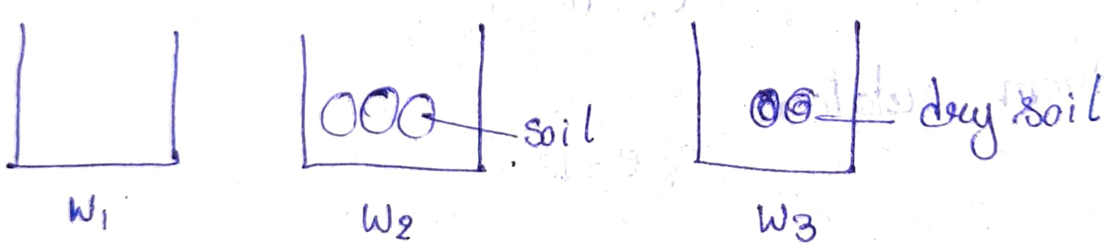
Water Content Determination: IS 2720 Part - II

1) Oven Drying Method } Important
2) Pycnometer Method }
3) Sand bath Method
4) Torsion Balance — Rapid Moisture Meter Method
5) Alcohol Method
6) Calcium Carbide method
7) Radiation Method

1) **Oven Drying Method**:- above $110^\circ C$ may break crystalline structure of clay particles
→ about $60^\circ C$ for organic soils

→ Test Temperature = $110^\circ C \pm 5^\circ C$ to avoid oxidation of organic matter present in the samples

→ Duration : 24 hrs.



W_1 - empty Container

W_2 - wt of wet soil + Container

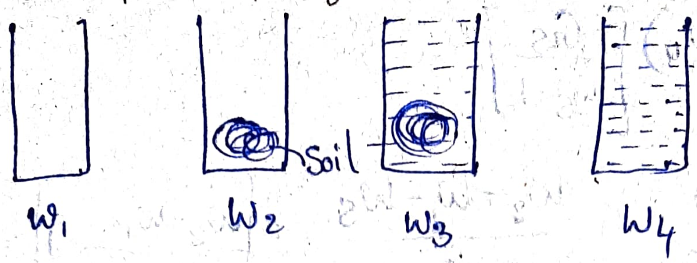
W_3 - wt of dry soil + Container

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

$$\boxed{w = \frac{W_2 - W_3}{W_3 - W_1} \times 100}$$

2) Pycnometer Method :- very fast Method.

↳ 900ml capacity provided with conical top (6mm dia)



$10^{-3} \times 10^{-2} \times 10^{-3}$

$1 \times 10^{-6} \times 10^{3}$

$\frac{1}{1000}$

0.001

W_1 - empty wt of Pycnometer

W_2 - empty wt of pycnometer + soil sample

W_3 - empty wt of " + soil sample + water

W_4 - wt of container + wt of water

$$\text{Water content } (w) = \frac{W_w}{W_s} \times 100$$

$$W_w = (W_2 - W_1) - W_s$$

Now, replace V_s in W_3 with W_4 [water]

$$V_{solid} = V_w$$

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

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

$$W_w = \frac{W_s}{G_s}$$

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

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

$W_w = \frac{W_s}{G_s}$

$W_3 - W_s + \frac{W_s}{G_s} = W_4$

$$\Rightarrow \left[W_3 - W_s + \frac{W_s}{G_s}\right] = W_4 \quad \text{[water added in place of solid]}$$

$$W_3 - W_4 = W_s - \frac{W_s}{G_s}$$

$$W_3 - W_4 = W_s\left[1 - \frac{1}{G_s}\right]$$

$$W_3 - W_4 = W_S\left[\frac{G_S - 1}{G_S}\right]$$

$$\Rightarrow W_S = (W_3 - W_4)\left[\frac{G_S}{G_S - 1}\right]$$

$$w\% = \frac{W_W}{W_S} = \frac{W_2 - W_1 - W_S}{W_S} = \left[\frac{W_2 - W_1}{W_S} - 1\right] \times 100$$

$$w\% = \left[\frac{(W_2 - W_1)}{(W_3 - W_4)\left[\frac{G_S}{G_S - 1}\right]} - 1\right] \times 100$$

$$\boxed{w\% = \left[\frac{(W_2 - W_1)(G_S - 1)}{(W_3 - W_4)\, G_S} - 1\right] \times 100}$$

Specific Gravity: IS 2720-Part-III at 4°C:

weight of material to weight of water ~~to~~ of an the equivalent volume.

→ For coarse grained soils low G. (Presence of organic matter)

$$G = \frac{W_m}{W_W}$$

→ Soil high in iron/mica high values.

→ Specific gravity of water (G_W) = 1

→ Specific gravity of ice (G_i) = 0.92

→ Sands & gravel - 2.65 - 2.68

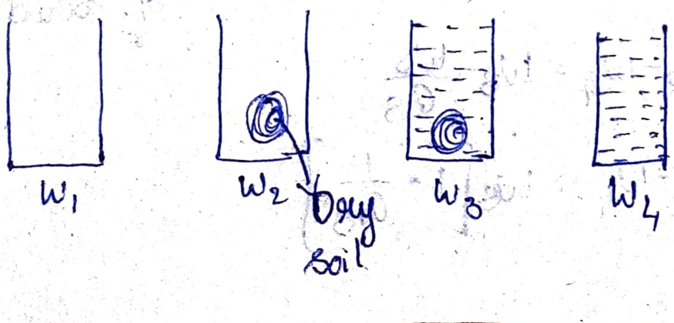
→ Silt - 2.66 - 2.70

→ Inorganic clays - 2.70 - 2.80

→ Soils high in mica, iron - 2.75 - 2.85

→ Organic < 2.0

1) Pycnometer:-



$$G = \frac{W_{soil}(\text{dry soil})}{W_w}$$

$W_s = W_2 - W_1$

$W_w = (W_4 - W_1) - (W_3 - W_2) \Rightarrow W_4 - W_1 - W_3 + W_2$

$(W_2 - W_1) - (W_3 - W_4)$

$$G = \frac{(W_2 - W_1)}{(W_4 - W_1) - (W_3 - W_2)}$$

$$G = \frac{(W_2 - W_1)}{(W_2 - W_1) - (W_3 - W_4)} \;//$$

17/9/16

Determination of density: IS 2720 : Part. 28

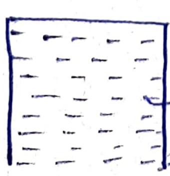
1. Sand Replacement Method
2. Core cutter Method - Cohesive Soils is used.
3. Water Displacement Method

2) Core cutter Method :-

→ Volume of core cutter = Volume of soil.

→ We use cohesive soil bcz it has interlocking b/w particles, so it cannot easily remove from core cutter.

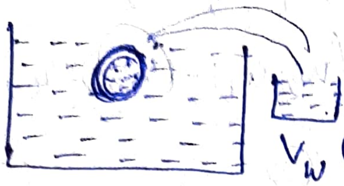
$$\text{density} = \frac{W_{soil}}{V_{soil} = V_{core\ cutter}}$$



3) Water Displacement Method :- Cohesive soils

Here, we use Paraffin wax to soil to remain as volume.

→ Soil sample W_1 Paraffin wax of known density W_2



V_w (displaced water)

$$V_w = V_{soil} + V_{p.w}$$

→ Weight of Paraffin wax $= W_2 - W_1$

$$V_{p.w} = \frac{wt}{density} = \frac{W_2 - W_1}{\gamma_{pw}}$$

$$V_{soil} = V_w - V_{p.w} = V_w - \frac{W_2 - W_1}{\gamma_{pw}}$$

$$\boxed{V_{soil} = V_w - \frac{W_2 - W_1}{\gamma_{pw}}}$$

$$density\ (\gamma) = \frac{W_1}{V_{soil}}$$

1) Sand Replacement :- Hard & Gravelly soils

→ Cone is used for Pouring of sand in calibrating Cylinder to find wt of soil in cylinder.

⇒ For bouldery soils, a large hole 30 cm to 1m dia

Soil Structures :- means the mode of arrangement of Soil particles relative to each other

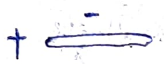
→ soil fabric only to describe geometry of mineral particles

1) Single grained - Ex:- Gravel, Sand.

2) Honey Comb Structures -  - fine Sand & Silts

→ It is very weak structure, if small disturbance takes place, it collapse.

3) Flocculated Structure - Clay particles.

 → Clay Particle flaky Shape

flocculated Structure	Dispersed Structure.
→ Edge to face orientation	→ face to face Orientation.
	
→ Net Attractive forces	→ Net Repulsive forces.
→ high strength, more void ratio, high permeability, low Compressibility	→ low strength, high Compressibility, low permeability, low void ratio.
→ More stable.	→ less stable.

Thixotropy :- Regaining of lost strength due to dispersed structure changing into flocculant structure with passage of time, with no change in water content.

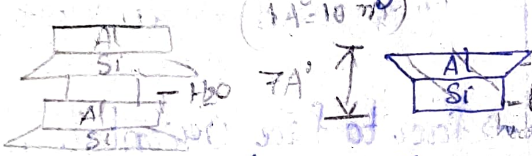
20/9/16

Clay Mineralogy :- are composed of tiny crystalline substances of one or more members of a small group of minerals. These minerals are evolved hydrous aluminosilicates.

Minerals

- ↓ Rock Minerals
  - → No surface activity
  - → eg :- Mica, Feldspar, Quartz etc
- ↓ due to Chemical weathering of rocks forming minerals. Clay Minerals.
  - → Surface activity
  - → eg :- Kaolinite, Illite, Montmorillonite.

1) **Kaolinite** :- 2 layered mineral, It is stable mineral.

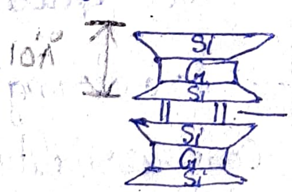


The stacking of one layer of each of two basic sheets sometimes called 1:1 clay mineral.

→ No swelling, No shrinkage.

→ These stru

2) **Illite** :- 3 layered mineral, Medium swelling & shrinkage

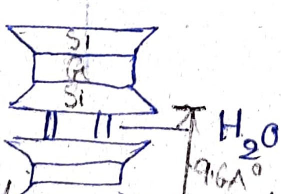


Pottassium ions.

→ The clay mineral containing Montmorillonite called Black cotton soils. Ex, Bentonite (volcanic ash)

3) **Montmorillonite / Smectite** :- 3 layered Mineral → two silica & 1 alumina (gibbsite) sheet, a 2:1 mineral

The thickness is about 9.6 A°



→ It has large specific area so water can easily enter b/w layers and separate due to volume changes occurs.

→ high swelling & shrinkage → water enters swelling takes place and shrink when water removed

augite
ocyte

4) **Halloysite** :- Same as Kaolinite, but it is more randomly stacked than Kaolinite

* Kaolinite < Hallocyte < Illite < Montmorillonite.

Specific Surface Area (SSA):-

$$SSA = \frac{\text{Surface area}}{\text{Unit volume}}$$

Consider a sphere

$$SSA = \frac{4\pi r^2}{\frac{4}{3}\pi r^3} = \frac{3}{r}.$$

$$\boxed{SSA \propto \frac{1}{r}}$$

Particles SSA - Colloidal > Clay > Silt > Sand > Gravel.

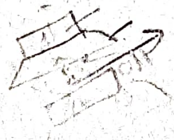
Soil Properties:-

Soil Properties

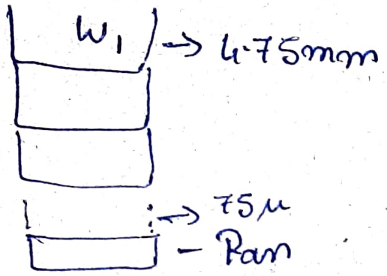
- Index Properties
  - Coarse Grain
    - → Grain Size Distribution
    - → Relative Density
  - F.G.
    - → Consistency limits
- Engineering Properties
  - → Shear strength, Compressibility, Permeability etc.

Grain Size Distribution:- IS

1) Sieve Analysis
2) Hydrometer Analysis.

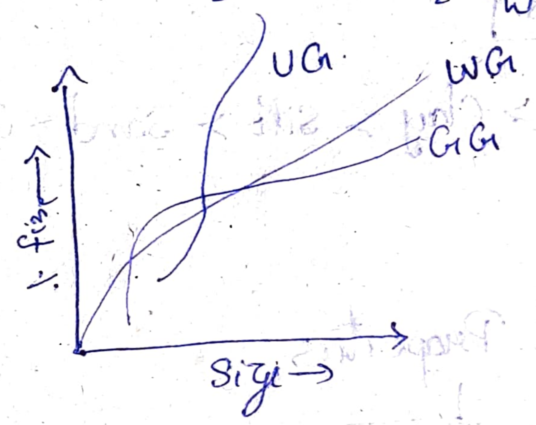


1) Sieve Analysis :-



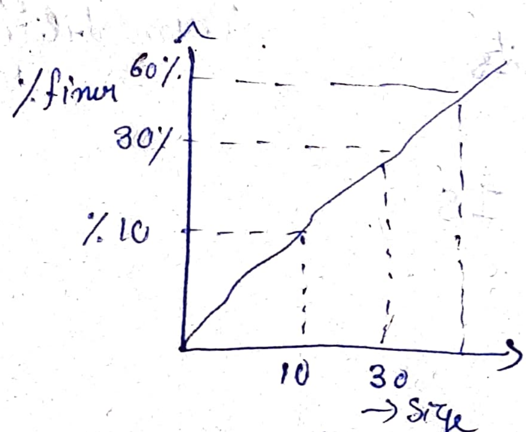
Particles < 75μ are called fines

Size	wt retained	% wt retained	Cumulative % wt retained	% Finer
4-75	W_1	$w_1 = \frac{W_1}{W} \times 100$	w_1	$100-w_1$
2.36	W_2	$w_2 = \frac{W_2}{W} \times 100$	$w_1 + w_2$	$100-(w_1+$



Well Graded
Gap Graded } Poorly graded
Uniformly Graded

1) Well Graded → All size of particles are available
2) Gap Graded → Some particle sizes are missing.
3) Uniformly Graded → Same particle sizes are exists.



→ D_{10} → Percentage of fines below 1mm

→ C_u, C_c, C_s (sorting)

→ $C_u > 6$ → Well Graded Sand

→ $C_u > 4$ → Well Graded Gravel

→ $C_u < 2$ → Uniformly Graded.

$$C_u = \frac{D_{60}}{D_{10}}$$ → D_{10} is also called effective diameter.

→ C_c is also called as Coefficient of gradiation.

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

for well graded $1 < C_c < 3$

Coefficient of Sorting (C_s):-

$$C_s = \sqrt{\frac{D_{75}}{D_{25}}}$$

→ For Uniformly Graded soil

$$D_{10} \simeq D_{30} \simeq D_{60}$$

→ Another Methods are Pipette, Sieve Analysis.

2) Hydrometer Analysis:-

→ Particles of size $< 75\mu$.

→ 50g of soil passing 75μ.

→ We add dispersing Agents because in soil sample clays particles are present and forms flocs due to cohesive forces and forms turbulence, to avoid this we add.

→ Dispersing Agent

- Sodium oxalate or Sodium hexa meta phosphate
- Sodium Carbonate = 100ml.

→ The law is Stoke's Law. 10^{-6}

→ Size - 0.2mm - 0.2μ

→ > 0.2 mm - It causes turbulence in solution.

→ < 0.2μ - It causes Brownian motion / Zig Zag motion.

effective depth $H_e = h_1 + \frac{1}{2}\left[h - \frac{h}{2A_j}\right]$

Settling velocity $(V_s) = \frac{g d^2 (G-1)}{18\nu}$ dyne-s/cm², cm/s², g/cm³

ν = Kinematic viscosity $\left(\frac{\mu}{\rho} \cdot cm^2/sec - \text{Stoke}\right)$

Types of corrections:-

1) Temparature:-

- If the temp is above the standard temp then the correction is positive.
- If the test temp is below the standard temp then the correction is negative.

2) Meniscus:-

correction of meniscus is always positive.

3) Dispersing agent:-

Correction due to dispersing agent is always negative.

23/9/16 Relative Compaction (or) Density index :-

$$I_D = \frac{e_{max} - e_{nat}}{e_{max} - e_{min}}$$

$$I_D = \frac{\frac{1}{(\gamma_d)_{min}} - \frac{1}{(\gamma_d)_{natural}}}{\frac{1}{(\gamma_d)_{min}} - \frac{1}{(\gamma_d)_{max}}}$$

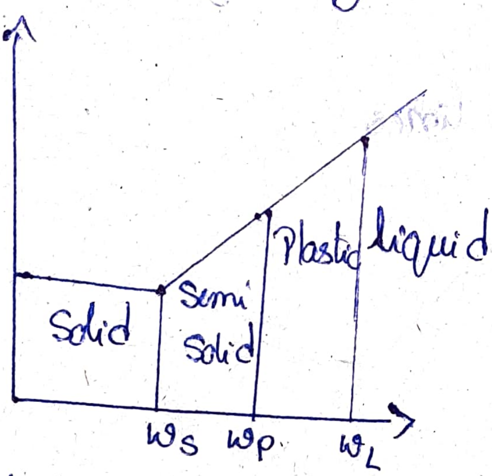
$I_D \leq 15\%$ - very loose
$15 < I_D < 35$ - loose
$35 < I_D < 65$ - medium dense
$65 < I_D < 85$ - Dense
> 85 - very dense

If density index of two samples are same they become identiful.

Consistency Limits : Fine grained soils.

→ Consistency limits are , when water combines it changes from one state to other state.

→ Consistency limits are determined by Atterberg. They are also called as Atterberg Limits.



$\gamma_{max} = 2.1413$

$\gamma_{min} = 1.906$

→ when liquid to plastic - liquid limit
→ when plastic to semi solid - Plastic limit
→ when semi solid to solid - Shrinkage limit

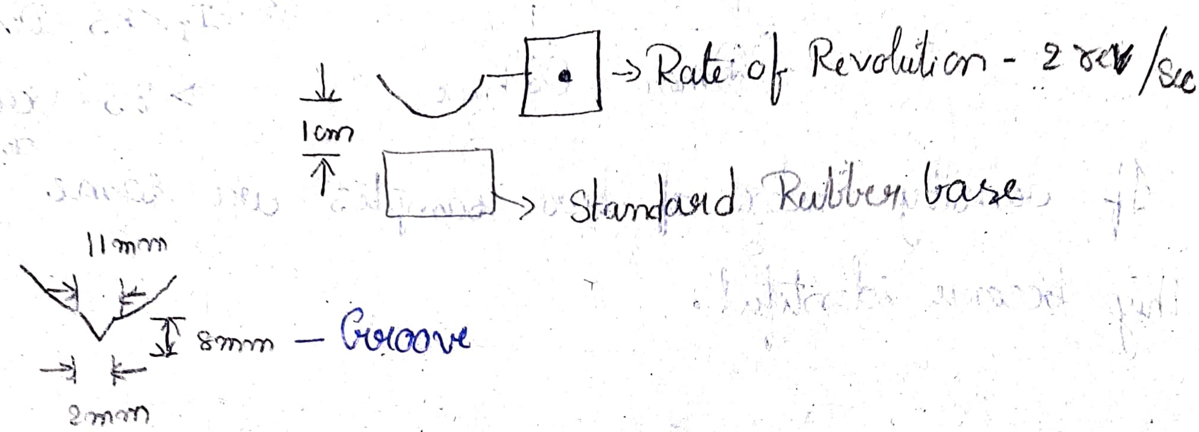
These values are nothing but water content values

1) Liquid Limit :- IS : 2720 Part-5

→ Apparatus : Casagrande.

→ Soil passing from 425 μ.

→ from code soil sample taken as 120g, but we take 200g.

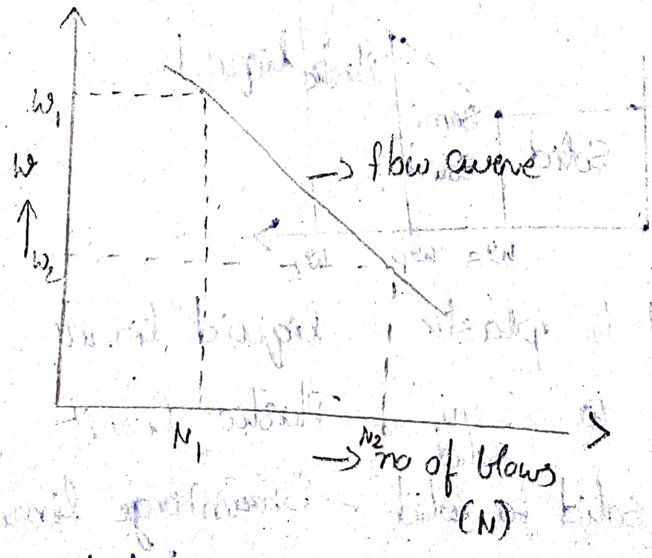


→ Between 10 and 40 blows liquid limit value is taken $\left[\frac{10+40}{2}\right] = 25$ blows.

The uses of Log Scale :-

→ If the observations are very wide [large variation in values]

→ To get straight lines.

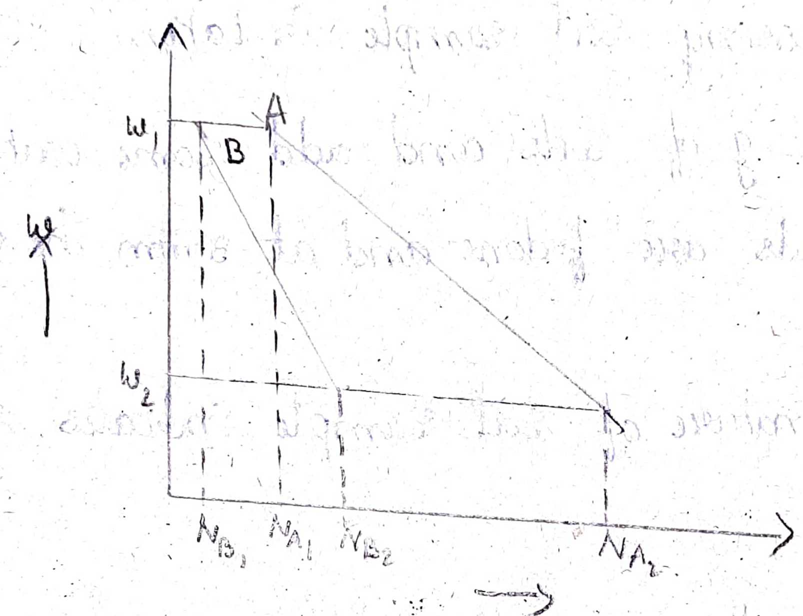


→ The graph between no. of blows to water content is called flow curve.

