GEO TECHNICAL ENGINEERING (20 Q)

UNIT - I

ORIGIN OF SOILS (1M)

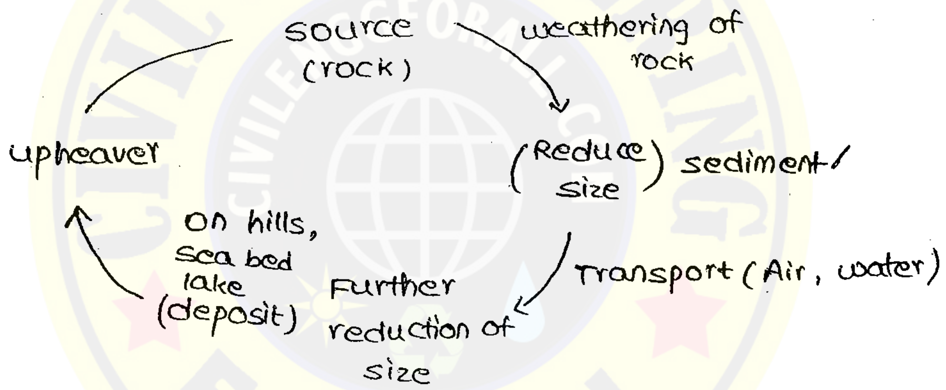
Soil:-

Soil is an assemblage of discrete (discollected) solid particles (organic or inorganic) with a composition of air and or water occupying the void space amoung the particles.

The solid particles may be varying size and shape like clay soils, silt soils, gravel soil etc are may be a mixture.

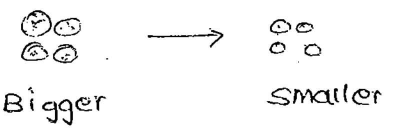
Formation of soils:-

Geological cycle



Types of weathering:-

1. physical weathering:-



No change in chemical composition.

2. chemical weathering:-

1. size reduction takes place
2. change in chemical composition.
3. It results in fine grained and cohesive soils.

	Agency/	deposited place	Name of soil
1.	Water	River (banks)	Alluvial (or) Fluvial soils (river sand)
2.	water	Lake	Lacustrine soils (varved clay)
3.	Water	sea	Marine soils (soft clays)
4.	Air/wind	–	Aeolian soils → Loess (silt), Sand dunes (Fine sand)
5.	Ice/Glaciers	–	Glacier soils (Drift, Till)
6.	Gravity	–	Colluvial soils (Talus).

Coarse grain soils	Fine grain soils
1. Sand, Gravel	1. silt, clay (Rock floor → Stone dust)
2. 3-dimensional size	2. 2-Dimensional particle
3. gravity force predominate	3. surface charge predominate
4. Friction is present (ϕ)	4. cohesion is present ($kN/m^2 = C$)
5. pervious	5. less pervious.
6. specific surface area is low.	6. specific surface area is high

Some important soils:-

1. Varved clay → 

} combination of these two is called Varved soils.

2. Moorum → It is a gravel mixed with red clay.

3. Loam → Mixture of sand, silt and clay (in equal proportion)
(Triangular soil classification chart)

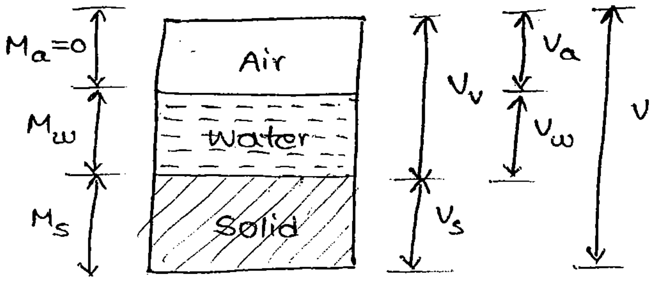
4. Bentonite → It is a decomposed volcanic ash.
It is a highly compressibility, swelling and shrinkage.

UNIT - 2

②

DEFINATION AND PROPERTIES OF SOILS (2M)

Phase diagram :-



1. Void ratio (e) :-

$$e = \frac{\text{volume of voids}}{\text{volume of solids}} = \frac{V_v}{V_s}$$

a. $e_{cgs} < e_{fgs}$

b. $e < 1$ (coarse grain soils)

c. $e \leq 1$ (fine grain soils)

d. $e > 0$

e. $e = 0.91$ (max in spherical grains arranged in cubical array

f. $e = 0.35$ (least in spherical grain arranged in diagonally)

2. porosity :- (n)

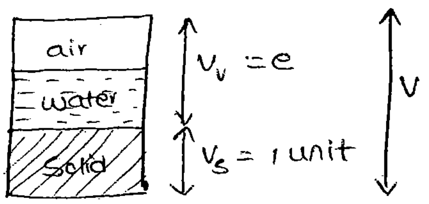
It is also called percentage of voids.

$$\%\text{ of voids} = n = \frac{\text{volume of voids} \times 100}{\text{total volume of soil}}$$

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

Range : $0 < n < 100\ \%$

Relation between 'n' and 'e' is $n = \dfrac{V_v}{V}$



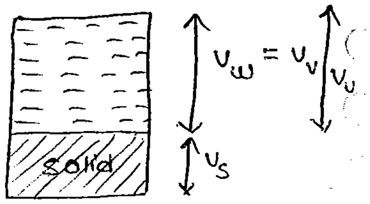
$$\boxed{n = \frac{e}{1+e}}$$

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

3. Degree of saturation (S) :-

$$S = \frac{\text{volume of water}}{\text{volume of voids}} \times 100\ \%$$

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



For dry soil, $V_w = 0$

$\therefore\ S = 0\%$

For fully saturated $V_w = V_v$

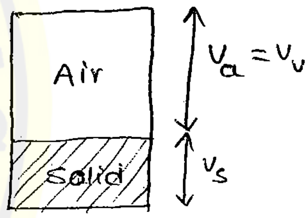
$\therefore\ S = 100\%$ (or) 1

Range: $\boxed{0\% \le S \le 100\%}$

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

4. Air content (a_c) :-

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



For a saturated soil, $V_a = 0$

$\therefore\ a_c = 0\%$

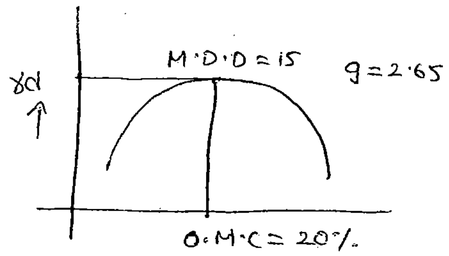
For dry soil, $V_a = V_v$

$\therefore\ a_c = 100\%$

Range: $\boxed{0\% \le a_c \le 100\%}$

$\boxed{0 \le a_c \le 1}$

Ex:-



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

$$15 = \frac{2.65\,(10)}{1+e}$$

$e = 0.76$

$G\,w = S \cdot e$

$2.65\,(20\%) = S \cdot (0.76)$

$S = 70\%\ \Rightarrow\ S + a_c = 100\%\ \Rightarrow\ a_c = 30\%$

5. percentage air voids (n_a) :-

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

For saturated soil, $V_a = 0$

$$\therefore n_a = 0\ \%$$

For dry soil, $V_a = V_v$

$$\therefore n_a = \frac{V_v}{V} \times 100$$

$$\therefore n_a = n\ \%$$

Range : $\boxed{0 \leq n_a \leq n}$

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

6. specific volume (v') :-

$$v' = \frac{\text{Total volume}}{\text{volume of solids}}$$

$$v' = \frac{V}{V_s}$$

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

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

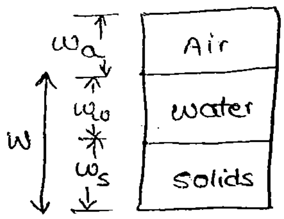
$$\boxed{v' = 1 + e}$$

7. Water content (or) Moisture content (w) :-

$$w = \frac{\text{weight of water}}{\text{weight of solids}} \times 100$$

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

$$w = \frac{W_{wet} - W_{dry}}{W_{dry}} \times 100$$



Range : greater than (w) ≥ 0

sometime it can be greater than 100 %.

Methods of water content:-

1. Oven dry method:-

Take 80gm of wet soil.

weight of container $= W_1$

container + wet soil $= W_2$

weight of dry soil + container $= W_3$

After 24 hrs, 105°c

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

$$= \frac{W_{water}}{W_{dry}} \times 100.$$

It is a laboratory method.

2. pycnometer:- (Lab method)

It is also called density bottle method.

pycnometer (coarse)	density bottle (fines)
1. sand	1. cement
2. gravel	2. clay
3. coarse aggregate	3. stone dust
4. bitumen	4. Bentonite.

Empty wt. of pycnometer $= W_1$

pycnometer $+ \frac{1}{3}$ wet soil $= W_2$

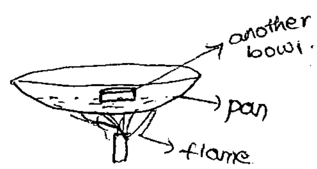
pycnometer + wet soil + water $= W_3$

pycnometer + water only $= W_4$

Water content, $w = \left[\frac{W_2 - W_1}{W_3 - W_4} \left(\frac{G-1}{G} \right) - 1 \right] \times 100$

Sp. gravity $\uparrow$ G = solids

3. Sand bath method:-



Empty wt. of container $= W_1 = 80$ gms

Container + wet soil $= W_2 = 200$ gms.

container + dry soil = 160 gm

$$\text{Water content, } w = \frac{200-160}{160-80} \times 100$$

$$w = 50\%$$

4. Torsion balance method:-

It is used in Research laboratories. It consumes more power.

8. Specific gravity (G):-

Specific gravity are two types:

1. True sp. gravity (or) sp. gravity of grains (or) sp. gravity of solids = G (or) G_s = constant.
2. Mass sp. gravity (G_M)

$$G = \frac{\gamma_{solid}}{\gamma_{water}} = \frac{W_{solid}}{W_{water}}$$

~~Ex~~:- 9. Density (ρ):-

$$\rho = \frac{Mass}{volume} = \frac{gm}{c.c}$$

10. Unit weight (γ):-

$$\gamma = \frac{weight}{volume} = \frac{W}{V} \quad KN/m^3$$

$$\gamma = \rho . g$$

$$\gamma_{dry} = \frac{W_{dry}}{volume} = \frac{W_{dry}}{V} \text{ (or) } \frac{W_{solid}}{V}$$

$$\text{partially} \rightarrow \gamma_{wet} = \frac{W_{wet}}{volume} = \frac{W_s + W_w}{V}$$

$$\text{fully} \rightarrow \gamma_{saturated} = \frac{W_s + W_w}{V}$$

buoyant (or) Effective (or)

$$\gamma' \text{(submerged)} = \gamma_{sat} - \gamma_{\cancel{dry}\ water}$$

$$= 22 - 10 \Rightarrow 12 \frac{k}{m}$$

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

γ_{solids} = 27 kN/m³ > R.C.C (24 kN/

$$W_{solid} = W_{dry} = 50g$$

$$V_s = \frac{\text{weight of water expelled}}{\gamma_{water}} = \frac{W_w}{\gamma_w} \rightarrow 1 \cdot gm/c.c$$

$$\gamma'_{submerged} < \gamma_{dry} < \gamma_{wet} < \gamma_{saturated} < \gamma_{solids}$$

Some important relationships

1. $$\gamma_{saturated} = \frac{W_s + (W_w + 0)}{V_s + (V_w + V_a)}$$

$$= \frac{W_s + W_w}{V_s + V_v}$$

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

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

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

2. $$\gamma_{wet} = \frac{G\gamma_w(1+w)}{1+e} \rightarrow ①$$ (partially saturated)

If water content is zero

3. $$\gamma_{dry} = \frac{G\gamma_w}{1+e} \rightarrow ②$$, sub in eq ①

4. $$\gamma_{wet} = \gamma_{dry}(1+w)$$

5. $$\boxed{\gamma_{dry} = \frac{\gamma_{wet}}{1+w}}$$

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

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

$G\,w_{sat} = S_r \cdot e$ ∴ $S_r = 1$ for saturated

∴ $G\,w_{sat} = 1 \cdot e$.

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

7. $$\gamma' = \gamma_{sat} - \gamma_{water}$$

γ' = submerged

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

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

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

5

8. $\gamma_{wet} = \gamma = \gamma_d(1+w)$

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

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

$$\boxed{\gamma_{wet} = \frac{\gamma_w}{1+e}(G+se)}$$

9. $\gamma_{wet} = \gamma_{dry} + s(\gamma_{sat} - \gamma_{dry}) \Rightarrow$ $\gamma_{sat} - \gamma_{dry} = \left(\frac{G+e}{1+e}\right)\gamma_w - \frac{G\gamma_w}{1+e}$

$$\gamma_{sat} - \gamma_{dry} = \frac{e\gamma_w}{1+e} \therefore$$

proof:-

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

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

$$\gamma_{dry} + s\left(\frac{e}{1+e}\right)\gamma_w = \gamma_{wet}$$

$$\boxed{\gamma_{dry} + s[\gamma_{sat} - \gamma_{dry}] = \gamma_{wet}}$$

For $s = 0$ (for dry soil)

$$\gamma_{dry} = \gamma_d$$

For $s = 100\%$ (fully saturated soil).

$$\gamma_{sat} = \gamma_{dry} + 1(\gamma_{sat} - \gamma_d)$$

10. $$\boxed{\gamma_{dry} = \frac{(1-n_a)G\gamma_w}{1+Gw}}$$

*** 11. $e = \frac{V_v}{V_s}$

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

$$s = \frac{V_w}{V_v} = \frac{V_w}{e}$$

$$\boxed{es = V_w}$$

$$\therefore \boxed{es = wG_s}$$

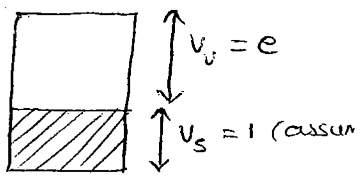
$$w = \frac{W_w}{W_s}$$

$$w = \frac{\gamma_w \cdot V_w}{\gamma_s \cdot V_s}$$

$$= \frac{\gamma_w \cdot V_w}{\gamma_s \cdot (1)}$$

$$w = \frac{V_w}{(\gamma_s/\gamma_w)} = \frac{V_w}{G_s}$$

$$V_w = w G_s$$



12. $\gamma_{dry} = \frac{W_{dry}}{V} = \frac{G\gamma_w}{1+e}$

$\gamma_{d_1} V_1 = \gamma_{d_2} \cdot V_2$

$\frac{\gamma_{d_1}}{\gamma_{d_2}} = \frac{V_2}{V_1} \longrightarrow (1)$

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

$\gamma_{d_1}(1+e_1) = \gamma_{d_2}(1+e_2)$

$\gamma_{d_1}(1+e_1) - \gamma_{d_2}(1+e_2) \longrightarrow (2)$

$$\boxed{\frac{V_1}{V_2} \propto \left(\frac{1+e_1}{1+e_2}\right)}$$

Pg No:-9

5 ~~Ex~~:- $e = 1$ (Before compaction)

$e = 0.6$ (After compaction)

$$\frac{V_{BC}}{V_{AC}} = \frac{1+e_{BC}}{1+e_{AC}}$$

$$V_{AC} = V_{BC}\left(\frac{1+e_{AC}}{1+e_{BC}}\right)$$

$$= V_{BC}\left(\frac{1+0.6}{1+1.0}\right)$$

$$V_{AC} = 0.8\, V_{BC}$$

P.9 No:- 8

11. $n = 50\% = 0.05$

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

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

$$e = 1$$

15. $e = 0.5$

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

$$= \frac{0.5}{1+0.5}$$

$$n = \frac{5}{15} = 0.333$$

Additional questions:-

P.g No:- 9

1. $\gamma_{bulk_1} = 1.8\ gm/c.c$ $\quad w_1 = 5\%$, $\quad e =$ constant

$\gamma_{bulk_2} = ?$ $\quad w_2 = 10\%$

$$\gamma_{bulk} = \gamma_{dry}(1+w)$$

$$\gamma_{bulk_1} = \frac{G \cdot \gamma_w}{1+e}(1+w_1)$$

$$\gamma_{bulk_2} = \frac{G\,\gamma_w}{1+e}(1+w_2)$$

$\therefore$ If void ratio is const γ_{dry} also constant

$$\Rightarrow \frac{\gamma_{bulk_1} = \frac{G\,\cancel{\gamma_w}}{\cancel{1+e}}(1+w_1)}{\gamma_{bulk_2} = \frac{G\,\cancel{\gamma_w}}{\cancel{1+e}}(1+w_2)}$$

$$\Rightarrow \gamma_{bulk_2} = \gamma_{bulk_1}\left(\frac{1+w_2}{1+w_1}\right)$$

$$= 1.8\left(\frac{1+0.1}{1+0.05}\right)$$

$$\gamma_{bulk_2} = 1.88\ gm/c.c$$

(or)

$$\gamma_{bulk_1} = \gamma_d(1+w_1)$$

$$1.8 = \gamma_d(1+0.05)$$

$$\gamma_d = 1.71\ gm/c.c.$$

$$\gamma_{bulk_2} = \gamma_d(1+w_2)$$

$$= 1.71(1+0.1)$$

$$= 1.88\ gm/c.c.$$

3. Given, $G = 2.5$, $e = 1$

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

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

$$= \frac{2.5+1}{2.5-1}$$

$$\frac{\gamma_{sat}}{\gamma'} = 2.3$$

4. $\gamma_{bulk} = 2.4\ gm/c.c$

$w = 20\%$

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

$$= \frac{2.4}{1+0.2}$$

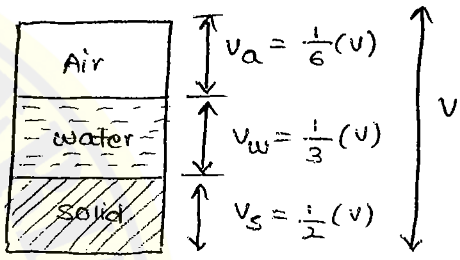
$$= \frac{2.4}{1.2}$$

$$\gamma_d = 2\ gm/c.c.$$

6. $V_v = V_a + V_w$

$$= \left(\frac{1}{6} + \frac{1}{3}\right)V$$

$$= \frac{1}{2}V$$



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

$$= \frac{\frac{1}{2}V}{\frac{1}{2}V}$$

$$e = 1$$

7. $V_v = V_s$

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

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

previous A.P.P.S.C questions:-

P.g NO:- 9

1. $w = 10\%$ $S_r = 90\%$, $G = 2.7$, $\gamma_{dry} = ?$

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

$$e = \frac{wG}{S} = \frac{0.1\,(2.7)}{0.9} = 0.3$$

$$\gamma_{dry} = \frac{2.7(10)}{1+0.3} = \frac{27}{1.3}$$

$$\gamma_{dry} = 20.76 \ kN/m^3$$

2. $n = 20\%$

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

$$= \frac{0.2}{1-0.2} = \frac{0.2}{0.8} = \frac{1}{4} = 0.25$$

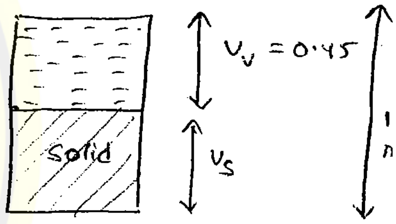
3. $w = 15\%$ $G = 2.67$ $e = 0.49$ $S = ?$

$$Se = Gw$$

$$S = \frac{Gw}{e} = \frac{2.67(0.15)}{0.49}$$

$$= 0.817 \text{ or } 81.7\%$$

4. $n = 45\%$ $V = 1 \ m^3$



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

$$(0.45 \times 1) = V_v$$

$$V_v = 0.45 \ m^3$$

$$V_w = V_v = 450 \text{ lit}$$

$$\rho_w = \frac{M_w}{V_w}$$

$$M_w = 1000(0.45) \text{ kg}$$

$$= 450 \text{ kg.}$$

6. Moist soil + tin lid = 24 gms.

Empty wt. of tin lid = 14 gms

Moist soil = 24 − 14 = 10 gms

dry soil + lid = 22 gms

Dry soil = 22 − 14 = 8 gms

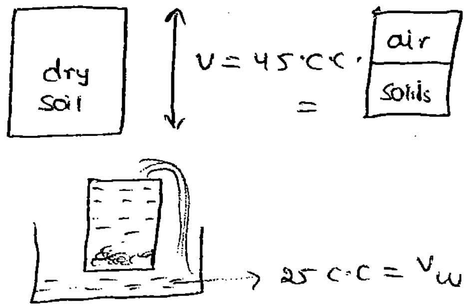
$$w = \frac{W_{wet} - W_{dry}}{W_{dry}} \times 100$$

$$= \frac{10-8}{8} \times 100$$

$$w = 25\%.$$

Ex:-) A sampler with a volume of 45 c.c. is filled with dry soil sample. When the soil is poured into the graduated cylinder. It displaces 25 c.c. of water. What is a void ratio of soil.

A) Given, $V = 45\ cm^3$, $V_s = 25$ c.c.



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

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

$$e = \frac{45 - 25}{25} = \frac{20}{25} = 0.8$$

Ex:- A saturated clay has water content of 39.3% and bulk sp. gravity of 1.84. The sp. gravity of soil and void ratio ?

A. $w = 39.3\%$ $\quad G_m = 1.84$ (bulk or mass)

$$e = \frac{wG}{S} \quad (\text{saturated},\ S = 1)$$

$$e = \frac{1.84(0.393)}{1}$$

$$e = 0.723$$

$$G_{m\,(dry)} = \frac{\gamma_d}{\gamma_w}$$

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

$$1.84 = \frac{\gamma_{sat}}{10} = \cancel{\left(\frac{G+e}{1+e}\right)}$$

$$\gamma_{sat} = 18.4\ kN/m^3$$

$$\gamma_w\left(\frac{G+e}{1+e}\right) = 18.4$$

$$\frac{G+e}{1+e} = \frac{18.4}{10}$$

$$G = 1.84(1+e) - e \Rightarrow 1.84(1+0.723) - 0.723$$

$$G \Rightarrow 2.44$$

18-11-2014

(8)

UNIT - 3

SOIL STRUCTURES AND CLAY MINEROLOGY (I M)

Types of structures:-

1. single grained structure:-

$e_{c.G.s} < e_{f.G.s}$ coarse grained soil

Fine grained soil.

$e_{sand} < e_{clay}$

1. when c.G.s material are arranged in loose packing the density is low and void ratio is maximum.

$e_{max} = 0.91$

void is taken in sand or gravel

$e = 0.91$

Loose packing

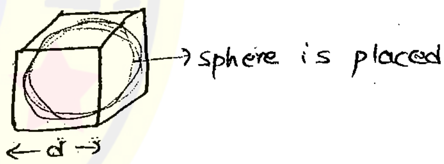
$e_{dense} = 0.35$

dense packing

$V_T = d^3$

$V_s = \frac{\pi d^3}{6}$

$V_v = V - V_s$



2. Honey comb structure:-

1. present in fine sands or silt.
2. Under vibrations and shocks, the structure collapse and large deformation takes place.

Clay Mineralogy:-

Kaolinite:-

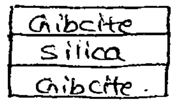
1 : 1 Clay mineral

silica
Gibcite / Alumina

1. Silica sheet and Gibcite/Allumina sheet with equal proportion.
2. It is a stable mineral. No swelling and No shrinkage
3. China clay.

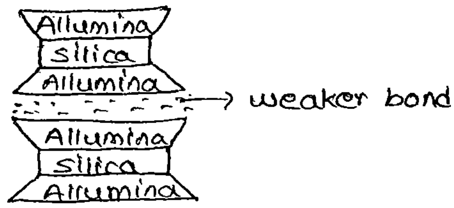
Illite Mineral:-

1. It is a 2:1 clay
2. It exhibit medium swelling and shrinkage.

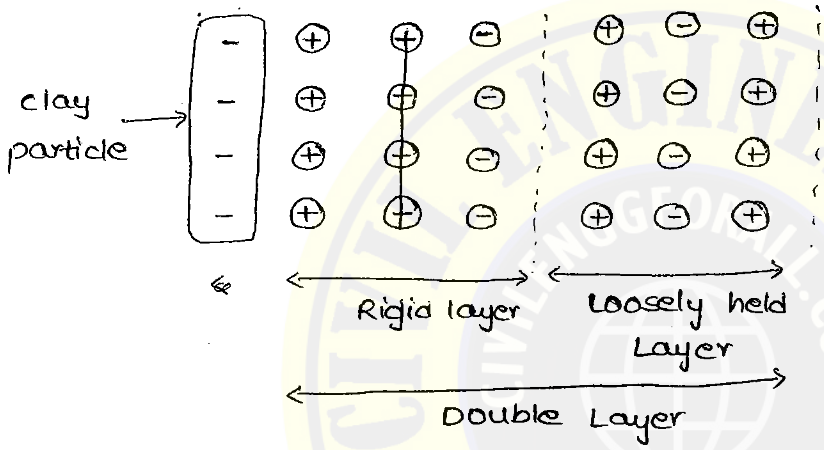


Montmorillonite:-

1. It is a 2:1 clay
2. It exhibits large swelling and large shrinkage.



Diffuse Double Layer and absorbed water:-



19-11-2014

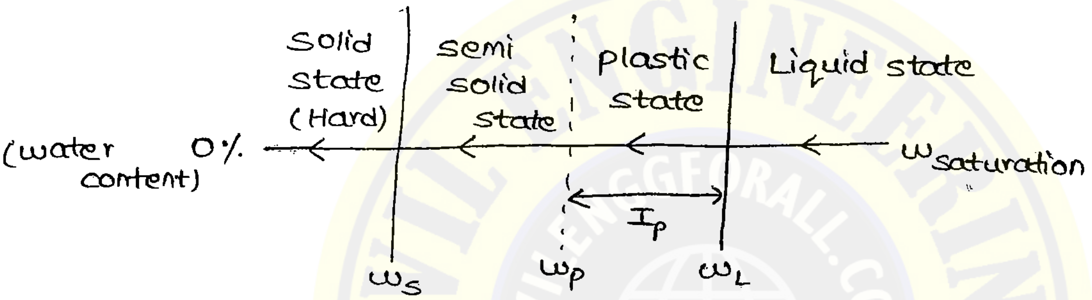
UNIT - 4

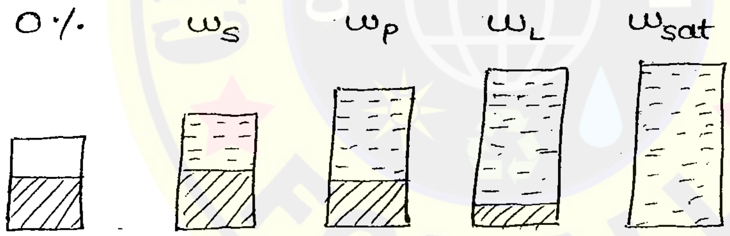
INDEX PROPERTIES OF SOILS (3M)

Properties of soil
→ Index properties
→ Engineering properties

Index properties of Fine grained soils:-

Limit of consistency (Atterberg's limit):-



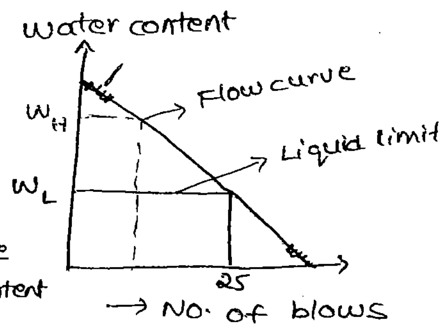


Liquid Limit:-

(flow curve) $I_f = \dfrac{(w_1 - w_2)}{\log_{10}\left(\frac{n_2}{n_1}\right)} = \dfrac{(w_H - w_L)}{\log_{10}\left(\frac{N_H}{N_L}\right)}$

I_f = Flow index

H - Higher value of water content



Grooving tool
→ ASTM
→ casagrandes

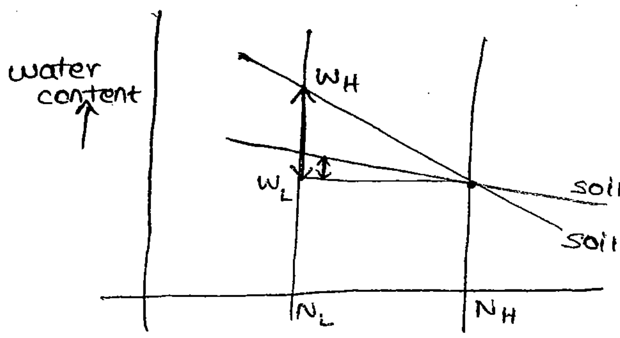
used for silt

used for sticky silts. (clays) (or) plastic

$\therefore I_f \propto \dfrac{1}{\text{shear strength of soil}}$

$$I.F \propto \frac{1}{S.S}$$

If I.F is increase, shear strength is decrease.