→ Flow curve represents loss of shear strength upon increasing water contents (w).

→ The slope of flow curve is flow index

$$I_F = \frac{w_1 - w_2}{\log\left(\frac{N_2}{N_1}\right)}$$



Consider two different soils A & B

→ A has more strength because in A contain cohesive forces to separate particle, more number of blows is applied.

→ Slope of flow curve is flow Index $(I_F) \propto \frac{1}{\text{Shear strength}}$

Empirical formula :-

$$w_L = w_n \left(\frac{N}{25}\right)^x$$

where $x \rightarrow 0.068 - 0.121$

$N \rightarrow$ no. of blows in natural state.

→ For any type of soil, at liquid state shear strength is zero.

→ At liquid limit any type of soil has same shear strength that is 27 g/cm².

2) Plastic limit :- IS 2720 (Part-5)

→ 425μ passing soil sample is taken.

→ Take 15 g of soils and add some water.

→ Some threads are done and at 3mm the crumbles occurs.

→ Crumbling nature of soil sample indicates semi solid state.

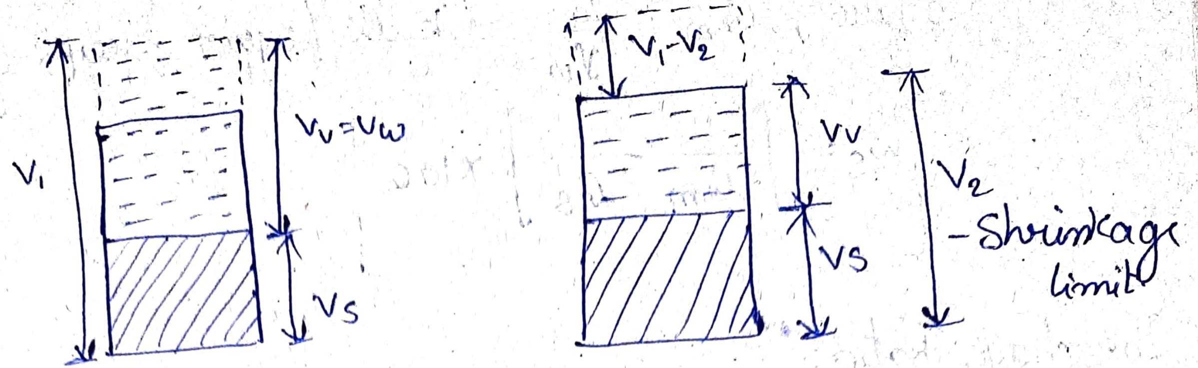
3) Shrinkage Limit:

→ It is a boundary water content where soil changes from semi solid state to solid state.

→ At shrinkage limit, degree of saturated is 100%.

→ Shrinkage limit value is nothing but the water content at which the soil is just full saturated.

→ Below shrinkage limit there is no change in volume of ~~solids~~ soil sample.



→ Some extra water is added to get solids.

→ The extra water is removed, then we get shrinkage limit value (initial how much water is present in voids).

$$\text{Shrinkage limit } (w_s) = \frac{\overset{W_w}{W_1 - W_d}}{W_d\ (W_s)} - \frac{\overset{(W_w)}{(V_1 - V_2)\gamma_w}}{W_d\ (W_s)}$$

$\left\{ \gamma_w = \frac{W_w}{V_w} \quad \gamma_w V_w = W_w \right.$

where, W_1 - wt of wet soil

W_d - wt of dry soil

→ Any non observing agent like mercury is used to measure the volume.

$$w_s = \left[\frac{W_1 - W_d}{W_d} - \frac{(V_1 - V_2)\gamma_w}{W_d}\right] \times 100$$

For dry sample is Consider:

$$w_s = \left[\frac{V_d - V_s}{W_d}\right] \times 100$$

$$= \left[\frac{V_d}{W_d} - \frac{V_s}{W_d}\right]\gamma_w$$

$\left\{ \gamma_d = \frac{W_d}{V_d} \quad \frac{V_d}{W_d} = \frac{1}{\gamma_d} \right.$

$$= \left[\frac{\gamma_w}{\gamma_d} - \frac{\gamma_w}{\gamma_s}\right] = \left[\frac{1}{G_m} - \frac{1}{G_s}\right] \times 100$$

where, $G_m = \frac{\gamma_d}{\gamma_w}$ (Bulk / upper Specific gravity)

$$\boxed{w_s = \left[\frac{1}{G_m} - \frac{1}{G_s}\right] \times 100}$$

Shrinkage Ratio :-

$$SR = \frac{\frac{V_1 - V_2}{V_d}}{w_1 - w_2} \times 100 = \frac{\text{change in Volume}}{\text{weight}} \times 100.$$

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

Plasticity Index :-

→ Represents degree of plastic behaviour.

→ If plastic ~~behav~~ index is more it can behave plastic at a wide range of water content values.

$$I_p = w_L - w_p$$

→ If plastic behaviour is more Plasticity index is more.

Shrinkage Index :-

→ The Range ~~of~~ water content b/w which the sample is in semi solid state.

$$I_s = w_p - w_s$$

→ If $I_p = 0$ → non-plastic behaviour

→ If $I_p < 7$ → low plastic behaviour

→ If $7 < I_p < 17$ → Medium Plastic "

→ $I_p > 17$ - highly Plastic behaviour.

Toughness Index:

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

→ Toughness index represents a shear strength of a soil in Plastic state.

Consistency Index:-

$$I_c = \frac{w_L - w_n}{(w_L - w_p) I_p} \quad [w_{nat} > w_p]$$

→ The ratio between natural water limit and Plastic limit.

→ $I_c < 1$ - when $w_{nat} > w_p$ - Plastic state

→ $I_c = 1$ - Plastic limit at Plastic state

→ $I_c > 1$ - below Plastic limit (w_p)

→ The Consistency is nothing but resistance to deformation.

$w_L - w_n + w_n$ $- w_p$

Liquidity Index:-

→ The ratio of difference between natural moisture Content to Plasticity index.

$$I_L = \frac{w_{nat} - w_p}{w_L - w_p}$$

→ Liquidity index represents, how much likely to be behave like liquid.

→ $I_L = 1$ - liquid limit ($w_n = w_L$)

→ $I_L > 1$ - If $w_n > w_L$.

→ $I_L = 0$ - at plastic limit.

Relation b/w I_c and I_L

$$I_c + I_L = \frac{w_L - w_n}{I_p} + \frac{w_n - w_p}{I_p}$$

$$= \frac{w_L - \cancel{w_n} + \cancel{w_n} - w_p}{I_p}$$

$$= \frac{w_L - w_p}{I_p} = \frac{\cancel{I_p}}{\cancel{I_p}}$$

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

Activity number :-

$$A = \frac{I_p}{\% \text{ clay fraction}}$$

→ It represents swelling and shrinkage characteristics.

→ when % clay fraction ↑, swelling & shrinkage ↓.

→ If $A < 0.75$ - inactive.

→ If $0.75 < A < 1.25$ - normally active.

→ If $A > 1.25$ - active.

* For Coarse grained soils plasticity index is zero

1) If a soil has void ratio of 0.4, what will be the density Index? Assume all the particles are perfectly spherical.

$$I_D = \frac{e_{max} - e_{nat}}{e_{max} - e_{min}}$$

2) A sample of clay was coated with wax and total mass of soil sample without wax is 690 gms, with wax 697.5 gms, the sample was immersed in water and the volume of water displaced was found to be 355 ml. The water content of soil specimen was 18%. Determine Bulk density, Void ratio, Porosity and Degree of saturation. ($G_s = 2.7$, $G_{pw} = 0.89$)

$$G_{pw} = \frac{W_{pw}}{W_w}$$

Sol) Given $W_1 = 690$ gms

$V_w = 355$ ml

$W_2 = 697.5$ gms, $w = 18\%$

$$V_{soil} = V_w - \frac{W_2 - W_1}{\gamma_{pw}}$$

$$= 355 - \frac{(697.5 - 690)}{0.89}$$

$$V_{soil} = 355 - 8.426 = 346.57 \text{ cm}^3$$

$$\gamma = \frac{690}{346.57} = 1.99 \text{ g/cc}$$

$$\gamma_d = \frac{\gamma_t}{1+w} = \frac{1.99}{1+0.18} = 1.68 \text{ g/cc}$$

$$e = \frac{G_s \gamma_w}{\gamma_d} - 1 = 0.6$$

$$n = \frac{0.6}{1+0.6} = 0.37$$

$$eS_r = w \times G_s$$

$$S_r = \frac{0.18 \times 2.7}{0.6} = 0.81$$

24/9/16

Soil Classification

1) IS Grain size Classification:

Criteria : Grain size

2μ	75μ	4.75mm	80mm	300 mm	
Clay	(Silt) Silt	Sand	Gravel	Cobbles	Boulders.

Draw back: Rock flour < all about 2μ but has no plasticity.

2) Highway Research Board or AASHTO Classification:

AASHTO - American Association of state highway and Transport officials. (1978)

Criteria: 1) Grain size Distribution

2) Consistency limits.

$A_1 - A_7$ with an additional group A_8

→ Divided soils into several groups: $A_1, A_2, \ldots A_8$

$A_1 > A_2 > A_3 \ldots A_8$ A_8 - peat or muck.

→ mainly used in Pavement Design

→ A_1 is strong soil, A_8 is recently added Highly organic soil, A_7 is black cotton soil.

→ Based on quality they divided groups into sub categories.

$A_n(x)$ → group index (0-20) (GI)

↳ Group no. (1-8)

$$\boxed{GI = 0.2a + 0.005ac + 0.01bd}$$

(0-40) a = % Soil passing 75μ sieve > 35% and not exceeding 75%. a = 60-35 = 25%

(0-40) b = % soil passing 75μ sieve > 15% and not exceeding 55%.

(0-20) c = liquid limit value > 40%, should not exceed 60%.

(0-20) d = plasticity index value > 10%, should not exceed 30%.

(Q) Percentage of soil passing 75μ sieve is 60%. liquid limit 55, plastic limit 35.

Sol) $I_p = W_l - W_p$

$= 55 - 35$

$I_p = 20 \Rightarrow d = 20 - 10$

$= 10\%$

$a = 60 - 35 = 25\%$

$b = 55 - 15 = 40\%$

$c = 55 - 40 = 15$

$GI = 0.2 \times (25) + (0.005 \times 25 \times 15) + 0.01 \times 40 \times 10$

$GI = 10.875$

3) Unified soil Classification system (USCS) IS : 1498

→ Casagrande (1948) - used in airfield. Construction.

This is based on their criterias

1) Grain size distribution
2) Consistency
3) Compressibility characteristics.

4) IS: Soil Classification System : IS : 1498.

This is based on their criterias

1) Grain size distribution

2) Consistency

3) Compressibility characteristics.

Classifying the soils:-

1) Coarse grained	2) Fine grained	3) Highly organic soils.
→ Soil retained from 75μ sieve can called Coarse fraction. → C.F > 50%. → If CF = F.F = 50% then we always favour Coarse grained. Gravel (G), Sand (S) } C.G [illegible]]-4.75mm retained > 50% - Gravel % fine > 50% - Sand → 50% / more CF is retained on 4.75mm - Gravel	→ Sieving through 75μ Sieve. → Soil retained from 75μ Sieve called coarse fraction. → Soil passing from 75μ Sieve called fine fraction. → C.F > 50%, F.F > 50%. → If CF = FF = 50% then always favour Coarse grained ① Gravel (G) ② Sand (S) } C.G	Colour test: Black or Dark Brown → odour: very bad smell → Feel of touch: Spongy Ex - Peat & muck → Peat (Pt): Soil formed from decayed vegetables. → muck: - mixture of organic & inorganic soil.

→ otherwise - Sand → Silty clay

Peat & muck
cumulose soils

→ Well graded soil has wide distribution of grain size present

Fine Grained:-

→ FF > 50%.

① Silt →

② clays

③ organic soil

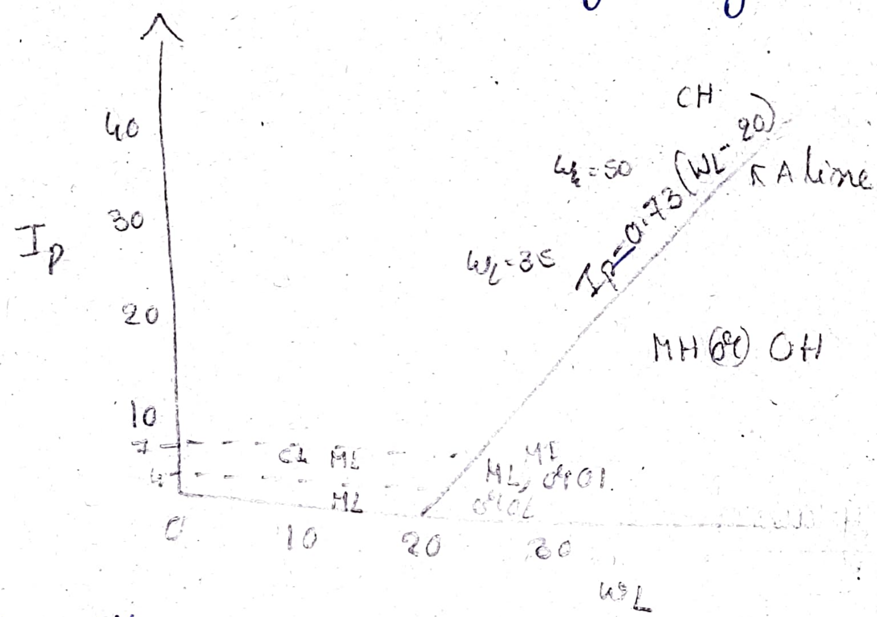
% retained > 50% - Silt

% passed > 50% - clay.

ISSCS

Plasticity Chart:-

Plasticity chart determined by Casagrande



→ If $w_L < 35\%$.

→ low Compressibility soil

→ If $35 < w_L < 50\%$.

→ It is intermediate Compressible soil.

→ If $w_L > 50\%$.

→ It is high Compressible soil.

→ Soil above A line is clay (∵ M-mho)

→ Soil below A line is silt (M) or organic soil (O)

2) Liquid limit value is 60% and Plastic limit is 20%. Determine soil type based plasiticity chart.

α = I_p = 60 - 20 = 40% - 10 = 30%

a = 60 - 35 = 25%

b = 55 - 15 = 40%

c = 60 - 40 = 20%

d = 30%

<u>Coarse grained</u>:-

<u>Gravel</u>:-

1. well graded gravel (GW)
2. Poorly " " (GP)
3. Silty gravel (GM)
4. Clayy gravel (GC)

Sand:-

1. well graded sand (SW)
2. Poorly graded sand (SP)
3. Silty sand (SM)
4. Clayed sand (SC)

→ silty sand
silt % < Sand %

→ Clayey silty sand
Sand % > Silt > clays.

Fine grained

Silt :-

① Low Compressible silt (ML)
② Intermediate Compressible silt (MI)
③ High Compressible silt (MH)

clay (c) :-

① Low Compressible clay (CL)
② Intermediate compressible clay (CI)
③ High Compressible clay (CH)

Organic soil (O) :-

① Low Compressible Organic soil (OL)
② Intermediate " " " (OI)
③ High " " " (OH) - $\underbrace{\text{Peat \& Swamp}}_{\text{Put together } P_t}$

Gravel :-

① Well graded gravel :-
→ % fines < 5% Passing 75μ
→ $C_u \geq 4$
→ $1 < C_c < 3$

② Poorly graded gravel :-
→ % fines < 5%
→ $C_u \geq 4$
→ If it is satisfies $1 < C_c < 3$ then it is WG otherwise (if not satisfied PG)

3) Silty gravel :-

→ % fines > 12 %.

→ Ip value should fall below A-line.
(Ip < 4)

4) Clayey gravel :-

→ % fines > 12 %.

→ Ip value should fall above A line (Ip > 7)

→ 5 < % fines < 12 % then dual symbols are occur that is GW-GM, GP-GM, GW-GC, GP-GC.

→ If Ip value is 4 < Ip < 7 then duals symbols are occur that is GM-GC.

In dual symbol, 1st part indicate gradation

Sand :- GMGS - silty gravel with s... 2nd part indicate nature of fines.

1) Well graded sand :-

→ % fines < 5%.

$C_u \geq 6$

$1 < C_c < 3$

2) Poorly graded sand

→ % fines < 5%.

$C_u \geq 6$

$1 \nless C_c \nless 3$

3) Silty sand

→ % fines > 12%

→ Ip value should fall below A-line (Ip < 4)

→ 5 < % fines < 12 then dual symbols occur SW-SM, SP-SM, SW-SC, SP-SC.

→ $4 < I_p < 7$ → Dual symbols.

SM-SC.

→ For Organic soils moisture content decreases by more than 25% of the initial value.

Boulders		Rounded to angular, bulky, hard dia more than 300mm.
Cobbles		Rounded to angular " " dia smaller than 300mm but retained on 80mm IS Sieve.
i) Coarse-grained soils		
Gravel	G	Passing 80mm & retained on 4.75mm Sieve. Coarse: 80mm to 20mm Fine : 20mm to 4.75mm.
Sand	S	Passing from 4.75mm, retained on 75μ Coarse - 4.75mm to 2mm Medium - 2mm to 425μ Fine - 425μ to 75μ.
2) Fine grained soils		
Silt	M	Smaller than 75μ, identified by behaviour it slightly plastic/non plastic little strength
Clay	C	Smaller than 75μ, identified by behaviour exhibit plastic properties, strength.
3) Organic matter	O	Organic matter in various sizes & stage of decomposition (no specific grain)
4) Peat	P_t	Fibrous, spongy (no specific grain).

26/9/16

2. Permeability

1) Bernoulli's Equation:-

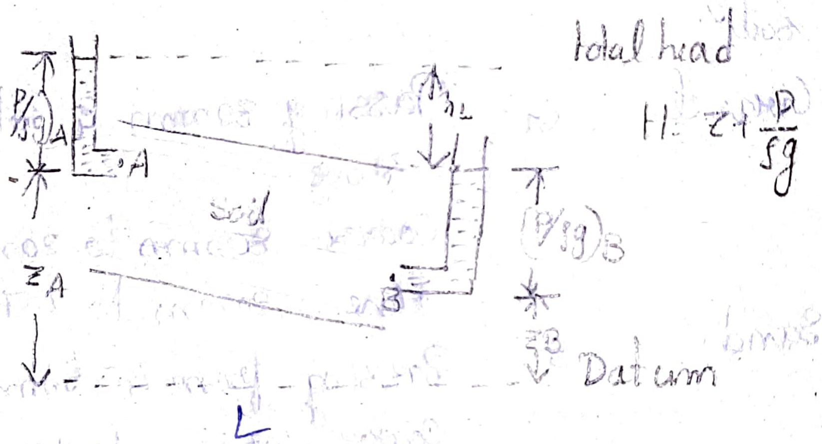
A B

$$H = Z + \frac{P}{\rho g} + \frac{V^2}{2g}$$ total head ($H_A > H_B$)

Permeability:- Permeability is a soil property which describes the ease with which a fluid can pass through

① It should be porous

② Pores should be interconnected.



→ when a flow through soil is considered because of velocity being small, velocity head is neglected.

→ Total head A is greater than total head B.

2) Darcy's Law:- Velocity of fluid through soil is proportional to hydraulic gradient.

$V \propto i$, $i = \frac{h}{L}$ = head lost per unit seepage length.

$\boxed{V = Ki}$ where, V - discharge velocity (or) Apparent velocity.

where, K - hydraulic Conductivity / Coefficient of Permeability.

→ Discharge $Q = AV$

$Q = Aki$

$\boxed{Q = kiA}$ where, A - Area of C/s.

→ $Q = AV = A_v V_s$ where, A_v - area of voids

V_s = Actual velocity / Seepage velocity.

$(A_s + A_v)V = A_v V_s$

$V_s = \frac{(A_s + A_v)V}{A_v}$

$V_s = \frac{(A_s + A_v)L}{A_v L} \times V$

$\left\{ \frac{V_v}{V_s} = e \right.$

$\left. \frac{1+e}{e} = \frac{1}{n} \right.$

$V_s = \frac{(V_s + V_v)}{V_v} \times V$

$V_s = \frac{V_s\left[1 + \frac{V_v}{V_s}\right]}{V_v} \times V = \left[\frac{1+e}{e}\right] V$

$\boxed{V_s = \frac{V}{n}}$

Superficial velocity

V_s - Seepage velocity

V - discharge velocity

n - Porosity

⇒ Darcy's Law. $V = ki$

$V_s = \frac{ki}{n}$

$\boxed{V_s = \frac{k}{n} i}$

$\frac{k}{n}$ = Coefficient of Percolation (k_p)

$\boxed{V_s = k_p i}$

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

where; C - shape constant - 2.3

D_{10} = effective size.

e = Void ratio

γ_w = Unit wt of water

μ = Dyanmic viscosity = 0.001 Pa-Sec

1 Poise = 0.1 Pa-sec

0.01 Poise = 0.001 Pa-sec.

$\rightarrow \nu = \frac{\mu}{\rho} = \frac{1\times10^{-3}}{10^3} = 1\times10^{-6}$

Factors effecting permeability (K):-

1) <u>Shape</u>:- Among rounded and Angular, rounded Particles are more permeable because they have more void volume (or) void size is more.

2) <u>Void ratio</u>:- There is no perfect relation between void ratio and permeability. By equations they are directly proportional but in clays & Coarse are less permeable and high void ratio.

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

3) <u>Size</u>:- Permeability is directly proportional to size.

$$K \propto D_{10}^2$$

$$\boxed{\underset{cm/sec}{K} = 100 \cdot \underset{cm}{D_{10}^2}}$$

4) Viscosity :- $K \propto \frac{1}{\mu}$

$\left\{ \mu \propto \frac{1}{T^\circ c} \right\}$ for liquids

→ If viscosity is more, then K is less.

5) Temperature :- $K \propto T^\circ c$

6) Porosity :- $K \propto n$

7) Degree of saturation :- Among fully saturated and partially saturated, saturated soils are more permeable because in partially saturated soils, air present in void spaces blocks the passage of water.

8) Specific Surface Area (SSA) :-

$\frac{\text{Surface area}}{\text{Volume}}$

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

→ for clay SSA is more, but less permeable.