Fall cone test:-

1. It is used for non plastic (or) silt (loose) and also plastic soils (or) clay (sticky silt).

Inorganic soils L.L < 50%

Organic soils of volcanic ash is High L.L

Fat clays L.L > 50% (% of clay is larger) (CH are fat clays)

Bentonite L.L ≈ 786%

Plastic Limit test:-

1. soil passing 425 μ

$$P.I = L.L - P.L$$

$$I_P = w_L - w_P$$

I_P	plasticity
0	Non-plastic
< 7	Low plastic (Lean clays, ~~clays~~ having a less sticky nature is called low plastic)
7–17	Medium plastic
>17	Highly plastic

(Lean clays — % of clays is less and silt is more)

Shrinkage limit test:-

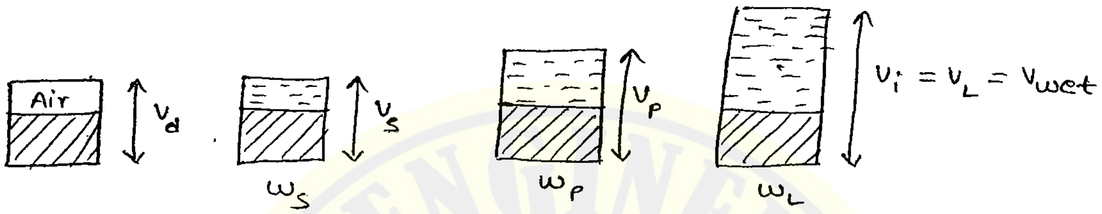
wt. of only dish $= w_1$

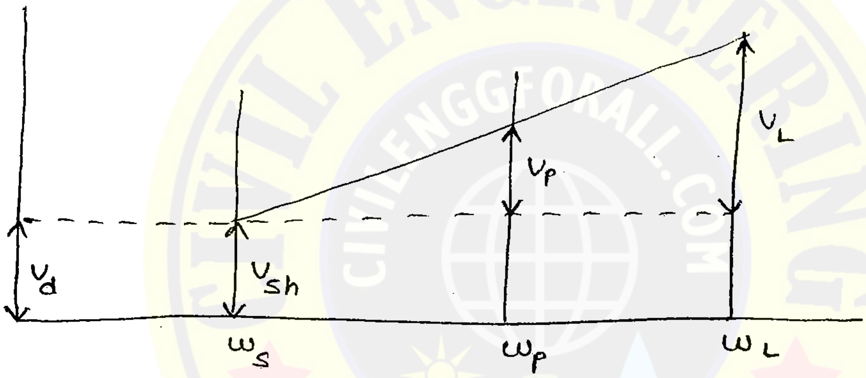
wt. of wet soil + dish $= w_2$

wt. of dry soil + dish $= w_3$

$$w = \frac{w_2 - w_3}{w_3 - w_1} \times 100\%$$

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





(a) $w_s = [w_i - \text{loss of water from } w_L \text{ to } w_s]$

$$= \left[w_i - \frac{(V_L - V_d)\cdot \gamma_w}{W_s}\right] \times 100$$

$$w_i = \frac{W_{wet} - W_{dry}}{W_{dry}}$$

(b) $$w_s = \frac{(V_d - V_s)\gamma_w}{W_s} \times 100$$

(c) $$w_s = \frac{(V_d - V_s)\gamma_w}{W_s}$$

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

$$= \frac{\left(V_d - \frac{W_s}{\gamma_s}\right)\gamma_w}{W_s}$$

$$= \frac{V_d \gamma_w}{W_s} - \frac{\gamma_w}{\gamma_s}$$

$$= \frac{\gamma_w}{\left(\frac{W_s}{V_d}\right)} - \frac{\gamma_w}{\gamma_s} \Rightarrow \left[\frac{\gamma_w}{\gamma_d} - \frac{\gamma_w}{\gamma_s}\right] \Rightarrow \left[\frac{1}{(\gamma_d/\gamma_w)} - \frac{1}{(\gamma_s/\gamma_w)}\right]$$

$$\Rightarrow \left[\frac{1}{G} - \frac{1}{G_s}\right]$$

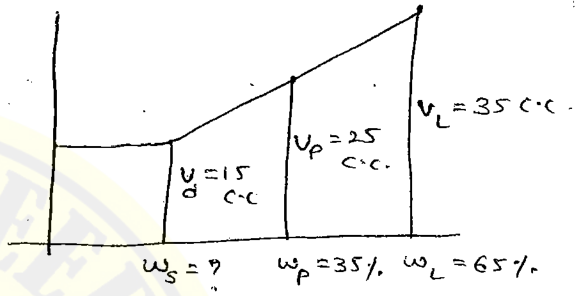
d. $w_s = e/G$

Shrinkage Ratio:-

$$\boxed{S.R = \frac{\frac{\Delta V}{V_d} \times 100}{w_H - w_L}}$$

$$S.R = \frac{\frac{(V_L - V_P)}{V_d} \times 100}{w_L - w_P}$$

Ex:-



i) $S.R = \frac{\frac{V_L - V_P}{V_d} \times 100}{w_L - w_P}$

$= \frac{\frac{35-25}{15} \times 100}{65-35}$

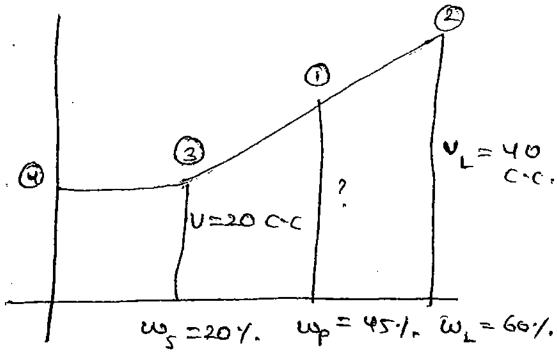
$S.R = 2.23$

ii) $S.R = \frac{\frac{V_P - V_S}{V_d} \times 100}{w_P - w_S}$

$2.23 = \frac{\frac{25-15}{15} \times 100}{35 - w_S}$

$w_s =$

Ex:-



$S.R = \frac{\frac{V_L - V_S}{V_d} \times 100}{w_L - w_S} = \frac{\frac{V_L - V_P}{V_d} \times 100}{w_L - w_P}$

$= \frac{\frac{40-20}{20} \times 100}{0.6 - 0.2} = \frac{\frac{40 - V_P}{20} \times 100}{0.6 - 0.2}$

$V_P =$

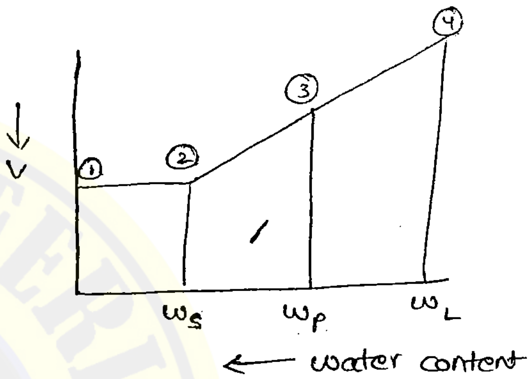
Volumetric shrinkage :-

$$V_s \neq S.R = \frac{\frac{\Delta V}{V_d} \times 100}{W_H - W_L}$$

$$SR\,(W_H - W_L) = \frac{\Delta V}{V_d} \times 100$$

$$SR\,(W_H - W_L) = \frac{\Delta V}{V_d} \text{ (volumetric shrinkage)}$$

$$SR\,(W_H - W_L) = V_s \text{ (Volumetric shrinkage)}$$



1. w_s upto w_L , $S = 100\%$.
2. $w = 0$, $S = 0$
3. $w = 0$ and w_s , $0 \leq S_r < 100\%$.
4. ① & ②

   e = constant , $\rho_d = \frac{M_s}{V} = \frac{G \cdot \rho_w}{1+e}$
5. ② & ④

   e = variable , $S_r = \frac{V_w}{V_v}$
6. ② & ④

   ΔV = variable , G_s = constant.
7. ① & ④

   water content = variable
8. ① & ④

   S_r = constant

Appsc :-

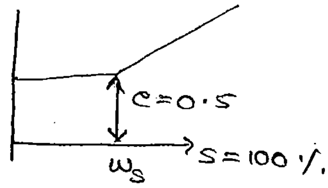
1. If $W \cdot C_{natural} = 27\%$ is w_s & w_p
2. If $W \cdot C_{natural} = 45\%$ is w_p

Ex:-) $G w_s = se$

$G w_s = 1 \cdot e$

$w_s = \frac{e}{G} = \frac{0.5}{2.72} \times 100$

$=$



$G_s = 2.72$

Consistancy Index (I_c):-

$$I_c = \frac{w_L - w_{natural}}{w_L - w_p} = \frac{w_L - w}{I_p}$$

i) $w_{nat} = 0$ (dry)

$$= \frac{w_L}{w_L - w_p} \quad (I_c > 1)$$

ii) $w_{nat} \ngtr w_s$

$$I_c = \frac{w_L - w_s}{w_L - w_p} \quad (I_c > 1)$$

Greater

iii) $w_{nat} = w_p$

$$I_c = \frac{w_L - w_p}{w_L - w_p} = 1 \quad (I_c = 1)$$

iv) $w_{nat} = w_L$

$$I_c = \frac{w_L - w_L}{w_L - w_p} = 0 \quad (I_c = 0)$$

v) $w_{nat} > w_L$

$$I_c' = \frac{w_L - (w_{sat})}{w_L - w_p} \quad (I_c = -ve)$$

$$(w_{sat} > w_L)$$

Liquidity Index :-

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

i) $w_{nat} = 0$, $I_L = -ve$

ii) $w_{nat} = 0$, $I_L = -ve$

iii) $w_{nat} = w_p$, $I_L = 0$

iv) $w_{nat} = w_L$, $I_L = 1$

$w_{nat} > w_L$

$I_L > 1$

Toughness Index:- (I_T):-

$$I_T = \frac{I_P}{I_f} \quad , \quad I_f \propto \frac{1}{S.S}$$

Activity:-

$$\text{Activity (A)} = \frac{\% \, I_P}{\% \text{ clay fraction}}$$

If activity, $A < 0.75$ (In active)

$A = 0.75 - 1.25$ (Normal)

$A > 1.25$ (Active)

EX:- Soil Bentonite — High swelling and shrinkage

$A > 4$ For montmorillonite

C.G.S. (Index properties):-

Density Index (I_D) (or) Relative density :-

$$I_D = \frac{e_{max} - e}{e_{max} - e_{min}} \times 100$$

V. loose ($R_D = 0\%$)

Loose

Mud

Dense

V. Dense ($R_D = 100\%$)

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

Note :-

1. Relative density test is suitable for <u>Friction soil.</u>

2.

Relative density	Description
0% - 15%	very loose material (this soil is used for ground improveme
15 - 35%	loose material
35 - 65%	Medium material
65 - 85%	Dense material
> 85%	very dense material.

3. General shear failure occur when $R_D > 30\%$.

Shape of particle:-

Engineering properties depends on shape of particle (coarse grained soil).

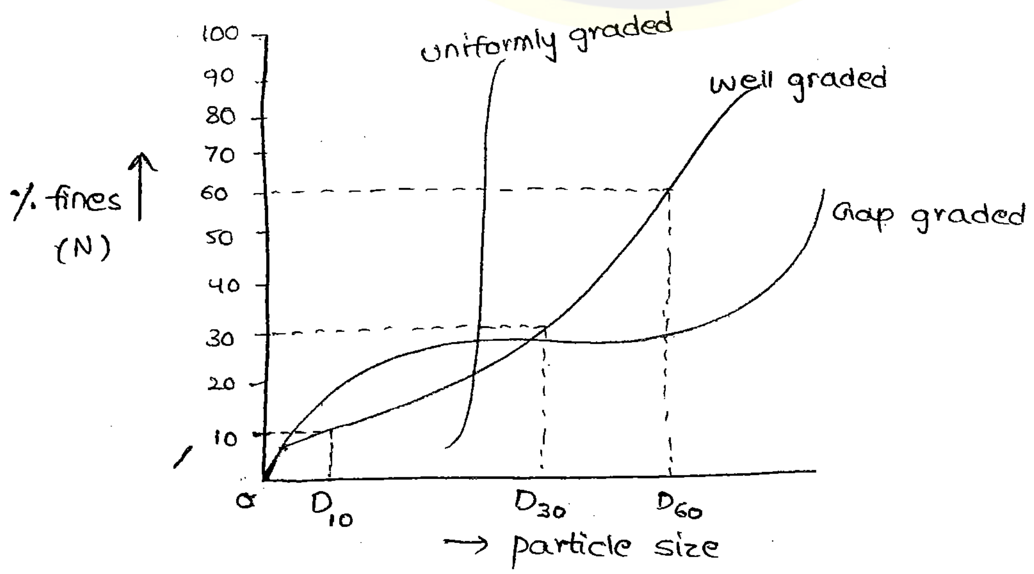
- Angular — Angle of friction (ϕ) is more
- Sub-angular
- Rounded
- Well rounded — Angle of friction (ϕ) is less

C.G.S material are generally called Bulky material (or) 3D particle.

Particle size distribution:-

Soil
- → C.G.S
  - → Clean sand (or) gravel (Dry sieve analysis is used)
  - → Fines ~~(Silt)~~ (Silt (or) clay) (wet sieve (or) sedimentation analysis is conducted)
- → F.G.S
  - → Finers (Silt or clay) (Hydrometer test, Pippette test is conducted)

* In 75 μ sieve there are 200 openings in one inch.



Coefficient of uniformity (C_u):-

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

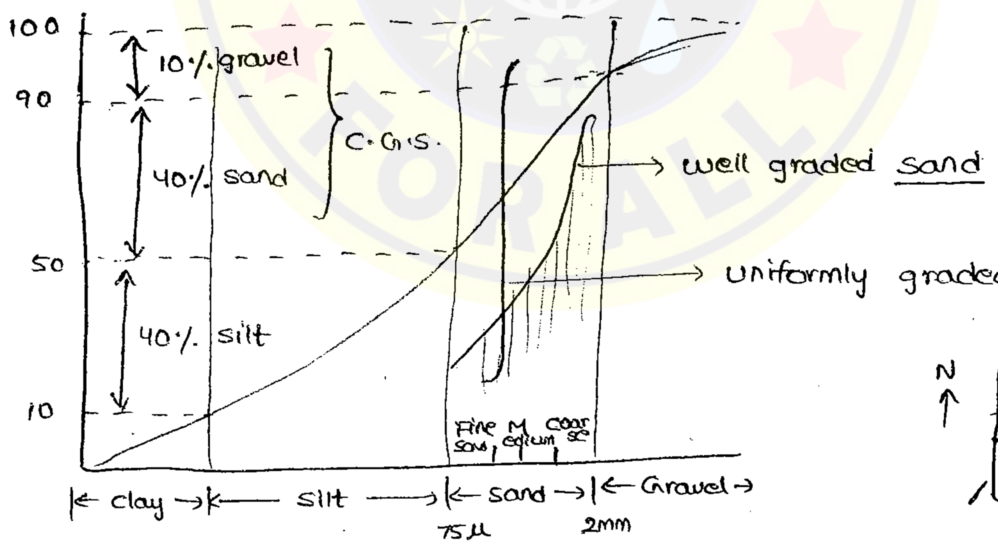
D_{10} is also called effective size (mm)

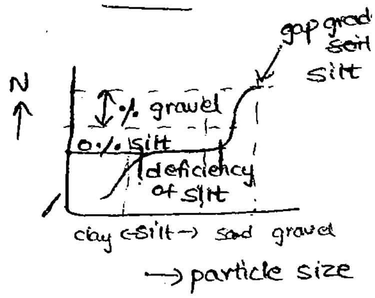
1. C_u cannot be less than '1'.
2. A uniformly graded soil, C_u lies between 1 and 2
3. A well graded ~~soil~~ gravel C_u must be greater than 4.
4. For well graded sand, C_u must be greater than 6.
5. ~~Uniformly graded soil is also called~~ poorly graded soil.

Coefficient of curvature (or) concavity (or) gradation (C_c):-

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

1. It represents particle size curve.
2. For well graded soil, C_c lies between 1 and 3
3. For uniformly graded soils, $C_c \approx 1$





→ If curve is present in Gravel zone then the curve is called "Uniformly graded Gravel" (or) some other curve.

→ well graded are "wide Range" ($C_u > 6$)

→ poorly graded are "Narrow or steep Range". ($C_u = 1$)

(well sorted soil)

1. Well graded soil — Aeolian soil, Talus, Glacier soils (Till)

2. poorly graded soil (uniformly graded soil) — Marine deposit

EX:- Marine depost, Beach sand

P.g NO:- 19

1. Given $C_u = 4$ $C_c = 1$, $D_{30}/D_{10} = ?$

$$C_u = \frac{D_{60}}{D_{10}} \Rightarrow 4D_{10} = D_{60}$$

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

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

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

$$D_{30}^2 = 4D_{10} \times D_{10}$$

$$\left(\frac{D_{30}}{D_{10}}\right)^2 = 4$$

$$\frac{D_{30}}{D_{10}} = 2$$

Stoke's law:-

Terminal velocity, $V_s = \frac{9}{18}(G-1)\frac{d^2}{\eta}$

% of finer obtained by $N = \left[\frac{R_h (\text{corrected})}{1000} \times \frac{\left(\frac{G}{G-1}\right)}{M/V}\right] \times 100$

V = volume of water (1 lit)

M = mass of finer

R_h = hydrometer reading corrected.

22-11-2014

UNIT - 5

SOIL CLASSIFICATION

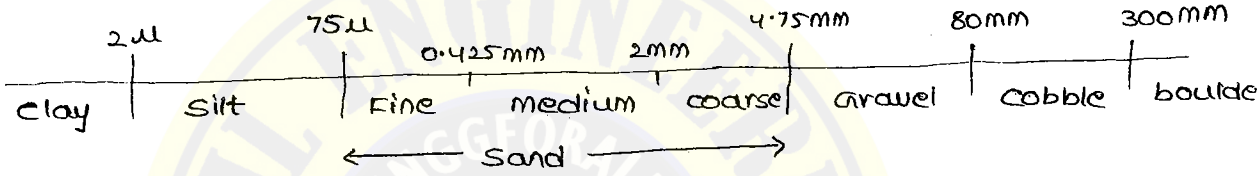
Labelling of soil is soil classification.

Criteria :-

1. Method used should be simple
2. It should give more information
3. Easy to understand.

I.S. particle size classification:-

Classification is done based on grain size only.



AASHTO (American Association of State Highways and Transpo Officials):-

1. It is also called Public Road Administration (PRA) or Highway Research Board (HRB).
2. AASHTO classification is used for Road purpose not gener purpose.

Soil classification:-

1. soil is classified into groups like $A_1, A_2, \ldots A_7$.
2. A_1 is good quality soil, A_7 is poor quality soil.
3. Black cotton soils (or) clay comes under A_7 group.
4. stone fragments (or) Gravel (or) sand coumes under A_1, A_2, A_3
5. Fine sand, silt, clay comes A_4, A_5, A_6, A_7.

Group Index:-

$$G.I = 0.2(a) + 0.005(a \cdot c) + 0.01(b \cdot d)$$

a = % of soil passing through 75μ sieve, > 35% and no exceeding 75%. [The range is 0 to 40]

b = % of soil passing through 75 µ sieve, > 15% and
not exceeding 55% [Range is 0 to 40%]

c = a portion of L·L > 40% and not exceeding 60%
[Range is 0 to 20%]

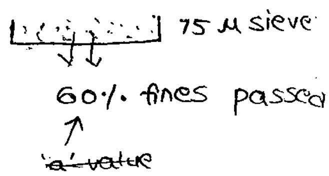
d = a portion of P·I > 10% not exceeding 30%
[Range is 0 to 20%]

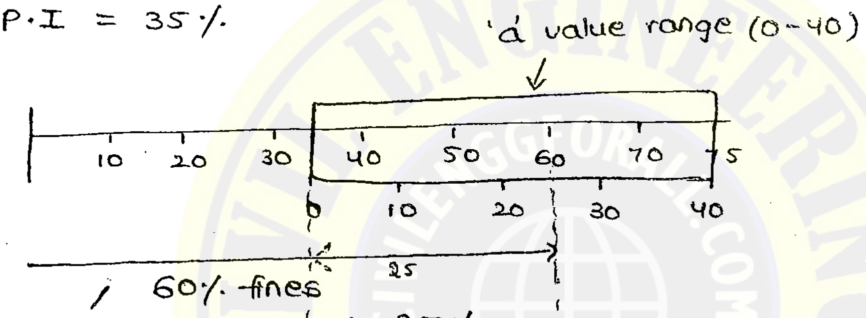
Group index is used to indicate quality of soil within its own group.

Ex:-

L·L = 60%

P·I = 35%





∴ 25%

60 − 35 = 25%

∴ a = 25%

Ex:-) Soil passing through a 75 µ is 78% and L·L = 55%, P·I = 24%. Find Group Index (G·I).

43
63
15
14

A) G·I = 0·2(a) + 0·005 (a·c) + 0·01 (bd)

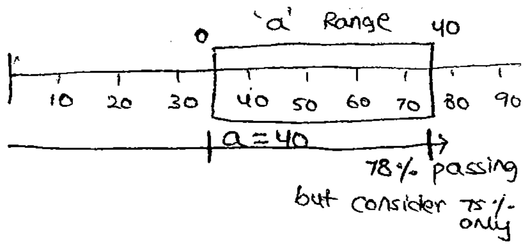
∴ a = 40 b = 40 c = 15% d = 14%

G·I = 0·2(40) + 0·005 (40×15) + 0·01 (40×14)

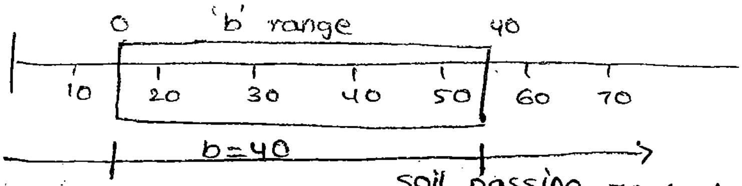
= 8 + 3 + 5·6

= 16·6

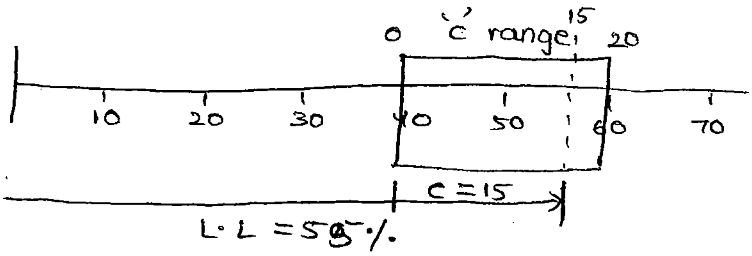
≅ 17

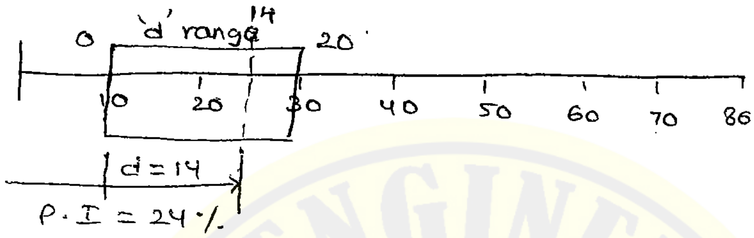


Note:- Actually 'a' value is 43 but in G·I 'a' Ranges 0 to 40, so take 40 (not exceed 40). Actually 'b' value is 63, but 'b' Ranges is 0 to 40. so consider 'b' is 40 (not 63).



soil passing 78%. but consider 'b' max range is 55%.





Plasticity chart :-

C → clay   L → Low compressibility

M → silt   I → Intermediate compressibility

O → Organic silt   H → High compressibility.

Ex:- $L \cdot L = 30\%$

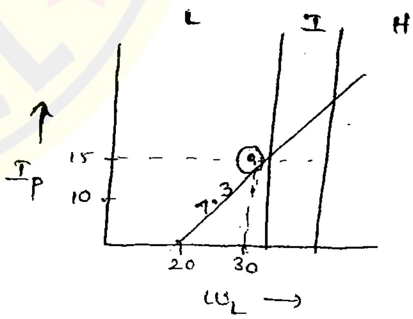
$P \cdot L = 15\%$

$I_p = 30 - 15 = 15\%$

$A\text{-line} = 0.73(L \cdot L - 20)$

$= 0.73(30 - 20)$

$= 7.3$



⇒ CL

Note:- If I_p value is above the A-line (7.3) then soil comes under clay. Liquid limit is given 30%. it is in L-zone (Low zone). Then the soil classification is CL

∴ plasticity chart is used only for Fine grained soils.

50% of soil is retained on 75 μ sieve

C·G·S (>50% retained)   F·G·S (<50% retained)

Coarse fraction :-

(1000 gm) → 600 gm

(1000) 60% retained — $75\,\mu$ sieve — 40% fines

(600) 400 gm gravel — 4.75 mm sieve — 200 gm are sand

If 50% is retained on 4.75 mm sieve then the coarse fraction is gravel.

∴ Gravel > sand.

Note:- SC – clayey sand (sand is major part, sand > clay)

clayey, silty whichever is end with ey, ty (y) that is minor part.

Ex:- clayey silty gravel

clay < silt < Gravel.

Gravel :-

GW	F < 5%	C_u > 4%	C_c = 1 to 3
GP	F < 5%		
GM	F > 12%	silt > clay	
GC	F > 12%	clay > silt	

Sand :-

SW	F < 5%	$C_u > 6$, $C_c = 1-3$	
SP	F < 5%		
SM	F > 12%	silt > clay	
SC	F > 12%	clay > silt	

If F = 5–12%. Dual symbol

Gravel :-

GW – GC, GP – GC } clay > silt (Atterberg limits) $C_u > 4$, $C_c = 1-3$

GW – GM, GP – GM } silt > clay $C_u > 4$, $C_c = 1$ to 3

Sand :-

clay > silt

SW – SC, SP – SC } $C_u > 6$, $C_c = 1-3$

silt > clay

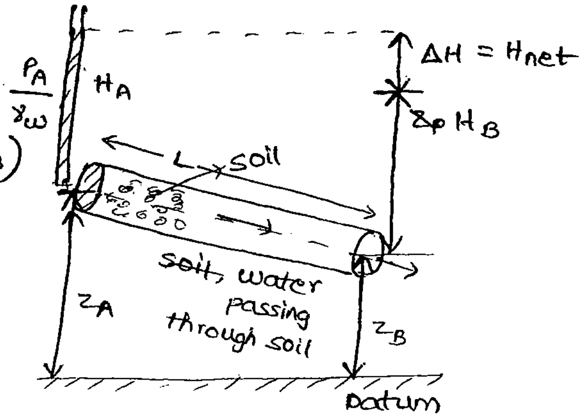
SW – SM, SP – SM } $C_u > 6$, $C_c = 1-3$

29-11-2017

UNIT - 6

PERMEABILITY

Darcy's Law:-



$$\frac{V_A^2}{2g} + \left(\frac{P_A}{\gamma_w} + Z_A\right) \qquad \frac{V_B^2}{2g} + \left(\frac{P_B}{\gamma_w} + Z_B\right)$$

↓

potential head

(or)

Total head

$$\Delta H = \left[\frac{P_A}{\gamma_w} + Z_A\right] - \left[\frac{P_B}{\gamma_w} + Z_B\right]$$

For flow through soils

$$V \propto i$$

V = average velocity (or) Discharge velocity (or) superficial velocity

i = hydraulic gradient = $\frac{\Delta H}{L}$ (no units)

$V = ki$

Multiply both sides by Area(A)

$V \cdot A = kiA$

$\boxed{Q = kiA}$

$$\frac{m^3}{sec} = \frac{m}{sec} \times m^2$$

when $i = 1$, $V = K$

K = coefficient of permeability

A = c/s of soil fabric = $(A_v + A_s)$

$AV = A_{\cancel{seepage}\ voids} V_{seepage}$

$$V_{seepage} = V \cdot \frac{A}{A_{voids}}$$

$$= (V_{discharge}) \cdot \frac{1}{\left(\frac{A_v}{A} \cdot \frac{H}{H}\right)}$$

$$\boxed{V_{seepage} = \frac{V_{discharge}}{\frac{V_v}{V}} = \frac{V_{discharge}}{n}}$$

$$0\% < n < 100\%$$

Note:- seepage velocity always greater than Discharge velocity

$V_{seepage} > V_{discharge}$

$$V_{seepage} \propto V_{discharge} \propto i$$

$$V_{seepage} \propto i$$

$$\frac{V_{seepage}}{V_{discharge}} = \frac{K_p\, i}{k\, i}$$

$$K_{percolation} = \frac{V_{seepage}}{V_{discharge}} \cdot K$$

$$K_p = \frac{k}{n}$$

$\therefore$ K_p is always greater than 'k'.

$\rightarrow$ $Q = KiA$

$$Q = \left[C \cdot D_{10}^2 \cdot \frac{\gamma_w}{\mu_w} \cdot \frac{e^3}{1+e}\right] iA$$

D = effective grain size

μ = viscosity of percolating fluid

e = void ratio

C = shape constant depends upon the type of soil.

property of percolating fluid :-

$$K \propto \gamma_w \propto \frac{1}{\mu_{fluid}}$$

$$K_1 \mu_{1\,(27^\circ)} = K_2 \mu_{2\,(40^\circ)}$$

$$K_{40^\circ} = K_{27^\circ} \cdot \frac{\mu_{27^\circ c}}{\mu_{40^\circ c}}$$

Void ratio:-

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

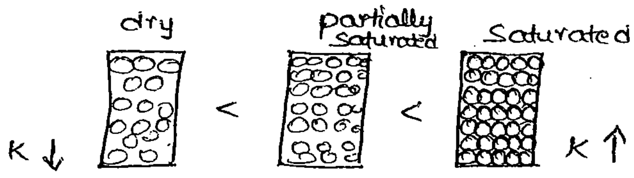
The above equation is not valid for clays, it is valid for Sand and Gravel (C.G.S material)

$10^0 = 1$

Before compaction	After compaction
$K = 10^0$ cm/sec	$K = ?$
$e = 0.9$	$e = 0.35$

$$\frac{K_{BC} \propto \frac{e^3_{BC}}{1+e_{BC}}}{K_{AC} \propto \frac{e^3_{AC}}{\ldots}} \Rightarrow K_{AC} = 0.08$$

permeability of different types of soil :-



$K_{dry} < K_{p.s} < K_s$

'k' is high in saturated soil.

In clay permeability is low because it having a adsorbed water.

Testing (or) Determination of permeability (k):-

permeability (k)

- Lab test (small samples) (Un-disturbed) (Less time for conducting test)
  - constant head
  - variable (or) Falling head
  - Horizontal capillary permeability test
- Field test (insitu) (More accurate method for 'k'.) (small samples) (Un-disturbed)
  - Open well (pumping out) This test is accu, used for soil
  - Tube well (pumping in) This test is accurately used for rock.
- Indirect method ~~(Lab)~~ (consolidation)

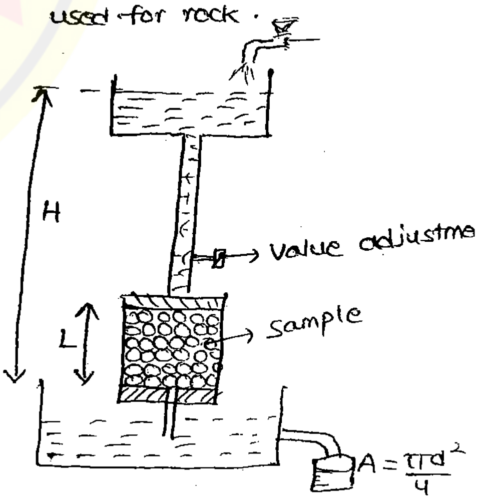
Constant head test :-

$$Q = kiA$$

$$Q = k \cdot \frac{H}{L} \cdot A$$

$$\therefore k = \frac{QL}{AH}$$

$$k = \frac{vol.}{t}\left(\frac{L}{AH}\right)$$



P.g NO:- 28.

4. Given $A = 10\ cm^2$, $i = 0.5$, $t = 60\ sec$ $V = 600\ c.c.$

$$Q = kiA$$

$$k = \frac{Q}{iA}$$

$$= \frac{\left(\frac{V}{t}\right)}{0.5 \times \frac{\pi(10)^2}{4}}$$

$$= \frac{600/60}{0.5 \times \pi(10)^2 \ldots} = 2.0\ cm/s$$

1. Temperature ↑ $\mu \downarrow = 0.75$

Temperature ↑ $\gamma_w \downarrow = 0.9$

condition - ① | condition - ②

Low temp. | High temp

$\mu = \mu_1$ | $\mu_2 = 0.75(\mu_1)$

$\gamma_w = \gamma_{w_1}$ | $\gamma_{w_2} = 0.90(\gamma_{w_1})$

$k = k_1$ | $k = k_2$

$$k_2 \propto \frac{\gamma w_2}{\mu w_2}$$

$$k_1 \propto \frac{\gamma w_1}{\mu w_1}$$

$$\frac{k_2}{k_1} = \frac{\overset{\to 0.9\gamma w_1}{\gamma w_2}}{\gamma w_1} \times \frac{\mu w_1}{\underset{\to 0.75\mu_1}{\mu w_2}}$$

$$k_2 = k_1 (1.20)$$

Variable head test (or) Falling head test :-

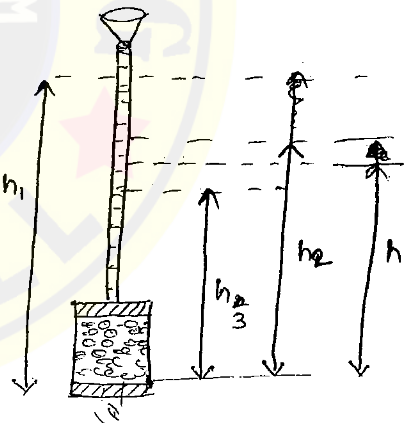
$$\Delta q = k \, i A$$

$$-\left(a \cdot \frac{dh}{dt}\right) = k \cdot \frac{h}{L} \cdot A$$

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

$$[\log_e h]_{h_1}^{h_2} = -k \cdot \frac{A}{a \cdot L}(t_2 - t_1)$$

$$\boxed{k = \frac{aL}{At} \log_e\left(\frac{h_1}{h_2}\right)}$$



Highly pervious soil use larger diameter pipe. Low pervious soil small diameter pipe is used.

P. Q No:- 28

2. $$\frac{a\cancel{L}}{At} \cancel{\log_e}\left(\frac{h_1}{h_2}\right) = \frac{a\cancel{L}}{At} \cancel{\log_e}\left(\frac{h_2}{h_3}\right)$$ ∴ time is same

$$\frac{h_1}{h_2} = \frac{h_2}{h_3}$$

$$h_2^2 = h_1 \times h_3$$

$$h_2 = \sqrt{h_1 \times h_3}$$

Note:-

Fine sand is suitable for both constant head, and variable (or) Falling head test.

Ex:-)

constant head

$V = 600\ cc$

$t = 60\ sec$

$A = 100\ cm^2$

$H = 100\ cm$

$L = 10\ cm$

Same sample is tested on variable head test

$h_1 = 100\ cm$

$h_2 = 75\ cm$

$a = 10\ cm^2$

$t = ?$

$$K = \frac{a \cdot L}{A t} \log_e\left(\frac{h_1}{h_2}\right)$$

$$\frac{Q}{iA} = \cancel{10}\left(\frac{aL}{At} \log_e\left(\frac{h_1}{h_2}\right)\right.$$

$$\frac{Q\cancel{L}}{h\cancel{A}} = \frac{a\cancel{L}}{\cancel{A}t} \log_e\left(\frac{h_1}{h_2}\right)$$

$$\frac{\left(\frac{V}{t}\right)}{H} = \frac{a}{t} \log_e\left(\frac{h_1}{h_2}\right)$$

$$\frac{\frac{600}{60}}{100} = \frac{10}{t} \log_e\left(\frac{100}{75}\right)$$

$$t = \cancel{12.49\ sec}\ 28.76\ sec.$$

Capillary permeability test:-

$$\frac{x_2^2 - x_1^2}{t_2 - t_1} = \frac{2 \cdot K_{u.s}}{s \times n} \cdot h_1 + h_c$$

$$\frac{x_2^2 - x_1^2}{t_2 - t_1} = C \times K_{u.s} \times (h_2 + h_c).$$

Note:-

Acquifer is used for constructing of wells.

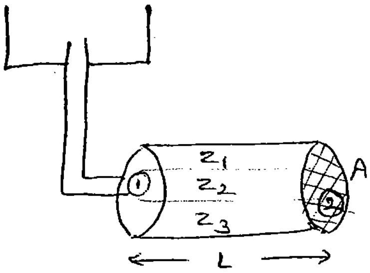
Acquifer - Gravel, sand

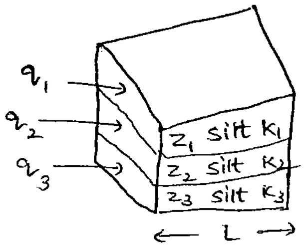
Aquiclude - clay

$$K_{acquifer} = \frac{Q \ln\left(\frac{r_2}{r_1}\right)}{\pi(h_2^2 - h_1^2)} = \frac{Q \ln \frac{R}{r_w}}{\pi(H^2 - h_w^2)} \rightarrow \text{pumping out test}$$

permeability of stratified soil deposits:-

a. Flow is parallel to planes of stratification.





Loss of head or hydraulic gradient is same for all layers

$$i = \frac{H_{net}}{L}$$

$$i = i_1 + i_2 + i_3 + \cdots$$

$$q = q_1 + q_2 + q_3 \cdots$$

$$\cancel{q_T = \Sigma (q}$$

$$q_T \text{ (through entire soil)} = q_1 + q_2 + q_3 + \cdots$$

$$k i A = k_1 i_1 A_1 + k_2 i_2 A_2 + k_3 i_3 A_3 + \cdots$$

'i' is same

$$K_{\text{entire soil}} = K_1(A_1) + K_2(A_2) + K_3(A_3) + \cdots$$

$$K_{\text{horizontal}} = K_1(z_1 \times 1) + K_2(z_2 \times 1) + K_3(z_3 \times 1) + \cdots$$

$$K_h (A_T \times 1) = K_1 z_1 + K_2 z_2 + K_3 z_3 + \cdots$$

$$K_{avg} (1 \times \Sigma z_1) = K_1 z_1 + K_2 z_2 + K_3 z_3 + \cdots$$

$$K_{avg} = \frac{\Sigma K_1 z_1}{\Sigma z_1}$$

$$K_H = \frac{\Sigma (K_1 z_1)}{z_1}$$

$$\boxed{K_H = \frac{K_1 z_1 + K_2 z_2 + K_3 H_3 + \cdots}{z_1 + z_2 + z_3 + \cdots}}$$

b. Flow through perpendicular to stratification:

$$\boxed{K_V = \frac{z_1 + z_2 + z_3 + \cdots}{\frac{z_1}{K_1} + \frac{z_2}{K_2} + \frac{z_3}{K_3} + \cdots}}$$

$\therefore$ K_H is always greater than K_V.

P.g NO:- 27

Ex:- 3)

$$K_H = \frac{K_1 z_1 + K_2 z_2 + K_3 z_3}{z_1 + z_2 + z_3}$$

$$= \frac{10^{-4}(1) + 10^{-3}(1) + 10^{-4}(1)}{1+1+1}$$

$$= \frac{10^{-4}[1 + 10 + 1]}{3}$$

$$K_H = 4 \times 10^{-4} \text{ cm/s}$$

$z = 1$	10^{-4} cm/sec
$z = 1$	10^{-3}
$z = 1$	10^{-4}

$\therefore 10^{-3} \Rightarrow 10^{1} \times 10^{-4}$

P.g NO:- 26

3. Given $K_1 = 4 \times 10^{-5}$ cm/sec

$\mu = \mu_1$ $\mu_2 = 0.5\mu_2$ (half reduced)

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

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

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

$$= K_1 \left(\frac{\mu_1}{0.5\mu_2} \right)$$

$$= \frac{4 \times 10^{-5}}{0.5}$$

$$K_2 = 8 \times 10^{-5}$$

30-11-2014

UNIT-8

SEEPAGE PRESSURE AND CRITICAL HYDRAULIC GRADIENT

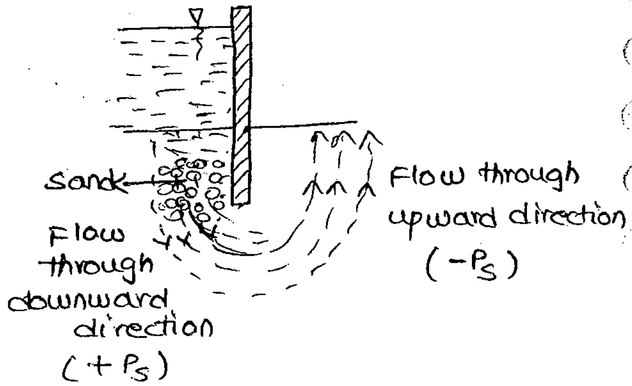
Seepage pressure (P_s):-

$P_s = \pm \gamma_w h$

$P_s = i z \cdot \gamma_w = \gamma_w i L$

σ' is for upward flow is decreases

σ' for downward flow is increases



Critical hydraulic gradient and quick sand condition:-

$\sigma' = 0 = \gamma' L - P_s$

$0 = \gamma' L - P_s$

$\gamma' L = P_s$

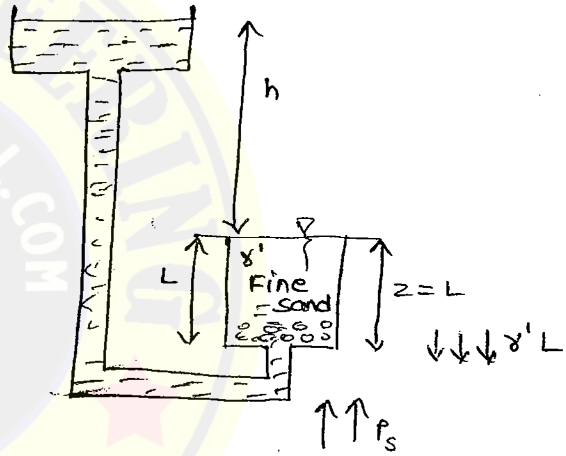
$\gamma' L = \gamma_w i_c L$

$\gamma' = \gamma_w i_c$

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

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

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



Void ratio in dense state = 0.35

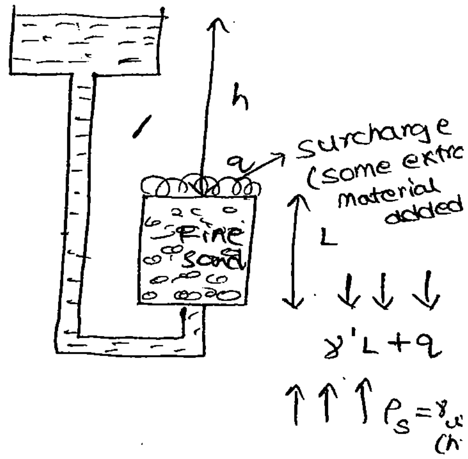
Medium state =

Loose state = 0.91

Quick sand condition occurs when the soil having a low permeability (fine sands). Quick sand condition not occur for coarse sand or gravel.

Seepage pressure increases Quick sand condition will be reduced.

Effect of surcharge on quick condition :-



$$\sigma' = 0$$

$$q + \gamma' L = (h + L)\gamma_w$$

$$q + \gamma' L = \gamma_w h + \gamma_w L$$

$$\boxed{\frac{q + \gamma' L - \gamma_w h}{\gamma_w} = h}$$

Piping:-

1. $\dfrac{i_c}{i_{exit}\ (\text{smaller})} = F \cdot S$ (Factor of Safety)

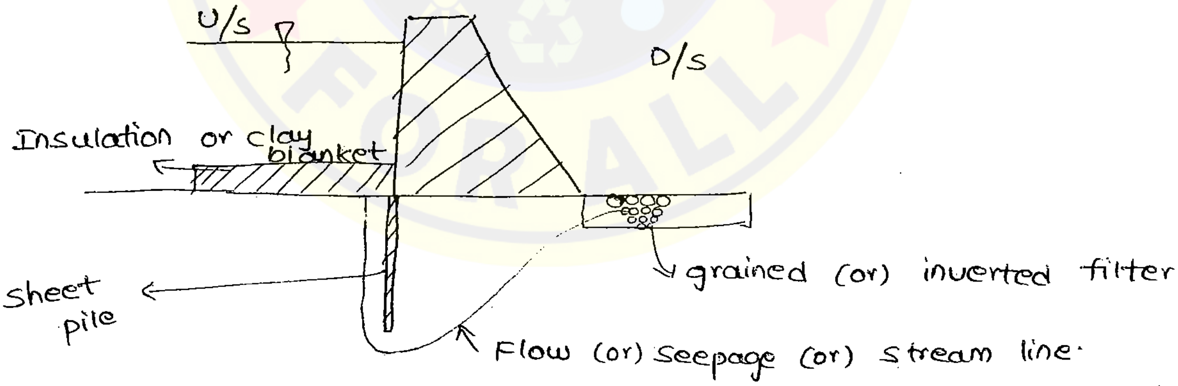
   $\therefore$ $F \cdot S > 1$ (No piping) (or) safe criteria

2. $\dfrac{i_c}{i_{exit}\ (\text{greater})} = F \cdot S$.

   $\therefore$ $F \cdot S < 1$ (piping occur) (or) Unsafe criteria

3. $\dfrac{i_c}{i_{exit}} = F \cdot S$

   $\therefore$ $F \cdot S = 1$ (critical)



P.g No:- 35

1. Given $e = 0.62$ $G = 2.62$ $F \cdot S = 5$

$$F \cdot S = \frac{i_c}{i_{exit}}$$

$$F \cdot S = \frac{\frac{G-1}{1+e}}{i_{exit}}$$

$$i_{exit} = \frac{\frac{2.62 - 1}{1 + 0.62}}{5} = \frac{1}{5} = 0.2$$

2. Given, $G = 2.7$, $e = 0.7$, $L = 2m$

Quick sand condition is occur then $F.S = 1$

$$F.S = \frac{i_c}{i_{exit}}$$

$$1 = \frac{i_c}{i_{exit}}$$

$$i_c = i_{exit}$$

$$\frac{G-1}{1+e} = i_{exit}$$

$$\frac{2.7-1}{1+0.7} = i_{exit}$$

$$1 = i_{exit} = \frac{h}{L}$$

$$h = L = 2m.$$

22-11 (21)

UNIT - 12
COMPACTION

compaction:-

Compaction is artificial process in which partially saturated soils will be compacted by Mechanical means to expell most of the air voids instantly. These is one of the economical ground improvement technique.

Benefits of compaction:-

1. void ratio decreases
2. permeability decreases
3. Future compressibility will be lesser
4. Erodability will reduce.
5. Durability of structure increases
6. Stability increases

Objective of compaction:-

compaction test are done to determine

a. amount of compaction

b. the optimum moisture content (OMC)

P.9 NO:- 55

8. $$\frac{\text{Energy}}{cm^3} = \frac{\text{wt. of hammer} \times \text{ht. of fall} \times \text{No. of layers} \times \text{blows per each lay}}{\text{volume of soil compaction}}$$

$$\frac{\text{Energy}}{cm^3} = \frac{2.6 \times 30.48 \times 3 \times 25}{1}$$

$= 5,952$ kg-cm.

$= 59.52$ kg-m

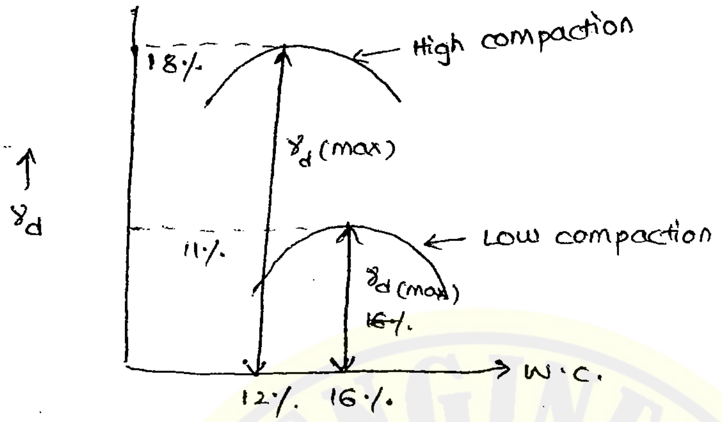
Note:-

$$\frac{\text{Energy}_M}{\text{Energy}_S} = \frac{49 \times 45 \times 5 \times 25 / 1000}{2.6 \times 30.48 \times 3 \times \cancel{25} / \cancel{1000}} = \frac{1102.5}{237.74} = 4.637$$

Compaction Test (lab):-

1. To judge the suitability of soil for specific use.

Comparison between Low compaction and High compaction (or) Standard proctors test and Modified proctor test.



Dry density increases, OMC decreases

→ 0% air voids line and 100% saturated line are <u>same</u>.

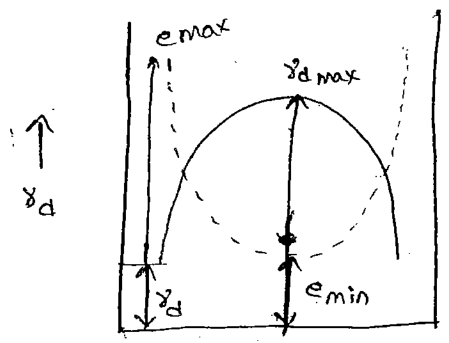
$$\gamma_{dry\,theoritical} = \frac{G\gamma_w}{1+Gw} = \gamma_d \text{ (saturated line)}$$

(0% line)

$$\gamma_{d(sat)} = \frac{G\gamma_w}{1+\frac{Gw}{S}} \quad \therefore S = 1 \text{ (saturated)}$$

→ 95% of Saturation line and 5% of Air content are <u>same</u>

→ 95% of saturation line and 5% of air voids are <u>different</u>



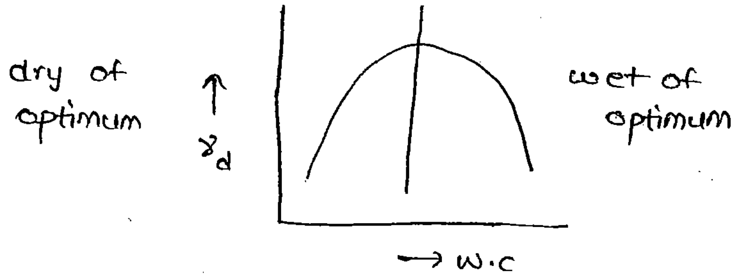
$$e = \frac{G\gamma_w}{\gamma_d} - 1$$

Relative compaction:-

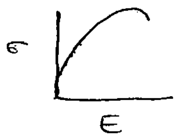
$$= \frac{\text{dry density in the field}}{\text{Max. dry density obtained in the lab}} \times 100$$

$$= \frac{\gamma_d (field)}{\gamma_d (max)} \times 100$$

Dry of optimum:-



1. Floculated	1. Dispersion
2. water is deficient	2. water is excess
3. permeability is more	3. permeability is less
4. swelling is more.	4. swelling is less



5. pore water pressure ↑ (U)	5. pore water pressure ↓ (U)

23-11-201

UNIT - 7

EFFECTIVE STRESS

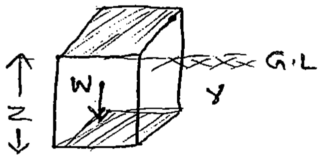
Stress :-

$W = \gamma V$

$= \gamma [A \cdot Z]$

$\sigma = \frac{F}{A} = \frac{W}{A} = \frac{\gamma A Z}{A}$

$\boxed{\therefore \sigma = \frac{F}{A} = \gamma Z}$



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

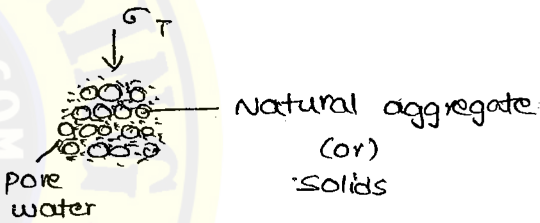
$\gamma V = W$

Stresses

- Geo stress
- stress due to applied load

Saturated soil :-

$\sigma_T = \sigma' + U$

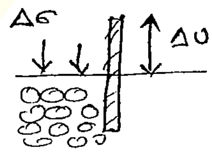


σ_T, U are measurable

σ' = non measurable

U = pore water pressure (or) Neutral pressure (or) hydrostatic pressure.

$\boxed{\sigma' = \sigma_T - U}$



ΔU = Hydro dynamic pressure (or) excess of pore water pressure Occur when applied the load.

$\Delta \sigma$ = Applied stress

σ' = effective stress.

Note:-

1. When there is no water table (or) soil is completely dry

$\boxed{\sigma' = \sigma_T}$ $\therefore (U = 0)$

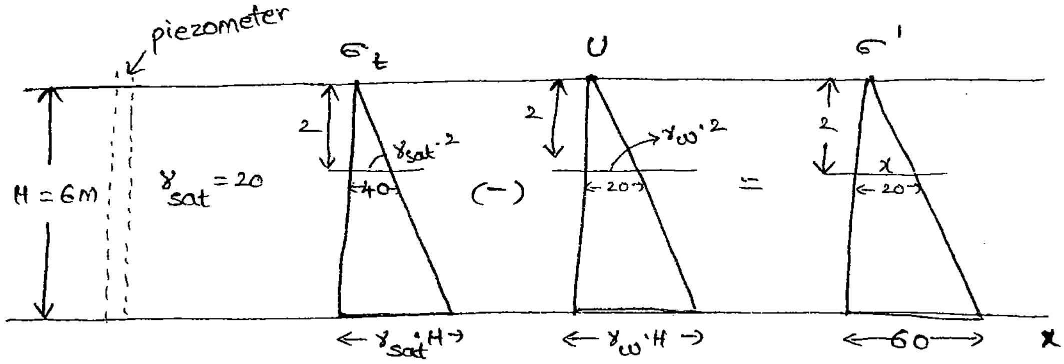
$\downarrow$

$\sigma (\gamma_{dry} \cdot H)$

2. when soil is wet, $\boxed{\sigma_T = \sigma'}$ $\rightarrow \gamma_{wet} \times H$

3. when soil is saturated (Non capillary), $\boxed{\sigma' = \sigma_T}$ $\rightarrow \gamma_{sat} \times H$

Ex:-)



20×6
$= 120$

10×6
$= 60$

$\frac{2}{x} = \frac{6}{60}$

$x = 20$

(or)

$x = \gamma_{sat} \cdot H - \gamma_w \cdot H$

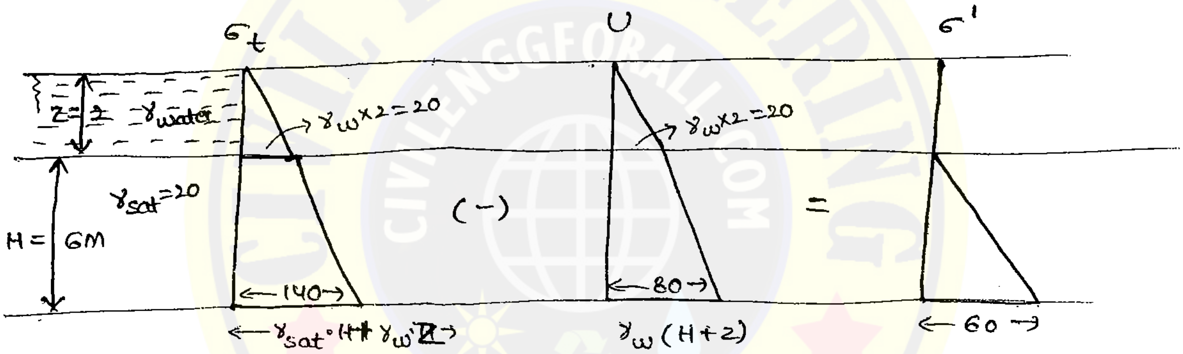
$x = (\gamma_{sat} - \gamma_w) H$

$x = \gamma' \cdot H$

$\gamma' = 20 - 10 = 10$

$x = 10 \times 2 = 20$

Ex:-)

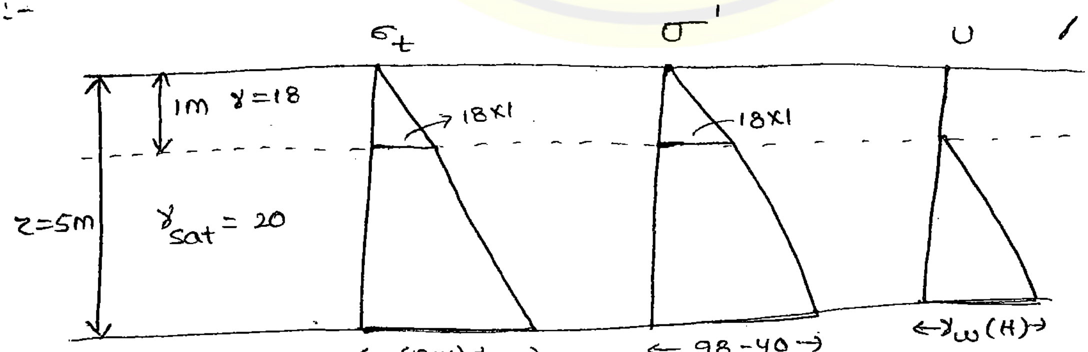


$\leftarrow \gamma_{sat} \cdot H + \gamma_w \cdot 2 \rightarrow$
$= 20 \times 6 + 20$
$= 120 + 20$
$= 140$

$\gamma_w (H + 2)$
$= 10(6 + 2)$
$= 80$

$\Rightarrow 140 - 80$
$= 60$

Ex:-



$\leftarrow (18 \times 1) + \rightarrow$
(20×4)
$= 98$

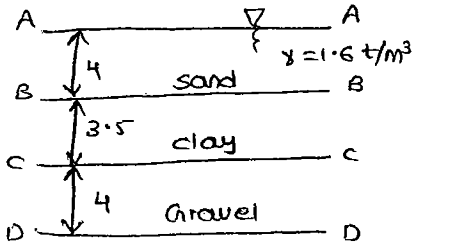
$\leftarrow 98 - 40 \rightarrow$
$= 58$

$\leftarrow \gamma_w (H) \rightarrow$
$10(4)$
$= 40$

EX:- 1) $\sigma_{cc} = (\gamma')(4) + (\gamma')(3.5)$

$= (0.6)(4) + (0.9)(3.5)$

$= 5.5\ T/m^2$



$\sigma_{DD} = (5.5) + (0.925)(4)$

$= 9.25\ T/m^2$

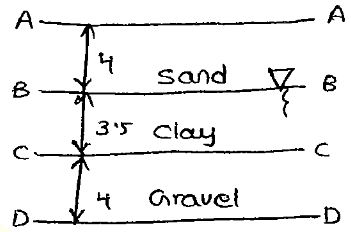
$\therefore \gamma_{sub} = \gamma_{sat} - \gamma_{water}$

$\sigma_{cc} = (1.6)(4) + (0.9 \times 3.5)$

$= 9.55\ T/m^2$

$\sigma_{DD} = 9.55 + 0.925(4)$

$= 13.25\ T/m^2$

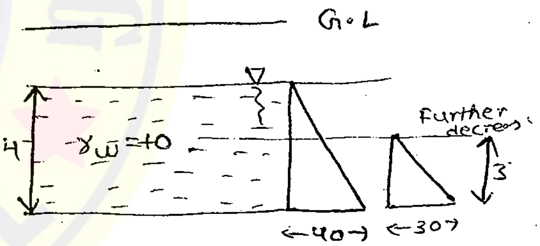


Note:-

$\gamma_w = 1\ T/m^3$ (or) $10\ KN/m^3$ (or) $62.5\ lb/m^3$

8. $\therefore \uparrow \sigma' = \sigma_T + U \downarrow$

If water table decreases below the ground level "pore water decreases" and effective stress increases.



(Assume)

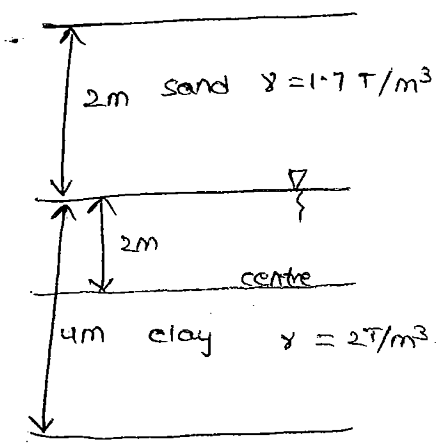
$\therefore \downarrow \sigma' = \sigma_T + U \uparrow$

If water table is rise gradually above the ground then pore water increases and effective stress decreases.

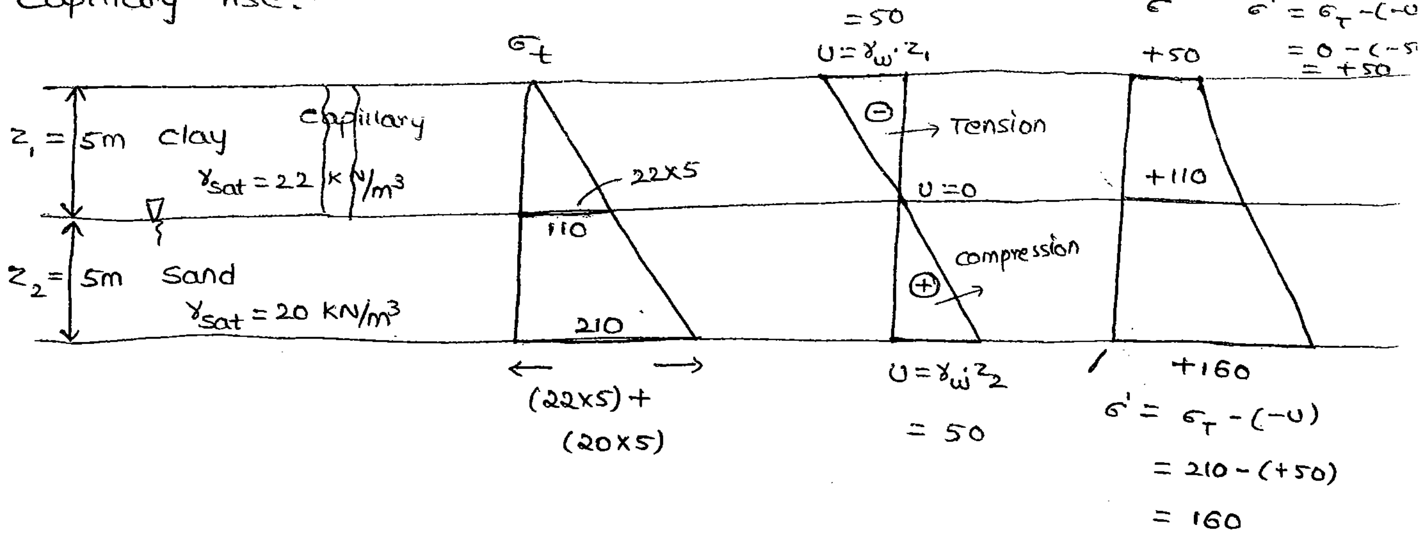
19. $\sigma'_{\text{centre}} = (1.7)(2) + \underset{\text{submerged}}{(2-1)}(2)$

$= 3.4 + 2$

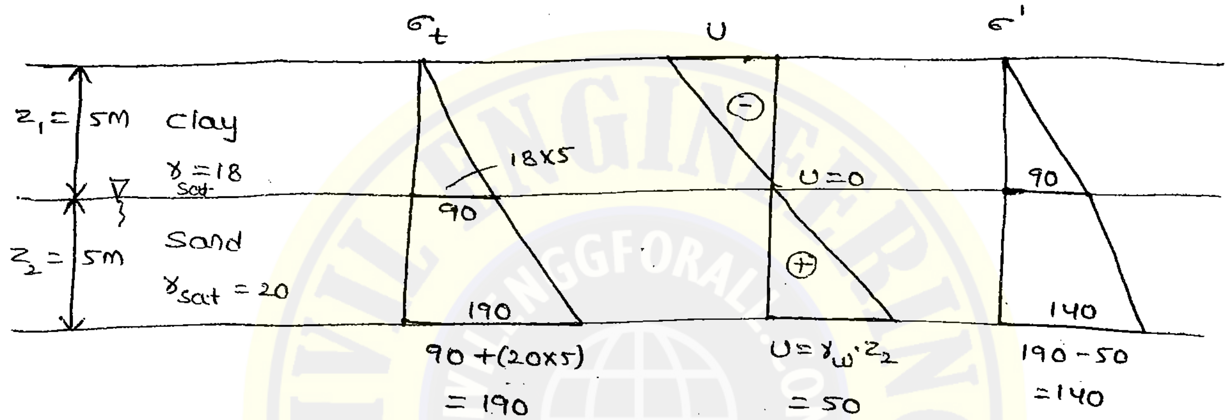
$= 5.4\ t/m^2$



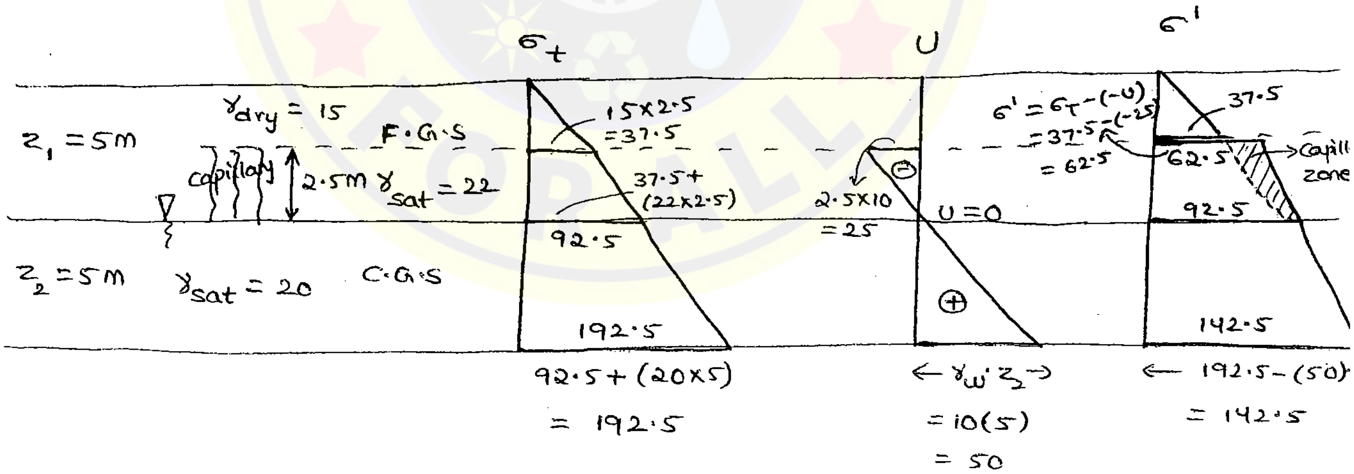
Capillary rise:-



Without capillary



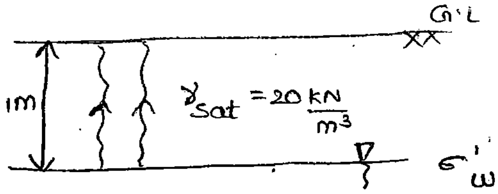
Ex:-



Note:-

When capillary is increasing up then effective stress is increasing. If capillary is decreasing downwards then effective stress decreasing.