9) Adsorbed water :-

$$K \propto \frac{1}{\text{adsorbed water}}$$

→ due to formation of layer on the particle, it decreases.

10) Density Properties of flowing liquid :-

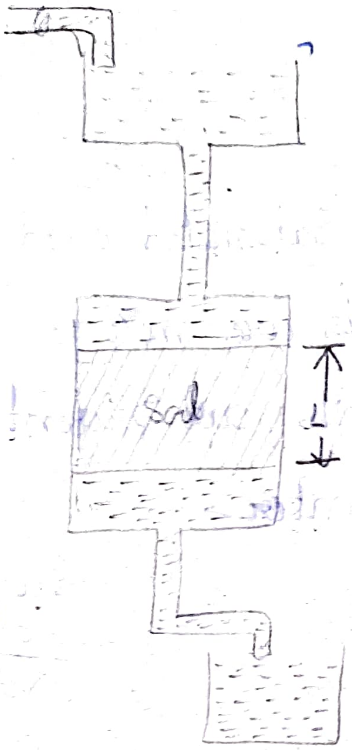
$$K \propto \frac{\gamma}{\mu}$$

$$\boxed{\frac{K_1}{K_2} = \frac{\gamma_1}{\mu_1} \cdot \frac{\mu_2}{\gamma_2}}$$

Permeability :-

1) Constant Head Test :-

→ It is used in Coarse grained soils to get measuring water.



$$Q = kiA$$

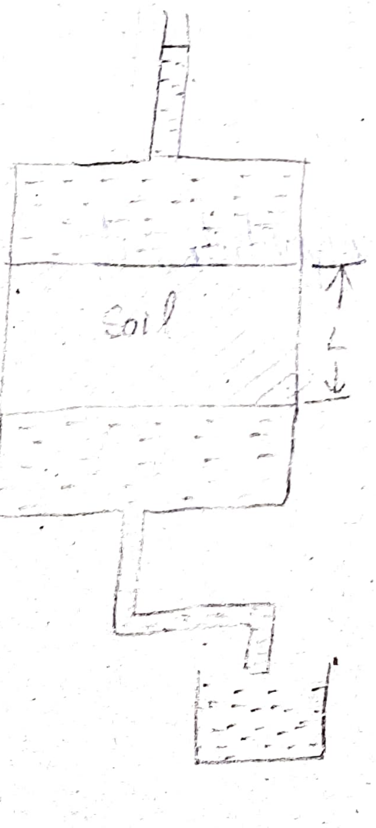
$$\frac{V}{t} = kiA$$

$$\frac{V}{t} = k \cdot \frac{h}{L} A$$

$$\boxed{K = \frac{VL}{Aht}}$$

V = Volume of water collecting in time t.

2) Variable Head Test / Falling Head Test :-



$$Q = kiA = k \cdot \frac{h}{L} \cdot A$$

$$V = -\frac{dh}{dt} \text{ (velocity)}$$

$$Q = -a \cdot \frac{dh}{dt}$$

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

$$\frac{kA}{L} \cdot dt = -a \frac{dh}{h}$$

Integrate on both sides.

$$-a \int_{h_1}^{h_2} \frac{dh}{h} = \frac{kA}{L} \int_{t_1}^{t_2} dt$$

$$-a \log_e [h]_{h_1}^{h_2} = \frac{KA}{L}[t_2 - t_1]$$

$$-a[\log_e(h_2) - \log_e(h_1)] = \frac{KA}{L} t$$

$$a[\log_e(h_1) - \log_e(h_2)] = \frac{KA}{L} t$$

$$a \cdot \log_e (h_1/h_2) = \frac{KA}{L} t$$

$$K = \frac{aL}{At} \times \log_e (h_1/h_2)$$

$\left\{ \log_e^x = 2.303 \log_{10}^x \right.$

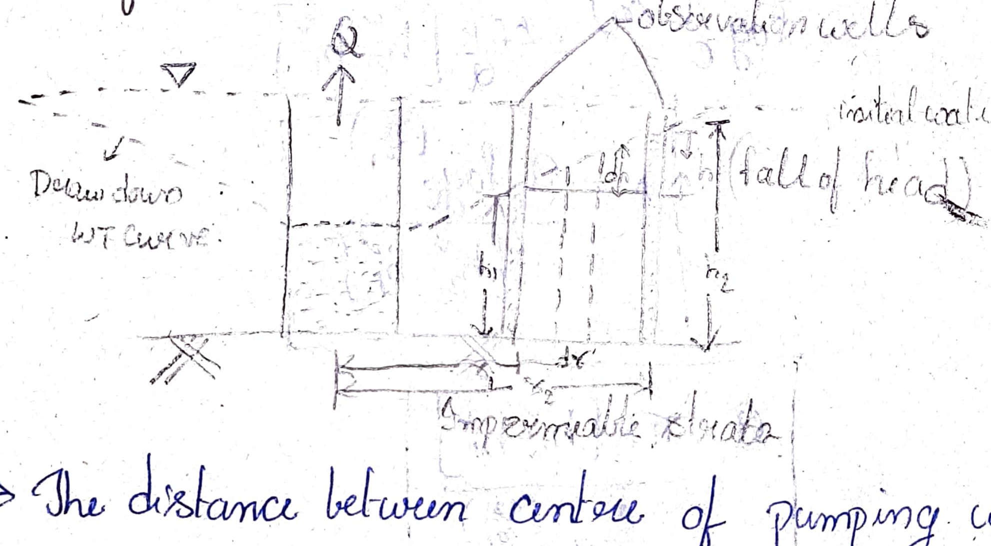
$$\boxed{K = 2.303 \times \frac{aL}{At} \times \log_{10} (h_1/h_2)}$$

27/9/16

Field Test:-

a = area of stand pipe

1) Unconfined well test:-



→ The distance between centre of pumping well to the point where initial water table and draw down curve (or) draw down water table coincide is called zone of Influence (or) Radius of Influence

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

$$Q = kiA$$

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

well

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

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

$$\log_e(r)\Big]_{r_2}^{r_1} = \frac{2\pi}{Q}k\cdot\frac{1}{2}\left[h^2\right]_{h_2}^{h_1}$$

$$\log_e(r_1) - \log_e(r_2) = \frac{2\pi}{Q}k\,\frac{1}{2}\left[h_1^2 - h_2^2\right]$$

multiply with minus

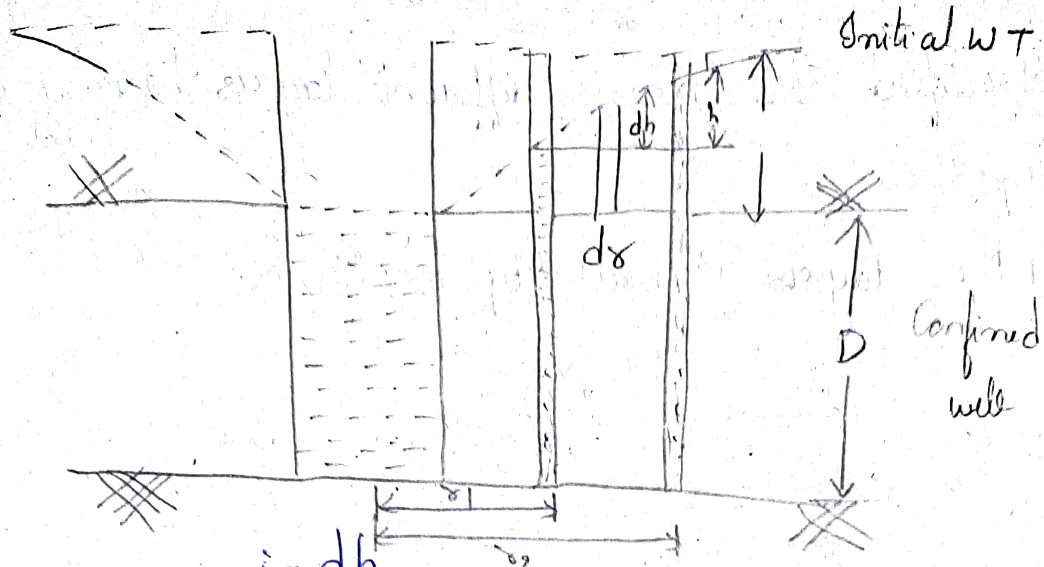
$$\log_e(r_2) - \log_e(r_1) = \frac{\cancel{2}\pi}{Q}k\cdot\frac{1}{\cancel{2}}\left[h_2^2 - h_1^2\right]$$

$$\log_e\left(\frac{r_2}{r_1}\right) = \frac{\cancel{2}\pi k}{Q}\left[h_2^2 - h_1^2\right]$$

$$\boxed{k = \frac{Q\cdot\log_e\left(\frac{r_2}{r_1}\right)}{\pi\left[h_2^2 - h_1^2\right]}}$$

$$\boxed{Q = \frac{\pi k\left[h_2^2 - h_1^2\right]}{\log_e\left(\frac{r_2}{r_1}\right)}}$$

2) Confined well test:- The water is in pressure between two impermeable strata.



$i = \frac{dh}{dr}$, $A = 2\pi r D$

From Darcy's law, $Q = KiA$

$$Q = K\frac{dh}{dr}\cdot 2\pi r D$$

$$\frac{dr}{r} = \frac{KDdh}{Q}\cdot 2\pi$$

$$\int_{r_2}^{r_1}\frac{dr}{r} = \frac{2\pi K D}{Q}\int_{h_2}^{h_1} dh$$

$$\log_e(r)\Big]_{r_2}^{r_1} = \frac{2\pi KD}{Q}\Big[h\Big]_{h_2}^{h_1}$$

$$\log_e(r_1) - \log_e(r_2) = \frac{2\pi KD}{Q}[h_1 - h_2]$$

multiply with minus

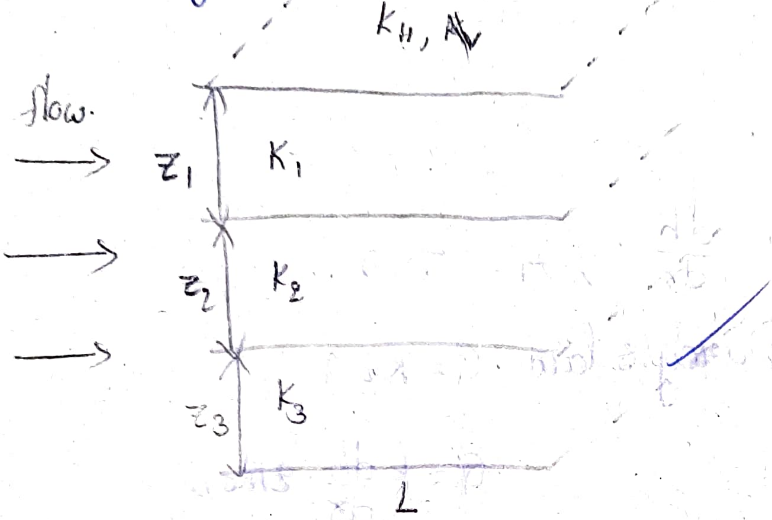
$$\log_e(r_2) - \log_e(r_1) = \frac{2\pi KD}{Q}[h_2 - h_1]$$

$$\boxed{K = \frac{Q\cdot \log_e(r_2/r_1)}{2\pi D[h_2 - h_1]}}$$

Permeability in Stratified Soils:-

→ Stratified Soils means different layers having different properties.

→ In this layers Permeability is K_H, K_V.



→ **Horizontal Flow**:- When flow takes place parallel to bedding planes.

$$Q_{eq} = q_1 + q_2 + q_3$$

$$i_{eq} = \frac{h}{L} = i_1 = i_2 = i_3 \qquad A = Z \times 1$$

From Darcy's law

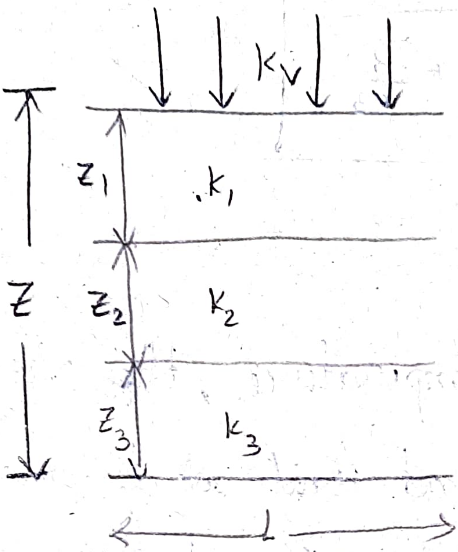
$$Q_{eq} = K_{eq}\, i_{eq}\, A_{eq}$$

$$K_H \cancel{i_{eq}} A_{eq} = K_1 \cancel{i_1} A_1 + K_2 \cancel{i_2} A_2 + K_3 \cancel{i_3} A_3$$

$$K_H (Z_1 + Z_2 + Z_3) \times 1 = K_1 Z_1 + K_2 Z_2 + K_3 Z_3$$

$$\boxed{K_H = \frac{K_1 Z_1 + K_2 Z_2 + K_3 Z_3}{Z}}$$

If flows in Vertical :- If the flow is ⊥er to bedding planes, it is called vertical flow.



$Q = q_1 = q_2 = q_3$

$i = \frac{h}{Z}, \quad i_1 = \frac{h_1}{Z_1}, \quad i_2 = \frac{h_2}{Z_2}, \quad i_3 = \frac{h_3}{Z_3}$

$h = iZ, \quad h_1 = i_1 Z_1, \quad h_2 = i_2 Z_2, \quad h_3 = i_3 Z_3$

Total head $(h) = h_1 + h_2 + h_3$

$h = i_1 Z_1 + i_2 Z_2 + i_3 Z_3$ $\quad [V = ki]$

$k_{eq} \cdot i_{eq} = k_1 i_1 = k_2 i_2 = k_3 i_3$

$k_v \cdot \frac{h}{Z} = k_1 i_1 = k_2 i_2 = k_3 i_3$

$k_v \frac{h}{Z} = k_1 \cdot i_1$

$$k_v = \frac{Z(k_1 i_1)}{i_1 Z_1 + i_2 Z_2 + i_3 Z_3}$$

$$k_v = \frac{Z}{\frac{\cancel{i_1} Z_1}{k_1 \cancel{i_1}} + \frac{i_2 Z_2}{k_1 i_1} + \frac{i_3 Z_3}{k_1 i_1}}$$

$$K_V = \frac{Z}{\frac{i Z_1}{K_1 i} + \frac{\cancel{i}_2 Z_2}{K_2 \cancel{i}_2} + \frac{\cancel{i}_3 Z_3}{K_3 \cancel{i}_3}}$$

$[K_1 i_1 = K_2 i_2 = K_3 i_3]$

$$\boxed{K_V = \frac{Z}{\frac{Z_1}{K_1} + \frac{Z_2}{K_2} + \frac{Z_3}{K_3}}}$$

Problem:-

1) Due to raise in temperature, the viscosity of percolating liquid and unit wt of percolating liquid are reduced to 75% and 90% respectively. If other things remains constant. The coefficient of permeability is.

Sol) Given

viscosity of percolating liquid reduced = 75%

Unit wt of liquid = 90%

$V = K \rho i$

$$K = C D_{10}^2 \frac{e^3}{1+e} \cdot \frac{\gamma_\omega}{\mu} = C D_{10}^2 \frac{e^3}{1+e} \cdot \frac{90}{75}$$

$K =$ ~~83%~~ reduced.

$$\frac{K_2}{K_1} = \frac{\gamma_2}{\gamma_1} \cdot \frac{\mu_1}{\mu_2}$$

$\gamma_2 = 0.9 \gamma_1$

$\mu_2 = 0.71 \mu_1$

$$\frac{K_2}{K_1} = 0.9 \times \frac{1}{0.75} = 1.2$$

$$K_2 = 1.2 K_1$$

$\rightarrow K \uparrow 20\%$

2) A drainage pipe is beneath the dam has clogged with sand, the coefficient of permeability is found to be 8 m/day. The average difference in head = 21.6 mts and it has been observed that there is a flow of 162 lt/day through the pipe. The pipe is 96.3 mts long and has c/s area 180 cm^2. What percentage of length of pipe is filled with sand.

Sol) From Darcy's Law

$$Q = kiA$$

$$\frac{162 \times 10^{-3}}{day} = 8 \times \frac{21.6}{L} \times 180 \times 10^{-4}$$

$$L = 19.2 \text{ mts}$$

$$\text{Percentage of Length of pipe} = \frac{19.2}{96.3} \times 100$$

$$= 19.93\% \simeq 20\%$$

$$\frac{K_2}{K} = \frac{\gamma_1}{\mu_1} \cdot \frac{\mu_2}{\gamma_2}$$

$\frac{0.75}{0.9}$

29/9/16

Effective Stress

1) Total Stress: $\dfrac{\text{Resisting force}}{\text{unit area}} = \dfrac{\text{load}}{\text{area}}$

weight of soil (w) = load

$$W = \gamma \times V$$

$$\text{stress} = \frac{\gamma \times V}{A}$$

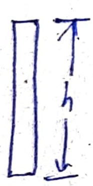
$$= \frac{\gamma \times \cancel{A} \times H}{\cancel{A}}$$

$$\boxed{\sigma = \gamma H}$$

2) Neutral stress (or) neutral pressure or pore pressure.

→ This neutral stress is due to liquid column or liquid weight.

Pore pressure (u) = Ht of liquid column × γ_w



$$u = h \times \gamma_w$$

→ It has no shear component.

→ In compaction we add water because by adding water in b/w the particles there is a lubricating effect occur and the particles settle and density increases.

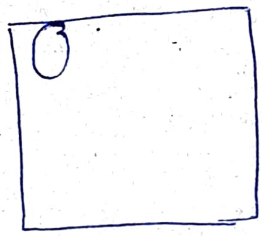
3) Effective Stress:

→ Load resisted due to contact between solid particles only.

→ Effective stress is also known as Inter granular

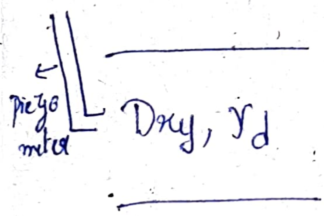
Pressure

→ Father of Soil mechanics Terzaghi.



$\sigma' = \sigma - U$

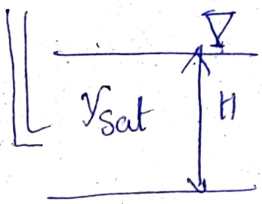
= Total stress - pore pressure



$\sigma = \gamma_d H$

$U = \gamma_w \cdot h = 0$ (there is not water column)

$\boxed{\sigma' = \sigma = \gamma_d H}$



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

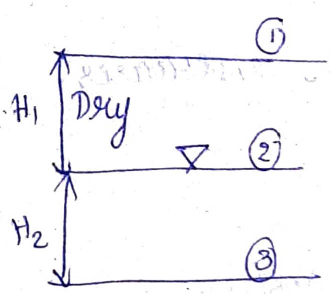
$U = \gamma_w H$

$\sigma' = \gamma_{sat} H - \gamma_w H$

$\sigma' = (\gamma_{sat} - \gamma_w) H$

$\boxed{\sigma' = \gamma' H}$

γ' - Submerged unit wt.



At plane ①

$\sigma = 0$

$U = 0$

$\boxed{\sigma' = 0}$

At plane ②

$\sigma = \gamma_d H_1$

$U = 0$

$\sigma' = \sigma - U$

$\boxed{\sigma' = \gamma_d H_1}$

At plane ③:

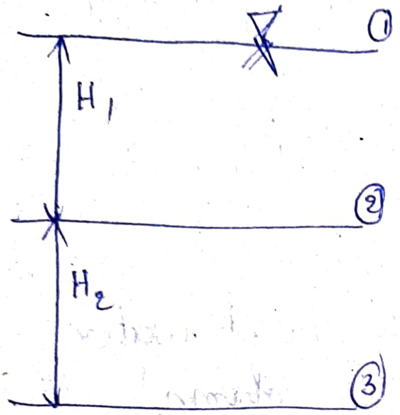
$\sigma = \gamma_d H_1 + \gamma_{sat} H_2$

$U = \gamma_w H_2$

$\sigma' = \gamma_d H_1 + \gamma_{sat} H_2 - \gamma_w H_2$

$= \gamma_d H_1 + [\gamma_{sat} - \gamma_w) H_2$

$$\boxed{\sigma' = \gamma_d H_1 + \gamma' H_2}$$



The water table is at plane ①

At plane ①

$\sigma = 0$

$U = 0$

$\sigma' = 0$

At plane ②

$\sigma = \gamma_{sat} \cdot H_1$

$U = \gamma_w H_1$

$\sigma' = \gamma_{sat} \cdot H_1 - \gamma_w H_1$

$\sigma' = \gamma' H_1$

At plane ③

$\sigma = \gamma_{sat} [H_1 + H_2]$

$U = \gamma_w [H_1 + H_2]$

$\sigma' = \sigma - U = \gamma' [H_1 + H_2]$

* If water table rises the effective stress decreases.
* If water table rises then total stress increases.

Capillary water:

→ Capillary is due to cohesion and adhesion

→ Surface tension is only due to cohesion

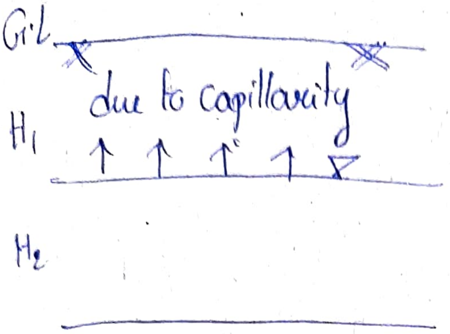
$$h_c = \frac{4\sigma \cos\theta}{\rho g d}$$

$\theta = 0°$ for water.

$\theta = 130°$ for mercury.

$h_c = \frac{0.3}{d}$, where d is dia of void in cm.

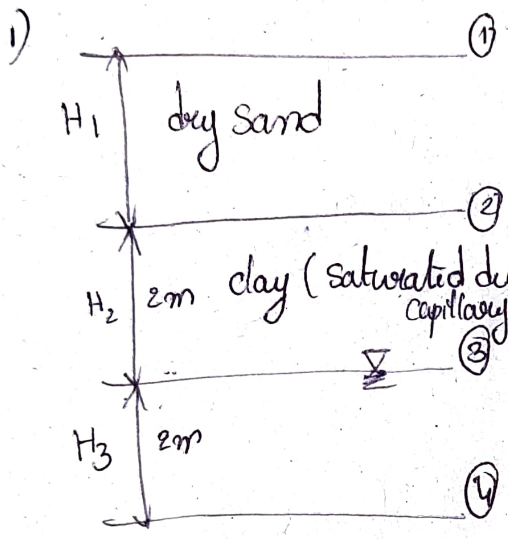
$d = (e D_{10}^3)^{1/3}$



→ Above water table the forces are tensile in nature.