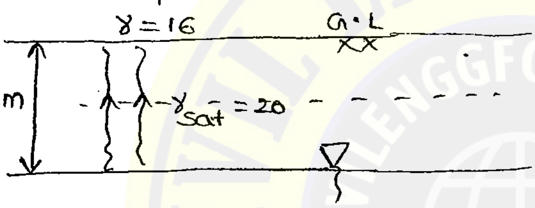
22.



$$\sigma'_{W.T} = (\sigma_T) - (u)$$
$$= \sigma_T - 0$$
$$= \gamma_{sat}(z) - 0$$
$$= 20 \times 1$$
$$\sigma'_{W.T} = 20 \ kN/m^2$$

$$\sigma'_{G.L} = \sigma_T - u$$
$$= 0 - (-10)$$
$$= 10 \ kN/m^2$$

EX:-)

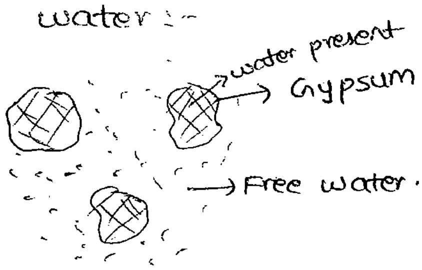


$$\sigma'_{0.5m \text{ from } G.L} = 16(0.5) - (-(0.5 \times 1))$$
$$= 9 + 0.5$$
$$= 9.5$$

Water in soil :-

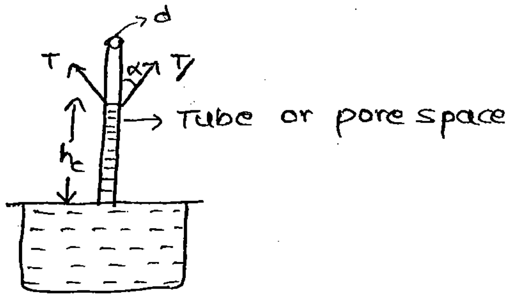
- Held water (against gravity) :-
2. structural water (any soil)
3. Adsorbed water (clay)
4. capillary water (Fine, Medium, Coarse sand, silt and clay)

Structural water :-



In Gypsum (water present + free water) can be removed in oven dry at ~~110~~ 105°c. But other minerals water is not water present in structure cannot be removed in oven dry at 105°c. If want to remove uses higher temperature (>105°c)

Capillary water :-



$$\gamma_w \cdot V_w = \underbrace{(T\cos\alpha)(\pi d)}_{\text{surface tension force}}$$

$$\gamma_w \cdot \left(\frac{\pi}{4} d^2 \times h_c\right) = (T\cos\alpha)(\pi d)$$

$$h_c = \frac{4\,T\cos\alpha}{\gamma_w\, d}$$

$$h_c \propto \frac{1}{d_{\text{pore space}}}$$

Clays $\quad h_c \uparrow \propto \dfrac{1}{d_{\text{pore space}} \downarrow}$

For clays, $h_c \approx 25 - 30\ m$

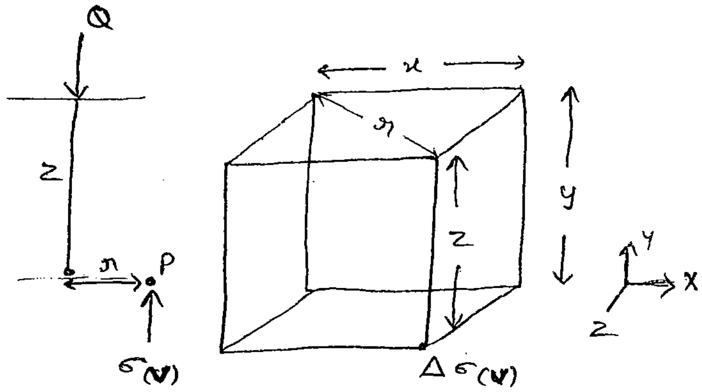
27-11-2014

UNIT - 10

STRESS DISTRIBUTION

Boussinesq's Equation:-

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



$$\Delta\sigma_{V(P)} = \frac{Q}{z^2}(I_B)$$

I_B = Boussinesq's influence factor

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

$I_B = \frac{3}{2\pi}$ (or) 0.4775 (under point load)

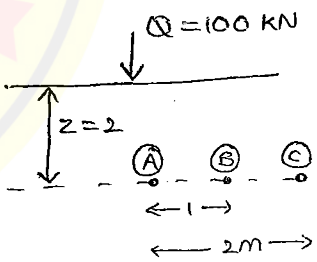
This is applicable for weightless soil.

Westergaard's theory:-

$$\Delta\sigma'_{(V)} = \frac{Q}{z^2}(I_W)$$

$$I_W = \frac{1}{\pi}\left(\frac{1}{1+2\left(\frac{r}{z}\right)^2}\right)^{3/2}$$

Ex:- Calculate the verticle stress at a point by using Boussinesq's equation.



A. Given Q = 100 KN, z = 2

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

$r = 0$ at point A.

$$I_B = \frac{3}{2\pi}$$

$$\Delta\sigma_{V(A)} = \frac{Q}{z^2}(I_B)$$

$$= \frac{100}{2^2}(0.4775)$$

$$\Delta\sigma_{V(A)} = 11.93\ KN/m^2$$

At point B:-

$r = 1$, $z = 2$

$I_B =$

$$\Delta \sigma_{v(B)} = \frac{Q}{z^2}(I_B)$$

$$= \frac{100}{4}($$

$$= 6.825 \text{ KN/m}^2$$

At point c:-

$r = 2, \quad z = 2$

$$I_B = \frac{3}{2\pi}\left(\frac{1}{1+\left(\frac{2}{2}\right)^2}\right)^{5/2}$$

$$I_B =$$

$$\Delta \sigma_{v(c)} = \frac{Q}{z^2}(I_B)$$

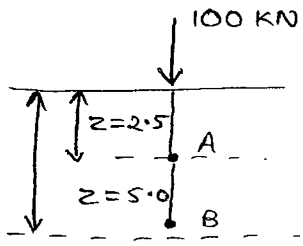
$$= \frac{100}{4}($$

$$= 2.11 \text{ KN/m}^2$$

Note:-

1. If $\frac{r}{z} = 0$, $I_B = \frac{3}{2\pi}$ (or) 0.4775
2. If $\frac{r}{z} = 10$, $I_B = 0$

Ex:- Calculate the stresses by using Boussinesq's equation.



A.

$$\frac{\Delta \sigma_A}{\Delta \sigma_B} = \frac{\frac{Q}{z^2}(I_B)}{\frac{Q}{z^2}(I_B)}$$

$r = 0$, then $I_B = \frac{3}{2\pi}$, $Q = 100$

$$= \frac{\frac{100}{2.5^2}\left(\frac{3}{2\pi}\right)}{\frac{100}{5^2}\left(\frac{3}{2\pi}\right)}$$

$$= \left(\frac{5}{2.5}\right)^2$$

$$\frac{\Delta\sigma_A}{\Delta\sigma_B} = 4$$

10. Given $Q = 700\ KN$, $z = 2m$, $r = 1.0m$, $\sigma_v = ?$

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

$$= \frac{3}{2\pi}\left(\frac{1}{1+\left(\frac{1}{2}\right)^2}\right)^{5/2}$$

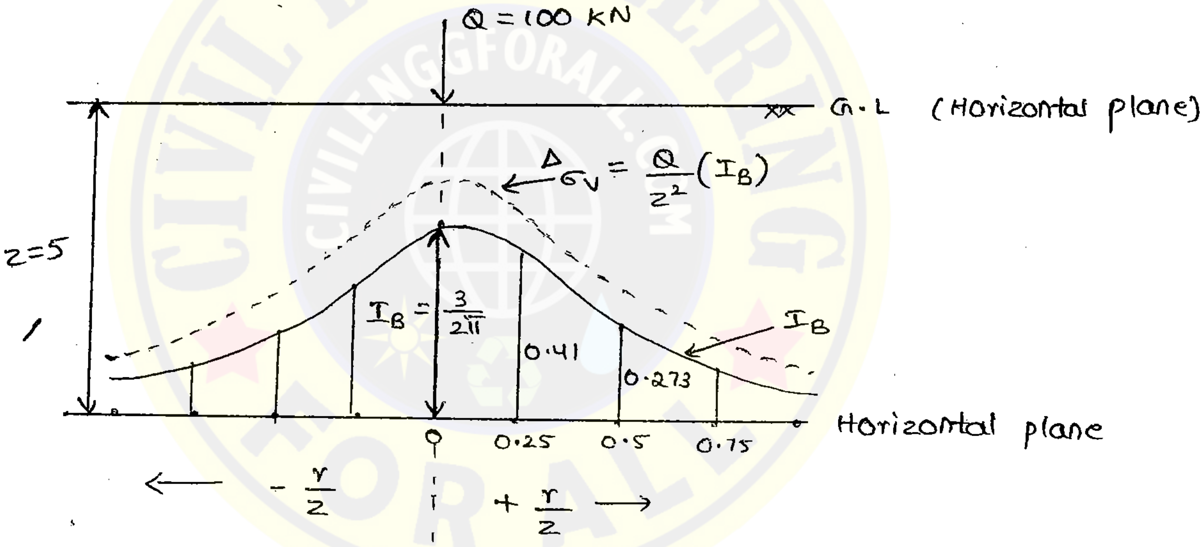
$$I_B = 0.275$$

$$\sigma_v = \frac{Q}{z^2}(I_B)$$

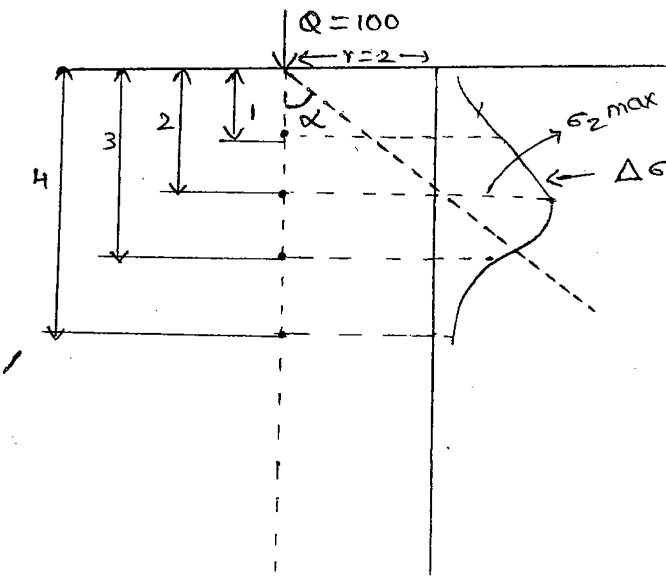
$$= \frac{700}{2^2}(0.27)$$

$$\sigma_v = 47.8\ K.pa\ (or)\ K.N/m^2$$

~~Ex~~:- Verticle stress distribution on horizontal plane:-



Verticle stress distribution on verticle plane:-



$\Delta\sigma_v$ along vertical plane (r=2m)

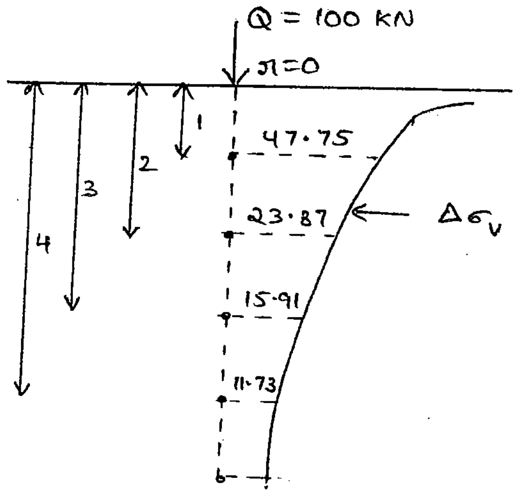
$\therefore \frac{r}{z} = 0.817$ (σ_z max)

$\alpha = 39^\circ 15'$

$\frac{r}{z} = \tan 39^\circ 15'$

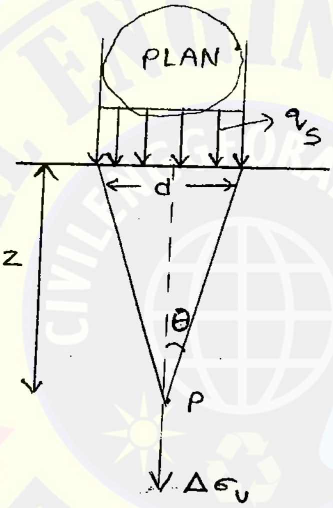
$= 0.817$

Verticle stress distribution on verticle plane $(r=0)$:-



$$\Delta\sigma_v = \frac{3}{2\pi}\left(\frac{Q}{z^2}\right)$$

Verticle stress due to circular loaded area :-



$$\tan\theta = \frac{r}{z}$$

$$\Delta\sigma_v = q_s(I_{circular})$$

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

$$\Delta\sigma_v = q_s\left[1-\cos^3\theta\right]$$

q_s = load intensity.

P.g NO:- 44

8. Given $d = 2m$, $q_s = 100\ kN/m^2$, $z = 1m$, $r = 1m$

$$\tan\theta = \frac{r}{z} = \frac{1}{1} = 1$$

$$\theta = 45^\circ$$

$$\Delta\sigma_v = q_s(1-\cos^3\theta)$$

$$= 100(1-\cos^3 45)$$

$$= 64.65\ kN/m^2$$

Note:-

If $Q = 750$ kN is given then $q_s = ?$

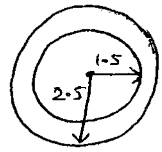
$d = 2$ m

$$q_s = \frac{Q}{A_f}$$

$$q_s = \frac{750}{\frac{\pi(2)^2}{4}} = 238\ kN/m^2$$

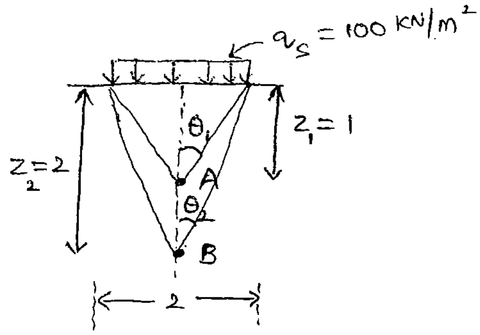
Ex:- For hallow surface

$$\Delta\sigma_v = q_s\left[(1-\cos^3\theta)_{outer} - (1-\cos^3\theta)_{inner}\right]$$



Ex:-

$$\boxed{\frac{\Delta\sigma_A}{\Delta\sigma_B} = \frac{q_s(1-\cos^3\theta_1)_A}{q_s(1-\cos^3\theta_2)_B}}$$



$$\tan\theta_1 = \frac{r}{z_1} = \frac{1}{1}$$

$$\theta_1 = 45^\circ$$

$$\tan\theta_2 = \frac{1}{2} = 0.5$$

$$\theta_2 = 26^\circ\ 33'$$

$$\frac{\Delta\sigma_A}{\Delta\sigma_B} = \frac{100(1-\cos^3 45)}{100(1-\cos^3 26^\circ 33')}$$

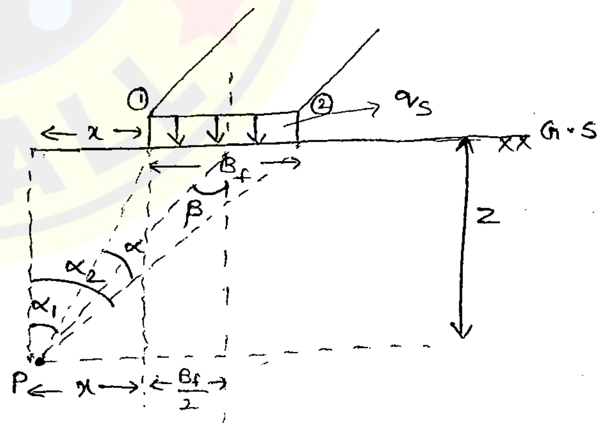
$$= 2.27$$

Strip footing:-

$$\tan\alpha_1 = \frac{x}{z}$$

$$\tan\alpha_2 = \frac{B_f + x}{z}$$

$$\tan\beta = \frac{x + \left(\frac{B_f}{2}\right)}{z}$$

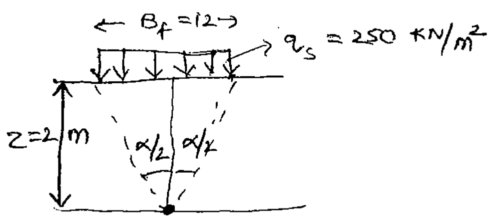


$$\sigma_{z(P)} = \frac{q_s}{\pi}\left[\alpha + \sin\alpha\cos 2\beta\right]$$

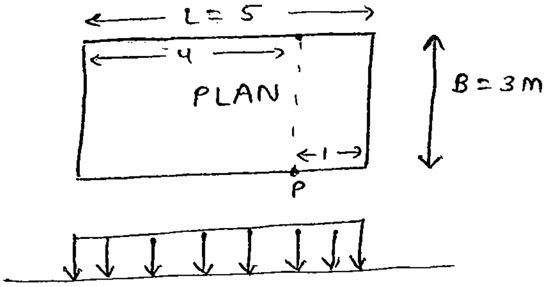
$$\Delta\sigma_{v(centre)} = \frac{q_s}{\pi}\left[\alpha + \sin\alpha\right]$$

Ex:-

$$\sigma_{z(P)}\ \sigma_{v(centre)} = \frac{q_s}{\pi}(\alpha + \sin\alpha)$$

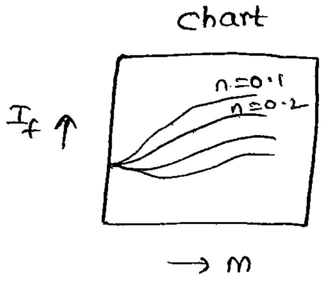


Ex:-)



$Z = 2m$

Find at 'P'.



$$\Delta\sigma_v = q_s\,(I_{f_1} + I_{f_2})$$

For I_{f_1}

$$m = \frac{L_1}{2} = \frac{4}{2}$$

$$n = \frac{B_1}{2} = \frac{3}{2}$$

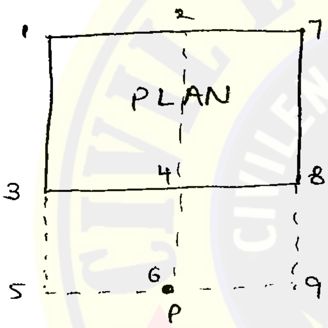
For I_{f_2}

$$m = \frac{L_2}{2} = \frac{1}{2}$$

$$n = \frac{B_2}{2} = \frac{3}{2}$$

using this m, n values in chart we can found I_f

Ex:-)

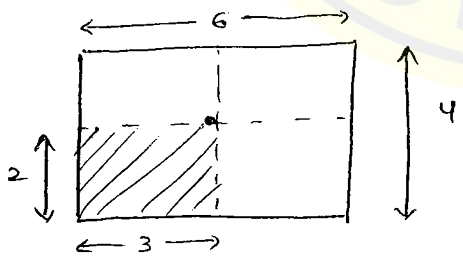


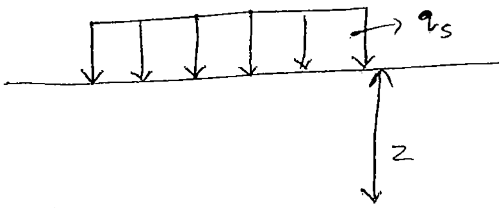
point is given outside the plan so you can drawn imaginary line

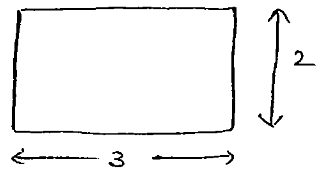
$$\Delta\sigma_v = 2\left[q_s\left(I_{f(1-2-3-4-5-6)} - I_{f(3-4-5-6)}\right)\right]$$

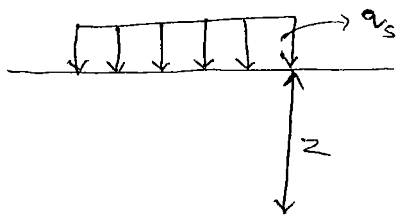
P.g. No:- 44

12.









$\Delta\sigma_v = 100\ kN/m^2$

$$\Delta\sigma_{v\,(centre)} = 4\left[q_s\,(I_f)_{shaded}\right]$$

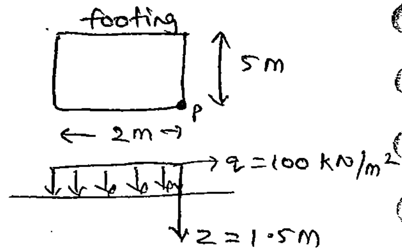
$$= 4\left[\Delta\sigma_{shaded}\right]$$

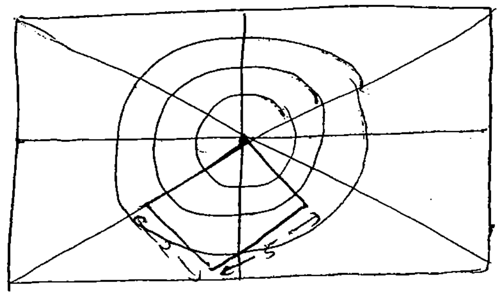
$$= 4\,(100)$$

$$= 400\ kN/m^2$$

Newmark's influence chart:-

$$\Delta\sigma_{v(P)} = q_s (N)(I_N)$$





$N_{max} = \frac{1}{I_N} = 200$ sectors

$N_{min} = 0$ (when point is far away from footing then sectors is zero).

P.g No:- 43

3. Given $I_N = 0.005$, $q = 40\ t/m^2$, $n = 6$

$$\Delta\sigma_{v(c)} = q_s \cdot N \cdot I_N$$

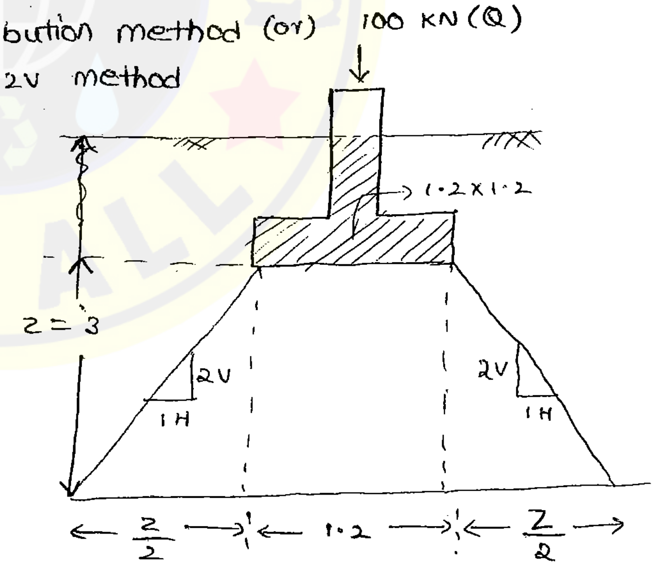
$$= 40 \times 6 \times 0.005$$

$$= 1.2\ t/m^2 \ [\text{centre}]$$

If point is located at corner the $\Delta\sigma_v$ value decreases

$\therefore < 1.2\ t/m^2$ [corner]

Approximate method:- (~~or~~) 26° distribution method (or) 1H : 2V method



$$\Delta\sigma_v = \frac{Q}{A_f}$$

$$= \frac{100}{1.2 \times 1.2} = 69\ kN/m^2$$

$$\Delta\sigma_v = \frac{Q}{(B+z)^2} = \frac{100}{(1.2+3)^2}$$

$$= 5.69\ kN/m^2$$

P.g No:- 44

13. Given $z = 1\ m$

$$\sigma = \frac{F}{A}$$

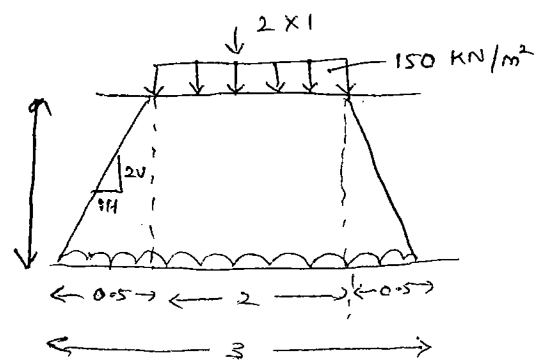
$$F = \sigma \times A$$

$$= 150 \times (2)$$

$$Q = F = 300\ kN$$

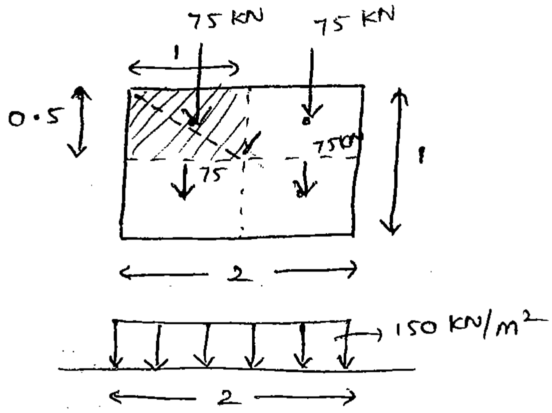
If $2V = 1H$

$1V = 0.5\ H$



$$\Delta \sigma_{v(1m)} = \frac{300 \text{ kN}}{3 \times 2} = 50 \text{ kN/m}^2$$

2) Equivalent point load method :-

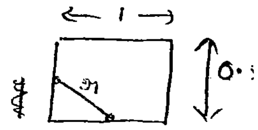


$$F = \sigma \times A$$
$$= 150 \times (0.5 \times 1)$$
$$= 75 \text{ kN}$$

$$r = \sqrt{\left(\frac{0.5}{2}\right)^2 + \left(\frac{1}{2}\right)^2}$$

$$r = 0.56$$

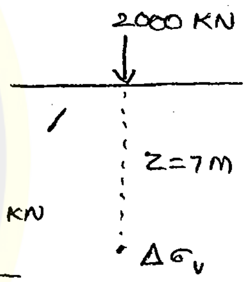
$$\frac{r}{z} = \frac{0.56}{1} = 0.56$$

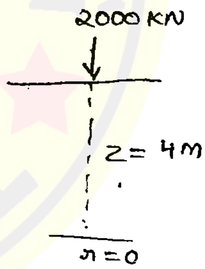


I_B (Bossineq equation formula use) $= 0.24$

$$\Delta \sigma_{v(1m)} = 4\left[\frac{Q}{z^2}(I_B)\right]$$

4. $$\Delta \sigma_v = (I_B) \cdot \frac{Q}{z^2}$$
$$= 0.4775 \times \frac{2000}{7^2}$$
$$= 20 \text{ kN/m}^2$$



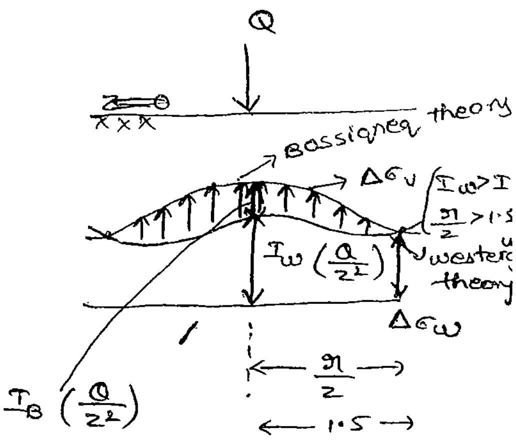


5. $$\tau_{(r/z)} = [\Delta \sigma_v]\frac{r}{z}$$
$$= \left[\frac{3}{2\pi}\left(\frac{1}{1+\left(\frac{r}{z}\right)^2}\right)^{5/2}\frac{Q}{z^2}\right] \cdot \left(\frac{0}{4}\right)$$
$$= 0$$

6. $$I_B\left(\frac{Q}{z^2}\right) = 0.4775$$
$$= \frac{3}{2\pi}$$

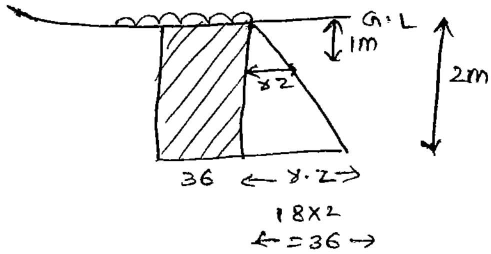
$$I_B = \frac{3}{2\pi}\left(\frac{1}{1+(1.5)^2}\right)^{2.5}$$

$$I_w = \frac{1}{\pi}\left(\frac{1}{1+2(1.5)^2}\right)^{2.5}$$



Bossinesq	Westergaard's
1. Isotropic	1. Anisotropic
2. $\frac{r}{z}=0$, $I_B > I_w$ $\Delta\sigma_{v(B)} > \Delta\sigma_{v(w)}$	2. $\frac{r}{z}=0$, $I_w < I_B$
3. $\frac{r}{z}=1.5$, $I_B = I_w$	3. $\frac{r}{z} > 1.5$, $I_w > I_B$

11. $q = 2 \times 18 = 36\ kN/m^2$



$36\ kN/m^2$

P.g NO:- 44

10.

$$\begin{array}{cccc} & x & y & z \\ & \left[\begin{array}{c} \sigma_x \\ \tau_{xy} \\ \tau_{xz} \end{array}\right. & \begin{array}{c} \tau_{yx} \\ \sigma_y \\ \tau_{xy} \end{array} & \left.\begin{array}{c} \tau_{zx} \\ \tau_{zy} \\ \sigma_z \end{array}\right] \end{array}$$

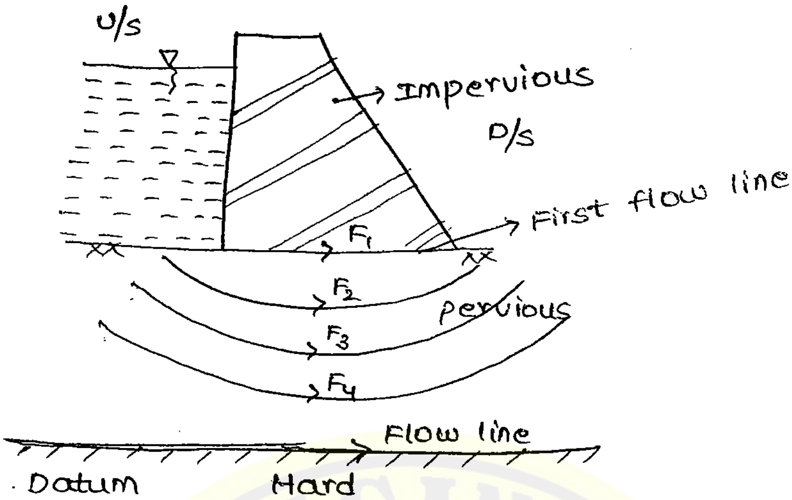
Three normal stresses $(\sigma_x, \sigma_y, \sigma_z)$ and 6 shear stresses

11-12-2014

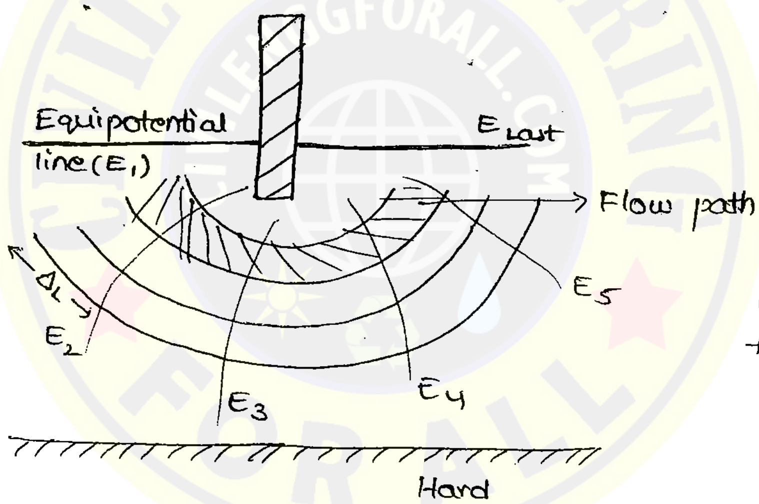
UNIT - 9

SEEPAGE ANALYSIS

Flow line :-



Flow path :-



Discharge is same for all chambers

$H_1 - (H_2) =$ Head capacity seepage

$H_{net} = H_1$

$H_{net} = N_D(\Delta H)$

$\Delta H = \dfrac{H_{net}}{N_D}$

$$\boxed{\Delta H = \frac{H_1 - H_2}{N_D}}$$

Seepage quantity (q):-

$$q = k \cdot i \cdot A$$

$$= k \cdot \frac{\Delta h}{\Delta x} \cdot (1 \times \Delta y)$$

$$= k \cdot \Delta h \cdot \frac{\Delta y}{\Delta x}$$

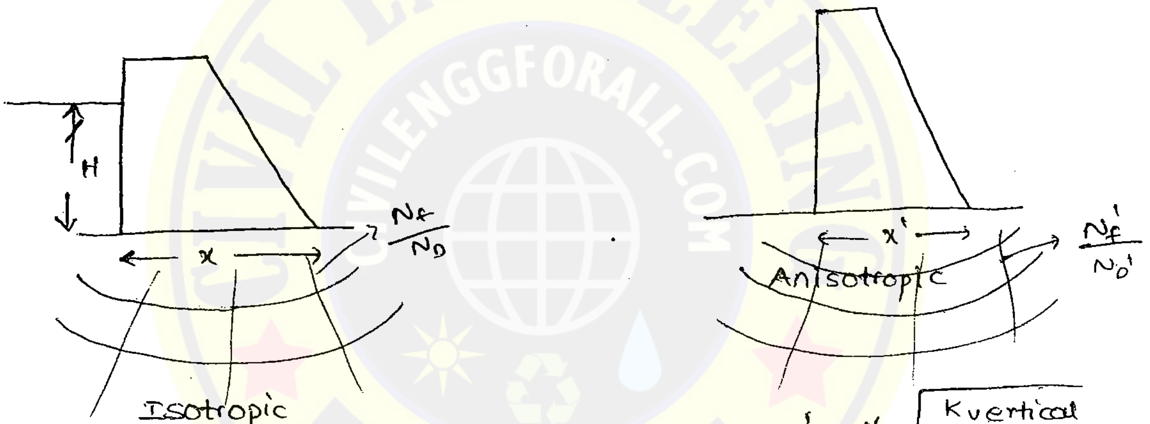
$$q = k \cdot \Delta h \cdot N_f$$

$$q_T = N_f (q_{each})$$

$$= k \cdot \frac{H_{net}}{N_D} (N_f)$$

$$\boxed{q_{Loss} = k \cdot H \cdot \frac{N_f}{N_D}} \quad m^3/sec$$

Transformed section:-



$$x' = x\sqrt{\frac{k_{vertical}}{k_{horizontal}}}$$

Seepage pressure:-

$$P_s = \gamma_w \cdot h$$

$$h = H_{net} - N_D(\Delta h)$$

$$= (H_2 - H_1) - N_D(\Delta h)$$

Uplift pressure:-

$$P_U = (\text{Hydraulic potential}) - \text{elevation head}$$

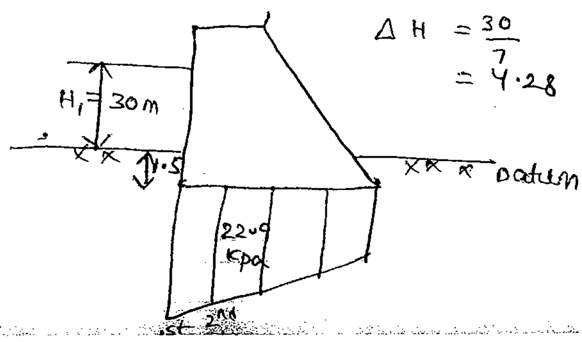
$$= [30 - (\overset{\rightarrow 2^{nd}\,drop}{2})(4 \cdot 28)] - (-1 \cdot 5\,m)$$

$$= 22 \cdot 9\,m$$

$N_D = 7$

$\Delta H = \frac{30}{7}$

$= 4 \cdot 28$



Use of flow net:-

$$i_{exit} = \frac{\Delta h}{\Delta L_{avg}}$$

$$i_{exit} < i_{critical}$$

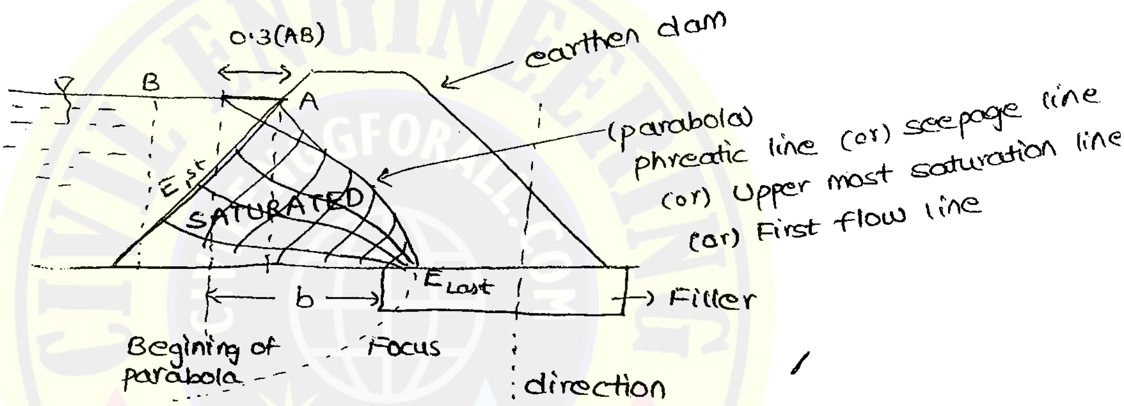
$$F.S = \frac{i_{critical}}{i_{exit} \downarrow} > 1$$

(No piping)

$F.S < 1$ (piping occurs)

$F.S = 1$ (critical)

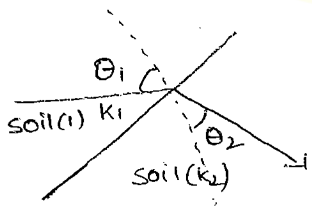
Phreatic line (or) seepage line (or) top flow line:-



Discharge through body of the dam, $q = KS$

$$S = \sqrt{(b^2 + H^2)} - b$$

Seepage in anisotropic soils:-



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

$$\frac{D_{15}\ (\text{filter})}{D_{15}\ (\text{soil})} \geq 5 \quad (\text{to allow water to pass})$$

$$\frac{D_{15}\ (\text{filter})}{D_{85}\ (\text{soil})} \leq 5 \quad (\text{To prevent soil particle movement})$$

P.g NO:- 39

8. Given $N_f = 4$, $N_D = 15$, $\Delta H = 3m$, $K = 2\times10^{-5}$ m/s

$$q = K \cdot H \left(\frac{N_f}{N_d}\right)$$

$\Delta H = H_1 - H_2$

$\Delta H = H_{net}$

$$= 2\times10^{-5} \times 3 \times \left(\frac{4}{15}\right)$$

$$= 1.6\times10^{-5}\ m^3/s$$

2. Given $N_f' = 4$, $N_D' = 16$, $K_H = 4\times10^{-7}$ m/sec, $K_V = 1\times10^{-7}$ m/sec

$$q = K_{equivalent} \cdot (H_{net}) \left(\frac{N_f'}{N_d'}\right)$$

$$= \sqrt{4\times10^{-7} \times 1\times10^{-7}}\ (20) \left(\frac{4}{16}\right)$$

$$= 2\times10^{-7}\ (5)$$

$$= 10\times10^{-7}$$

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

13-12-2014

UNIT - II

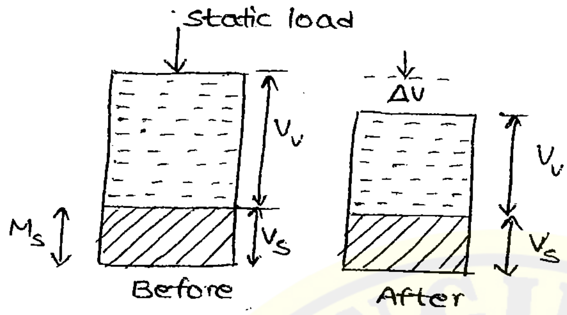
CONSOLIDATION

Compressibility:-

Expulsion of 'any voids' compressibility occurs.

Ex:- Air is the highly compressible material.

Consolidation:-



Expulsion of the water from the satuyated soil due to static load is called consolidation.

1. saturated soil
2. static load
3. water voids are expelled
4. Long process
5. After making or building structures, consolidation occurs after construction as well as before construction.

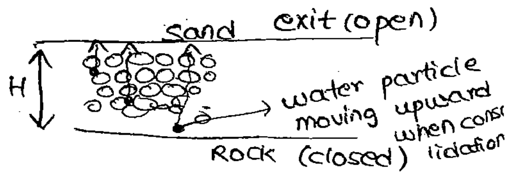
Factors affecting consolidation :-

a. For coarse grained soils (Gravel sand) :-

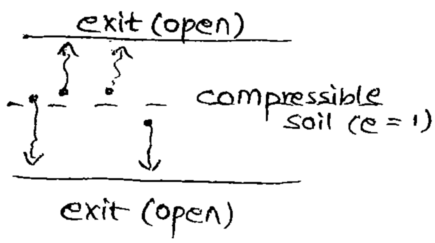
k (permeability) $= 10^{0}$ cm/sec (Fast consolidation)

$k = 10^{-7}$ (cm/sec (slow consolidation)

Drainage (d) has two types



1. single drainage, $\boxed{d = H}$
2. Double drainage, $\boxed{d = \frac{H}{2}}$



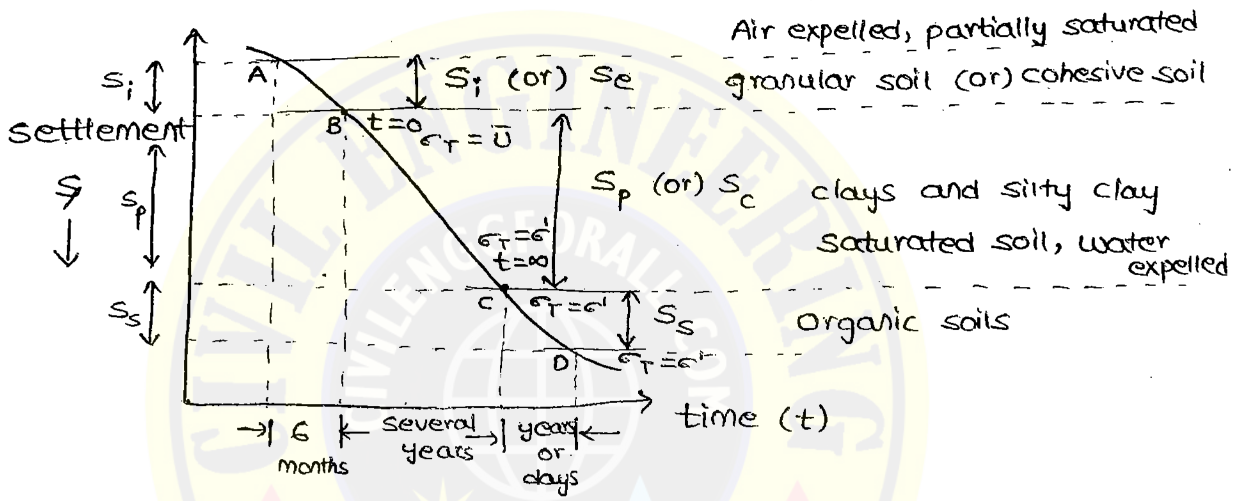
In double drainage, water particle are moving upward or ~~and~~ downward because both sides are open.

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

(b) For clays :- (F.G's = silt, clay)

1. permeability is low (t → high, several years)
2. d' → S.D and Double drainage
3. Expulsion of adsorbed water
4. slipping of grains.
5. Sand drain technique is a one of the ground improvement technique for clay to avoid the large settlement after construction.

Stages (or) Types of consolidation (or) compression :-



$$S_{total} = \left[S_{immediate} \text{ (or) } S_{elastic}\right] + \left[S_{primary} \text{ (or) } S_{consolidation} \text{ (or) } S_{time\ dependent}\right] + \left[S_{secondary\ compression} \text{ (or) } S_{creep}\right]$$

$$\boxed{S_{total} = S_i + S_p + S_s}$$

For sand

$S_T = S_i + 0 + 0$

$\boxed{S_T = S_i}$

For clays

$S_T = 0 + S_p + 0$

$\boxed{S_T = S_p}$

For Organic soils

$S_T = 0 + 0 + S_{secondary}$

$\boxed{S_T = S_s}$

In consolidated soil (or) primary consolidation :-

→ $t = 0$

$\sigma_T = \sigma' + \bar{U}$

$= 0 + \bar{U}$

$\boxed{\sigma_T = \bar{U}}$

∴ ~~If~~ In begining of the consolida settlement $t=0$, then the load is carried by excess pore wat

→ $t = \infty$

$\sigma_T = \sigma' + \bar{U}$

$= \sigma' + 0$

$\boxed{\sigma_T = \sigma'}$

∴ At the end of the consolidati settlement $t = \infty$ (several yea then the load is carried by effective stress.

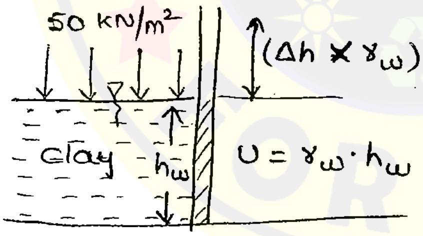
$\bar{U}$ = excxess pore water pressure

In secondary consolidation :-

$\sigma_T = \sigma'$ (constant)

Terzaghi's piston and spring analoghy model :-

1. Excess pore pressure $(\bar{U})$:-



$q = \sigma = 50\ kN/m^2$

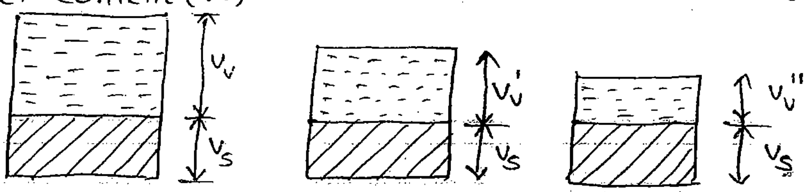
$50 = \Delta h \times \gamma_w$

$\Delta h = 5\ m$

Hydro dynamic lag :-

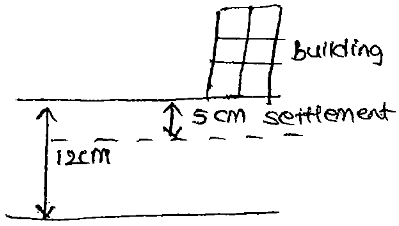
During consolidation (time is intermediate)

Decreasing	Increasing	constant
1. volume of voids (V_v)	1. weight of solids (γ)	1. Total stress (σ_T)
2. voids ratio (e)	2. Shear strength	2. Volume of solids
3. Excess pore pressure$(\bar{U})$	3. Effective stress (σ')	3. Degree of saturatio
4. volume (v)	4. Degree of consolidation (U)	/
5. Water content (W)		



Degree of consolidation (U) :-

After some years, the settlement occur



$$U = \frac{5}{12} \times 100$$

$$= 42\%$$

Again some more years, the settlement will be more. so Degree of consolidation is increasing.

Coefficient of volume compressibility (m_v):-

$$m_v = \frac{\frac{\Delta V}{V}}{\Delta \sigma'}$$

$$\boxed{m_v = \frac{\Delta V}{V \cdot \Delta \sigma'}}$$

$$a_v \text{ and } M_v = \frac{m^2}{kN} = \frac{1}{\sigma}$$

$$m_v = \frac{\left(\frac{\Delta H}{H} \cdot \frac{\cancel{A}}{\cancel{A}}\right)}{\Delta \sigma'}$$

$$m_v = \frac{\Delta H}{H}\left(\frac{1}{\Delta \sigma'}\right) \qquad \therefore \frac{\Delta H}{H} = \frac{\Delta e}{1+e}$$

$$= \left(\frac{\Delta e}{1+e}\right)\left(\frac{1}{\Delta \sigma'}\right)$$

$$= \frac{\Delta e}{\Delta \sigma'}\left(\frac{1}{1+e}\right)$$

$$\boxed{m_v = \cancel{a_v}\ \frac{a_v}{1+e}}$$

Coefficient of compressibility (a_v):-

$$\boxed{a_v = \frac{\Delta e \downarrow}{\Delta \sigma' \uparrow}} = \frac{e_i - e_f}{\Delta \sigma'}$$

Ex:- For same $\Delta\sigma'$, which of soil will have more (a_v) [ ]

a) Gravel b) Sand c) clay d) None

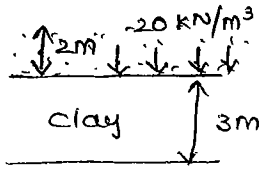
Consolidation Settlement (S_f):-

1. $m_v = \dfrac{\frac{\Delta V}{V}}{\Delta \sigma'}$

E_c = compression modulus

$m_v = \dfrac{\Delta H}{H}\left(\dfrac{1}{\Delta \sigma'}\right)$

$S_f = \boxed{\Delta H = m_v \cdot H \cdot \Delta \sigma'} = \dfrac{1}{E_c}(H)(\Delta \sigma')$



Ex:- $q = \sigma' = 20\gamma \cdot z_{fill}$

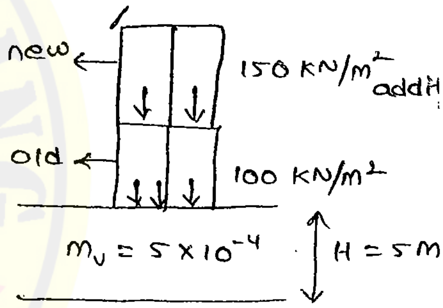
$= 20(2)$

$\Delta \sigma' = 40 \text{ kN/m}^2$

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

$= 5 \times 10^{-4} \times 3 \times 40$

$= 60 \text{ mm}$



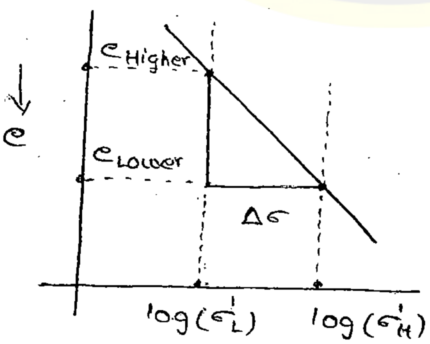
Ex:-

$\dfrac{S_{new}}{S_{old}} = \dfrac{\cancel{m_v} \cdot \cancel{H}\, \Delta \sigma'}{\cancel{m_v} \cdot \cancel{H} \cdot \Delta \sigma'}$

$= \dfrac{(150 + 100)}{100}$

$= 2.5$

Coefficient of laterally confined soils :-



C_c = compression index

$C_c = \dfrac{\Delta e}{\log \dfrac{\sigma'_H}{\sigma'_L}}$

$= \dfrac{e_H - e_L}{1}$ (compute log cycle)

$$\boxed{C_c = \frac{\Delta e}{1}}$$

$$C_c = \frac{e_H - e_L}{\log\left(\frac{\sigma_f'}{\sigma_i'}\right)}$$

$$C_c = \frac{\Delta e}{\log\left(\frac{\sigma_0' + \Delta\sigma}{\sigma_0'}\right)}$$

$$\Delta e = C_c \log\left(\frac{\sigma_0' + \Delta\sigma}{\sigma_0'}\right)$$

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

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

$$S_{consolidation} = \boxed{S_{max} = \frac{C_c}{1+e_0}(H) \log\left(\frac{\sigma_0' + \Delta\sigma}{\sigma_0'}\right)}$$

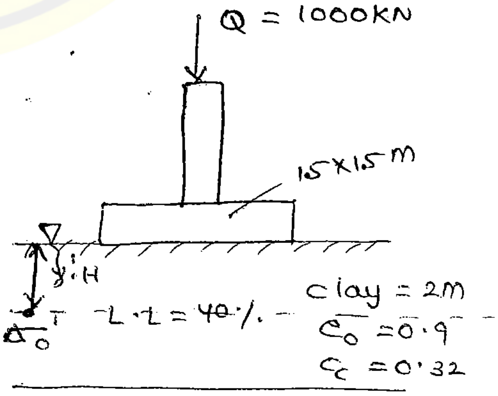
where,

C_c = coefficient of compression index of clay

$= 0.009(W_L - 10\%)$

$$\boxed{\begin{aligned} S_{consolidation} &\propto C_c \\ S_{consolidation} &\propto \frac{1}{e} \\ S_{consolidation} &\propto H \end{aligned}}$$

Ex:-



$$S_{consolidation} = \frac{C_c}{1+e_0} H \cdot \log\left(\frac{\sigma_0' + \Delta\sigma}{\sigma_0'}\right)$$

$$\sigma_0' = \gamma' \cdot H$$

$$= 10\ (1)$$

$$= 10\ kN/m^3$$

$$\Delta\sigma = \frac{1000\ kN}{225\ m^2} = 440\ kN/m^2$$

$$S_c = \frac{0.32}{1+0.9} \times \overset{2000}{\cancel{2}} \times \log\left(\frac{10+440}{10}\right)$$

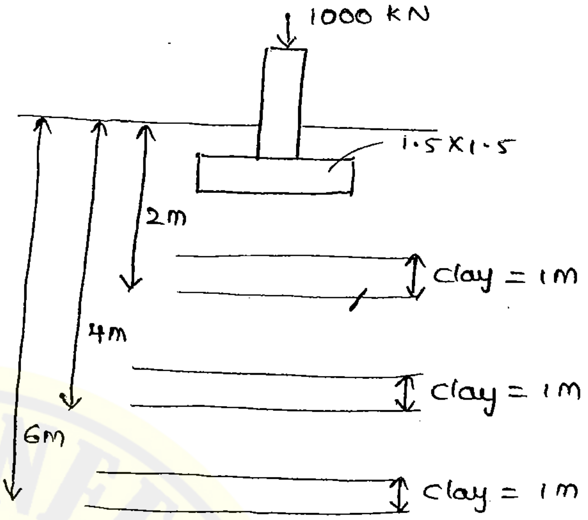
$$=$$

Note :-

1. Max settlement occurs when applied load ($\Delta\sigma$) is more
2. Max settlement occurs when 'C_c' is more
3. Max settlement occurs when 'H' is more

Ex :-
For the following cases
C_c , e_0 , H = same
In which case settlement is minimum ?

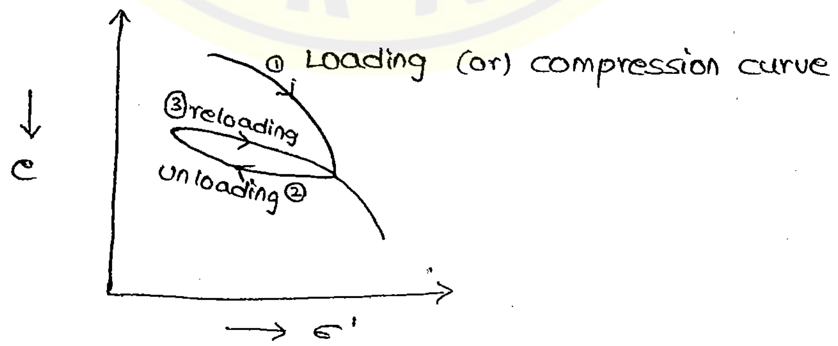
a) When clay is 2m below footing
b) When clay is 4m below
c) When clay is 6m below
d) same

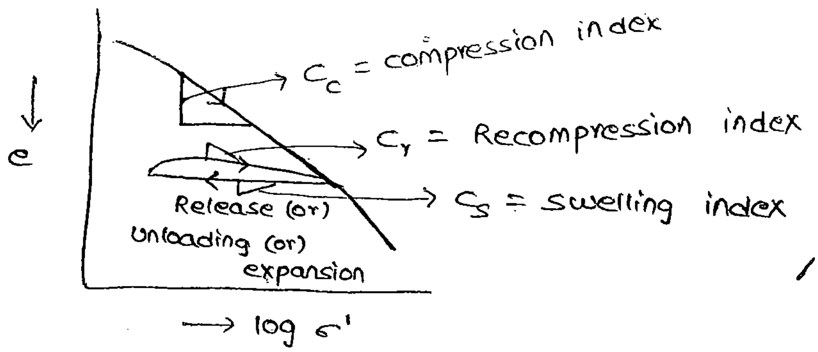


Ans :- (c) When goes down settlement is decreasing, settlement is depends upon $\Delta\sigma$ value.

Note :-
When settlement is zero (s = 0) such footing are called compensated footing (or) Floating foundation.

Coefficient of laterally confined soils :-





$\therefore C_r < C_c$

Settlement C_r < Settle C_c

Consolidation of undisturbed specimens :-

1. Preconsolidated or Over consolidated soil:-
   a. Void ratio decreased
   b. settlement decreased
   c. Unit weight increased
   d. Bearing capacity increased.

2. Under consolidated soil:-
   a. Void ratio increases
   b. Unit weight decreases
   c. Bearing capacity decreases
   d. settlement increases

Ex:-

1. Excess pore pressure (or) Hydro dynamic pressure (or) hydro static pressure is more at Middle
2. Excess prore pressure is minimum at the end of the consolidation is bottom (time = ∞) ~~and maximum at top(t=0)~~

Degree of consolidation (U) and Time Factor (T_v):-

$$T_v = \frac{\pi}{4}\left(\frac{U\%}{100}\right)^2, \quad \text{for } U \leq 60\%$$

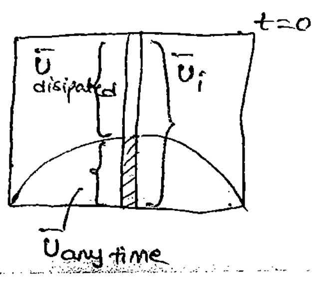
$$T_v = 1.781 - 0.933 \log_{10}(100 - U\%), \quad \text{for } U > 60\%$$

U%	T_v
0%	0
⋮	
100%	∞

$$U = \left(\frac{\text{settlement at any time}}{\text{Ultimate settlement}}\right) \times 100$$

$$\boxed{U = \left(\frac{S}{S_f}\right) \times 100}$$

$$U = \frac{\bar{U}_{\text{initial}(t=0)} - \bar{U}_{\text{any time}}}{\bar{U}_{\text{initial}}\ (t=0)} \times 100$$



$$U = \frac{\bar{U}_{\text{disipated}}}{\bar{U}_{\text{initial}}} \times 100$$

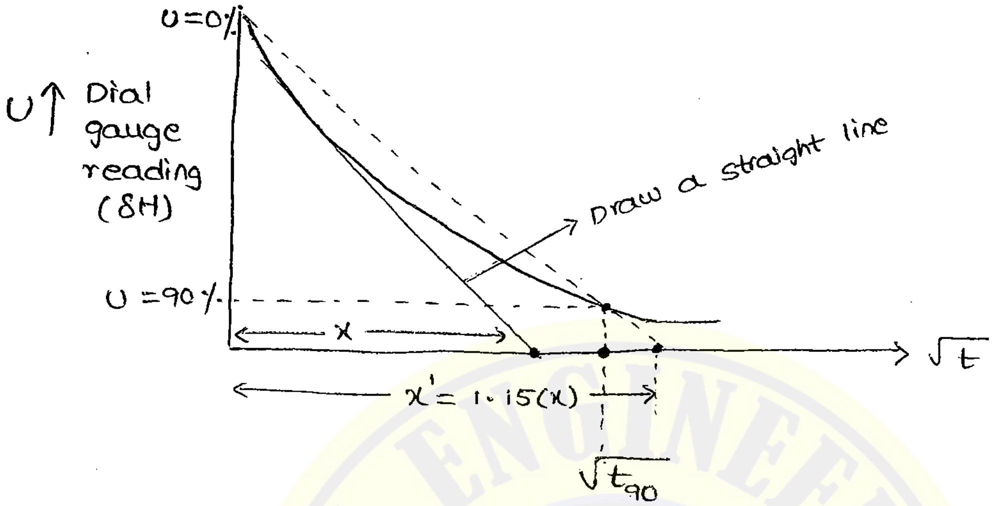
Determination of coefficient of consolidation (c_v):-

1. Time fitting method

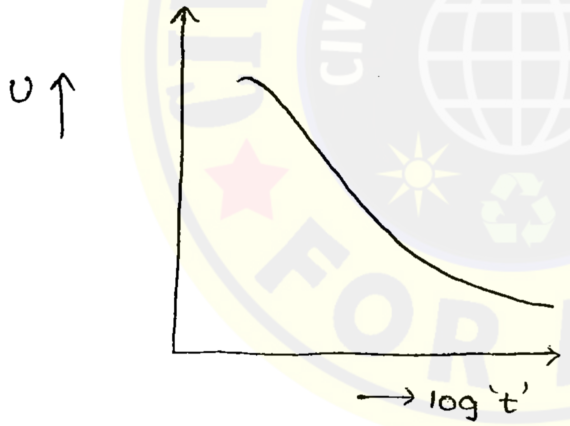
   a. square root of time fitting method (Taylor)

   b. logarithm of time fitting method (casagrande method)

a. square root of time fitting method :-



b. Logarithm of time fitting method:-



Ex:- During consolidation load transfers from pores to grain

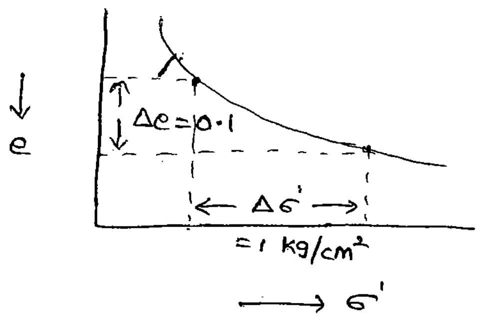
P.g No:-50

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

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

$= 700\left(\frac{0.1}{1+0.4}\right)$

$= 50$ cm



7. Given $e_o = 0.5$ $e_f = 0.2$

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

$$\Delta H = S = \frac{(e_o - e_f)}{1+e_o}(H)$$

$$= \frac{0.5 - 0.2}{1+0.5}(10)$$

$$\Delta H = 2.0$$

16.

A	B
single drainage	Double drainage
$d = H$	$d = \frac{H}{2}$
$U = 50\%$	$U = 50\%$
$t_1 = 10$ min	$t_2 = ?$

$$\frac{t_1}{t_2} = \left(\frac{d_1}{d_2}\right)^2 = \left(\frac{H}{H/2}\right)^2$$

$$\frac{10}{t_2} = 4$$

$$t_2 = 2.5 \text{ min}$$

17. settlement = 1 cm , $\sigma_o = 1\ kg/cm^2$, $\sigma_f = 2\ kg/cm^2$

settlement = ? , $\sigma_o = 2\ kg/cm^2$, $\sigma_f = 4\ kg/cm^2$

$$\frac{S_1 = \frac{C_c \cancel{H}}{1+e_o} \log_{10}\left(\frac{\sigma_f}{\sigma_i}\right)}{S_2 = \frac{C_c \cancel{H}}{1+e_o} \log_{10}\left(\frac{\sigma_f}{\sigma_i}\right)}$$

$$\frac{1 = \log\left(\frac{2}{1}\right) = 2}{S_2 = \log\left(\frac{4}{2}\right) = 2}$$

$$2S_2 = 2$$

$$S_2 = 1 \text{ cm}$$

20. Given $k = 2$, $m_v = \frac{1}{2}$ $C_v = ?$

$$C_v = \frac{K}{\gamma_w \cdot m_v} = \frac{2}{\frac{1}{2}} = 4 \text{ times increased}$$

γ_w = constant

(or)

$$C_{v_1} = \frac{K_1}{m_{v_1}}$$

$$C_{v_2} = \frac{2K_1}{0.5\,m_{v_1}}$$

$$C_{v_2} = 4\,(C_{v_1})$$

21.

A	B
double drainage	single drainage
$\frac{d}{2} = H$	$3d$
$t_1 = t$	$t_2 = ?$

$$\frac{t_1}{t_2} = \left(\frac{d_1}{d_2}\right)^2$$

$$\frac{t}{t_2} = \left(\frac{d/2}{3d}\right)^2$$

$$\frac{t}{t_2} = \frac{1}{36}$$

$$t_2 = 36t$$

23.