→ At water table Atmospheric pressure is there.

1)



$G = 2.67$, $e = 0.3$
$\gamma_w = 10\ KN/m^3$

$G = 2.5$
$e = 0.4$

Find out effective stress at each plane?

Plane ①

$\sigma = 0$

$U = 0$

$\sigma' = 0$

Plane ② above:-

$\sigma = \gamma_d H = \gamma_d \times 2$

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

$= \frac{2.7 \times 10}{1+0.3} = 20.76$

$\sigma = 20.76 \times 2 = 41.52\ KN/m^2$

$U = 0$

$\sigma' = 41.52\ KN/m^2$

Plane ② just below:-

$\sigma = \gamma_d \times 2 = 41.52\ KN/m^2$

$U = -\gamma_w \times 2$

$= -10 \times 2 = -20\ KN/m^2$

$\sigma' = 41.52 - (-20)$

$= 61.52\ KN/m^2$

Plane ③

$\sigma = \gamma_d \times 2 + \gamma_{sat} \times 2$

$\gamma_{sat} = \frac{G+e}{1+e} \gamma_w = \frac{2.5+0.4}{1+0.4} = 20.71\ KN/m^2$

$\sigma = (20.76 \times 2) + (20.71 \times 2)$

$= 82.84 \ kN/m^2$

$U = 0$

$\sigma' = 82.84 \ kN/m^2$

Plane ④

$\sigma = \gamma_d \times 2 + \gamma_{sat} \times 4$

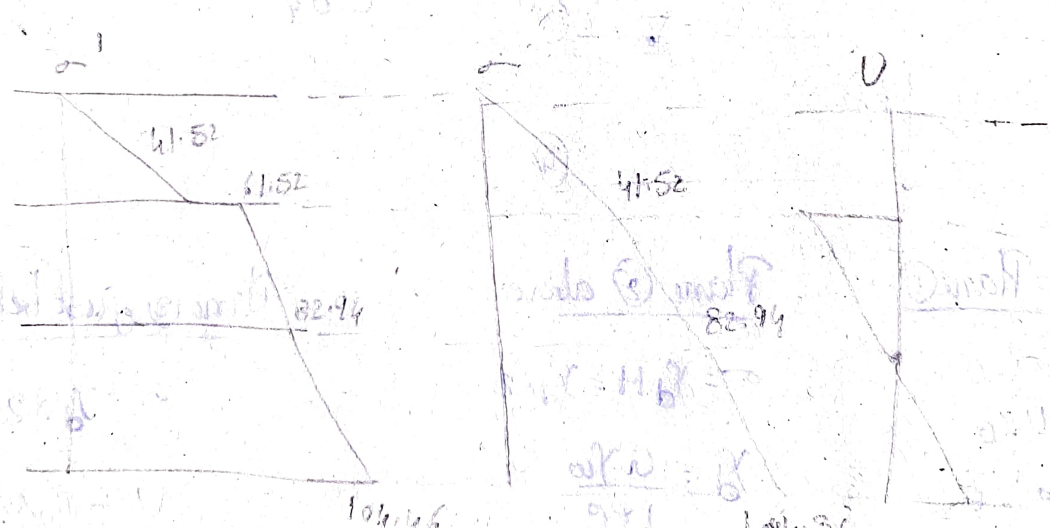
$= 20.76 \times 2 + (20.71 \times 4) = 124.36 \ kN/m^2$

$U = \gamma_w \times h$

$= 10 \times 2 = 20$

$\sigma' = 104.36 \ kN/m^2$

Dia



i) No flow Condition :-

At plane A

$\sigma = \gamma_w H_1$

$U = \gamma_w h = \gamma_w H_1$

$\sigma' = \sigma - U = 0$

$\sigma' = 0$

At B :-

$\sigma = \gamma_w H_1 + \gamma_{sat} H$

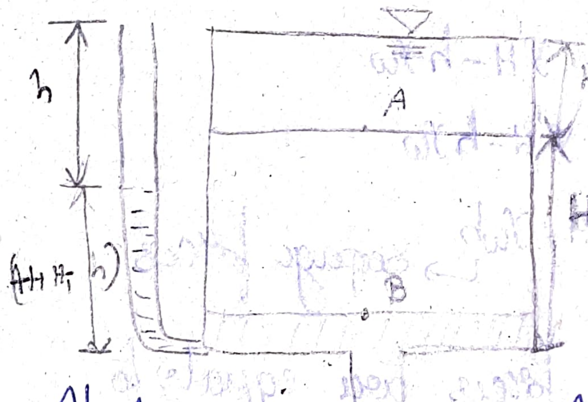
$U = (H + H_1)\gamma_w$

$\sigma' = \sigma - U = \cancel{\gamma_w H_1} + \gamma_{sat} H - H\gamma_w - \cancel{\gamma_w H_1}$

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

$\boxed{\sigma' = \gamma' H}$

2) Downward flow Condition :-



because of resisting force due to soil grains there is head loss occur.

At A :-

$\sigma = \gamma_w H_1$

$U = \gamma_w H_1$

$\sigma' = 0$

At B

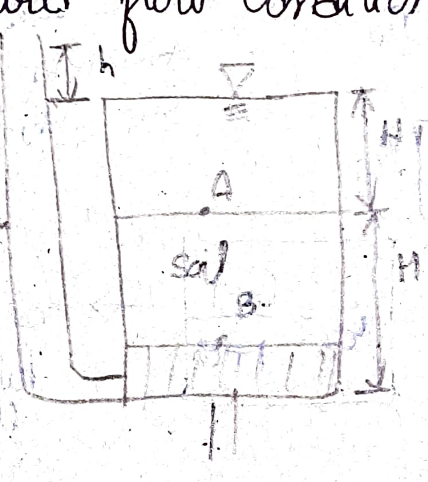
$\sigma = \gamma_w H_1 + \gamma_{sat} H$

$U = [H_1 + H - h]\gamma_w$

$\sigma' = \gamma_w H_1 + \gamma_{sat} H - H_1\gamma_w - H\gamma_w + h\gamma_w$

$\boxed{\sigma' = \gamma' H + \gamma_w h.}$

3) Upward flow Condition :-



At A :-

$\sigma = H_1\gamma_w$, $U = H_1\gamma_w$

$\sigma' = 0$

At B :-

$\sigma = H_1\gamma_w + \gamma_{sat} H$

$U = (H + H_1 + h)\gamma_w$

$\sigma' = \cancel{H\gamma_w} + \gamma_{sat}H - H\gamma_w - \cancel{H\gamma_w} - h\gamma_w$

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

No flow $\Rightarrow \sigma' = \gamma' z$

Downward flow $\Rightarrow \sigma' = \gamma' z + h\gamma_w$

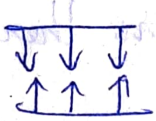
Upward flow $\Rightarrow \sigma' = \gamma' z - h\gamma_w$

* In downward flow effective stress increases.
* In upward flow effective stress decreases.

Quick Sand Condition:-

Inward direction flow $(\sigma') = \gamma' H - h\gamma_w$

$0 = \gamma' H - h\gamma_w$



$\gamma' H = \underbrace{\gamma_w h}_{\text{Seepage forces}}$

when the upward seepage forces are equals to downwards gravitational forces then effective stress is zero. In such situation Quick sand occurs.

→ Quick sand is not a special type of sand it is just a hydraulic condition.

Soil that experience quick sand condition would loss shear strength & Bearing capacity of soil.

$$i = \frac{h}{H} = \frac{\text{head loss}}{\text{Seepage height}} = \frac{\gamma'}{\gamma_w}$$

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

$$\boxed{\frac{\gamma'}{\gamma_w} = \frac{G-1}{1+e}} \qquad \boxed{i_{cr} = \frac{G-1}{1+e}} \rightarrow i_{cr} = \text{Critical hydraulic gradient}$$

→ $i \geq i_{cr}$ (Quick sand condition occurs)

Factor of safety at quick sand = $\frac{i_{cr}}{i}$

$i < i_{cr}$ $\qquad i_{cr} = 1 \qquad FOS = 4$

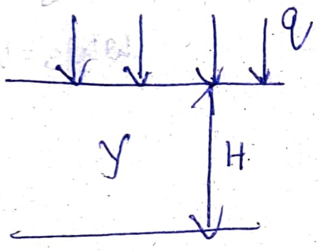
$i = 0.25 \times 4 = i_{cr}$

* Quick sand condition occurs in sand, If $\sigma' = 0$ the soil losses all the strength.

How to prevent Quick sand condition:-

→ lowering of water table in surrounding soil.

→ when making trench and keep some water in trench



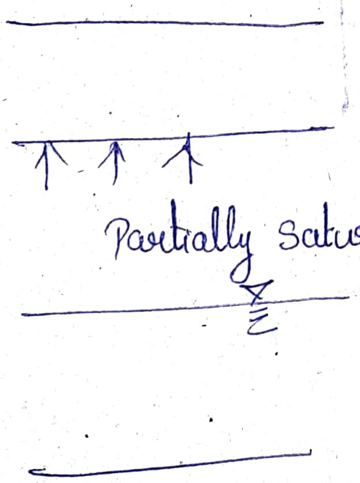
$\sigma = \gamma H + q$

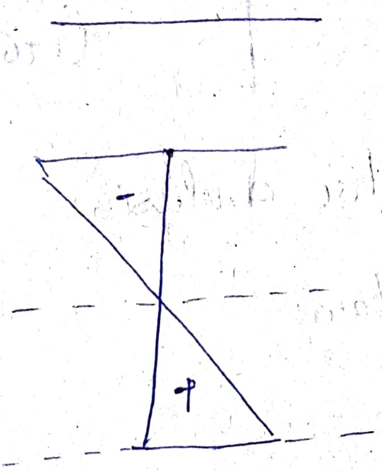
$U = h\gamma_w$

$\sigma' = \gamma H + q - h\gamma_w$

Flocculation of particles of cohesive soil soil. Due to vertical seepage flow.

Capillarity in partially saturated condition:-





30/9/16

Relation between b/w γ_d, n_a, G, w :-

$$V = V_s + V_w + V_a \qquad \left[n_a = \frac{V_a}{V}\right]$$

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

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

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

$$1 - n_a = \frac{W_s}{G_s \gamma_w V} + \frac{w \cdot W_s}{\gamma_w V}$$

$$1 - n_a = \frac{\gamma_d}{G_s \gamma_w} + \frac{\gamma_d w}{\gamma_w}$$

$$1 - n_a = \frac{\gamma_d}{\gamma_w}\left[\frac{1}{G_s} + w\right]$$

$$1 - n_a = \frac{\gamma_d}{\gamma_w}\left[\frac{1 + G_s w}{G_s}\right]$$

$$\boxed{\gamma_d = \frac{(1 - n_a)\gamma_w G_s}{(1 + G_s w)}}$$

$$\frac{W_s}{V_s} = \gamma_s$$

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

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

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

$$w W_s = W_w$$

Hydrometer Analysis :-

Stoke's Law :-

Gravity Force $= W = \gamma_S V$ $\qquad V = \frac{\pi}{6}D^3 / \frac{4}{3}\pi r^3$

$$= G_S \gamma_w \cdot \frac{\pi}{6} D^3$$

Buoyancy Force $= F_B = V \cdot \gamma_w$

$$= \frac{\pi}{6} D^3 \gamma_w$$

Drag Force $= F_D = 3\pi D \mu V_S$

$$F_g = F_B + F_D$$

$$F_g - F_B = F_D$$

$$G_S \gamma_w \frac{\pi}{6} D^{\cancel{3}\,2} - \frac{\pi}{6} D^3 \gamma_w = 3\cancel{\pi}\cancel{D}\mu V_S$$

$$\frac{\cancel{\pi}}{6} \cancel{D}^{\cancel{3}\,2} \gamma_w [G_S - 1] = 3\cancel{\pi}\cancel{D}\mu V_S$$

$\gamma_w =$

$$V_S = \frac{(G_S - 1)\gamma_w D^2}{18\mu}$$

$$V_S = \frac{[G_S - 1] \cdot g \cdot \rho_w D^2}{18\mu} \qquad \boxed{\nu = \mu/\rho}$$

$$V_S = \frac{[G_S - 1] g D^2}{18\,\mu/\rho} = \frac{g D^2 [G_S - 1]}{18\nu}$$

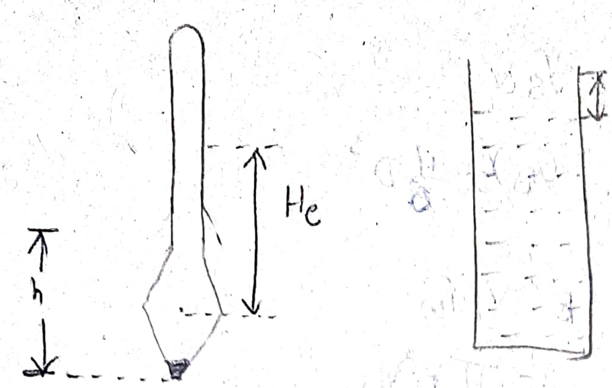
$$\boxed{V_S = \frac{g D^2 [G_S - 1]}{18\nu}}$$

Effective height (H_e):

H_e = distance between centre of hydrometer bulb to Hydrometer reading [R_H].

V_H = Volume of hydrometer bulb

A_j = Area of jar.



$V_H = A_j \times H$

$H = \frac{V_H}{A_j}$

$H_1 + h/2$

$H_e = \left(H_1 + \frac{h}{2} + \frac{V_H}{2A_j}\right) - \frac{V_H}{A_j}$

$= H_1 + \frac{1}{2}\{h + V_H/A_j\}$

$V_H = A_j \times H$

$H = \frac{V_H}{A_j} \Rightarrow \frac{H}{2} = \frac{V_H}{2A_j}$

effective height (H_e) $= H_1 +$ distance b/w neck to centre of hydrometer bulb

$$H_e = H_1 + \frac{h}{2} - \frac{V_H}{2A_j}$$

$$\boxed{H_e = H_1 + \frac{1}{2}\left[h - \frac{V_H}{A_j}\right]}$$

Diameter of particle :-

At $H_e, t \Rightarrow V_S = \frac{H_e}{t}$

Settling velocity (V_S) $= \frac{g d^2 (G-1)}{18\nu}$

$$\cancel{d = \sqrt{\frac{18\nu}{g(G-1)}}}$$

$$\frac{g d^2 (G-1)}{18\nu} = \frac{H_e}{t}$$

$$\boxed{d = \sqrt{\frac{18\nu H_e}{g(G-1)t}}}$$

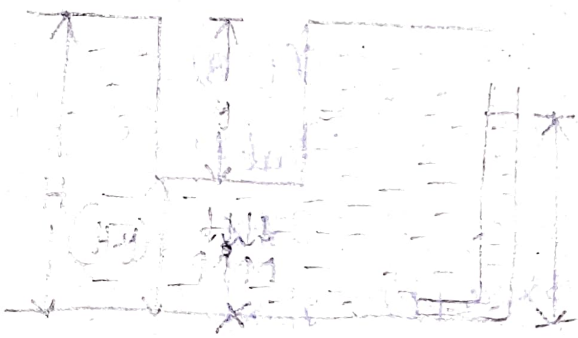
20/9/16

Frost Heave :- If the temperature is below freezing point water which is present in pore spaces freezes which leads to increase in volume. If there is any structure above that it exerts uplift forces. This is called Frost Heave. Soil - fine sands and silts.

Frost Boil :- Softening of soil is due to increase in water content because of melting or thawing of ice is called frost boil.

soil - Fine sands and silts.

Excavated soil :-



At critical condition uplift force is equal to doward force

$$\sigma = \gamma(H-y)$$

$$U = \gamma_w h$$

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

at critical condition, $\sigma' = 0$

$$\boxed{\gamma(H-y) = h\gamma_w}$$

1) Submerged unit weight of soil is 11 KN/m^3. Quick Sand Condition occured when excavating the soil reaches 4.2 mts : If the total depth of the soil is 6 mts. How much should be the water table be lowered or reduced to perform excavation upto 5 mts.

sol) $\gamma' = \gamma_{sat} - \gamma_w = 11\ KN/m^3$, $\gamma_w = 10\ KN/m^3$

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

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

y (depth of excavation) = 4.2 mts

$H = 6$ mts

$\gamma(H-h) = \gamma_w h$

$\Rightarrow \frac{\gamma'}{\gamma_w} = \left[\frac{G-1}{1+e}\right]$ Quick sand Condition.

total stress $(\sigma) = \gamma(H-y)$

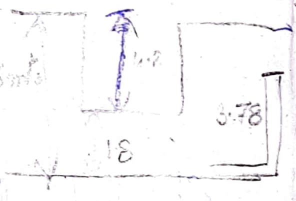
$= \gamma(1.8)$

$= \gamma_{sat} \times 1.8$

$\gamma_{sat} = 11+10$

$= 21$

$\sigma = 37.8$

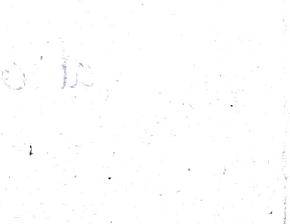


Pore pressure $(u) = \gamma_w \times h$

$= 10h$

Effective stress $\sigma' = 0$

$\sigma = u$



$37.8 = 10h$

$\boxed{h = 3.78}$

total stress $(\sigma) = \gamma_{sat}(H-y)$

$= 21 \times 1 = 21\ KN/m^3$

$U = \gamma_w \times h = 10 \times h$

$\sigma' = 0$

$\boxed{h = 2.1 m}$

Total water table be lowered is $3.78 - 2.1 = \underline{1.68 mts}$

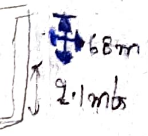
Piping:- Because of effective stress being zero on downstream side (or) exit gradient (i_{exit}) on downstream side exceeds critical gradient than erosion of soil particles occurs on downstream side because of seepage water. If this process occurs continuously, erosion of soil particles continuous to upstream side forming a pipe like structure. This phenomena is called Piping.

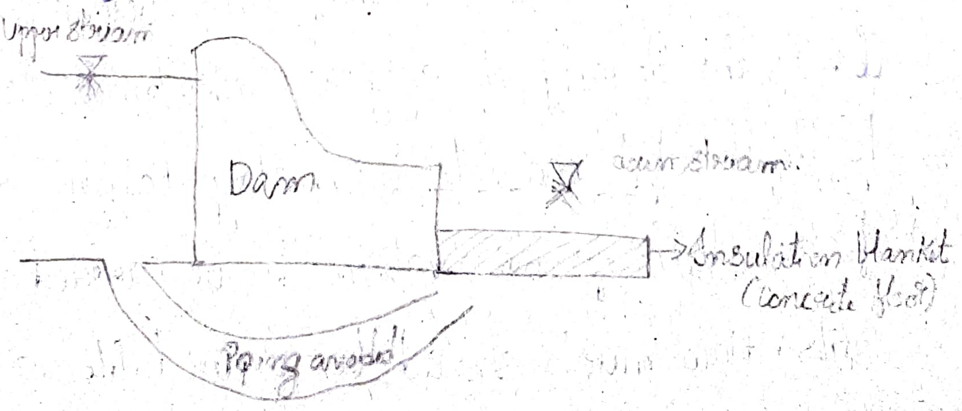
Loses of piping:-

1) Stability of Structure reduces due to piping.
2) Seepage loses will be more.

Prevention of Piping:-

→ Insulation blanket (Concrete floor) is provided to avoid piping.





-> By providing inverted filter, we can reduce pressure at downstream side.

-> Increasing seepage length by providing sheet piles

inverted filter.

Seepage pressure (P_s) :-

Pressure due to flowing fluid.

$$P_s = h \cdot \gamma_w$$

$$= iz \cdot \gamma_w$$

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

$$h = iL$$

$$\text{Seepage force} = P_s \times A$$

$$= iz \cdot \gamma_w \cdot A$$

$$= i \gamma_w V$$

$$\text{Seepage force / unit volume} = i \cdot \gamma_w$$

* Seepage pressure always acts towards the flow of fluid.
* Seepage force is downwards effective stress increases.
* Seepage force is upwards effective stress decreases.

2) A 9mts thick clay layer underlain by a bed of sand. Watertable is observed 3mts below the ground level. If the density of clay is 20 kN/m^3 a) Calculate safe depth of excavation in clay layer.

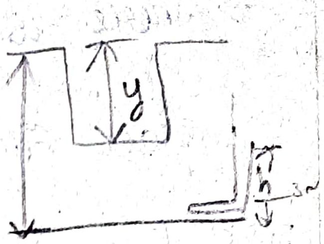
b) If the depth of excavation to be carried out upto 7mts. How much should be water table is reduced?

$\gamma_w = 10$ kN/m^3

sol) $H = 9$ mts

$y = ?$

$h = 6$ mts



$$\gamma(H-y) = \gamma_w h$$

$$20(9-y) = 10\times 6$$

$$180 - 20y = 60$$

$$120 = 20y$$

$$\boxed{y = 6.0 \text{ mts}}$$

b) $y = 7$ mts

$H = 9$ mts

$$20(9-7) = 10\times h$$

$$40 = 10h$$

$$h = 4 \text{ mts}$$

water table is reduced to 2mts $[6-4] = 2$mts

21/10/16

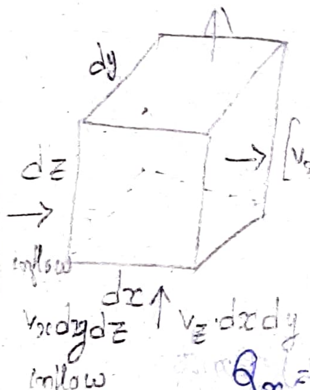
Seepage Analysis

↳ flow of water through pervious media.

→ There is an 2 Dimensional flow of Laplace Equation.

Laplace Equation assumptions for 2D-flow.

1) Soil under consideration is fully saturated.
2) Soil is homogeneous and isotropic.
3) During flow neither the soil grains nor the pore fluid is compressible.
4) Flow doesn't change with time i.e Steady State flow.