A	B
$t_1 = 4$ years	$t_2 = ?$
$U = 50\%$	$U = 50\%$
single drainage	Double drainage
$d_1 = H$	$d_2 = \frac{H}{2}$

$$\frac{t_1}{t_2} = \left(\frac{d_1}{d_2}\right)^2$$

$$\frac{4}{t_2} = \left(\frac{H}{H/2}\right)^2$$

$$t_2 = 1 \text{ year}$$

27.

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

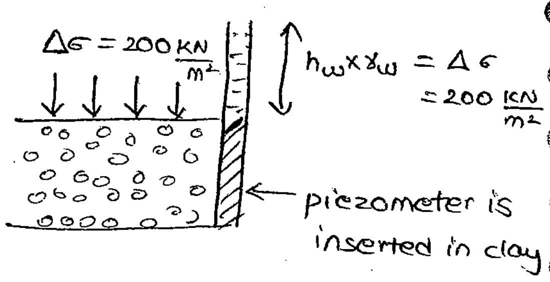
$$t = \frac{T_v \cdot d^2}{C_v}$$

$$t = \frac{T_v \cdot d^2}{\frac{K}{\gamma_w \cdot m_v}}$$

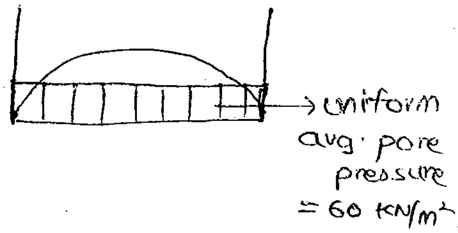
$$\uparrow t = \frac{T_v \cdot d^2 \cdot \gamma_w \cdot m_v}{k \uparrow}$$

29. At $t=0$, the load is carried by pore water pressure and effective stress is zero.

∴ In piezometer the water is raised is equal to applied load.



Degree of consolidation (or) stage of consolidation (U)



$$U = \frac{\bar{U}_i - \bar{U}_{any\ time}}{\bar{U}_i}$$

$$= \frac{200 - 60}{200} \times 100$$

$$U = 70\%$$

At $t=\infty$, $\bar{U}_i = 200$, $\bar{U}_{any\ time} = 0$

$$U = \frac{200 - 0}{200} \times 100\%$$

$$= 100\%$$

At $t=0$, $\bar{U}_i = 200$, $\bar{U}_{anytime} = 200$

$$U = \frac{200 - 200}{200} \times 100$$

$$U = 0\%$$

t	U%
0	0%
∞	100%

30. $S_1 = 80$ mm $\quad$ $S_2 = ?$

$t_1 = 4$ years $\quad$ $t_2 = 9$

$U = 60\%$ $\quad$ $U = 60\%$

In both cases $U < 60\%$

$$T_v = \frac{\pi}{4}(U)^2 \qquad U = \frac{S_{any\ time}}{S_{maximum}} \times 100$$

$$T_v = \frac{\pi}{4}\left[\frac{S_{any\ time}}{S_{max}}\right]^2$$

$$\frac{C_v t}{d^2} = \frac{\pi}{4}\left[\frac{S_t}{S_{max}}\right]^2$$

$$\frac{\dfrac{\cancel{C_v} t}{\cancel{d^2}} = \dfrac{\cancel{\pi}}{4}\left(\dfrac{S_1 t}{\cancel{S_1 max}}\right)^2}{\dfrac{\cancel{C_v} t}{\cancel{d^2}} = \dfrac{\cancel{\pi}}{4}\left(\dfrac{S_2 t}{\cancel{S_2 max}}\right)^2}$$

$$\frac{S_1^2}{S_2^2} = \frac{t_1}{t_2}$$

$$\frac{t_1}{t_2} = \left(\frac{S_1}{S_2}\right)^2$$

$$= \cancel{\left(\frac{4}{9}\right)^2} \quad \frac{4}{9} = \left(\frac{80}{S_2}\right)^2$$

$$S_2 = 120 \text{ mm}$$

31. $H = 4000$ mm $H = 4000$ mm

$S_{max} = 50$ mm $S_{max} = ?$

single drainage   Double drainage

$$U = \frac{\overset{\text{variable}}{S_{any time}}}{\underset{\text{constant}}{S_{max}}} \times 100$$

In single drainage settlement is same as well as in double drainage. But in S.D time varies compare to double drainage. In S.D, time is low, In D.D time is fast but settleme[nt] is same in both cases.

P-9 No :- 53

10. $$S = \frac{C_c}{1+e_0} H \log \frac{\sigma_0' + \Delta\sigma}{\sigma_0'}$$

$$\Delta H = H(\Delta\sigma)(m_v)$$

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

13. $$S_f = \Delta H = \left(\frac{\Delta e}{1+e_0}\right) H$$

$$S_f = \left(\frac{e_0 - e_f}{1+e_0}\right) H$$

$$S_f \; \cancel{2000} = \left(\frac{1.00 - 0.92}{1 + 1.00}\right)(2000)$$

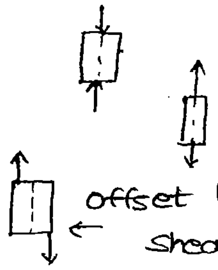
$H = 80$ mm

23-12-20

UNIT - 13

SHEAR STRENGTH

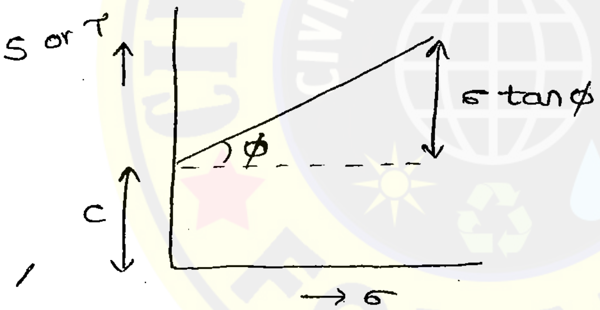
Applied load → compressive stress (σ_c), Tensile stress (σ_t), shear stress (τ)



Soil strength can be measured in shear strength (S).

Contributors	soil type	Example	significant	Minor contributor
1. Interlocking	ϕ - soils	sand, Gravel	① & ②	–
2. ϕ	c - soils	clay	③	①
3. c	c–ϕ	Murram	② & ③	①

Coulomb's Law:-



$$S = c + \sigma \tan\phi$$

S = shear strength KN/m^2

c = cohesion KN/m^2

ϕ = angle of internal friction (or) shear resistance

Note:-

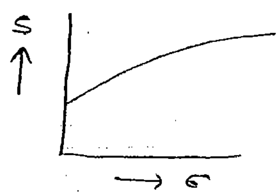
'c' and 'ϕ' are not constant for a particular soil.

Mohr's theory:-

Very much used for the determination of shear strength of soils.

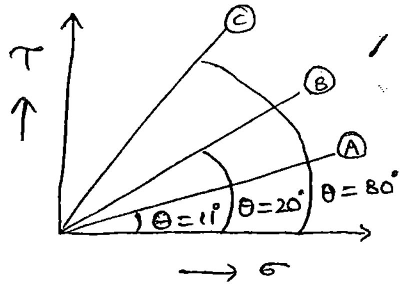
$$S = f(\sigma)$$

S = normal stress



Types of soil :-

1. ϕ - soil
2. C - soil
3. C-ϕ - soil

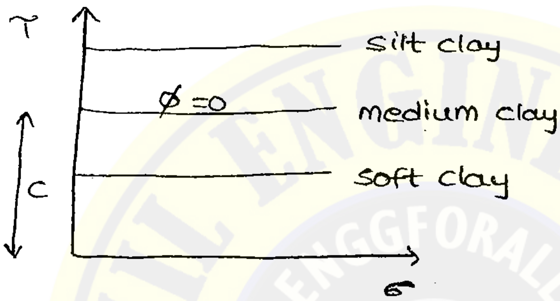


$\theta = 11^\circ \rightarrow$ size is small

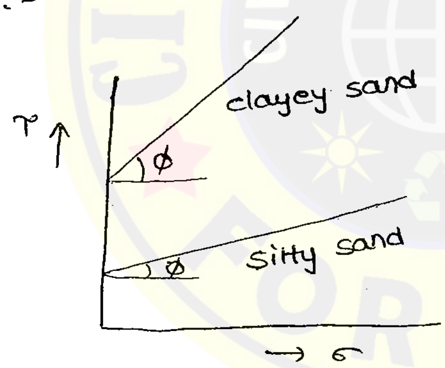
→ Dense state having a highest friction

→ Rounded particles having a low friction angle.

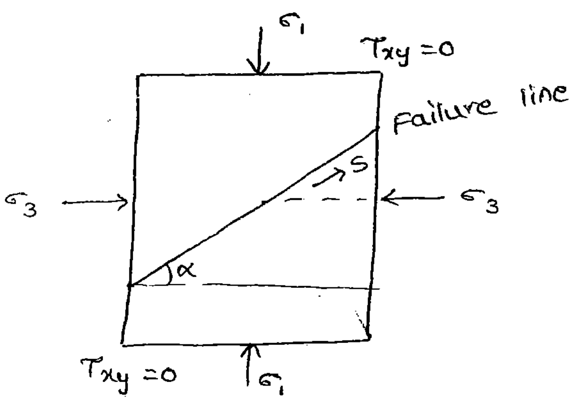
C-soil :-



C-ϕ soils:-



Shear stress (τ), Mohr's circle diagram:-



Major (σ_1) and minor (σ_3) principle planes (ab & ac) on which shear stress (or) Tangential ~~shear~~ stress acting is zero.

$\tau, \sigma_d \rightarrow$ induced stresses

σ_d = Deviatric stress = $(\sigma_1 - \sigma_3)$

Terzaghi's concept:-

$$s = c' + \sigma' \tan\phi'$$

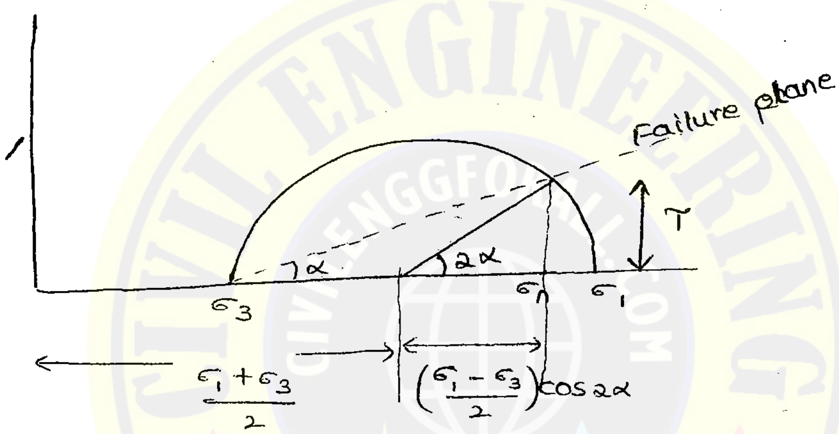
$$s = c' + (\sigma - u)\tan\phi'$$

c' = effective cohesion

ϕ' = effective angle of shear resistance

$$\left.\begin{array}{ll} \sigma_n(max) = \sigma_1, & \alpha = 0 \\ \sigma_n(min) = \sigma_3, & \alpha = 90^\circ \end{array}\right\} \tau_{min} = 0$$

$$\sigma_n = \frac{\sigma_1 + \sigma_3}{3}, \quad \alpha = 45^\circ \; \} \; \tau_{max} = \frac{\sigma_d}{2}$$



$$\sigma_n = \frac{\sigma_1 + \sigma_3}{2} + \left(\frac{\sigma_1 - \sigma_3}{2}\right)\cos 2\alpha$$

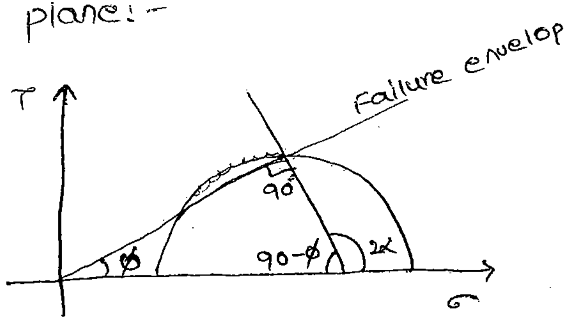
$$\tau = \frac{\sigma_1 - \sigma_3}{2}\sin 2\alpha$$

$$\sigma_R = \sqrt{\tau^2 + \sigma_n^2}$$

$$\beta = \tan^{-1}\left(\frac{\tau}{\sigma_n}\right)$$

$$\beta_{max} = \sin^{-1}\left(\frac{\sigma_1 - \sigma_3}{\sigma_1 + \sigma_3}\right)$$

Critical shear plane:-



$$180 = 2\alpha + (90 - \phi)$$

$$2\alpha = 180 - 90 + \phi$$

$$= 90 + \phi$$

$$\alpha_f = 45 + \frac{\phi}{2}$$

Most dangerous shear plane

Various tests to determined the strength of soil :-

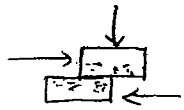
1. Direct shear test is suitable for ϕ - soils
2. Tri-axial compression test is suitable for $c-\phi$ soils
3. Unconfined compressed strength

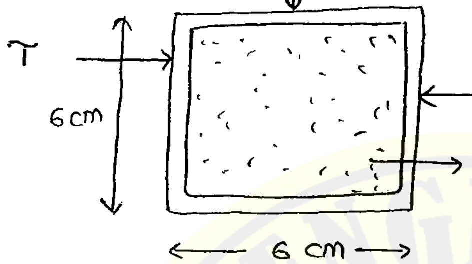
} Lab tests

4. Vane shear is suitable for soft clays } Lab and field tests

Direct shear (or) Box shear test :-

$$\frac{F_{normal}}{6 \times 6} = \sigma_n = \text{constant}$$





Advantages :-

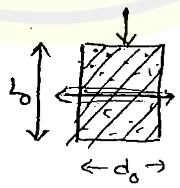
$$\tau = \text{Shear stress} = \frac{\text{shear load}}{\text{Area}}$$

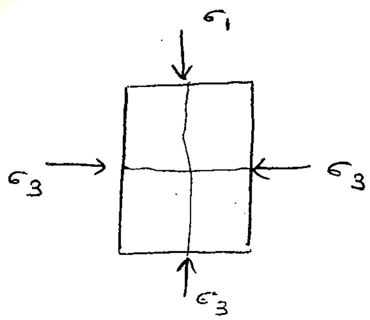
1. It is a quick test
2. Cheaper

Disadvantages :-

1. It is used only for ϕ soils (frictional soil) not cohesive soil
2. No facility for "pore water pressure" measurement.
3. Area corrected is not used
4. Failure plane is always horizontal

Triaxial compression test :-

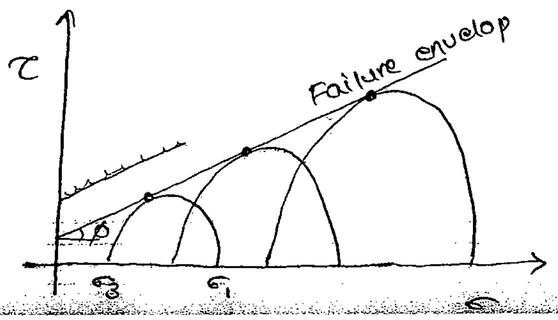


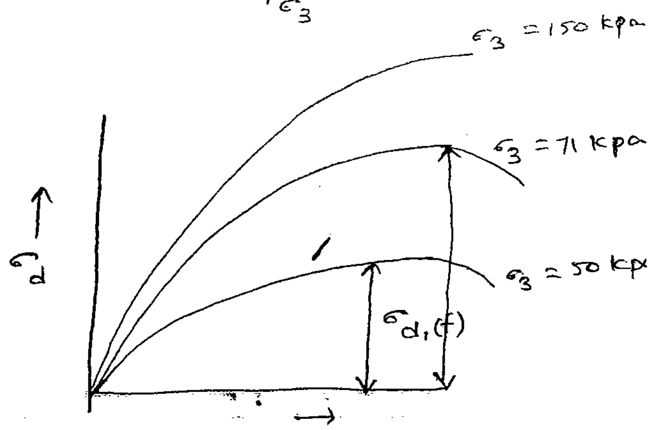


$$\sigma_1 = \sigma_d + \sigma_3$$

$$\sigma_d = \frac{\text{Load}}{\text{corrected Area} (A_c)}$$

$$A_c = \frac{A_0 \text{ (original area)}}{1 - \frac{\Delta L}{L_0}}$$





Shear characteristics of cohesionless soils:-

Bulging failure:-

1. It occurs in loose sand and silt clay
2. When 'ΔL' is large
3. It is a plastic failure

Brittle failure:-

1. It occurs in dense sand
2. It occurs in silt clay
3. Original area $(A_0) \approx$ corrected area (A_c)
4. $\Delta L \approx$ small
5. It occurs in stabilize of soils

Depending upon drainage conditions, there are three types of tests as explained below:-

	stage-I	stage-II	
σ_T	U	U	Quick test → Undrained (value is completely closed)
σ'	C	U	R-test → semi / partially drained ~~soil~~ test (value is closed and open)
σ'	C	D	slow test → drained (value is open throughout test)

1. U U — Stage - I → Not allowed
   Stage - II → Not allowed

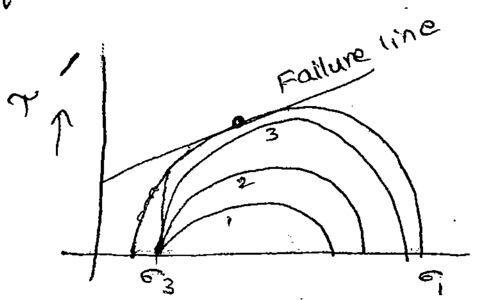
2. C U — stage - I → value opened
   stage - II → value closed

3. C D — stage - I → value opened
   stage - II → value opened

pore water pressure 'U' is measurable

Both test measures effective stress (σ') but 'CU' takes less time as compared to 'CD'.

Plastic equilibrium:-



1-2-3 circles are stable circles

P.g NO:- 71

1. Given $\sigma_T = 295$ k.pa

$U = 120$ k.pa

$c' = 12$ k.pa

$\phi' = 30°$

$s = c' + \sigma' \tan\phi'$

$= c' + (\sigma_T - U) \tan\phi'$

$= 12 + (295 - 120) \tan 30°$

$= 113.0$

2. Given $c = 0$ $\phi = 30°$ $\sigma_d = ?$ $\sigma_3 = 200$ k.pa

Method - I : $\sigma_1 = \sigma_3 (\tan(45° + \phi/2)) + 2c \tan(45 + \frac{\phi}{2})$

$= 200 (\tan(45° + \frac{30}{2})) + 0$

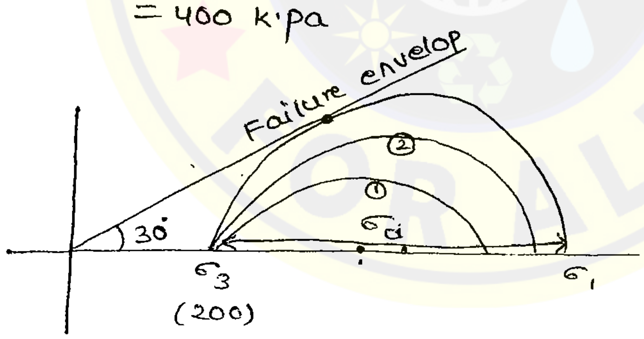
$\sigma_1 = 600$ k.pa

$\sigma_d = \sigma_1 - \sigma_3$

$= 600 - 200$

$= 400$ k.pa

Method - II



Remove ① & ② Mohr circ which is not touch and measure the touching circle distance

3. Given $c = 0$

$\sigma_n = 200$ k.pa

$\tau_f = 100$

$\phi Q = ?$

$\tau = \sigma_n \tan\phi$

$100 = 200 \times \tan\phi$

$\phi = 26.56°$

~~P.g No:- 64~~.

Unconfined compression test :-

1. $\sigma_3 = 0$
2. Axial stress $= \sigma_1$ (or) σ_d
3. U.C.S can be used for purely cohesive soils (clays)
4. U.C.S can also be used for 'c-ϕ' soils
5. U.C.S cannot be used for sand and Gravel
6. Brittle and plastic failure.
7. $\phi = 0$
8. $C = \frac{q_u}{2}$ (or) $\frac{\sigma_d}{2}$ (or) $\frac{U.C.S}{2}$
9. Mohr's circle dia $= (UCS = \sigma_1 = \sigma_d)$

$$U.C.S = \frac{F}{A_c} = \frac{F}{\left[\frac{\frac{\pi d^2}{4}}{1-\frac{\Delta L}{L}}\right]} = \frac{F}{\left(\frac{A_0}{1-\frac{\Delta L}{L}}\right)}$$

A_c = corrected area

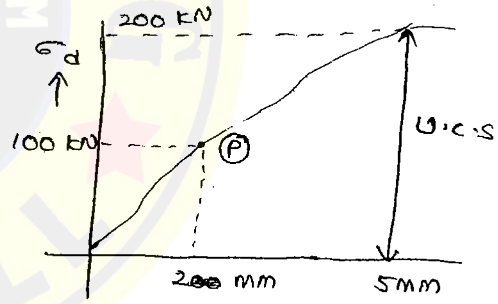
A_0 = original area

Ex:-) Given $d = 3.7$ cm, stress (P) = ?

$$\sigma = \frac{F}{A_c}$$

$$= \frac{100}{\left[\frac{\frac{\pi(3.7)^2}{4}}{1-\frac{2}{7.4}}\right]}$$

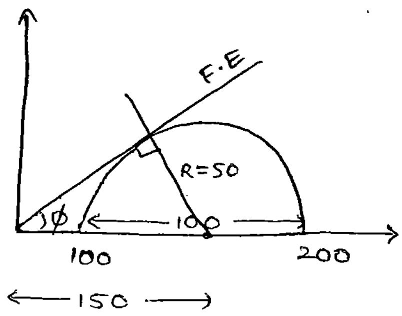
$\sigma =$



10. U.C.S can be used for stiff, medium, soft clays.
11. Vane shear can be used for very soft clays.

	Quick sand	Liquifaction
Material	Fine sand & silt	"S.P" (poorly graded soil) uniform soil
	seepage pressure (P_s)	Earthquake, vibration, pile driving
end results	$\sigma' = 0$	$\sigma' = 0$ $S = c + \sigma' \tan\phi$ $S = c + (\sigma_T - u) \tan\phi$

1.



$$\frac{100+200}{2} = 150$$

$$\sin\phi = \frac{50}{150}$$

$$\phi = \sin^{-1}\left(\frac{1}{3}\right)$$

4. Given $\sigma_3 = 100\ kN/m^2$

$\sigma_{d(f)} = 200\ kN/m^2$

$\sigma_1 = \sigma_3 + \sigma_d = 300\ kN/m^2$

$$\sigma_1 = \sigma_3 \tan^2\alpha_f + 2c\tan\alpha_f$$

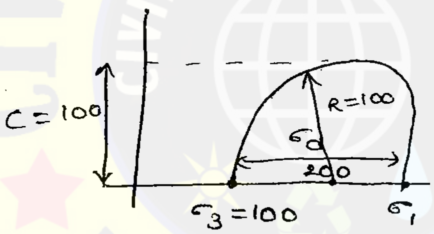
$$= 100\tan^2\left(45+\frac{\phi}{2}\right) + 2c\tan\left(45+\frac{\phi}{2}\right)$$

$\therefore$ saturated clay $\phi = 0$

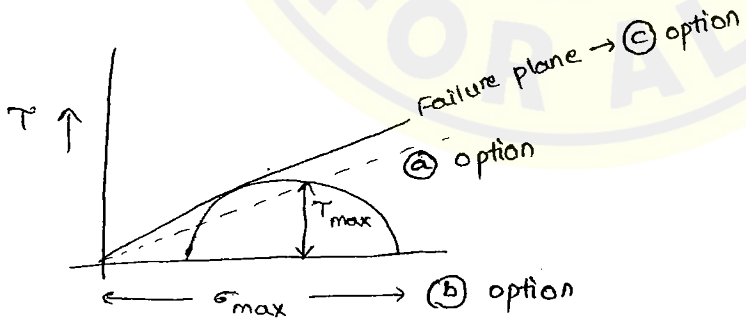
$$300 = 100\tan^2(45) + 2c\tan(45)$$

$$300 = 100 + 2c$$

$$c = 100$$



5.



26. Given $\sigma_1 = 200\ kN/m^2$

$$= \frac{200\times1000}{(1000)^2}\ \frac{N}{mm^2}$$

$\sigma_3 = 0$ (U.C.S test)

$\alpha_f = 45^\circ$

$$\sigma_1 = \sigma_3\tan^2\left(45+\frac{\phi}{2}\right) + 2c\tan\alpha_f$$

$$0.2\left(\frac{N}{mm^2}\right) = 0 + 2c\tan45^\circ$$

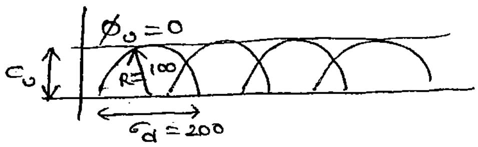
$$0.2 = 2c$$

$$c = 0.1\ N/mm^2$$

$\alpha_f = 45 + \phi/2$

$45 = 45 + \phi/2$

32. $\sigma_d = 200 \qquad \sigma_3 = 100$



$$c_u = \frac{200}{2} = 100$$

$$\therefore c_u = \text{Radius} = 100$$

33. Given $\sigma_3 = 50$ k.pa $\qquad \sigma_1 = \sigma_3 + \sigma_d = 150$ k.pa

$\sigma_d = 100$ k.pa

$\phi = ?$

$$\sigma_1 = \sigma_3 \tan^2\left(45+\frac{\phi}{2}\right) + 2c\tan\left(45+\frac{\phi}{2}\right)$$

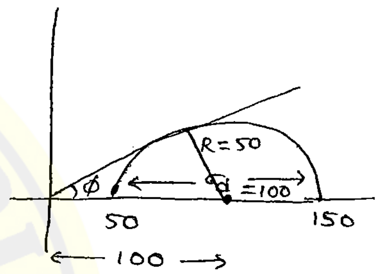
$$150 = 50\tan^2\left(45+\frac{\phi}{2}\right) + 0 \qquad \therefore c=0 \text{ (dry sand)}$$

$$\tan^2\left(45+\frac{\phi}{2}\right) = 3$$

$$\left(45+\frac{\phi}{2}\right) = \tan^{-1}(\sqrt{3})$$

$$\frac{\phi}{2} = 60^\circ - 45^\circ$$

$$\phi = 30^\circ$$



$$\sin\phi = \frac{50}{100}$$

$$\phi = \sin^{-1}\left(\frac{1}{2}\right)$$

$$\phi = 30^\circ$$

P.g NO:- 64

1. Given $\sigma_3 = 0.2$ M.Pa

$\phi = 30^\circ$

$c = 0$ (dry sand)

$$\sigma_1 = \sigma_3 \tan^2\left(45+\frac{\phi}{2}\right) + 2c\tan\alpha_f$$

$$\sigma_1 = 0.2\tan^2\left(45+\frac{30}{2}\right) + 0$$

$$= 0.2\tan^2(60)$$

$$= 0.2 \times 3$$

$$\sigma_1 = 0.6 \text{ M.pa}$$

2. $q_u = 2c_u$ (or) $\sigma_d = 2c_u$ (or) $U.C.S = 2c_u$

$$c_u = \frac{q_u}{2}$$

$$= 0.5\,q_u$$

3. Given $\sigma_1 = 50$ k.pa $\quad \phi = 30^\circ \quad c = 0$

$$\sigma_1 = \sigma_3 \tan^2\left(45+\frac{30}{2}\right) + 0$$

$$50 = \sigma_3 \tan^2(60^\circ)$$

$$\sigma_3 = \frac{50}{3} = 16.66 \text{ kpa}$$

24-12-2014

UNIT - 14

EARTH PRESSURE

Retaining wall can be classified into many types:-

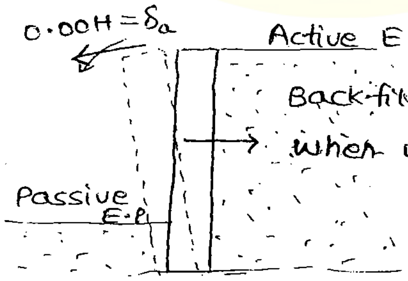
1. Based on material
   a. R.C.C
   b. Masonry
   c. Sheet pile (metal)

2. Based on type
   a. Gravity Retaining wall
   b. Cantilever retaining wall
   c. Counter-fort retaining wall

3. Based on movement of the wall w.r.t Back-fill
   a. Earth pressure at rest
   b. Earth pressure is active
   c. Earth pressure is passive

Only for sheet piles:-

1. When wall moves away from Back-fill is called Active Earth pressure.
2. When wall moves towards Back-fill is called passive earth pressure
3. No wall movement is called pressure at rest



H in mm

When wall moves that side active earth pressure is attracted and vice versa

Coefficient of Earth pressure (K):-

It is the ratio of horizontal stress (σ_h) to vertical stress (σ_v).

$$\boxed{K = \frac{\sigma_h}{\sigma_v}} = \frac{\sigma_{lateral}}{\sigma_{verticle}}$$

Earth pressure at rest:-

$$K_0 = \frac{\sigma_h}{\sigma_v}$$

$$\boxed{\sigma_h = K_0 \cdot \sigma_v}$$

$$\sigma_h = K_0 (\gamma z)$$

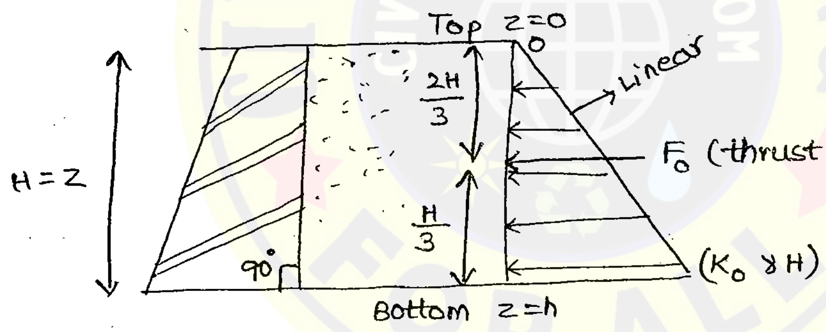
$$P_0 = K_0 (\gamma z)$$

$$\boxed{K_0 = \frac{\mu}{1-\mu}}$$

$K_0 = (1 - \sin\phi)$ for cohesionless soil

K_0 = coefficient of earth pressure at rest

→ Cohesive soil is not suitable for back-fill material
→ cohesive soil depends on kaolinite, Montmorillonite.
→ cohesive soil having a swelling, shrinking, and misalignment properties.
→ cohesionless less soil is suitable for back fill (ϕ soil and $c-\phi$ soil)



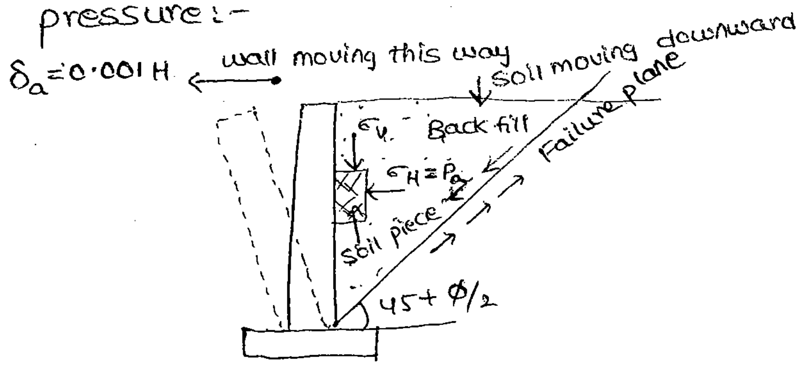
(kN/m²) (m) (m)

F_0 (thrust) $= \frac{1}{2}(K_0 \gamma H)(H) \times$ U.L (unit length)

$= \frac{1}{2} K_0 \gamma H^2$

pressure distribution diagram

Active pressure:-



H in mm

$\sigma_1 = \sigma_v = \gamma z$

$\sigma_H = \sigma_3 = P_a$ (active earth pressure)

1. It was proposed by Rankine's.
2. Retaining wall moving away from the back-fill.
3. Soil yields to be stretching (or) tension

Assumptions of Active pressure:-

1. Soil is homogeneous, semi-infinite, dry and cohesionless.
2. Retaining wall back is smooth and vertical.
3. plastic equilibrium condition.