Velocity in x-direction $(V_x) = K_x i_x$

Velocity in z-direction $(V_z) = K_z i_y$

Velocity in y-direction $(V_y) = 0$

$Q_x = K_x i_x (dz\,dy)$

$Q_z = K_z i_z (dx\,dy)$

$Q_y = 0$

Inflow = Outflow

$$V_x\,dy\,dz + V_z\,dx\,dy = \left[V_x + \frac{\partial V_x}{\partial x}.dx\right]dy\,dz + \left[V_z + \frac{\partial V_z}{\partial z}dz\right]dx\,dy$$

$$\cancel{V_x\,dy\,dz} + \cancel{V_z\,dx\,dy} = \cancel{V_x\,dy\,dz} + \cancel{V_z\,dx\,dy} + \frac{\partial V_x}{\partial x}dx\,dy\,dz + \frac{\partial V_z}{\partial z}dz\,dx\,dy.$$

$$\frac{\partial V_x}{\partial x} dx dy dz + \frac{\partial V_z}{\partial z} dx dy dz = 0$$

$$\left[\frac{\partial V_x}{\partial x} + \frac{\partial V_z}{\partial z}\right] = 0$$

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

$$V_z = K_z i_z = K_z \cdot \frac{\partial h}{\partial z}$$

$$\Rightarrow K_x \frac{\partial^2 h}{\partial x^2} + K_z \frac{\partial^2 h}{\partial z^2} = 0$$

If soil is isotropic so $K_x = K_z = K$

$$K\left[\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial z^2}\right] = 0$$

$$\boxed{\left[\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial z^2}\right] = 0}$$ Laplace equation for 2D-flow.

The solution of Laplace equation for 2D-flow yields two set of curves.

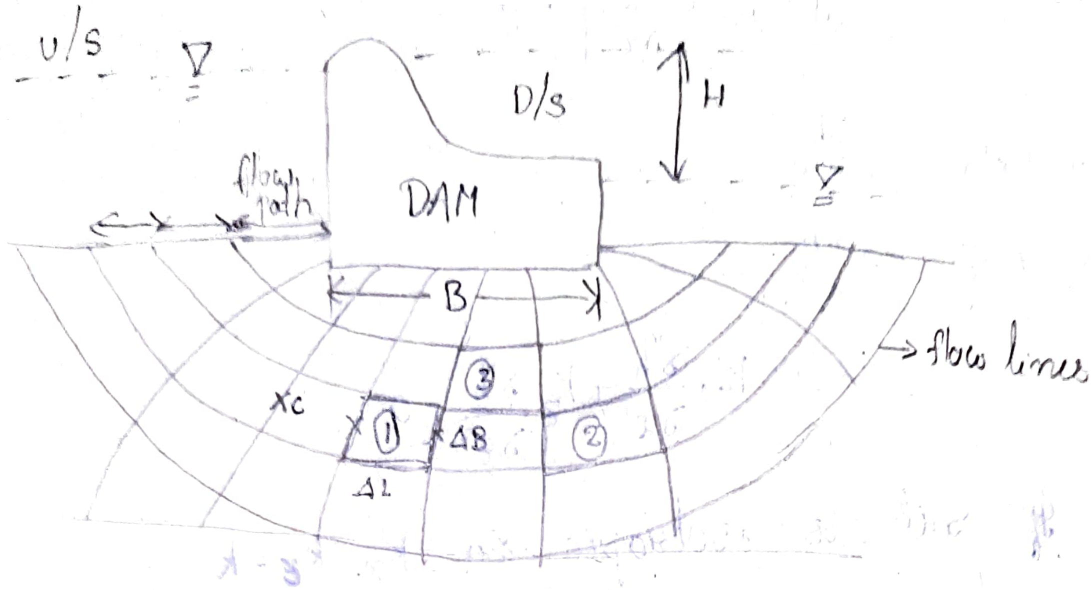
1) flow line
2) Equipotential line

Flow line:- The path assumed by a fluid particle during its flow in pervious medium.

Equipotential line: The line joining the points of equal and total heads.

* Equipotential and flow lines are $\perp$er to each other and except at stagnation point.

Flow net :- Diagram representing Combination of flow lines and equipotential lines.



* The slopes of equipotential and flow line is -1.

Flow Channel (or) Flow path :- The area between two flow lines is called flow path / channel.

Flow field :- The area between intersection of two adjacent flow lines and equipotential lines.

Properties of Flow net :-

1) The flow lines and equipotential lines are ⊥er.
2) Two adjacent flow lines will never intersect each other.
3) Flow field assumed to be square approximately Sometime rectangle also.
4) Two adjacent equipotential lines will never intersect each other.
5) Flownet is drawn for particular set of boundary

Conditions.

6) Flownet doesn't depend upon head causing flow. 7) On equipotential line at any point total head is same.

8) Between two adjacent equipotential lines head drop is constant.

Equation for Seepage loss:-

$$Q = KiA = K.\frac{\Delta h}{\Delta L}A$$

$$\Delta h = \frac{H}{N_D}$$ (no. of head drop)

For ① flow field

$$q_1 = K.\frac{H}{N_D \Delta L}.\Delta B \times 1$$ [for unit thickness]

$$q_2 = K.\frac{H}{N_D \Delta L}\Delta B \times 1$$

$$q_1 = q_2 = q$$

$$q_3 = K.\frac{H}{N_D \Delta L}.\Delta B \times 1$$

$$q_1 = q_2 = q_3 = q = K.\frac{H}{N_D \Delta L}\Delta B$$

Total discharge in flow field

$$\boxed{q = K.H.\frac{N_f}{N_D}.\frac{\Delta B}{\Delta L}}$$ No. of flow fields (N_f)

*flow field area is square, then $\boxed{q = K.H\frac{N_f}{N_D}}$

* For Rectangle, $\boxed{q = K \cdot \frac{N_f}{N_D} \cdot \frac{\Delta B}{\Delta L} H}$

Seepage Pressure:

$$P_s = h\,\gamma_w$$

$$P_s = (H - n_D \cdot \Delta h) \cdot \gamma_w$$

↳ upto that point (c)

$$P_s = \left(H - 2 \cdot \frac{H}{N_D}\right) \gamma_w$$

$N_D = 2$ drops upto point c

$$\boxed{P_s = \left[H - \frac{H}{N_D} n_d\right] \gamma_w}$$

Exit Gradient:

$$i_{exit} = \frac{\Delta h}{\Delta L}$$

* If exit gradient is greater than critical gradient than Quick sand and piping condition.

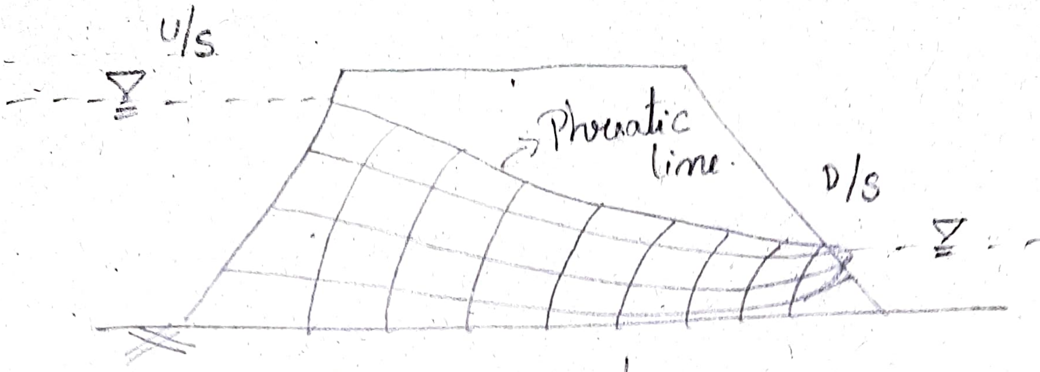
→ If the soil is anisotropic than reduced dimension of dam is

$$b = B\sqrt{\frac{K_z}{K_x}} \quad [K_z < K_x]$$

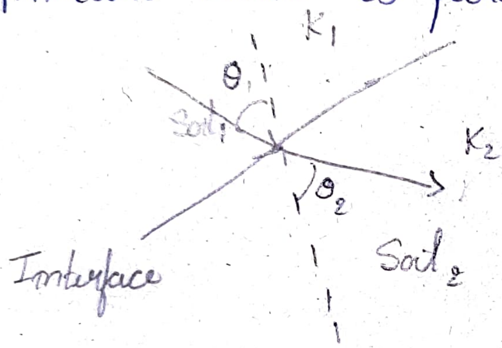
$$K = \sqrt{K_x \cdot K_z}$$

→ In the case of Anisotropic soil media, the horizontal dimension is reduced by multiplying with a reduction Coefficient $\sqrt{\frac{K_z}{K_x}}$.

Seepage loss in Eartheern dams:-



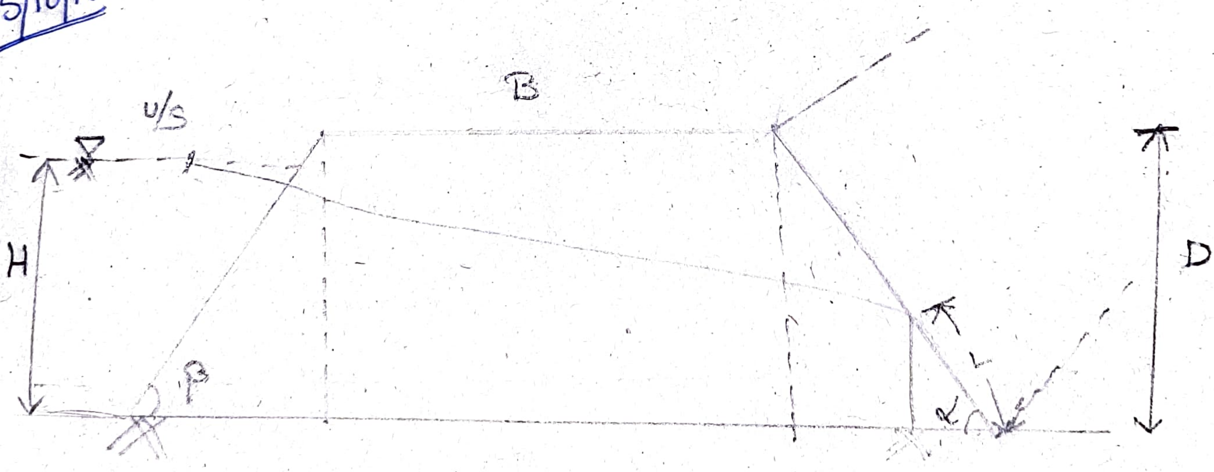
→ Phreatic lines is parabolic is shape.



By a relation

$$\boxed{\frac{\tan\theta_1}{\tan\theta_2} = \frac{K_1}{K_2}}$$

25/10/16



$$0.3\Delta + \frac{D-H}{\tan\beta} + B + \frac{D}{\tan\alpha}$$

Compaction

Compaction: Reducing the volume of soil by reducing volume of voids by the application of Mechanical Energy.

Consolidation: Expellition of pore water to reduce of volume of soil at constant load or gradual or static load.

Uses of Compaction:- 2.495 kg 2.6 kg

1) Increasing properties - density, Safe baring Capacity (SBC), shear strength.

2) Decreasing properties - Permeability, Volume, Compressibility, Swelling & Shrinkage.

Laboratory Procedures:- IS 2720 (Part-7)-1974 for Light compaction

1) Standard Proctor test (SPT):	IS equivalent to SPT:
Volume = 944 cm^3	Volume = 1000 cm^3
No. of layers = 3	No. of layers = 3
No. of blows = 25	No. of blows = 25
Weight of Hammer = 2.495 kg	Wt of Hammer = 2.6 kg
Height of fall = 30.48 cm.	Height of fall = 31 cm.

60.65

2) Modified Proctor test (MPT):-	IS equivalent to MPT:-
Volume = 944 cm^3	Volume = 1000 cm^3
No. of blows = 25	No. of layers = 5
No. of layers = 5	No. of blows = 25

275.625

developed for a scale 1:25 scale factor. The reduced base length of the dam will be?

Sol) Given

$K_x = 3.46\ m/day$

$K_y = 1.5\ m/day$

Base length $(B) = 100\ mts$

Scale factor $= \frac{1}{25}$

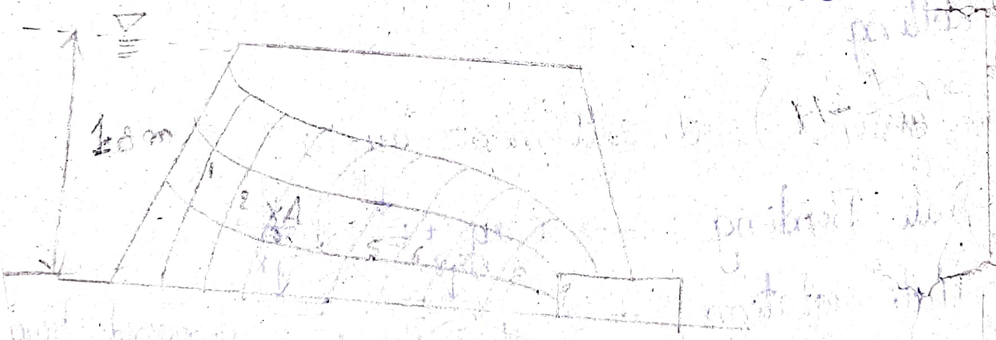
Reduced length $(b) = B\sqrt{\frac{K_y}{K_x}}$

$= 100\sqrt{\frac{1.5}{3.46}}$

$b = 65.84\ m.$

base length in flow net $= 65.84 \times \frac{1}{25}$

$= 2.63\ m.$



Calculate Hydraulic potential At "A"?

Hydraulic potential $(H_p) = H - (\Delta h . n_d)$

$\Delta h = \frac{H}{N_d} = \frac{1.8}{9} = 0.2$

$H_p = 1.8 - (0.2 \times 3) = 1.2\ m.$

29/10/16

Compressibility & Consolidation.

Three Important Engineering properties.

1) Permeability - to calculate seepage loss.
2) Compressibility & Consolidation - used in calculation of Settlement.
3) Shear Strength - used in stability related problems

→ due to Compression air from voids - Compaction.

→ Expulsion of pore water - Consolidation.

Coarse grained soil Settlement due to

2) De Elastic deformation

Sliding

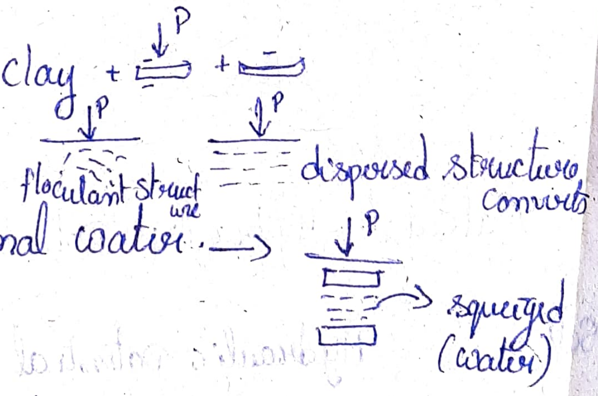
La Rolling

1) S grained soil settlement due to

Plate Bending →

2) Plate rotation →

3) Squeezing of Additional water →



Settlements :- These are divided into 3 categories.

1) Elastic Settlement (or) immediate Settlement :-

It occurs due to elastic deformation of soil grains. Settlement calculations are based on

elastic theory.

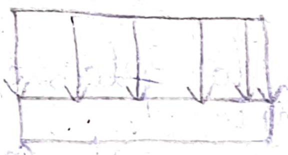
→ Elastic settlement depends on rigidity of footing and type of soil which is it is resting.

Rigid Pavement

→ clay :-

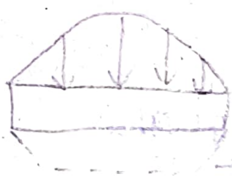
Flexible pavement

→ clay.



Rigid pavement

→ Sand



Flexible pavement

→ Sand.

$$\text{Elastic Settlement } (S_e) = \frac{qBI(1-\nu^2)}{E}$$

where, q - net intensity of pressure

B - least lateral dimension of f

Depends on type of footing, shape & $\frac{L}{B}$ ratio. ← I - Influence factor.

ν - poisson's ratio

E = Elastic modulus.

$(I_f)_{\text{Rigid footing}} = 0.8 \times (I_f)_{\text{flexible footing}}$

2) <u>Primary Consolidation Settelement</u>:-

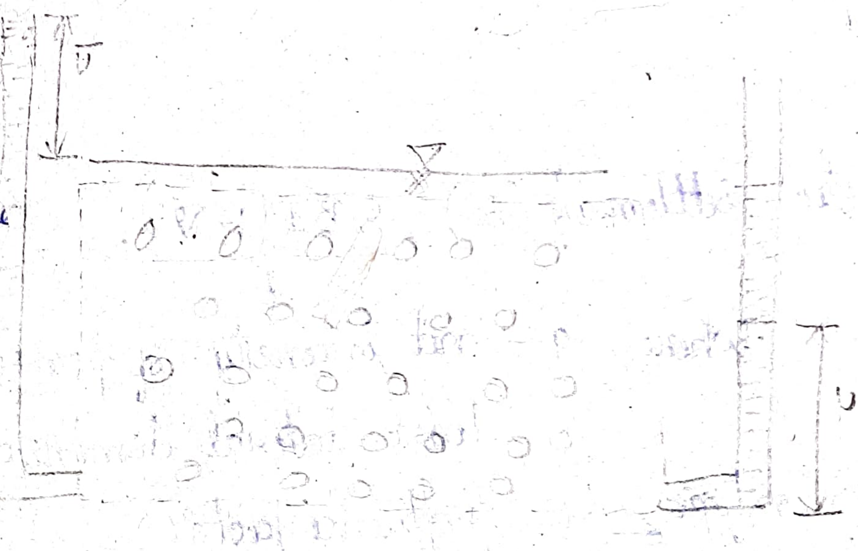
Primary Consolidation Settelement, it is due to expellusion of pore water under static and long term loading.

3) <u>Secondary Consolidation Settelement</u>:-

It is due to plastic rearrangement of particles under static long term load at constant effective stress.

* Consolidation Theory - <u>Terzaghi</u>.

→ This theory of Consolidation is applicable for saturated soils.



$\bar{U}$ - excess pore water

a) the footing is flexible - $I_f = 1.36$ for L/B of 1.5

b) the footing is rigid - $I_f =$

Sol)

$$S_e = \frac{qBI(1-\nu^2)}{E}$$

Pressure = 100 kN/m^2 (q)

$$\left\{ \begin{array}{l} \frac{F}{6} = 100\ kN/m^2 \\ \text{Force } (F) = 600\ kN \end{array} \right\} \times$$

$\Rightarrow$ B (least lateral dimension of footing) = 2 mts

a)

$$S_e = \frac{100 \times 2 \times 1.36\,(1-(0.5)^2)}{5 \times 10^4}$$

$$S_e = 4.08 \times 10^{-3} \text{ mts} = 4.08\ mm$$

b) For rigid pavements

$$(I_f)_{\text{rigid footing}} = 0.8 \times (I_f)_{\text{flexible footing}}$$

$$I_f = 0.8 \times 1.36$$

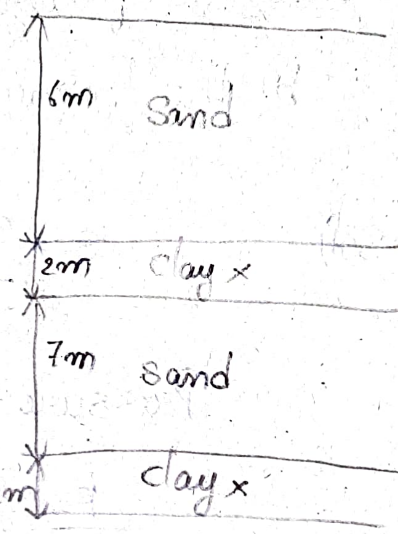
$$I_f = 1.088$$

$$S_e = \frac{100 \times 2 \times 1.088\,(1-(0.5)^2)}{5 \times 10^4}$$

$$S_e = 3.2\ mm.$$

1/11/16

2



3 1st layer:-

$\sigma_0' = \gamma'_{sand} \times 6m + \gamma'_{clay} \times 1$

$= (2-1)\times 6 + (1.82-1)\times 1$

$= 6.82 \; g/cm^2 = 68.2 \; KN/m^2$

$\Delta\sigma' = 400 \; KN/m^2, \; C_c = 0.5, \; e_0 = 1.08$

$$S_i = \frac{0.5}{1+1.08} \times 2 \times \log\left[\frac{400+68.2}{68.2}\right]$$

$= 0.4 \; m = 40 \; cm.$

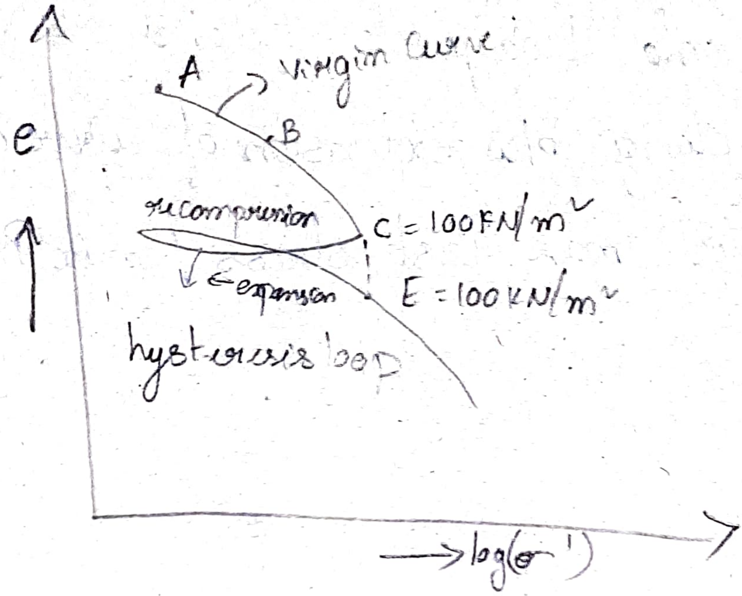
2nd layer:- $\sigma_0' = 6\times(2-1) + 2(1.82-1) + 7(2-1) + 1(1.82-1)$

$= 15.4 \; g/cc = 154 \; KN/m^2$

$\Delta\sigma_0' = 400 \; KN/m^2$

$S_1 = 26 \; cm.$

Total settlement $= 66$ cm.