$\sigma_h < \sigma_v$ (active pressure)

$\sigma_h > \sigma_v$ (passive pressure)

Rankine's Lateral pressure for dry cohesion less soils:

$$\sigma_1 = \sigma_3 \tan^2\alpha_f + 2c\tan\alpha_f$$

$$\therefore c = 0$$

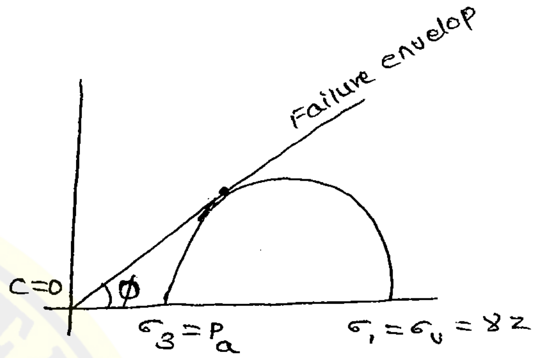
$$\sigma_1 = \sigma_3 \tan^2\alpha_f$$

$$\sigma_v = P_a \tan^2\alpha_f$$

$$P_a = \left(\frac{1}{\tan^2(45+\frac{\phi}{2})}\right)\sigma_v$$

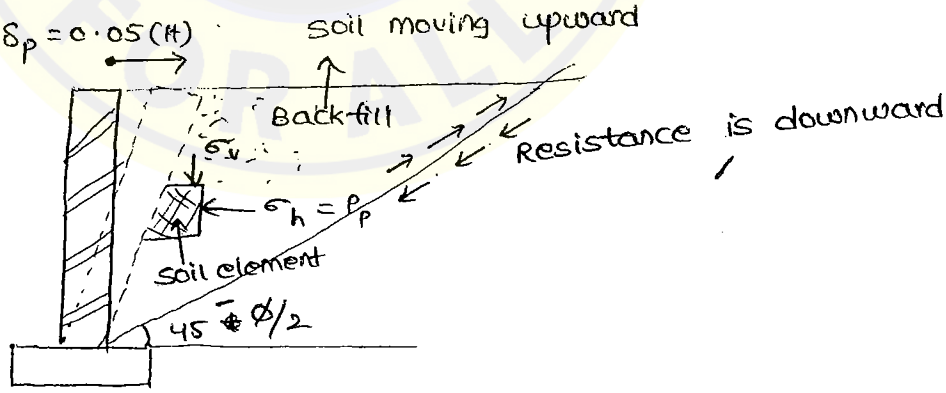
$$P_a = K_a \cdot \sigma_v$$

$$\boxed{P_a = K_a \cdot (\gamma z)}$$



Passive pressure (P_p):-

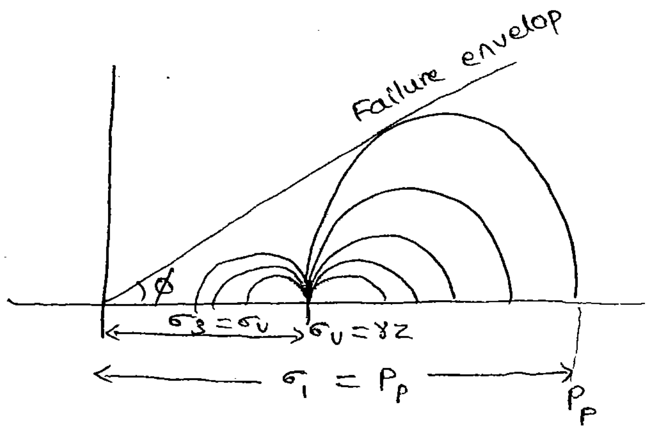
H in mm



$$\sigma_1 = \sigma_3 \tan^2(45+\frac{\phi}{2}) + 0$$

$$P_p = (\gamma z)\tan^2(45+\frac{\phi}{2})$$

$$\boxed{P_p = K_p(\gamma z)}$$



Relation between K_p and K_a :-

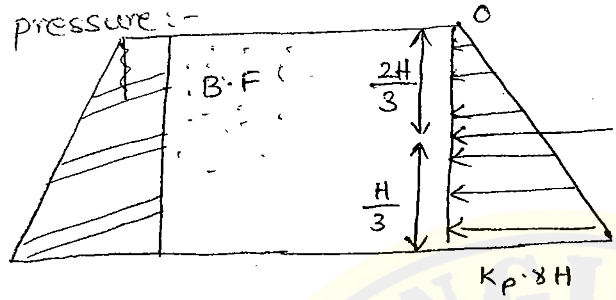
$$\boxed{K_p = \frac{1}{K_a}}$$

$$K_p \cdot K_a = 1$$

For $\phi = 30°$, $K_a = \frac{1}{3}$

$$K_p = 3$$

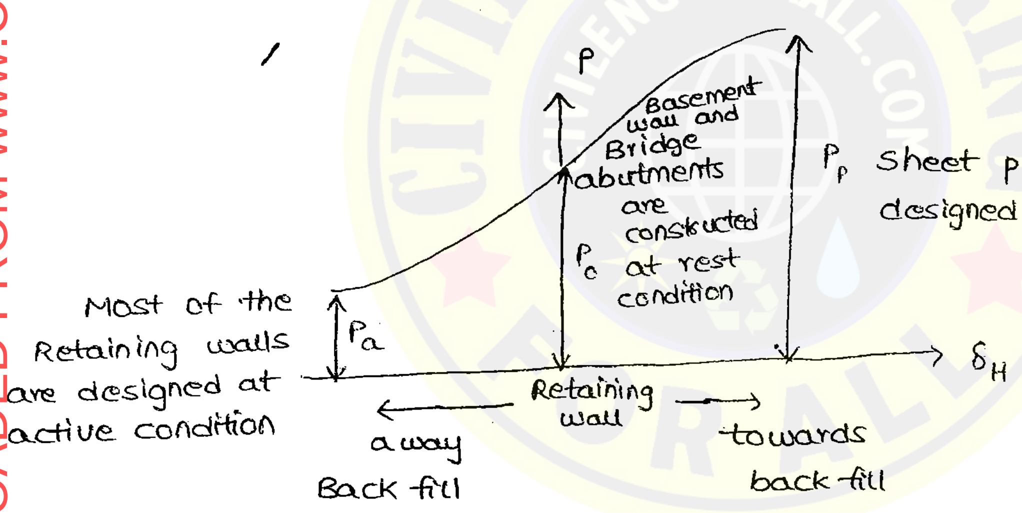
Passive pressure :-



$$F_p = \frac{1}{2}(K_p \gamma H)\cdot(H)(1)$$

$$= \frac{1}{2} K_p \gamma H^2$$

Comparison of K_p and K_a :-

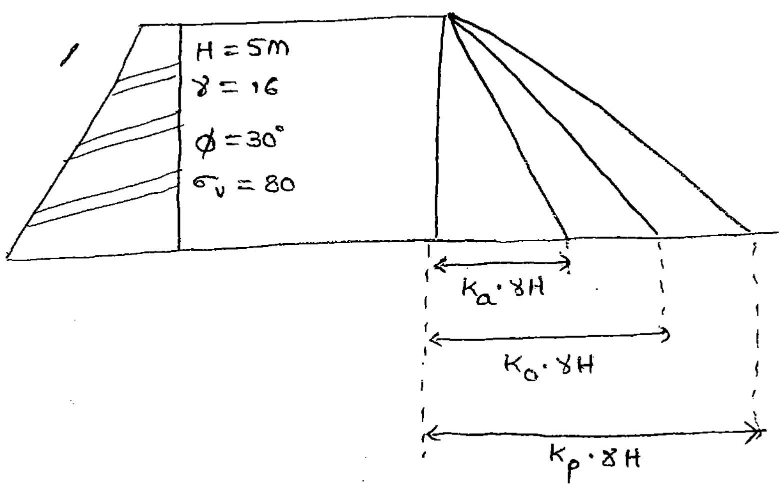


EX:- Given $K_a = \frac{1}{3}$, $K_o = 0.5$, $K_p = 3$

$P_a = K_a \gamma z$

$P_p = K_p \cdot \gamma z$

$P_o = K_o \cdot \gamma z$



$$\frac{P_p}{P_a} = ?$$

$$\frac{P_o}{P_a} = ?$$

$$\frac{P_p}{P_o} = ?$$

Note :-

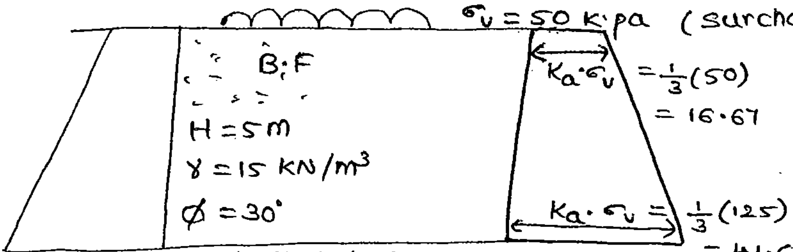
1. Any soil which is having a lesser frictional angle (ϕ), K_a will be less

2. Higher frictional angle is recommonded for construction.

EX :-

surcharge $q = 50$ k·pa (No verticle load)

$\sigma_v = 50$ k·pa (surcharge pressure is considered as verticle pressure)



If $\phi = 30$, $K_a = \frac{1}{3}$

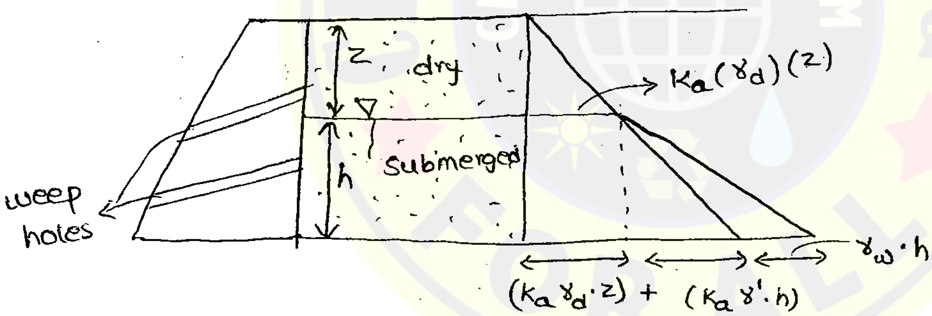
Total $\sigma_v = 50 + (\gamma H)$

$= 50 + 75$ (P.D.D)

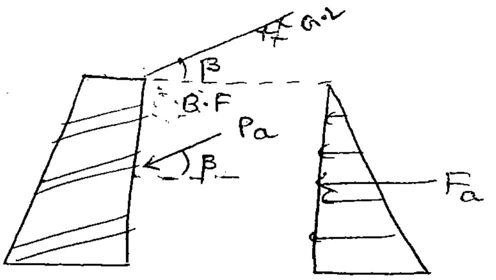
$= 125$

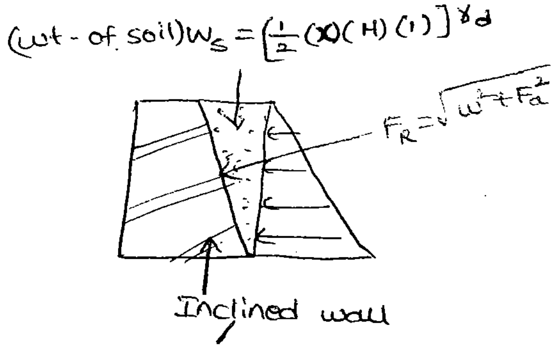
Due to (soil in Back fill + surcharge) the pressure distribution (P.D.D) diagram will be "Trapezium"

Variation of Lateral pressure :-



Rankine's earth pressure when ground is inclined (at surcharge angle) :-





$$K_a = \cos\beta \left[\frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}} \right]$$

$$K_p = \cos\beta \left[\frac{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}} \right]$$

Earth pressure in $c-\phi$ soil :-

1. Active pressure (P_a) :-

$$\sigma_1 = \sigma_3 \tan^2(45+\phi/2) + 2c\tan(45+\phi/2)$$

$$\sigma_1 = \gamma z, \quad \sigma_3 = P_a$$

$$\gamma z - 2c\tan(45+\phi/2) = P_a \tan^2(45+\phi/2)$$

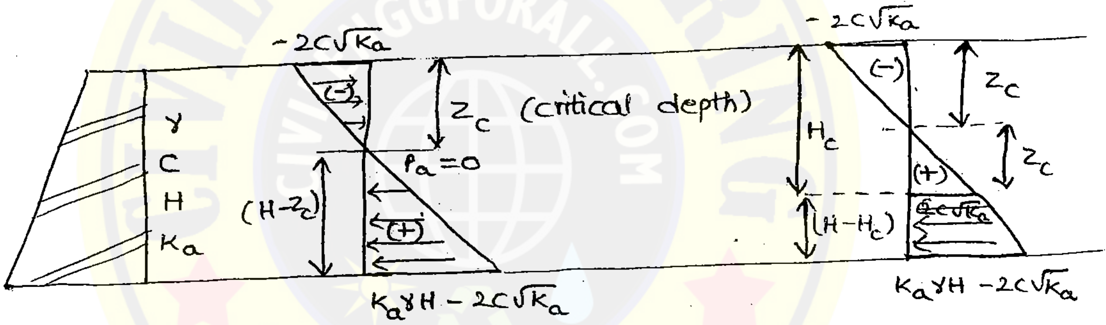
Divide both sides by $\tan^2(45+\phi/2)$

$$\frac{\gamma z}{\tan^2(45+\phi/2)} - \frac{2c\tan(45+\phi/2)}{\tan^2(45+\phi/2)} = \frac{P_a\tan^2(45+\phi/2)}{\tan^2(45+\phi/2)}$$

$$K_a(\gamma z) - 2c\sqrt{K_a} = P_a$$

$$\therefore \boxed{P_a = K_a(\gamma z) - 2c\sqrt{K_a}}$$

when $z > z_c$



$$P_a = K_a(\gamma z_c) - 2c\sqrt{K_a}$$

$$\therefore P_a = 0$$

$$K_a(\gamma z_c) = 2c\sqrt{K_a}$$

$$z_c = \frac{2c\sqrt{K_a}}{K_a \cdot \gamma}$$

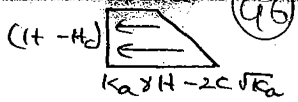
$$= \frac{2c}{\gamma}\left(\frac{\sqrt{K_a}}{K_a}\right) \qquad \therefore \frac{\sqrt{K_a}}{\sqrt{K_a}\cdot\sqrt{K_a}} = \frac{\sqrt{K_a}}{K_a}$$

$$\boxed{z_c = \frac{2c}{\gamma\sqrt{K_a}}} = \text{depth of tensile cracks.}$$

H_c = Height (or) depth of the unsupported excavation.

$\boxed{H_c = 2(z_c)}$ = depth of unsupported excavation

$\therefore$ No pressure is acting on 'H_c' area. pressure is acting below the 'H_c' area $(H-H_c)$.



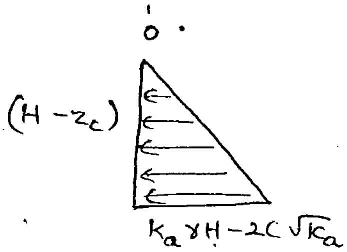
$\rightarrow$ Area of the trapezium, $F_a = \frac{h}{2}(a+b)$

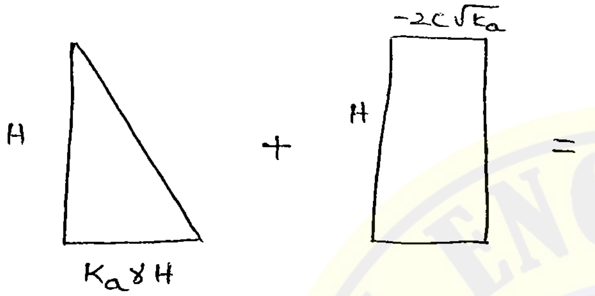
$$= \frac{H-H_c}{2}\left[2c\sqrt{K_a} + (K_a \gamma H - 2c\sqrt{K_a})\right]$$

This case is economical and less force because shorter area.

$\rightarrow$ $F_a = \frac{1}{2}\left[(K_a \gamma H - 2c\sqrt{K_a})(H - z_c)\right]$

This is the worst case and highest force (area is large)

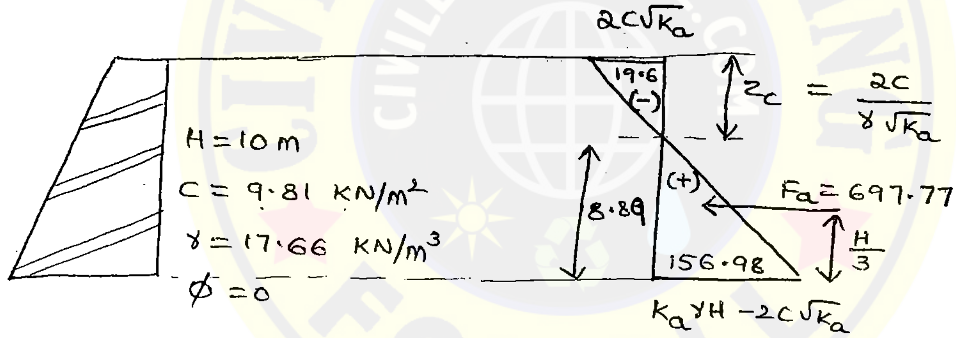




$$F_a = \left[\frac{1}{2}(K_a \gamma H)(H)\right] + \left[-2c\sqrt{K_a} \times H\right]$$

$$= \frac{1}{2} K_a \gamma H^2 - 2cH\sqrt{K_a}$$

Ex:-



$$z_c = \frac{2c}{\gamma\sqrt{K_a}} = \frac{2 \times 9.81}{17.66 \times 1} = 1.11$$

(or)

$$\frac{z_c}{19.6} = \frac{10 - z_c}{156.98}$$

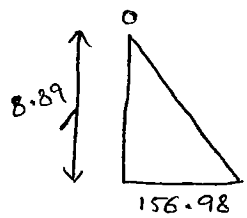
$$z_c = 1.11$$

If $\phi = 0°$ then $K_a = 1$

Neglect '-ve' zone

$F_a = \frac{1}{2}(156.98)(8.89)$ (unit length)

$= 697.77$ kN



$$= \frac{H}{2}[0 + 156.98]$$

Now consider whole diagram

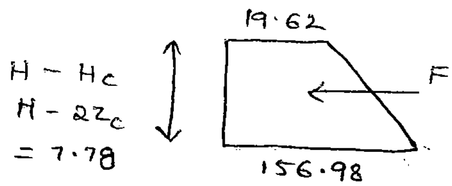
$F_{\text{whole diagram}} = 697.77 - \left(\frac{1}{2} \times 19.6 \times 1.11\right)$

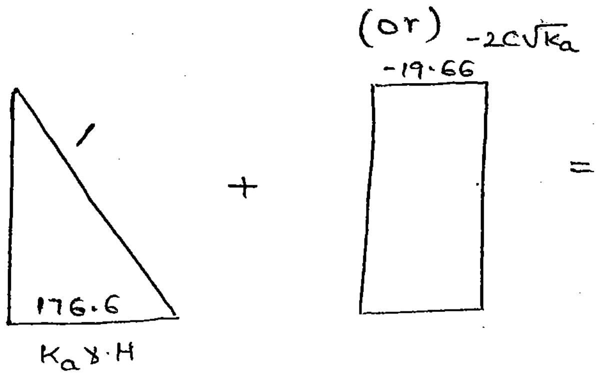
$= 686.$ kN

(or)

F_{whole} (or) $F_{\text{total}} = \frac{7.78}{2}(19.62 + 156.98)$

$= 686$ kN



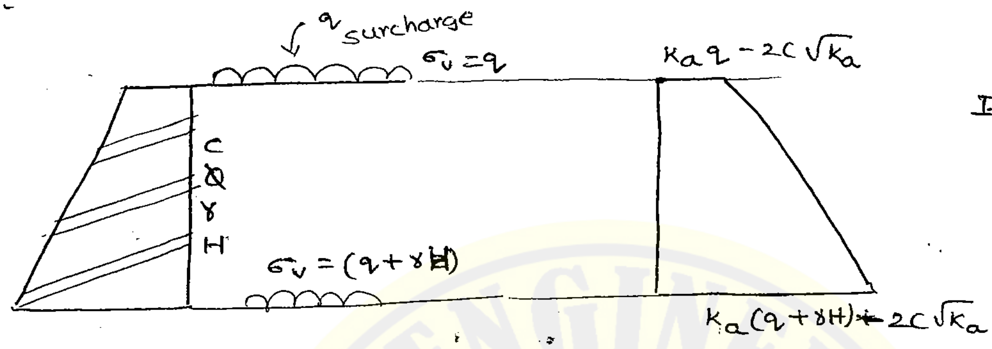


$$= \frac{1}{2}(176.6)(10) + (-19.66 \times 10)$$

$$= 686 \text{ KN}$$

Note:-

1.



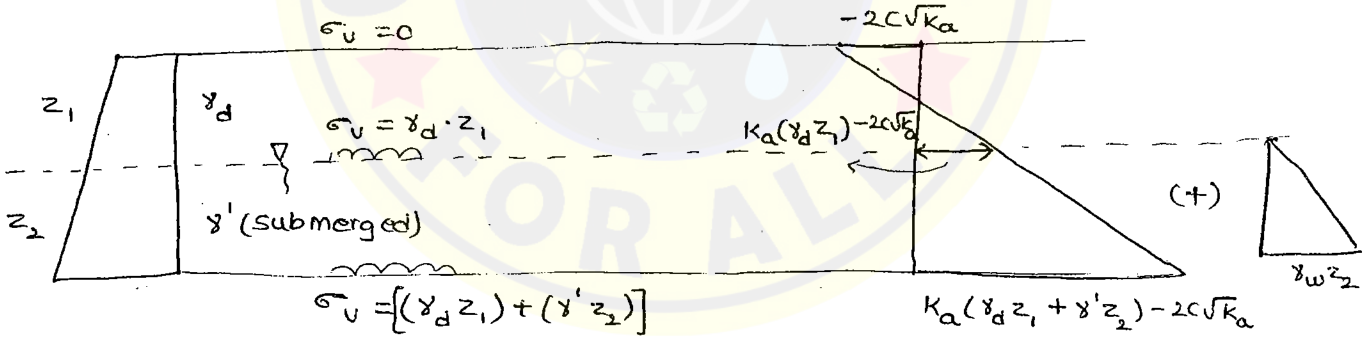
If there is a surcharge.

$$P_{a\,(top)} = K_a(\gamma z) - 2c\sqrt{K_a}$$

$$= K_a(q) - 2c\sqrt{K_a}$$

$$P_{a\,(bottom)} = K_a(q + \gamma H) - 2c\sqrt{K_a}$$

2.



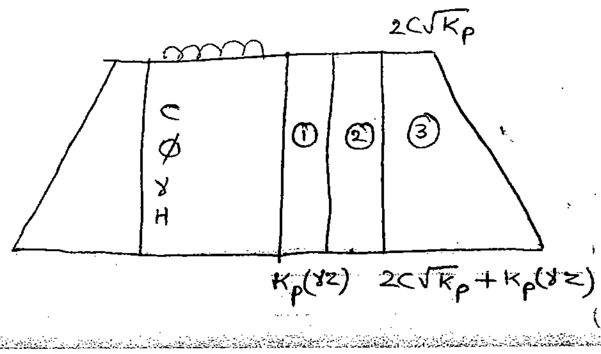
$$P_a = K_a(\sigma_v) - 2c\sqrt{K_a} = -2c\sqrt{K_a} \longrightarrow \text{top}$$

$$= K_a(\gamma_d z_1) - 2c\sqrt{K_a} \longrightarrow \text{middle}$$

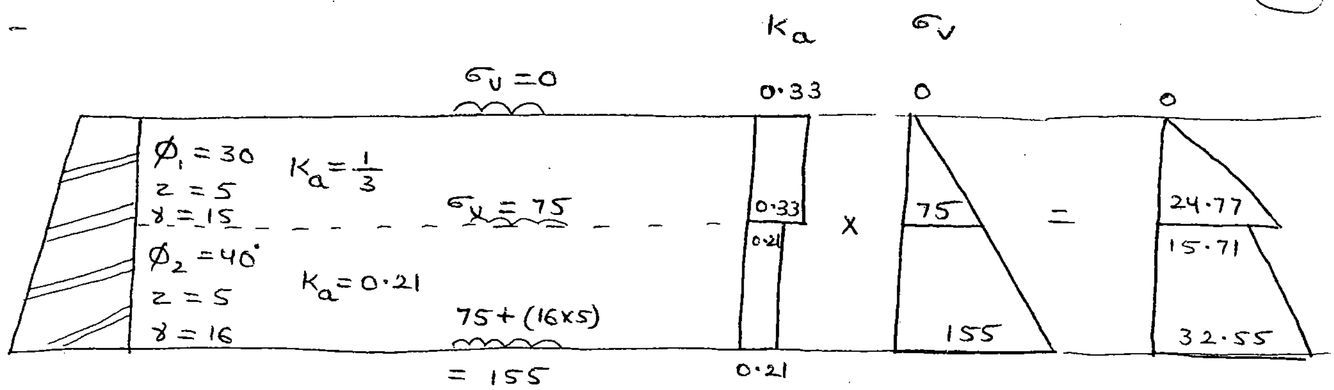
$$= K_a(\gamma_d z_1 + \gamma' z_2) - 2c\sqrt{K_a} \longrightarrow \text{bottom}$$

2. Passive pressure (P_p):- (c-ϕ soils)

$$P_p = K_p(\gamma z) + 2c\sqrt{K_p}$$

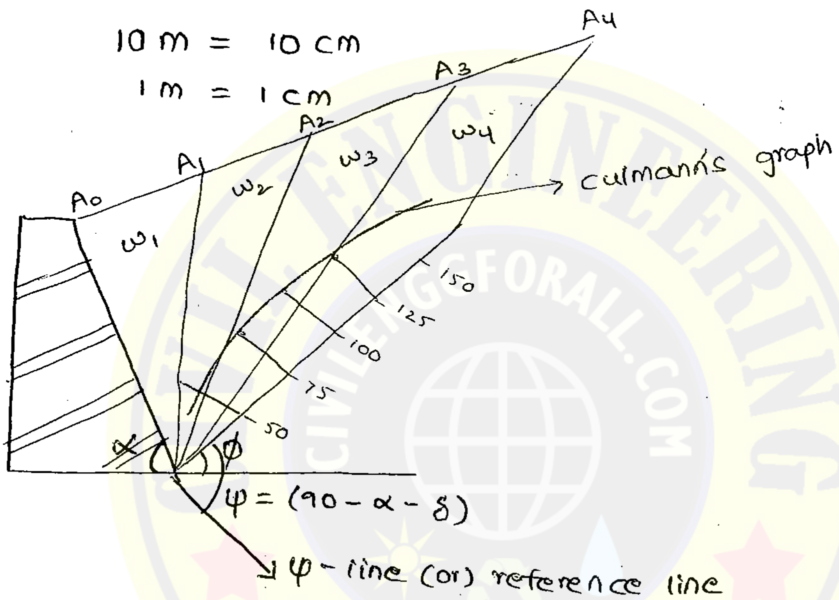


EX :-

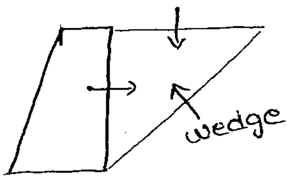
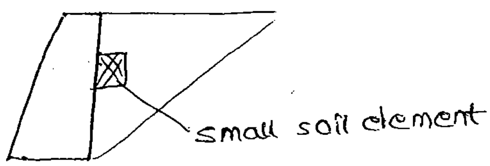


Graphical solution :-

1. Culmann's method :-



Difference between columb's theory and Rankine's theory

coloumb's	Rankine's
1. 	1. 
2. Rough walls ($\delta \neq 0$)	2. smooth walls ($\delta = 0$)
3. Masonry	3. R.C.C.
4. $K_a \Rightarrow \phi, \beta, \delta, \alpha$	4. $K_a \rightarrow \phi°, \beta°$

P.g NO:- 69

1. Given $c = 30 \ kN/m^2$

$\gamma = 20 \ kN/m^3$

$$H_c = 2 z_c$$

$$= \frac{4c}{\gamma \sqrt{K_a}}$$

$$= \frac{4 \times 30}{20 \times 1}$$

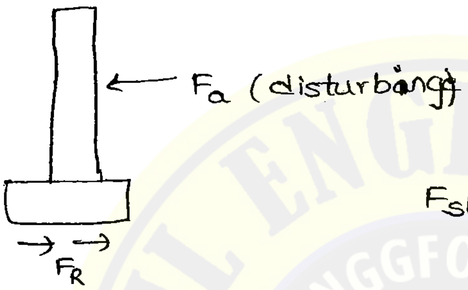
$$H_c = 6 \ m$$

For saturated clay

$\phi = 0$

$K_a = 1$

3.



$$F_{sliding} = \frac{F_R}{F_D}$$

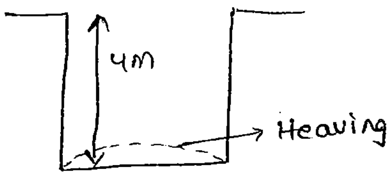
$$= \frac{\Sigma V \cdot \tan\delta}{F_D}$$

δ = angle of wall friction

10.

$$\frac{K_p}{K_a} = \frac{\frac{1+\sin\phi}{1-\sin\phi}}{\frac{1-\sin\theta}{1+\sin\phi}} = \frac{3}{\frac{1}{3}} = 3 \times 3 = 9$$

11.



$\gamma = 22$

$\phi = 0°$ for chohesion

$K_a = 1$

$$H_c = \frac{4c}{\gamma \sqrt{K_a}}$$

$$4 = \frac{4c}{22 \times 1}$$

$$c = 22 \ kN/m^2$$

12. Retaining wall is a R.C.C. structure. It can be determined based on Rankine's theory.

16. $Z_c = \frac{2C}{\gamma\sqrt{K_a}}$

$= \frac{2 \times 14}{16.5 \times \sqrt{1.373}}$

$= 2.33$

$\sqrt{K_a} = \sqrt{\frac{1-\sin\phi}{1+\sin\phi}} = \sqrt{\frac{1-\sin 18^\circ}{1+\sin 18^\circ}} = 1.373$

25. $q = 120 \text{ kN/m}^2$

$\phi = 30^\circ$

$K_a = \frac{1}{3}$

$F = K_a \cdot q$

$= \frac{1}{3}(120) = 40 \text{ kN/m}^2$

25-12-2014

UNIT - 20

FOUNDATIONS IN EXPANSIVE SOILS

Expansive soils:-

1. Black cotton soils is also called Expansive soils
2. Water is a polar liquid and causes swelling and shrinkage
3. Montmorillonite mineral is present in clay which causes more swelling and shrinkage. Expansive soils are also called cohesive swelling soil.

Identification of expansive soils :-

1. In dry soil cracks are develop. Absorb that cracks and measure the size and depth of the crack.

   size → inches

   depth → metres

2. Walk on wet soils, if soil is sticky (highly plastic soils) can be indicate as expansive soils.
3. Observe lightly loaded structure (only ground storey building, roadways, canal (empty)) if damage is severe soil can be expansive.