→ If the soil undergoes consolidation for the first time then (e vs log σ') curve is called virgin compressive curve.

→ Due to unloading effective stress decreases and soil tends to gain its initial properties but cannot reach initial properties.

→ If I apply load again after unloading (e vs log σ') curve cannot follow its initial path.

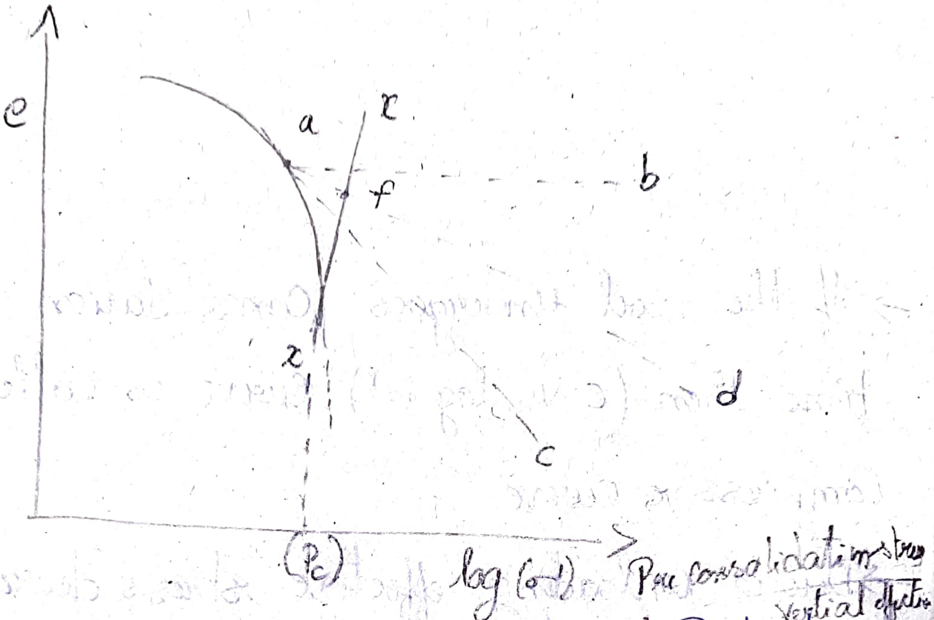
→ In reloading / recompression, if the effective stress reach initial max past stress beyond that point soil turns to virgin compression.

Casagrande method to find out maximum past stress:

→ e vs log σ' curve, take a point on which radius of curvature is minimum. Draw a horizontal line at that point and draw a tangent at same point.

→ Divides into two parts. Extend the curve. The point of intersecting b/w extension of curve and ad line.

2. It is called max Past stress / max Pre Consolidation stress.



Over Consolidation Ratio (OCR) = $\frac{\text{Max. Past stress}}{\text{Present overburden pressure}}$

OCR > 1 ⇒ Over Consolidated soils / pre,

OCR = 1 ⇒ Normally Consolidated

OCR < 1 ⇒ Under Consolidated.

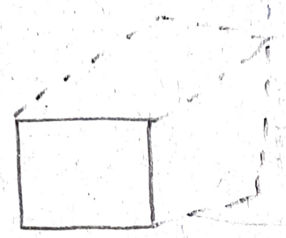
~~Two~~ One Dimensionless Parameters of Terzaghi:-

Assumptions:-

1) Soil is homogeneous.
2) Soil is Saturated.
3) Infinitely Small Area represents Some properties as entire Soil.
4) Flow is 1-D.
5) 1-D Compression is Valid.

6) Darcy's law is valid.

7) Soil grains and water are incompressible.



Seepage equation

$$\left[k_x \frac{\partial^2 h}{\partial x^2} + k_z \frac{\partial^2 h}{\partial z^2}\right] dx\,dy\,dz = 0$$

$$\left(k \cdot \frac{\partial^2 h}{\partial z^2}\right) dx\,dy\,dz = 0$$

For Consolidation

$$\left(k \cdot \frac{\partial^2 h}{\partial z^2}\right) dx\,dy\,dz = \frac{-dv}{dt}$$

From pore pressure $(u) = h\,\gamma_w$

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

$$\frac{k \cdot \partial^2 (u/\gamma_w)}{\partial z^2} \cdot V_0 = \frac{-dv}{dt}$$

$$\frac{k}{\gamma_w} \frac{\partial^2 u}{\partial z^2} = \frac{1}{V_0} \cdot \frac{-dv}{dt}$$

$V_{soil} = V_s + V_v$

$V_v = e$

$$= \frac{-1}{V_0} \cdot \frac{dv_v}{dt}$$

$$= \frac{-1}{V_0} \cdot \frac{de}{dt}$$

$$= \frac{-1}{V_0} \frac{de}{d\sigma'} \cdot \frac{d\sigma'}{dt}$$

$$\frac{k}{\gamma_w} \cdot \frac{\partial^2 u}{\partial z^2} = \frac{-1}{1+e} \cdot a_v \cdot \frac{d\sigma'}{dt}$$

$$\frac{k}{\gamma_w}\frac{d^2u}{dz^2} = \frac{+a_v}{1+e_0}\cdot\frac{d\sigma'}{dt}$$

$$\left[\frac{d\sigma'}{dt} = \frac{-du}{dt}\right.$$

2 $$\frac{k}{\gamma_w}\cdot\frac{d^2u}{dz^2} = \frac{a_v}{1+e_0}\cdot\frac{du}{dt}$$

$$\frac{k}{\gamma_w}\cdot\frac{d^2u}{dz^2} = m_v\cdot\frac{du}{dt}$$

$$\boxed{\frac{du}{dt} = \frac{k}{m_v\cdot\gamma_w}\cdot\frac{d^2u}{dz^2}}$$

3

$$\Longrightarrow \boxed{\frac{du}{dt} = C_v\cdot\frac{d^2u}{dz^2}}$$

where, $C_v = \dfrac{k}{m_v\cdot\gamma_w} = \dfrac{m/sec}{\cancel{m^2/kN}\cdot\cancel{kN}/m^3}$ *

C_v - Coefficient of Consolidation

Units - (m^2/sec)

3/11/16

· Settlement (S) = $\dfrac{C_c}{1+e_0} H \log\left(\dfrac{\sigma_f'}{P_c}\right) + \dfrac{C_r}{1+e_0}\cdot H \log\left(\dfrac{P_c}{\sigma_0'}\right)$

where, C_r = recompression index

P_c = pre-consolidation pressure.

Dimensionless parameters by Terzaghi :-

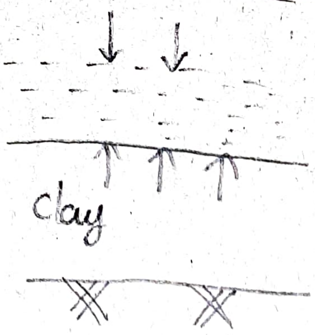
1) Time factor :-

$$\boxed{T_v = \frac{C_v t}{d^2}}$$

where, c_v = Coefficient of Consolidation

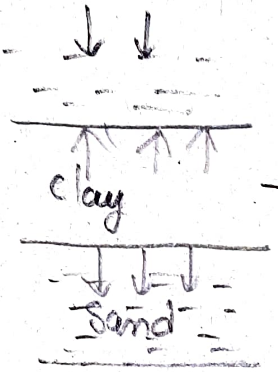
d = drainage path

t = time for Consolidation.



Single drainage (The travelling of water in one direction)

So, $d = H$ - for Single Drainage



- Double drainage, $d = \frac{H}{2}$.

*

2) Degree of Consolidation (U%):-

$$U\% = \frac{\text{Pore pressure dissipated}}{\text{Initial pore pressure}}$$

$$U\% = \frac{U_i - U_t}{U_i} \times 100$$

$$U\% = \frac{\Delta\sigma}{U_i} \times 100$$

Relation b/w Time factor & Degree of Consolidation.

$\Rightarrow U \leq 60\%$

$$T_v = \frac{\pi}{4}\left(\frac{U\%}{100}\right)^2$$

Empirical relation.

$\Rightarrow U > 60\%$

$$T_v = 1.781 - 0.933 \log(100 - U\%)$$

To Calculate void Ratio:-

1) Height of Solids Method:-

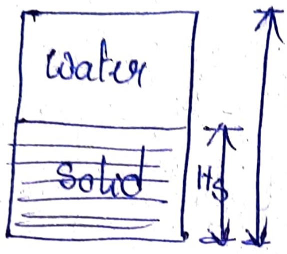
$$\gamma = \frac{W_d}{V}$$

$$\gamma_s = \frac{W_d}{A \cdot H}$$

$$G_s \gamma_w = \frac{W_d}{A \cdot H_s}$$

$$\boxed{H_s = \frac{W_d}{G_s \gamma_w A}}$$

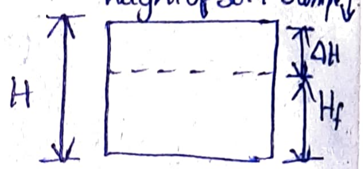
From phase diagram:-



$$e_0 = \frac{V_v}{V_s} = \frac{H_v \cdot \cancel{A}}{H_s \cancel{A}} = \frac{H_v}{H_s}$$

$$\boxed{e_0 = \frac{H - H_s}{H_s}}$$

In Consolidation height of soil sample.



$$(H - H_f) = \Delta H$$

$$e_f = \frac{H_f - H_s}{H_s}, \text{ where } H_f = H - \Delta H.$$

2) Change in Void Ratio Method:-

$$\frac{\Delta e}{1+e_0} = \frac{\Delta H}{H}$$

$$\Delta e = \frac{\Delta H}{H} \times (1+e_0)$$

$$\boxed{e_f = e_0 - \Delta e}$$

$$\Rightarrow \frac{\Delta e}{1+e_f} = \frac{\Delta H}{H_f}$$

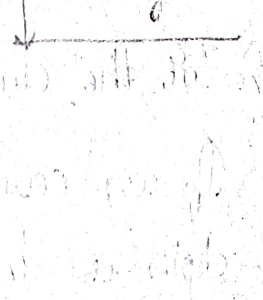
$$\Delta e = \frac{\Delta H}{H} \times (1+e_f)$$

$\boxed{e_t = e_f - \Delta e}$

Problems:-

1) An undistributed soil sample was taken from the middle of a clay layer which is 1.5m below ground layer. As shown in figure. The water table was at the top of clay layer. Laboratory test results are as follows natural water content of clay 25%, Pre-Consolidation 60 KPa, Compression index of clay 0.5, Pre-Compression index of clay 0.05, Specific gravity of clay 2.7, Bulk unit wt of sand 17 KN/m³. A Compacted fill of 2.5 mts hight with unit wt of 20 KN/m³ is placed at the ground level. assuming $\gamma_w = 10\ KN/m^3$. The Ultimate Consolidation settlement of clay layer in mm.

Sol) $S = \frac{C_c}{1+e_o} H \log\left(\frac{\sigma_f'}{P_c}\right) + \frac{C_r}{1+e_o} \cdot H \log\left(\frac{P_c}{\sigma_o'}\right)$



Given

$C_c = 0.5$

$C_r = 0.05$

$P_c = 60\ KPa$

$W = 25\%$

$G_c = 2.7 \Rightarrow \gamma_c = 27\ KN/m^3$

Step ①

$\sigma_o' = \gamma_1 H_1 + \gamma_2 \cdot \frac{H_2}{2}$

$= \cancel{17 \times 1 + 27 \times 1/2}$

$\sigma_o' = \cancel{30.5\ KN/m^2} + (2.5 \times 20)$

$\sigma_o' = 80.5\ KN/m^2$

$\sigma_0' = \gamma_{sand} \times H + (\gamma_{sat} - \gamma_w) \times 0.5$

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

$Se = wG$

$e = \frac{25}{100} \times 2.7 = 0.675$

$\gamma_{sat} = \left[\frac{2.7+0.675}{1.675}\right] \times 10 = 20.15\ kN/m^3$

$\sigma_0' = 17 \times 1 + [20.15 - 10] \times 0.5$

$\sigma_0' = 22.07\ kN/m^2$

$\Delta\sigma' = 2.5 \times 20 = 50\ kN/m^2$

$\sigma_f' = \Delta\sigma' + \sigma_0' = 50 + 22.07$

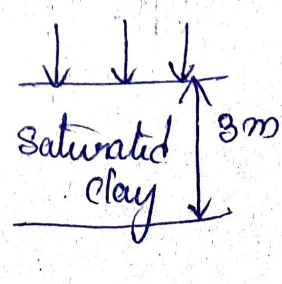
$= 72.07\ kN/m^2$

$S = \frac{0.5}{1+0.675} \times 1 \cdot \log\left[\frac{72.07}{60 \times \text{[illegible]}}\right] + \frac{0.05}{1+0.675} \times 1 \times \log\left[\frac{60}{22.07}\right]$

$S = 0.023 + 0.0129$

$S = 0.036 = 36\ mm$

2) For the clay layer shown in figure. $m_v = 5 \times 10^{-4}\ m^2/kN$ If an earth fill of unit wt $20\ kN/m^3$ and 2mts depth is dumped on the clay layer Then Ultimate settlement of clay layer.



Sol)

$S_f = m_v \Delta\sigma' \cdot H$

$\Delta\sigma' = 20 \times 2 = 40\ kN/m^2$

$H = 3m$

$S_f = 0.06$ mts

$S_f = 60mm$

$\Rightarrow \frac{\Delta e}{1+e_0} = \frac{\Delta H}{H}$

$S_f = \Delta H = \frac{\Delta e}{1+e_0} H$

$m_v = \frac{\Delta e}{(1+e_0)\Delta\sigma'}$

$\frac{\Delta e}{1+e_0} = m_v \Delta\sigma' \Rightarrow \underbrace{\boxed{\frac{\Delta e}{1+e_0} \cdot H}}_{S_f} = m_v \cdot \Delta\sigma' \cdot H$

$\boxed{S_f = m_v \Delta\sigma' \cdot H}$

3) A 2mts thick clay layer has $C_v = 2\times10^{-4} cm^2/sec$. If a building is constructed on it how long it will take to attain ~~off~~ the settlement under double drainage.

Sol) $\Rightarrow T_v = \frac{C_v t}{d^2}$

$d = \frac{H}{2}$ [double drainage]

$d = \frac{\cancel{2}}{\cancel{2}} = 1mts = 100cm$

$\Rightarrow U = \frac{S_t}{S_f} = \frac{0.5 S_f}{S_f} \times 100 = 50\%$

$T_v = \frac{\pi}{4}\left[\frac{U\%}{100}\right]^2 = 0.\cancel{8}9$

$0.19 = \frac{2\times10^{-4} \times t}{(100)^2}$

~~t = ...~~ $t = 9.5\times10^{6} sec = \underline{113.6 days}$.

4) Doing a pressure increament test specimen 20 mm thick under double drainage attain 50% primary

$$U\% = \frac{S_t}{S_f} = \frac{0.5 \cancel{S_f}}{\cancel{S_f}} \times 100 = 500$$

$$0.5 = \frac{S_t}{0.572}$$

$$S_{50\%} = 0.286 \text{ mts} //$$

2) A clay sample originally 30mm thick at a void ratio of 1.10 and subjected to a compressive load. After the clay sample was completed consolidated, its thickness is measured 25mm. Calculate final void ratio ?

Sol) Given

$e_o = 1.10$ $\quad$ $e_f = ?$

$H_o = 30\text{ mm}$ $\quad$ $H_f = 25\text{ mm}$

$$\frac{\Delta e}{1+e_o} = \frac{\Delta H}{H}$$

$$\frac{\Delta e}{1+1.10} = \frac{5}{30}$$

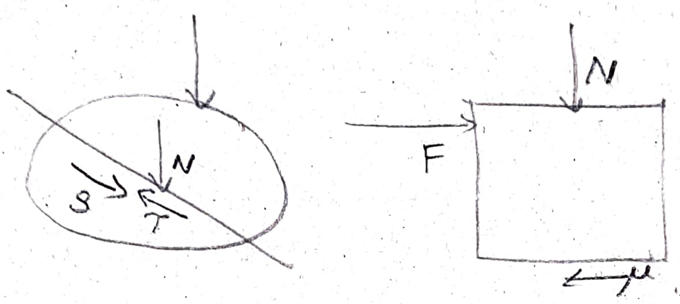
$$\Delta e = 0.35$$

$$e_o - e_f = 0.35$$

$$e_f = 0.75.$$

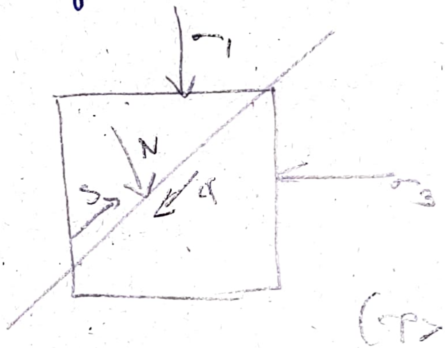
Shear Strength of a Soil

Shear Strength: Shear Strength is the capacity of a soil which resists shear stress.



$F = \mu N.$

→ Shear strength is the max shear stress that soil resist before it fails.



σ_1 = major

σ_2 = intermediate

σ_3 = minor.

($\tau > S$) failure occurs

S - resistance

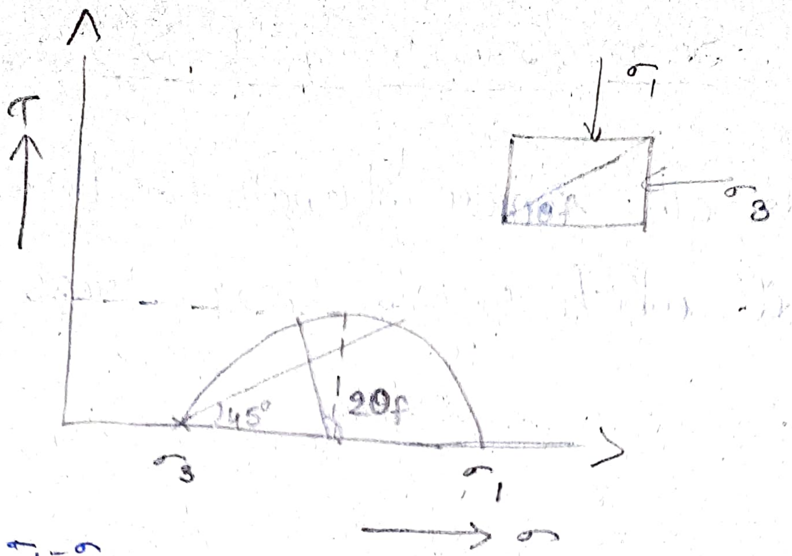
τ - shear force

$$\sigma_\theta = \frac{\sigma_1+\sigma_3}{2} + \frac{\sigma_1-\sigma_3}{2}\cos 2\theta$$

$$\tau = \frac{\sigma_1-\sigma_3}{2}\sin 2\theta$$

→ Shear strength is offered by Contact b/w solid particles in resistance to sliding

Mohr's Circle: To find principle stress and failure plane.

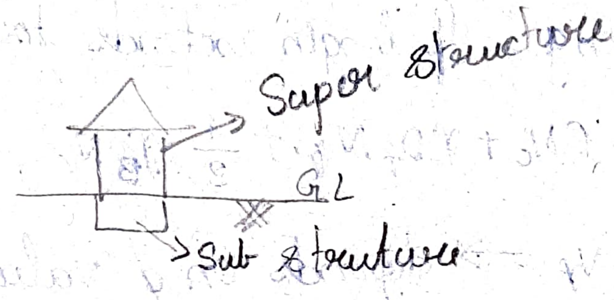


Radius $= \frac{\sigma_1 - \sigma_3}{2}$

$\tau_{max} = \frac{\sigma_1 - \sigma_3}{2}$

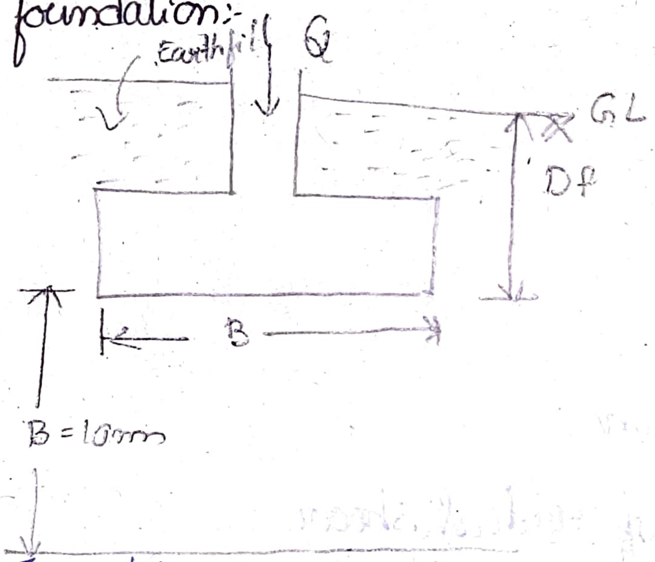
5/11/16

Foundation Engineering



Foundation:- Transfer of load from super structure to soil.

1) Shallow foundation:-



According to Terzaghi, If Depth of footing is less than width of footing is called Shallow foundation.

→ $K \frac{D_f}{B} < 15$ - Moderate foundation

→ $\frac{D_f}{B} > 15$ - Deep foundation.

Gross pressure:- Self weight of foundation + loads + Earthfill.

In direct shear, c & ϕ are shear parameters.

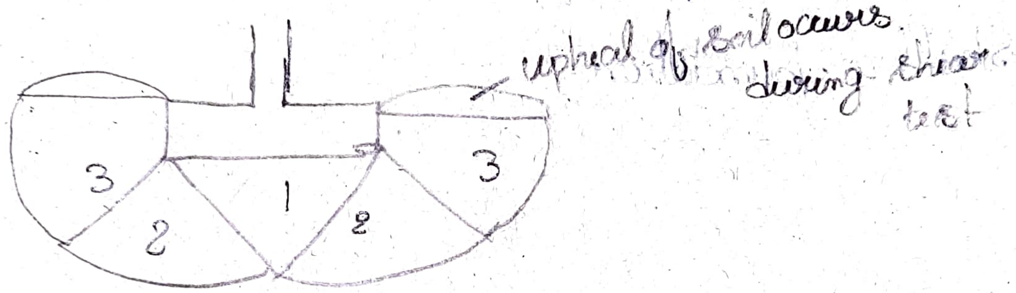
Ultimate Bearing Capacity - It is the minimum gross pressure the soil can withstand ~~before~~ below footing level.

For Strip footing - If length extends towards the wall.

$$q_u = CN_c + \gamma D_f N_q + \frac{1}{2}\gamma_B N_\gamma B$$

$N_c, N_q, N_\gamma \rightarrow$ depends on ϕ values.

If $\phi \geq 36^\circ$, it is called general shear failure.



$q_u = CN_c + N_q \gamma D_f + N_\gamma \gamma B 0.5$

1-zone - active zone

2-zone - zones of radial shear

3-zone - Rankains passive zone

~~4-zone~~ -

If $\phi < 28^\circ$, it is called local shear failure.

Net Ultimate Bearing Capacity:-

$$q_{nu} = q_u - \gamma D_f$$

$$q_{nu} = CN_c + \gamma D_f (N_q - 1) + \frac{1}{2}\gamma_B N_\gamma B$$

Net Safe Bearing Capacity:-

$$q_{\text{net safe bearing}} = \frac{q_{nu}}{FOS}$$

Gross Safe Bearing Capacity:-

$$q_{\text{gross safe Bearing capacity}} = q_{\text{net safe bearing}} + \gamma D_f$$

→ If $\phi < 28^\circ$, mobilise shear

$$C_m = \frac{2}{3} C$$

$$\tan \phi_m = \frac{2}{3} \tan \phi$$

$$\phi_m = \tan^{-1}\left[\frac{2}{3}\tan\phi\right]$$

If $\phi < 28^\circ$

$$\boxed{q_u = \frac{2}{3} CN_c' + \gamma D_f N_q' + \frac{1}{2} \gamma B N_\gamma'}$$

Footing is Circular:-

$$q_u = 1.3 CN_c + \gamma D_f N_q + 0.3 \gamma B N_\gamma$$

Footing is Square:-

$$q_u = 1.3 CN_c + \gamma D_f N_q + 0.4 \gamma B N_\gamma$$

Footing is Rectangular:-

$$q_u = \left[1 + 0.3 \frac{B}{L}\right] CN_c + \gamma D_f N_q + \left[1 + 0.2 \frac{B}{L}\right] \frac{1}{2} \gamma B N_\gamma$$

* $\phi = 0^\circ$ [Saturated clay] - base is rough.

$$q_u = CN_c + \gamma D_f N_q + \frac{1}{2} \gamma B N_\gamma \quad [N_c = 5.7] \quad [N_q = 1, N_\gamma = 0]$$

$$q_u = 5.7\,C + \gamma D_f$$

$$\Rightarrow q_{nu} = 5.7\,C$$

$$\Rightarrow q_{net\,saf\,bearing} = \frac{q_{nu}}{FOS} = \frac{5.7\,C}{FOS}$$

* base is smooth.

$N_c = 5.14$, $N_\gamma = 0$, $N_q = 1$

8/11/16

At a particular point if I draw Mohr circle, if it lies below Mohr's envelope that means shear stress on any plane at that point is less than shear strength.

-> If the Mohr's circle crosses Mohr's envelope which is imaginary why bcz shear stress cannot exceed shear strength.

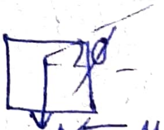
Mohr's-Coulomb failure/ Coulomb's strength theory :- The sliding force (F)

(F) $\tau = C + \sigma_n \tan\phi$

where, C - Apparent cohesion

σ_n - normal stress applied

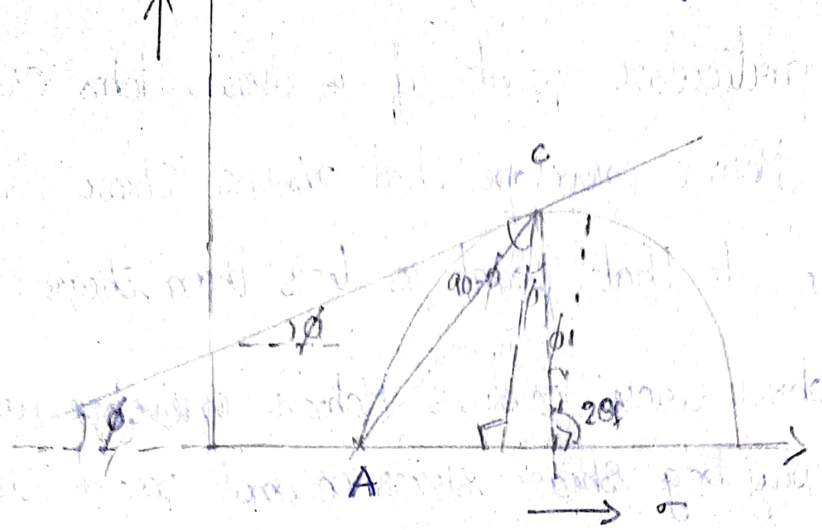
ϕ - Angle of internal friction/ Angle of friction b/w surface of sliding



Nμ

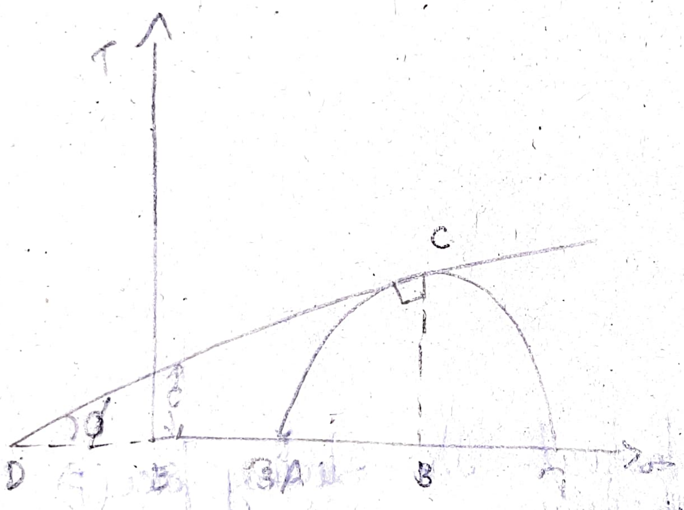
⇒ N tanϕ

AC- line is pole of mohr circle.



$$2\theta_f = 90+\phi$$

$$\theta_f = 45+\phi/2$$



Δ^{le} DBC $\Rightarrow$ $\sin\phi = \dfrac{BC}{BD} = \dfrac{(\sigma_1-\sigma_3)/2}{BE+ED}$

$$BE = \sigma_3 + \frac{\sigma_1-\sigma_3}{2} = \frac{\sigma_1+\sigma_3}{2}$$

$$ED = \tan\phi = \frac{c}{DE}$$

$$DE = \frac{c}{\tan\phi} = c\cot\phi$$

$$\Rightarrow \sin\phi = \frac{(\sigma_1-\sigma_3)/2}{BE+DE}$$

$$\sin\phi = \frac{(\sigma_1-\sigma_3)/2}{\frac{(\sigma_1+\sigma_3)}{2}+c\cot\phi}$$

$$\sin\phi\frac{(\sigma_1+\sigma_3)}{2}+c\cot\phi\sin\phi=\frac{\sigma_1-\sigma_3}{2}$$

$$\sin\phi\left[\frac{\sigma_1+\sigma_3}{2}\right]+\frac{c\cos\phi}{\cancel{\sin\phi}}\cdot\cancel{\sin\phi}=\frac{\sigma_1-\sigma_3}{2}$$

$$\left[\frac{\sigma_1+\sigma_3}{2}\right]\sin\phi+c\cos\phi=\left[\frac{\sigma_1-\sigma_3}{2}\right]$$

$$(\sigma_1+\sigma_3)\sin\phi+2c\cos\phi=\sigma_1-\sigma_3$$

$$\sigma_1-\sigma_1\sin\phi=\sigma_3+\sigma_3\sin\phi+2c\cos\phi$$

$$\sigma_1[1-\sin\phi]=\sigma_3[1+\sin\phi]+2c\cos\phi$$

$$\sigma_1=\sigma_3\left[\frac{1+\sin\phi}{1-\sin\phi}\right]+2c\frac{\cos\phi}{1-\sin\phi}$$

$$\sigma_1=\sigma_3\left[\frac{1+\sin\phi}{1-\sin\phi}\right]+2c\frac{\sqrt{1-\sin^2\phi}}{1-\sin\phi}$$

$$\sigma_1=\sigma_3\left[\frac{1+\sin\phi}{1-\sin\phi}\right]+2c\sqrt{\frac{(1+\sin\phi)(1-\sin\phi)}{(1-\sin\phi)^2}}$$

$$\sigma_1=\sigma_3\left[\frac{1+\sin\phi}{1-\sin\phi}\right]+2c\sqrt{\frac{1+\sin\phi}{1-\sin\phi}}$$

$$\sigma_1=\sigma_3\tan^2(45+\phi/2)+2c\tan(45+\phi/2)$$

10/11/18

Experimental Methods:-

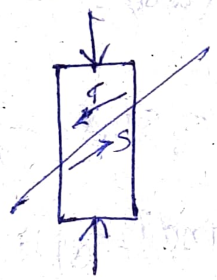
1) Direct shear test

2) Triaxial shear strength

3) Unconfined Compressive shear strength

4) Vane shear test

→ In Unconfined Compressive shear test only Compression force takes place. Due to shear, it fails



Shear strength depends on

1) friction - depends on stress level
2) Cohesion - depends on water content.

→ This is necessary for foundation, excavations, embankments, slopes etc

Mohr's Strength theory:-

Assumptions:-

→ Intermediate principal stress has no effect on the strength.

→ No variables other than the pressure have bending on the shearing strength.

① The principle stresses are $\sigma_1 = 500\ kN/m^2$

$\sigma_3 = 100\ kN/m^2$, $\phi = 45°$

$$\sigma = \left(\frac{\sigma_1 - \sigma_3}{2}\right)\cos 2\theta + \cancel{\sin 2\theta}\left(\frac{\sigma_1 + \sigma_3}{2}\right) = 300\ kN/m^2$$

$$\tau_{max} = \left(\frac{\sigma_1 - \sigma_3}{2}\right)\sin 2\theta = 200\ kN/m^2$$

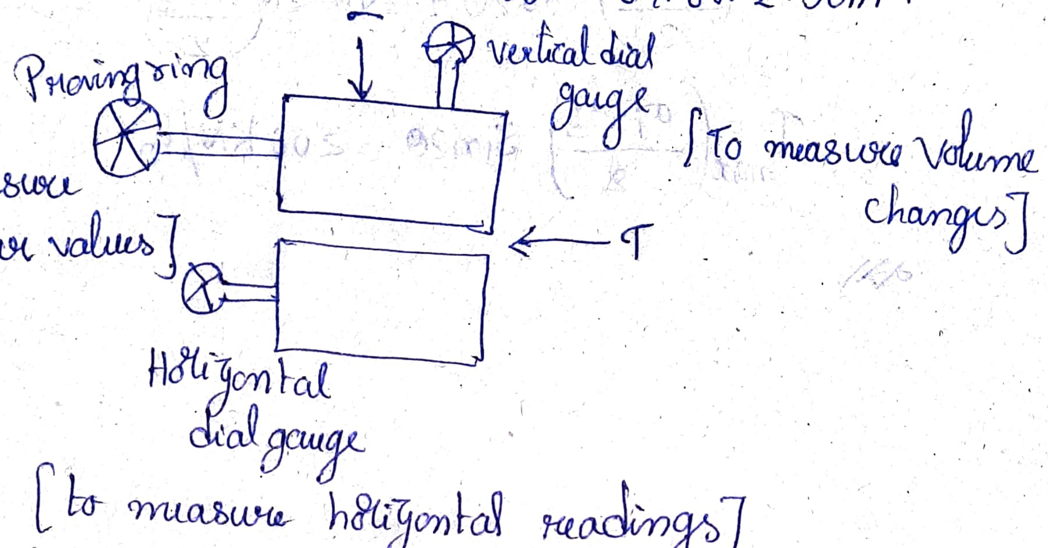
//.

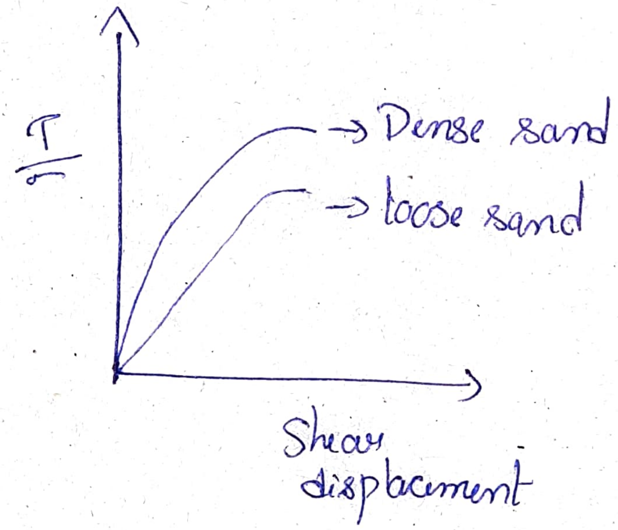
Modified Mohr's Coulomb equation:

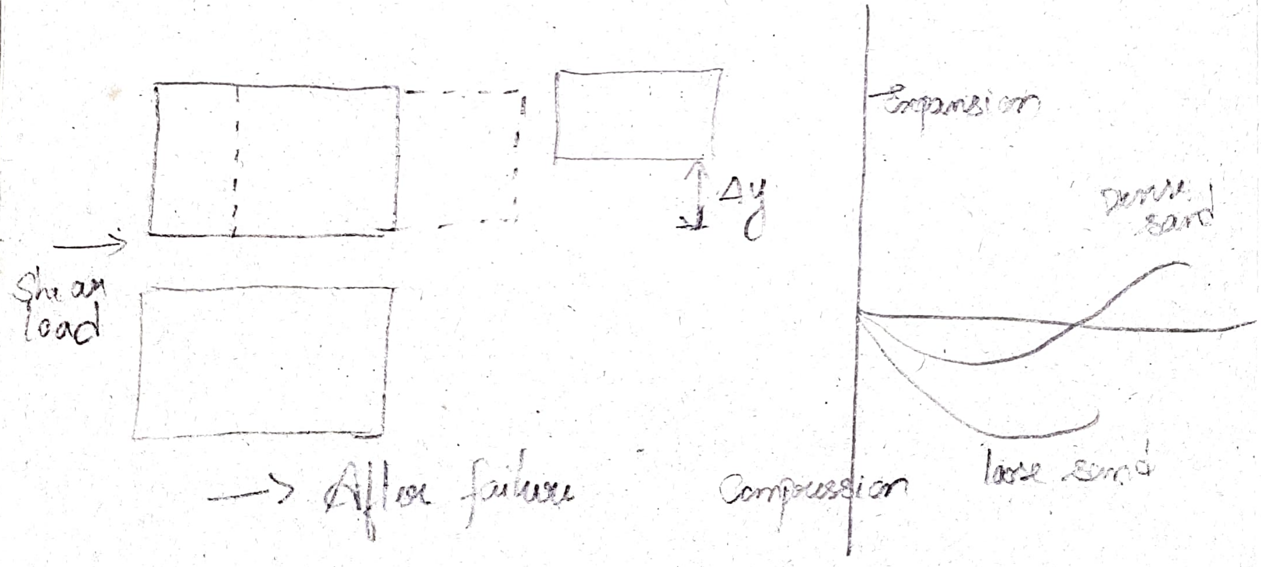
$$\tau = c' + \sigma' \tan\phi'$$

$$= c' + (\sigma - u) \tan\phi'$$

→ Shear box dimensions 6 × 6 × 2.5 cm.







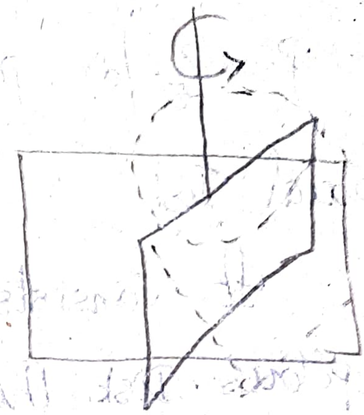
Vane Shear Test:-

In Laboratory:-

$d = 1.25\,cm$

$H = 2.5\,cm$

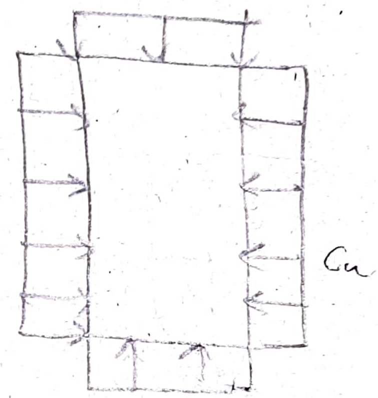
$H = 2d$



In field:-

$d = 50 - 100\,mm$

$H = 100 - 200\,mm$



Shear resistance of surface of Cylinder $= \pi D H \cdot C_u$

Shear resistance of Top & Bottom $= \int_0^{d/2} (2\pi r) C_u\, dr$

$T = F \times \perp$er distance

$$T = \pi D H\, C_u \frac{D}{2} + 2\int_0^{d/2} 2\pi r\, C_u\, dr \cdot r$$

$$= \pi D H\, C_u \cdot \frac{D}{2} + 2(2\pi C_u)\int_0^{d/2} r^2\, dr$$

$$T = \pi D H\, C_u \frac{D}{2} + 4\pi C_u \left[\frac{r^3}{3}\right]_0^{d/2}$$

$$= \pi D H\, C_u \frac{D}{2} + 4\pi C_u \left[\frac{D^3}{8\times 3}\right]$$

$$= \pi D^2 C_u \left[\frac{H}{2} + \frac{D}{6}\right]$$

$$C_u = \frac{T}{\pi\left[\frac{D^2 H}{2} + \frac{D^3}{4}\right]}$$

$$T = \pi D^2 C_u \left[\frac{H}{2} + \beta \frac{D}{4}\right]$$

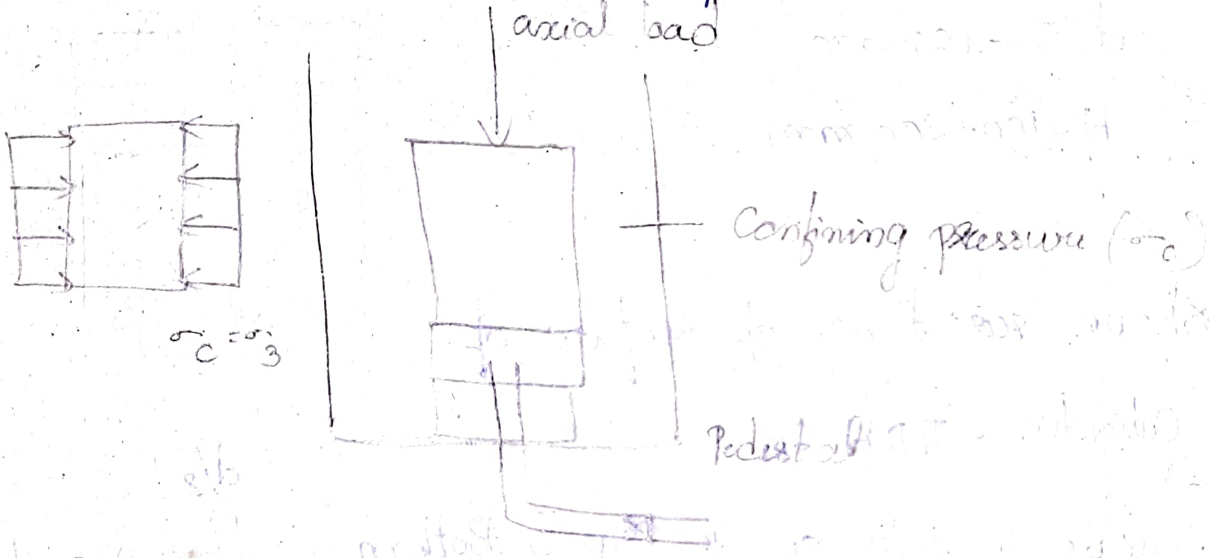
$\beta = \frac{1}{2}$ for Δ^{ler} mobilization

$\beta = \frac{2}{3}$ for uniform mobilization

$\beta = \frac{2}{5}$ for parabolic mobilization.

Triaxial Test:-

It consists on soil specimen, which is kept on a porous. Disk this attack to pedestal.



1. Confining pressure [to expulsion of pore water]
2. Deviator stress (or) axial load] [for shear failure]

There are 2 stages in triaxial test loading

1) Consolidation phase :- that is application of Confining pressure

2) Shearing phase :- that is during the application of deviator pressure.

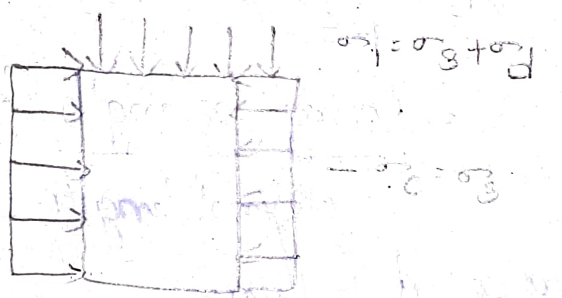
→ During the application of confining pressure if the drainage valve is closed we call it as

Unconsolidated.

→ During the application of Confining pressure if the drainage valve is open we call it as Consolidation phase.

→ During the application of deviator stress if the drainage valve is closed we call it as Undrained Condition.

→ During the application of deviator stress if the drainage valve is open we call it as Drained Condition.



Before shearing	After shearing	Symbol
Consolidated	Drained	CD
Consolidated	undrained	CU
Unconsolidated	undrained	UU

There is some special type of Compressive test that is undrained Compressive strength test.

Undrained Compressive strength test:

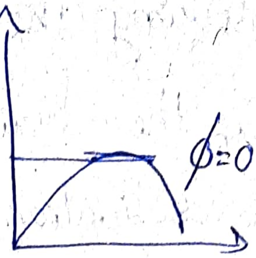
$\sigma_3 = 0$

$\Delta\sigma_d =$

$\sigma_1 = \sigma_3 \tan^2(45+\phi/2) + 2c \tan(45+\phi/2)$

$\sigma_1 = 2c \tan(45+\phi/2)$

It is applicable to $\phi = 0$

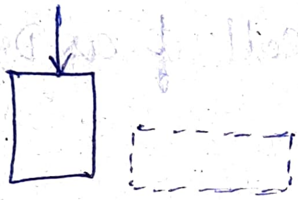


$$\boxed{\sigma_1 = 2c_u}$$

σ_1 = Unconfined Compressive strength

c_u = Undrained shear strength

$q_u = 2c_u$



$A_f = \dfrac{A}{1-\epsilon}$

$\epsilon = \dfrac{\text{Change in length}}{\text{original length}} = \dfrac{\Delta L}{L}$

A_f - area at failure.