Tests

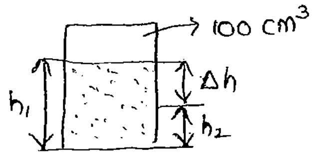
Less costly ~~test~~:

1. Free swell test
2. Differential free swell test
3. Swelling pressure test

complicated & expensive test:

1. X-Ray test
2. Microscopic test
3. chemical analysis

Free swell test :-



$$\Delta h = h_2 - h_1$$

$$\text{Free swell} = \frac{\Delta h}{h_1} \times 100\,\%$$

Free swell > 50% damage is caused

1. Life ~~Light~~ damage → It is nill
2. Structural damage
3. Cosmetic (or) architectural damage

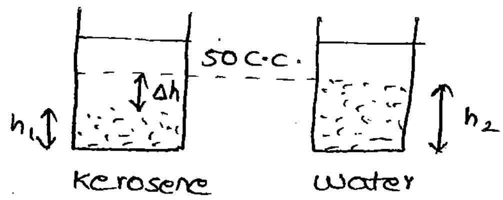
(2, 3) property damage

Most of the damage in the expansive soils are cosmetic damage cracks (flooring etc)

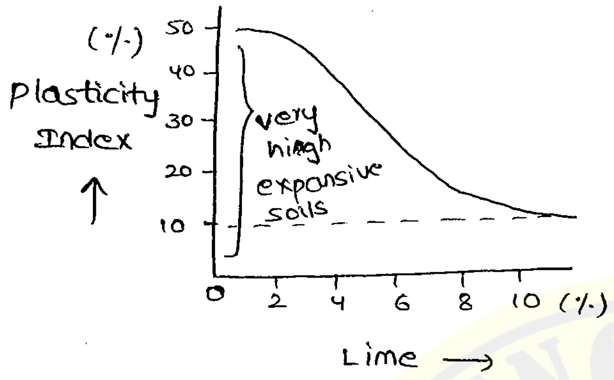
Differential free swell test :-

$$DFS = \frac{h_2 - h_1}{h_1} \times 100$$

$$DFS = \frac{\Delta h}{h_1} \times 100$$

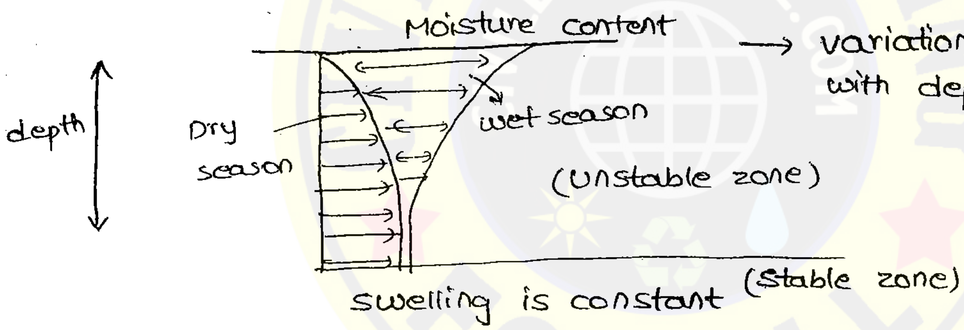


Relationship between swelling potential of soil and plasticity index :-



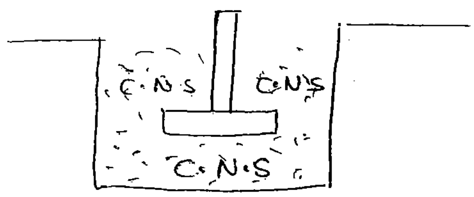
} Low expansiveness when lime is added, then the plasticity Index drop down to 10%.

~~Methods~~ Design of foundations in expansive soils :-



→ variation of Moisture content with depth

1. Shallow foundations are not recommended in expansive soils
2. Suitable for deep footing and under reamed pile footing. Min depth of foundation is 2m.
   ↓ best method
3.



cohesive non swelling soil
EX:- Moorum

25-12-2014

UNIT - 19

SHEET PILES

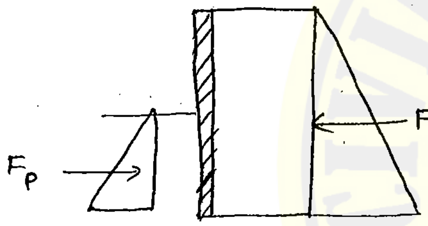
Sheet pile is one of the "flexible retaining wall". They are made of timber, steel, RCC, prefabricated unit (RCC) etc.

1. sheet piles always designed for lateral force.
2. Timber carrying less load. Durability is less.

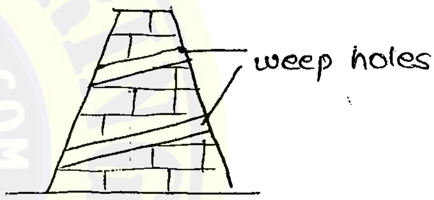
Steel pile : More load, less driving pile.
timber pile : Less load, temporary structure, durability less
Prefabricated pile : More thickness, more load, life is good.

3. In sheet piles, section is thinner to suitable low bearing capacity soils.

Sheet pile



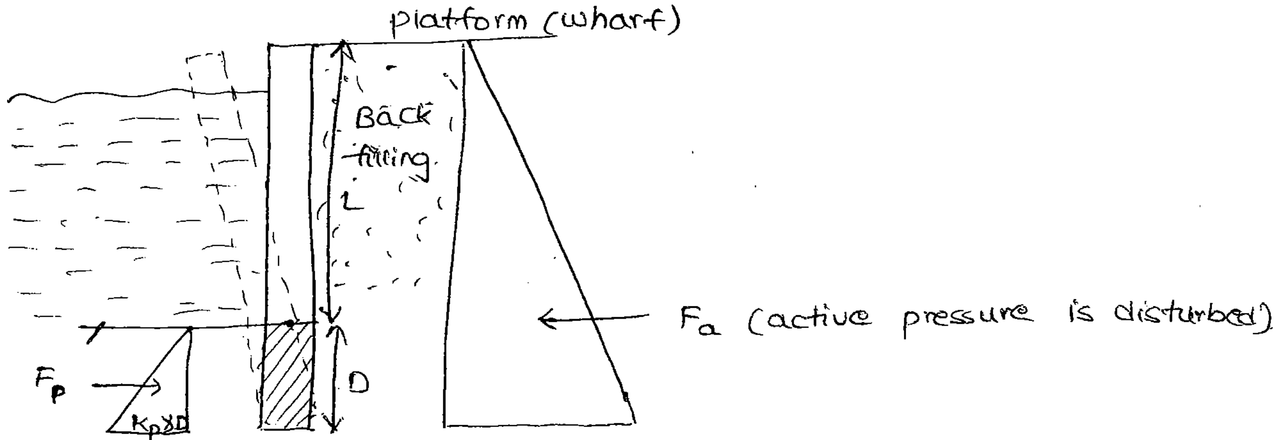
Retaining wall (R.C.C) (Masonry)



1. No weep holes in sheet piles
2. Anchored
3. Flexible

1. Bearing capacity increases
2. Rigid
3. Not anchored
4. Weep holes are provided.

Cantilever sheet pile:-



Passive is resistance

26-12-2019

UNIT - 18

SOIL EXPLORATION

Soil exploration is also called soil investigation (or) site investigation.

Exploration means to bring out the hidden things like rock, water table, type of soil, sound rock etc on earth.

Budget for site investigation is to spend maximum 2% of the total project cost

Site investigation is not done (or) adequate causes

1. Failure of the structure
2. Improper functioning of the structure
3. A Unable to decide altration or addition.

Seismic refractive method is the one of the method which is non destructive method.

Note:-

1. When bore holes are in closer spacing, no. of bore holes are increasing, time is increasing cost is increasing
2. Soil → Homogeneous, uniform, spacing is more, no. of Bore holes is less
3. Soil → Errotic soil, non uniform soil, spacing is less, no. of Bore holes is high.

Methods of exploration:-

1. Test pit method:-

Disadvantages:-

1. Soil is disturbed
2. Uneconomical for deeper depth

Advantages:-

1. Economical
2. Large quantity of soil is available.

2. Boring method:-

Making a small hole in soil is called Boring. There are two types of borings.

Auger Boring:-

1. Auger Boring gives highly disturbed soil.
2. It is economical for shallow depth.
3. It is a priliminary site investigation with a rapid time (quick)
4. There are two types of Auger Boring. They are

Hand Augers → suitable for shallow depths like Roads

power Augers → deeper depth.

Wash boring:-

Advantages:-

1. Information for deeper strata

Disadvantages:-

1. Destructive test
2. can't be used if Rock present
3. Disturbed soil.

percussion drilling or cable tool method:-

Advantages:-

1. Suitable for all soils
2. Brings soil sample

Disadvantages:-

1. Highly disturbed
2. progress is slow
3. Durability of Rock (or) "slack durability test"
4. Destructive test.

Factors affecting the sample disturbance:-

1. split spoon sampler (SSS) – Disturbed
2. shelby tubes
3. piston sampler
4. Thin wall sampler

} undisturbed

→ Static loads are good for disturbed sample.

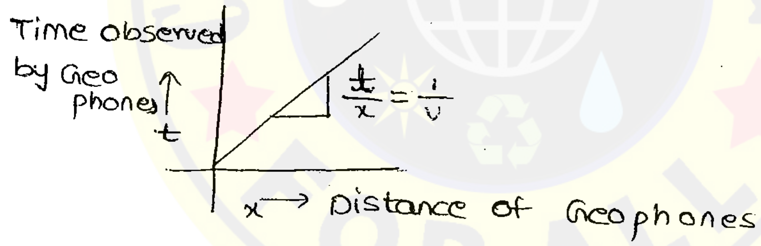
→ Dynamic loads are good for undisturbed sample.

pressure meter test:-

1. It is a field test
2. Deeper soil
3. Radial displacement
4. volume changes

Seismic refraction method:-

1. One of Non disturbed test
2. preliminary
3. No samples are taken out
4. surface or nearby surface zone (Deeper soil cannot be evaluate by this method)
5. suitable for runways, highways
6. Large area in short period of time
7. In seismic refraction method slope gives "inverse of velocit



Ground Improvement Engineering

- Geo synthetics
- Stabilization
- Grouting

1. Mechanical stabilization:-

By adding one soil + other soil is called Mechanical stabilization. No chemicals are used.

2. Chemical stabilization:-

soil + cement + water + compaction + curing. Quantity of cement required approximately, 5% for sand and 15% for clays

soil + chemical = chemical stabilization.

Lime stabilization:-

Lime + cohesive soils = Lime stabilization.

1. Binding increases
2. permeability decreases
3. Durability increases
4. Erosion decreases
5. Water proofing.

Ground improvement technique:-

1. Electro osmosis
2. vibroflotation method
3. Terraprobe method
4. Lime piles
5. stone columns
6. Geotextiles

Geotextitles:

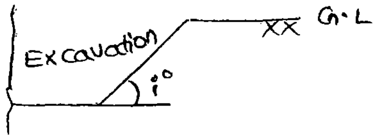
1. It is a synthetic material (or) artificial material.
2. It is used inside of soil like reinforcement, drainage.
3. It looks black in color.
4. It acts membrane (water proofing)

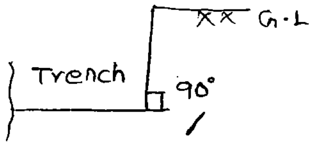
28-12-2017

UNIT-15

STABILITY OF SLOPES

Slopes are Embankment, Earthen dams, canals, Trenches, Leeves





Flat	steeper slope
1. More land	1. Less land
2. Stability is more	2. Stability is less
3. More material	3. Less material.

Slope failure may takes place due to

1. Gravitational force
2. Natural disasters
3. Rains (seepage)

Toe failure:-

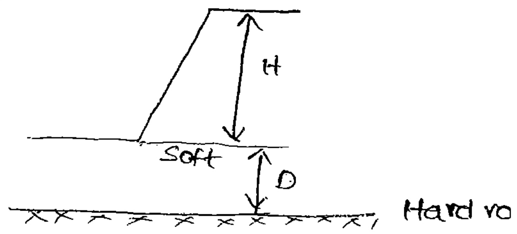
1. Toe failure is the common amoung the three failures

Base failure:-

1. It Occurs beyond the ~~toe~~ toe failure.

Depth factor (D_f):-

$$D_f = \frac{D+H}{H}$$



For face failure, $D_f < 1$

For toe failure, $D_f = 1$

For base failure, $D_f > 1$

Stability of infinite slopes:

Factor of safety, $F = \frac{S}{\tau} = \frac{\text{shear strength}}{\text{shear stress}}$

Infinite slope in $c-\phi$ soils:-

$$F = \frac{c + \sigma_n \tan\phi}{\tau}$$

$$F = \frac{c + \gamma z \cos^2 i \tan\phi}{\gamma z \cos i \sin i}$$

$$1 = \frac{c + (\gamma z_i \cos^2_i)\tan\phi}{\gamma z_c \cos i \sin i}$$

$$\frac{\gamma z_c \cos i \sin i}{z_c \gamma} = \frac{c}{z_c \gamma} + \frac{\gamma z_c \cos^2 i}{z_c \gamma}$$

$$\frac{c}{z_c \gamma} = \cos i \sin i - \cos i \tan\phi$$

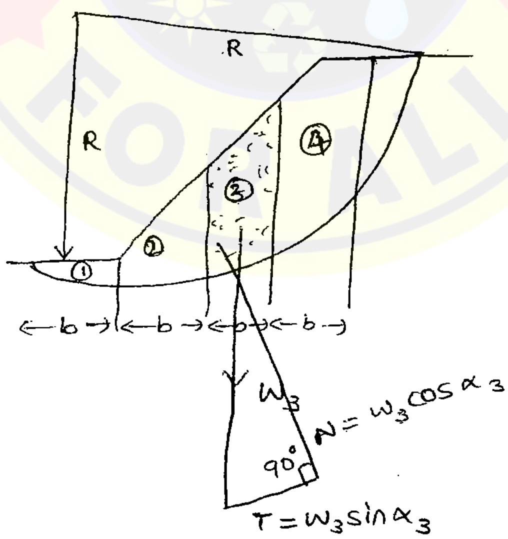
$$\frac{z_i}{z} = \frac{H_c}{H} = F_c$$

$$z_c = (F_e \cdot z)$$

$$\frac{c}{F_e \gamma z} = \cos^2 i(\tan i - \tan\phi) = S_N = \text{stability number.}$$

Swedish circle method (or) method of slices:-

1. It can be used for $c-\phi$ soils.



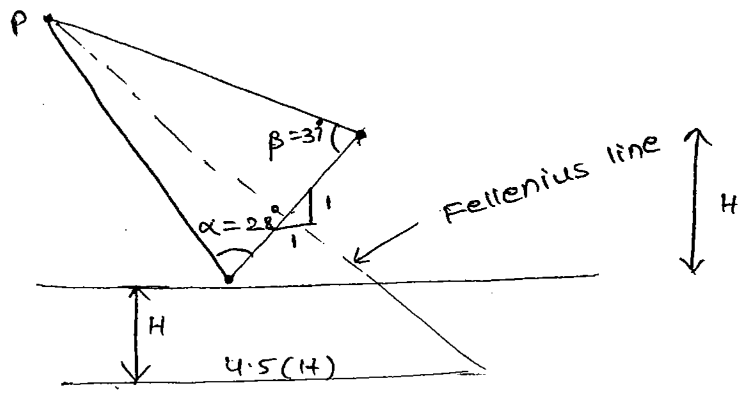
$$F_{c-\phi} = \frac{c(L_{arc} \times 1) + (\Sigma N)\tan\phi}{\Sigma T}$$

Fellenius method :-

For 1:1/ slope

$\alpha = 28^\circ$

$\beta = 37^\circ$



Factor of safety :-

$$F_c = \frac{c}{c_m} = \frac{H_c}{H} = \frac{\text{Critical height}}{\text{Actual height of slope}}$$

$$F_\phi = \frac{\tan\phi}{\tan\phi_m} = \frac{\text{friction angle}}{\text{mobilised friction angle}}$$

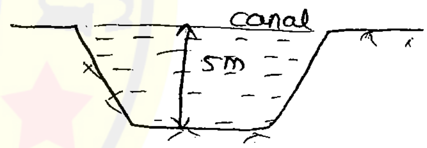
$$\phi_m = \tan^{-1}\left[\frac{\tan\phi}{F_\phi}\right]$$

P.g No:-79

33. Given $c = 1.5\ T/m^2$ $H = 5m$

$\gamma_{sat} = 2\ t/m^3$

$S_n = 0.1$



F_c $S_N = \frac{c}{F_c \cdot \gamma' \cdot H}$

$$F_c = \frac{c}{S_N \gamma' H}$$

$$= \frac{1.5}{0.1(2-1)(5)}$$

$$F_c = 3$$

34. Method of slices

$\Sigma T = 500$ KN

$\Sigma N = 900$ KN

$\Sigma U = 200$ KN

$L_{arc} = 27$ m

$c = 20$ KN/m^2

$\phi = 20^\circ$

$$F_{c-\phi} = \frac{c(L_{arc} \times 1) + \Sigma(N-U)\tan\phi}{\Sigma T}$$

$$= \frac{20(27\times1) + (900-200)\tan 20^\circ}{500}$$

$$= 1.5895$$

35. Given $c = 2T/m^2$ $\quad \gamma = 2T/m^3$, $\quad F = 2$ $\quad S_n = 0.1$

$$S_n = \frac{c}{F_c \cdot \gamma H}$$

$$0.1 = \frac{2}{2 \times 2 \times H}$$

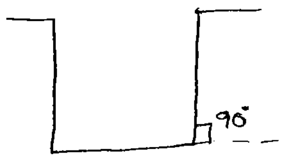
$$H = \frac{2}{0.4} = 5\,m$$

36. Given $c = 2.6\ t/m^2$

$i = 90^\circ$

$S_n = 0.261$

$\gamma = 2\ t/m^3$



$$H = \frac{c}{S_n \times \gamma \cdot F}$$

If F.S is not given then take safe F.S = 1

$$= \frac{2.6}{0.261 \times 2 \times (1)}$$

$$= 5.04\ m$$

Additional questions:-

1. Given $H = 7m$

$\gamma = 16\ KN/m^3$

$c = 25\ KN/m^2$

$i = 30^\circ$

$S_n = 0.178$

$$F_c = \frac{c}{S_n \cdot \gamma \cdot H}$$

$$= \frac{25}{0.178 \times 16 \times 7}$$

$$= 1.25.$$

UNIT - 16

BEARING CAPACITY

Shallow Foundations:-

A foundation is considered as shallow depth is called shallow foundation. If the depth is equal to or less than the width it is shallow.

Ex:- Spread footing, continuous footing, Raft foundation etc.

Deep Foundation:-

Load is transmitted at considerable depth below G.L.

Ex:- pile, pier, caisson etc.

Net pressure (q_n):-

Gross pressure - original over burden pressure

$$q_n = q - \gamma D$$

D = Depth of foundation

Ultimate bearing capacity:-(q_u)

The minimum gross pressure at the base of the foundation at which soil fails in shear.

Net ultimate bearing capacity (q_{nu}):-

$$q_{nu} = q_u - \gamma D$$

Net safe bearing capacity (q_{ns}):-

$$q_{ns} = \frac{(q_{nu})}{F}$$

F = Factor of safety, = 3

Gross safe bearing Capacity (or) Safe bearing capacity (q_s):-

$$q_s = q_{ns} + \gamma D$$

Rankine's Analysis:-

$$D_{min} = \frac{q}{\gamma}\left[\frac{1-\sin\phi}{1+\sin\phi}\right]^2$$

q = load intensity at the base

Terzaghi's general bearing capacity equation for continuous footing:-

$$q_{ult} = CN_c + \gamma D_f N_q + 0.5\,\gamma B N_\gamma \xrightarrow{\text{valid for}} C-\phi,\ G.S.F,\ \text{strip}$$

N_c, N_q, N_γ are bearing capacity factors for General shear failure. It depends upon angle of friction ϕ

Note:-

As the angle of internal friction increases bearing capacity factors also increases.

$$q_{net} = [q_{ult} - \gamma D_f]$$

$$q_{net} = CN_c + q(N_q - 1) + \frac{1}{2} B N_\gamma \cdot \gamma$$

$$q_{n(safe)} \text{ (or) } q_{n\,(allow)} = \frac{1}{F}\left[CN_c + q(N_q - 1) + \frac{1}{2}\gamma B N_\gamma\right]$$

$$q_{ult\,(gross)} = 1\cdot CN_c + 1\cdot q N_q + 1\cdot \frac{1}{2}\gamma B N_\gamma$$

$$S_c, S_q, S_\gamma = 1$$

S_c, S_q, S_γ are shape factors.

shape factors	strip	square	circular	rectangular
S_c	1	1.3	1.3	$1+0.3\frac{B}{L}$
S_q	1	1.0	1.0	1.0
S_γ	1	0.8	0.6	$1-0.2\frac{B}{L}$

Circular footing:-

$$q_{ult\,(gross)} = 1.3\,CN_c + 1\cdot qN_q + 0.6 \times \frac{1}{2}(D)\gamma \cdot N_\gamma$$

$$S_c = 1.3, \quad S_q = 1 \quad S_\gamma = 0.6$$

Square footing:-

$$q_{ult\,(gross)} = 1.3\,CN_c + 1.q\,N_q + 0.8 \times \frac{1}{2}\gamma B N_\gamma$$

Rectangular footing:-

$$q_{ult} = \left(1+0.3\frac{B}{L}\right)CN_c + 1\cdot qN_q + \left(1-0.2\frac{B}{L}\right)\frac{1}{2}\gamma B N_\gamma$$

Note:-

For pure cohesive soil ($\phi = 0$), $N_c = 5.7$, $N_q = 1.0$, $N_\gamma = 0$

$$q_{ult(g)} = cN_c + q(1) + 0$$

c-soils

$$q_{net} = [cN_c + q] - q$$

$$q_{net} = cN_c$$

$$\cong c \times 6$$

$$q_{net} \cong 6c$$

$$q_{safe\,(net)} = \frac{6c}{3} = 2c = U.C.S$$

$c = \frac{U.C.S}{2}$

$2c = U.C.S$

$$\boxed{q_{safe\,(c-soil)} = U.C.S}$$

∴ Most of the soils Factor of safety = 2 to 3.

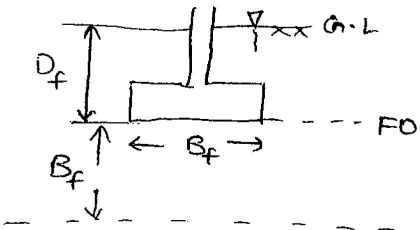
$$q_{net} = \left[q.N_q + \frac{1}{2}\gamma B N_\gamma\right] - q$$

$$= q(N_q - 1) + \frac{1}{2}\gamma B N_\gamma$$

$$\boxed{\underset{(\phi\text{-soil})}{q_{safe}} = \frac{1}{F}\left(q(N_q - 1) + \frac{1}{2}\gamma B N_\gamma\right]}$$

Effect of water table on Bearing capacity of soils :-

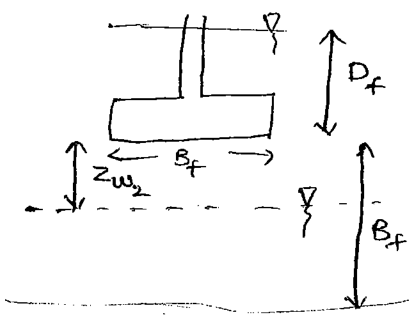
1. Bearing capacity will not affect when water table is $(D_f + B_f)$ from Ground surface and above.



2. If $D_f + B_f$

$R_{w_1} = R_{w_2} = 1$

$R_{w_2} = 0.5\left(1 + \frac{Z_{w_2}}{B}\right)$



3. W.T is at F.L

$R_{w_1} = 1$ $R_{w_2} = 0.5$

4. W.T is at G.L

$R_{w_1} = R_{w_2} = 0.5$

$\gamma_{sat} \times 0.5 = \frac{\gamma_{sat}}{2} = \gamma'$

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UNIT- 17

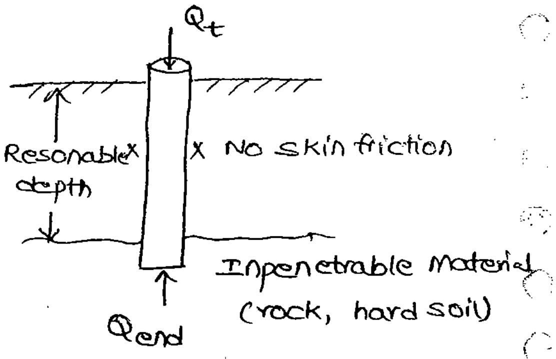
PILE FOUNDATION

Classification of piles:-

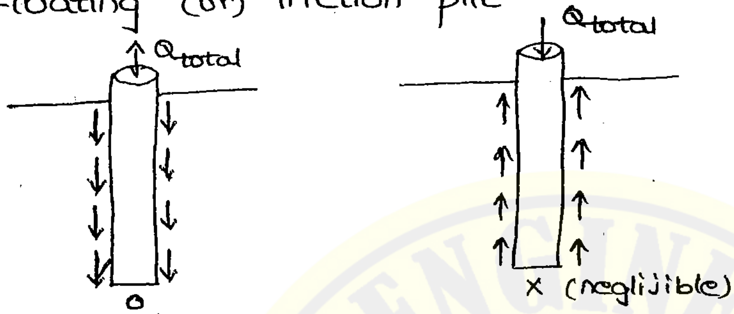
a. Based on load transfer:-

1. End bearing piles

   – No skin friction

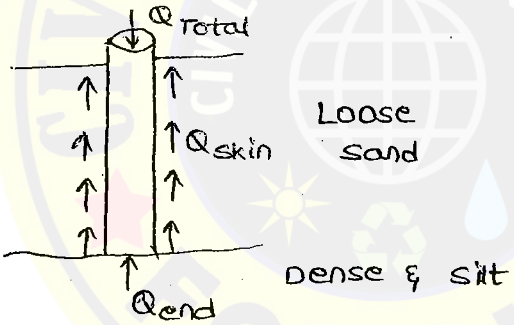


2. Floating (or) friction pile



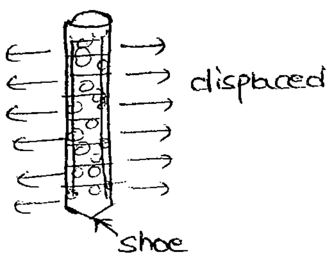
– No hard strata available in reasonable depth.

3. Combined end bearing and friction pile



b. Based on method of installation:-

1. Driven and cast-in-situ piles:-



Load carry capacity of piles:-

1. Static formula
2. Dynamic formula
3. penetration test (SPT, CPT)
4. pile load test

Static formula:-

a. piles in clay:-

$$Q_{ult} = Q_{end} + Q_{skin}$$

$$= (f_b) A_{tip} + (f_s) A_{surface}$$

$$= (C_{tip} \cdot N_c) A_{tip} + (C \cdot \alpha) A_{surface}$$

α = Adhesion factor (or) Shear mobilised factc

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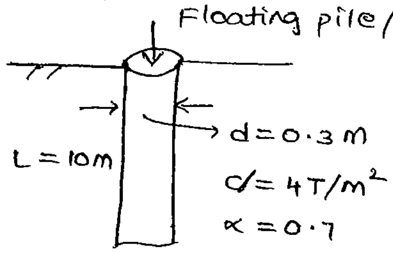
9. $Q_{ultimate} = C \cdot \alpha \cdot A_{surface}$

Floating pile/friction pile



F.S = 2.5

$Q_{end} = 0$

$$= 4 \times 0.7 \times \pi (0.3) 10$$

$$= 26.389 \text{ T}$$

$$Q_{safe} = \frac{Q_{ult}}{F.S} = \frac{26.389}{2.5}$$

$$= 10.55 \text{ T}$$

If top of the pile load is zero and end is given

$$Q_{ult} = Q_{end} + 0$$

$$= C N_c (A_s)$$

$$= 4 \times 9 \times \frac{\pi (0.3)^2}{4}$$

$$= 2.5 \text{T}$$

$$Q_{safe} = \frac{2.5\text{T}}{2.5\text{T}} = 1\text{T}$$

If both loads are given then $Q_{safe} = 10.55 + 1 = 11.55$ T

Ex:-

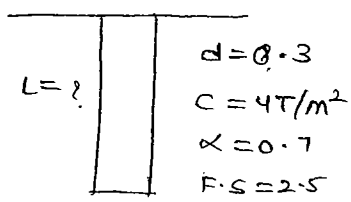
$$Q_{ult} = Q_{end} + Q_{skin}$$

$$Q_{ult} = 0 + Q_{skin}$$

$$(2.5 \times 25\text{T}) = c\alpha (\pi d L)$$

$$62.5\text{T} = 4 (0.7) (\pi \times 0.3 \times L)$$

$$L = 24\text{m}$$



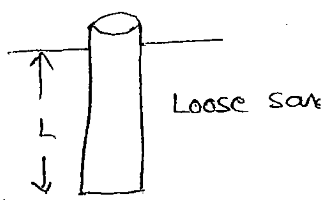
$\therefore Q_{safe} = \frac{Q_{ult}}{F.S}$

b. Piles in sand:-

$$Q_{ult} = Q_{end} + Q_{skin}$$

$$= (f_b) A_{tip} + (f_s) A_{surface}$$

$$= (q'_{tip} \cdot N_q) A_{tip} + (K \cdot \tan\delta \cdot q'_{avg}) \cdot A_s$$



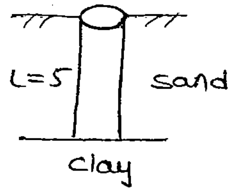
$N_q \downarrow$ $\phi, \frac{L}{D}$

$K \downarrow$ earth pressure coefficient

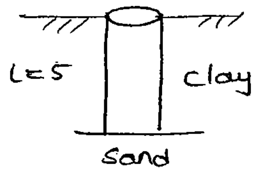
δ = angle of wall frictio

$\delta = \phi$

Ex:-) i) $Q_{ult} = CN_c A_{tip} + K \tan\phi\, q' A_s$



ii) $Q_{ult} = CN_c A_s + K \tan\phi\, q' A_{tip}$



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5. Given $W = 12$ kN $h = 2$ m $s = 10$ mm

$F.S = 6$ (assume)

$\eta_h = 1$ (assume) max

$c = 2.54$ cm for drop hammer

$$Q_s = \frac{W h \eta_h}{F(s+c)}$$

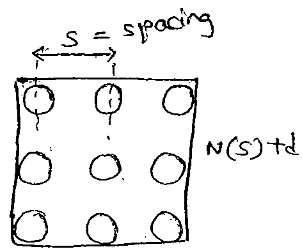
$$= \frac{12 \times 200 \times 1}{6(1 + 2.54)}$$

$$= 112.99 \text{ kN}$$

Group of piles:-

Efficiency of pile group, $\eta_g = \dfrac{Q_g \times 100}{N \times Q_{s.p}}$

Group capacity of piles in clay (Q_g):-



a. Individual failure:-

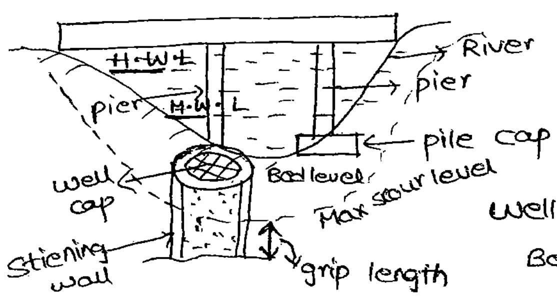
$$Q_{gi} = n \cdot Q_{usp}$$

$$= n\left[(C_{tip} N_c) A_{tip} + (c\alpha) A_s\right] \longrightarrow ①$$

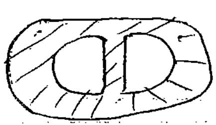
$$Q_{group/block/pier} = C_{tip} \cdot N_c \cdot A_{block\,(tip)} + C_{avg} \cdot A_{s\,(block)} \longrightarrow ②$$

$$= C_{tip} \cdot N_c \cdot (N(s)+D)^2 + C_{avg} \cdot A_{s\,(block}$$

Well foundation:-



Well cap is constructed at Bed level of river



Double - D shape

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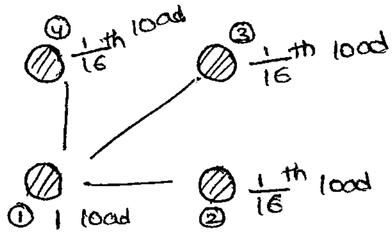
6.



1st pile carry a load something '1'

2nd pile carry a $\frac{1}{16}$th load of 1st pile

3rd, 4th also carry same $\frac{1}{16}$th load

$$\therefore 1 - 3\left(\frac{1}{16}\right)$$

$$= \frac{16-3}{16}$$

$$= \frac{13}{16}$$