1) Consolidated Drained test:-

During both the stages that is during Consolidation & shearing phase drainage valve is open means volume changes are permitted at every stage of test.

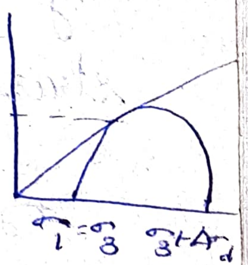
	σ	U	σ'
During Confining pressure (σ_3)	$\sigma_1 = \sigma_3$	$U=0$	$\sigma_1' = \sigma_3$
	$\sigma_3 = \sigma_3$	$U=0$	$\sigma_3' = \sigma_3$
During Deviator stress application ($\Delta\sigma_d$)	$\sigma_1 = \sigma_3 + \Delta\sigma_d$	$U=0$	$\sigma_1' = \sigma_3 + \Delta\sigma_3$
	$\sigma_3 = \sigma_3$	$U=0$	$\sigma_3' = \sigma_3$

In any soil

For normally consolidated soil Mohr's envelop pass through horizon.

If σ_3 < pre consolidation pressure (P_c) then the soil is over consolidated.

→ If the soil is normally consolidated.



i.e $\sigma_3 > P_c$

The envelop is Mohr's envelop. The envelop is linear and passes through origin.

→ An over consolidated soil will show higher strength upto $\sigma_3 = P_c$ and behaves like normally consolidated if σ_3 is greater than P_c.

→ This consolidated drained test is used for long term analysis.

2) Consolidated undrained test:-

During confining pressure	$\sigma_1 = \sigma_3$	$U = 0$	$\sigma_1' = \sigma_3$
	$\sigma_3 = \sigma_3$	$U = 0$	$\sigma_3' = \sigma_3$
During deviator stress application	$\sigma_1 = \sigma_3 + \Delta\sigma_d$	$U = \Delta\sigma_d$	$\sigma_1' = \sigma_3$
	$\sigma_3 = \sigma_3$	$U = \Delta\sigma_d$	$\sigma_3' = \sigma_3 - \Delta\sigma_d$

3) Unconsolidated Undrained test:-

During Confining Pressure $\quad \sigma_1 = \sigma_0' + \Delta\sigma_3 \quad U = \Delta\sigma_3 \quad \sigma_1' = \sigma_0'$

$\sigma_3 = \sigma_0' + \Delta\sigma_3 \quad U = \Delta\sigma_3 \quad \sigma_3' = \sigma_0'$

During deviator stress $\quad \sigma_3 = \sigma_0' + \sigma_3 + \Delta\sigma_d \quad U = \sigma_3 + \Delta\sigma_d \quad \sigma_1' = \sigma_0'$

$\sigma_3 = \sigma_0' + \sigma_3 \quad U = \sigma_3 + \Delta\sigma_d \quad \sigma_3' = \sigma_0' - \Delta\sigma_d$

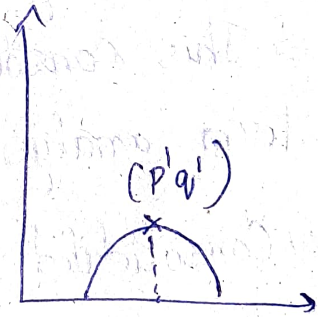
σ_0' = Intial over burden pressure.

Stress paths:- The results of triaxcial test can be represented in diagram is called stress path. It is a line connecting points which represents state stress in successive triaxcial test.

Introduced by Lambe.

$$p' = \frac{\Delta\sigma_1' + \Delta\sigma_3'}{2}$$

$$q' = \frac{\Delta\sigma_1' - \Delta\sigma_3'}{2}$$



CD test:- During Confining pressure application

$\sigma_1 = \sigma_3$

$\sigma_3 = \sigma_3$

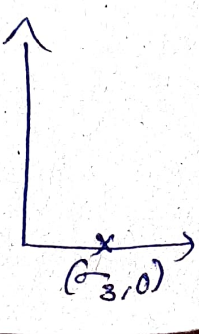
$$p' = \frac{\sigma_1 + \sigma_3}{2}$$

$p', q' = (\sigma_3, 0)$

$$p' = \frac{\sigma_3 + \sigma_3}{2} = \sigma_3$$

$$q' = \frac{\sigma_1 + \sigma_3}{2}$$

$q' = \sigma_3 - \sigma_3 / 2 = 0$



during application devitor stress

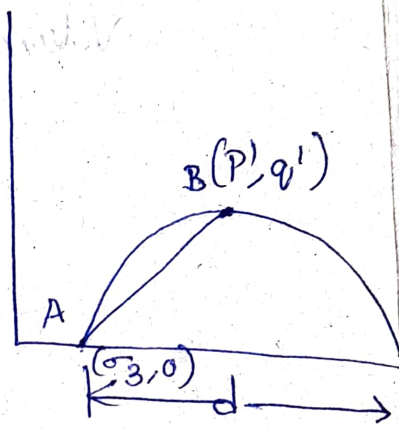
$$\sigma_1 = \sigma_3 + \Delta\sigma_d$$

$$\sigma_3 = \sigma_3$$

$$p' = \frac{\sigma_3 + \Delta\sigma_d + \sigma_3}{2}$$

$$= \frac{2\sigma_3 + \Delta\sigma_d}{2}$$

$$p' = \sigma_3 + \frac{\Delta\sigma_d}{2}$$



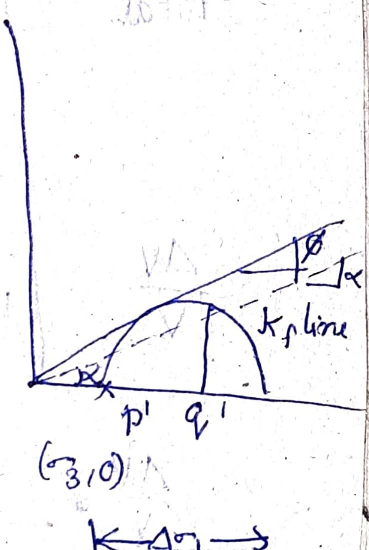
$$q' = \sigma_3 + \frac{\Delta\sigma_d}{2} - \sigma_3$$

$$q' = \frac{\Delta\sigma_d}{2}$$

$$(p', q') = \left[\left(\sigma_3 + \frac{\Delta\sigma_d}{2}\right), \left(\frac{\Delta\sigma_d}{2}\right)\right]$$

AB line is called stress path

$q' = p' \tan\alpha$ - eqn of K_f line



Relation b/w ϕ & α

$$\boxed{\sin\phi = \tan\alpha}$$

Sensitivity:

$$S = \frac{q_u \text{ (undistrubed)}}{q_u \text{ (distrubed)}}$$

Ranges (1-8)

Indicates loss of strength due to remoulding.

* Skempton's pore pressure parameters:-

→ Skempton's pore pressure parameter these are empirical pore pressure coefficient. which represents response of pore water due to increase total stress.

Volumetric strain $e = \frac{\Delta V}{V}$

$$\epsilon_1 = \frac{\Delta\sigma_1'}{E} - \mu\left(\frac{\Delta\sigma_2' + \Delta\sigma_3'}{E}\right)$$

$$\epsilon_2 = \frac{\Delta\sigma_2'}{E} - \mu\left[\frac{\Delta\sigma_1' + \Delta\sigma_3'}{E}\right]$$

$$\epsilon_3 = \frac{\Delta\sigma_3'}{E} - \mu\left[\frac{\Delta\sigma_1' + \Delta\sigma_2'}{E}\right]$$

$$\epsilon_{total} = \epsilon_1 + \epsilon_2 + \epsilon_3 = \frac{\Delta\sigma_1'}{E} - \mu\left(\frac{\Delta\sigma_2' + \Delta\sigma_3'}{E}\right) + \frac{\Delta\sigma_2'}{E} - \mu\left(\frac{\Delta\sigma_1' + \Delta\sigma_3'}{E}\right) + \frac{\Delta\sigma_3'}{E} - \mu\left(\frac{\Delta\sigma_1' + \Delta\sigma_2'}{E}\right)$$

$$\frac{\Delta V}{V} = \left[\frac{1-2\mu}{E}\right]\left[\Delta\sigma_1' + \Delta\sigma_2' + \Delta\sigma_3'\right]$$

$$\frac{\frac{\Delta V}{V}}{\frac{1}{3}(\Delta\sigma_1' + \Delta\sigma_2' + \Delta\sigma_3')} = \frac{3(1-2\mu)}{E} = C_v$$

C_v = Compressibility of soil skelton (It is Constant)

$$\boxed{\frac{\Delta V}{V} = C_v \cdot \frac{1}{3}(\Delta\sigma_1' + \Delta\sigma_2' + \Delta\sigma_3')}$$

$$C_w = \frac{\Delta V/V}{\Delta U}$$

C_w : Compressibility of water

$$C_w = \frac{\Delta V}{V \Delta U} \qquad \frac{V_v}{V} = n$$

$$\boxed{\Delta V = C_w \times n V \Delta U} \longrightarrow (1) \qquad Vn = V_v$$

$$\frac{\Delta V}{V} = \frac{C_v}{3}\left[(\sigma_1 - \Delta U) + (\sigma_2 - \Delta U) + (\sigma_3 - \Delta U)\right]$$

$$\frac{\Delta V}{V} = \frac{C_v}{3}(\sigma_1 + 2\sigma_3 - 3\Delta U)$$

$$\boxed{\Delta V = \frac{C_v}{3} V (\sigma_1 + 2\sigma_3 - 3\Delta U)} \longrightarrow (2)$$

Eq (1) & (2)

$$C_w n \cancel{V} \Delta U = \frac{C_v}{3} \cancel{V} (\sigma_1 + 2\sigma_3 - 3\Delta U)$$

$$C_w n \Delta U = \frac{C_v}{3}(\sigma_1 + 2\sigma_3 - 3\Delta U)$$

$$C_w n \Delta U = C_v\left[\frac{\sigma_1 + 2\sigma_3}{3} - \Delta U\right]$$

$$\frac{C_w n \Delta U}{C_v} = \frac{\sigma_1 + 2\sigma_3}{3} - \Delta U$$

$$\frac{C_w n \Delta U}{C_v} + \Delta U = \frac{\sigma_1 + 2\sigma_3}{3}$$

$$\Delta U\left(1 + \frac{C_w n}{C_v}\right) = \sigma_3 + \left[\frac{\sigma_1 - \sigma_3}{3}\right]$$

$$\Delta U = \frac{1}{1 + \frac{C_w n}{C_v}}\left[\sigma_3 + \frac{\sigma_1 - \sigma_3}{3}\right]$$

$$\Delta U = B\left[\sigma_3 + A(\sigma_1 - \sigma_3)\right]$$

A & B are skempton's pore pressure parameters

$$\Delta U_1 = B\Delta\sigma_3$$

$$\Delta U_2 = AB(\sigma_1 - \sigma_3)$$

$$B = \frac{\Delta U_1}{\Delta\sigma_3}$$

B indicates change in pore pressure (ΔU_1) due to change in confining pressure ($\Delta\sigma_3$)

$$AB = \bar{A} = \frac{\Delta U_2}{\sigma_1 - \sigma_3} = \frac{\Delta U_2}{\Delta\sigma_d}$$

$\bar{A}$ = change in pore pressure due to change in deviator stress.

For completely saturated soil as $C_w \simeq 0$, then $\frac{C_w}{C_v} \to 0$ then the value $B \to 1$

→ For dry soil $B = 0$

$C_w >>> C_v$

range of B - 0 to 1

$B = \frac{1}{\infty} = 0$

Liquification :- when saturated sandy soil is subjected to earthquake load the pore pressure suddenly increases and this decreases the shear strength of soil and may become zero also.

The soil momentarily liquifies behaves as a dense fluid. This phenomenon when sand losses shear strength due to oscilatory motion of soil is called Liquification.

Ex Saturated fine sand & saturated medium sand

Problem:- $\sigma_3 = 300$ $\Delta\sigma_d = 50$ Calculate c, ϕ

test results $\sigma_3 = 150$

$\Delta\sigma_d = 25$

$\sigma_1 = 300 + 50 = 350$

$\sigma_3 = 300 = 300$

$$(p', q') = \left[\left(300 + \frac{50}{2}\right), \left(\frac{50}{2}\right)\right]$$

$$= [325, 25]$$

$$(p', q') = [312.5, 12.5]$$

Consolidation

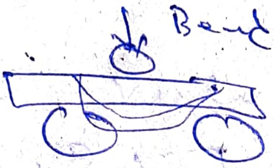
Deformations in Sands

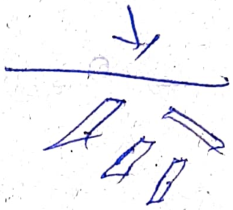
① Elastic compression of grains

② Sliding

③ Rolling.

Deformation in clays

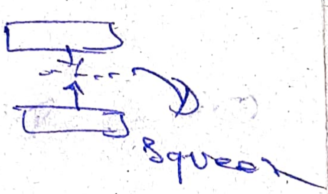


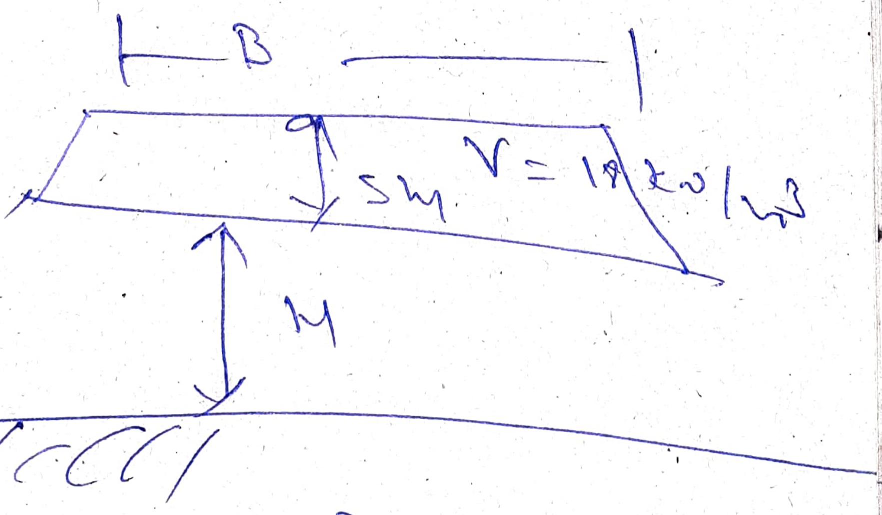


① plate bending

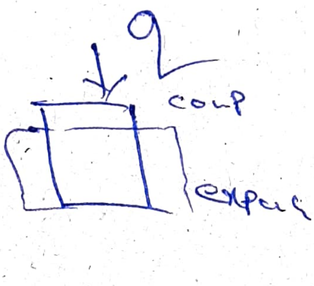
② plate rotation

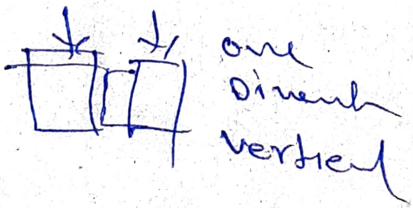
③ Squeezing of water



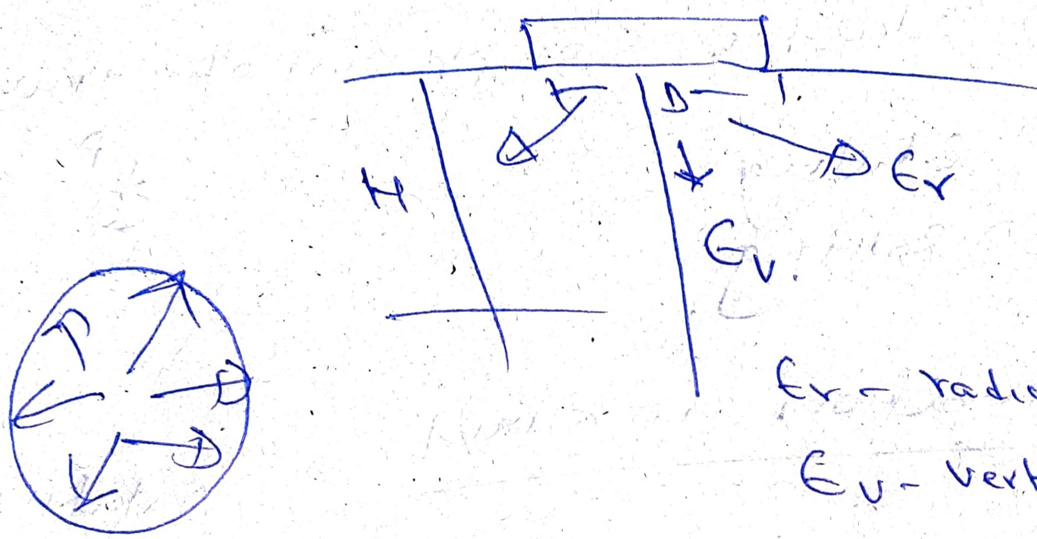


B >> M





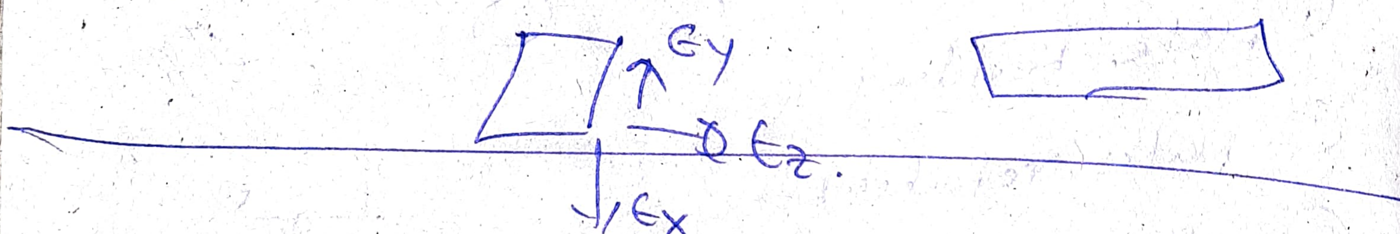
two Dimension



ϵ_r – radial strain

ϵ_v – vertical strain

– Rectangular – 3D.



One Dimensional comp

2) A soil sample with G_s - 2.70 has mass sp. gr (G_m) 1.84, Assuming the soil be perfectly dry. Determine void ratio.

Sol) $G_m = \frac{\gamma_d}{\gamma_w} = 1.84$

$\Rightarrow \gamma_d = 1.84 \times 1 = 1.84\, g/c.c$

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

$e = \frac{G_s \gamma_w}{\gamma_d} - 1 = \frac{2.7 \times 1}{1.84} - 1 = 0.47$

$G_m \times \gamma_w = \gamma_d \Rightarrow \frac{W_d}{V}$ (Assume $V = 100\,cc$

$W = G_m \times \gamma_w \times V = 1.84 \times 1 \times 100$

$= 184\,g.$

$W_s = 184\,g$

$\rightarrow V_s = \frac{W_s}{\gamma_s} = \frac{184}{2.7 \times 1} = 68.14\,cc$

$\rightarrow$ $V_v = V_a = V - V_s = 100 - 68.14$

$= 31.85\,cc$

Void ratio $(e) = \frac{V_v}{V_s} = \frac{31.85}{68.14}$

$e = 0.46$

3) A partially saturated sample from a borrow pit has a natural moisture content of 15% and $\gamma_b = 1.9\,g/cc$. The G_s is 2.70. Determine the S_r & e. What will be the unit wt of sample on saturation?

Sol) Given $w = 15\%$ $G_s = 2.70$

$\gamma_b = 1.9\,g/cc$

1) 1 cum of wet soil weighs 20KN. Its dry weight is 18KN. sp. gravity of solids is 2.67. Determine the water content, porosity, void ratio & S_r. Draw a phase diagram.

Sol) Given

volume of wet soil $(V_s) = 1\,m^3$

$W_d - 18\,KN$

$W_w - 20\,KN = W$

$G_s = 2.67$

$\gamma_s = G_s \gamma_w$

Water content $(w) = \frac{W_w}{W_s} = \frac{2}{18}$

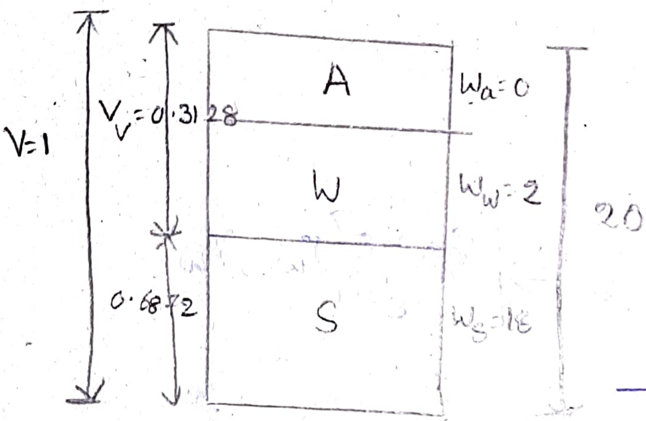
$\gamma_s = \frac{W_s}{V_s}$

$= 0.11$

$= 11.11\%$

$\gamma_w = 9.81\,KN/m^3$

$\gamma_w = 1\,g/cc$



void ratio
~~Porosity~~ (e) : $\frac{V_v}{V_s}$

$\rightarrow V_s = \frac{W_s}{\gamma_s} = \frac{W_s}{G_s \gamma_w} = \frac{18}{2.67 \times 9.81}$

$V_s = 0.6872\,m^3$

$\rightarrow V_w = \frac{W_w}{\gamma_w} = \frac{2}{9.81} = 0.2038\,m^3$

$\rightarrow V_a = V - V_s - V_w = 0.109\,m^3$

$\rightarrow V_v = V_w + V_a = 0.3128\,m^3$.

void ratio.
~~Porosity~~ (e) $= \frac{0.3128}{0.6872}$

$= 0.455$

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

$= \frac{0.3128}{1}$

$= 0.3121$

$S_r = \frac{V_w}{V_v} = \frac{0.2038}{0.3128} = 0.651$