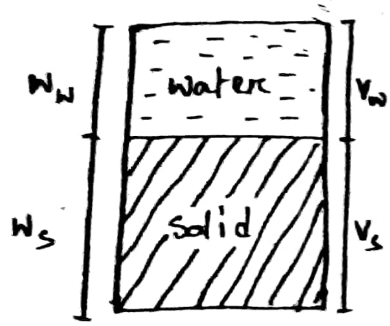
Roll : 1201032
Mob: 01742-940304

'phase Relationship'

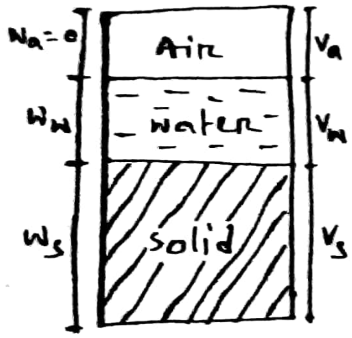
* **Geotechnical Engineering:** It is the branch of civil Engineering that deals with soil, rock and underground water and their relation to the design, contruction and operation of engineering projects.

* **Soil:** Soil is defined as an uncemented or weakly cemented aggregate of mineral grains formed by the weathering of rocks & decayed organic matters with liquid and gas in the empty spaces between the solid particles.

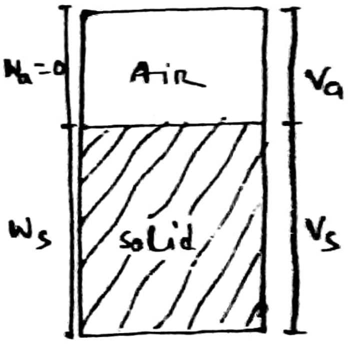
* Draw the phase diagram of Fully saturated soil, partially saturated soil & fully dry soil.



Fully Saturated soil

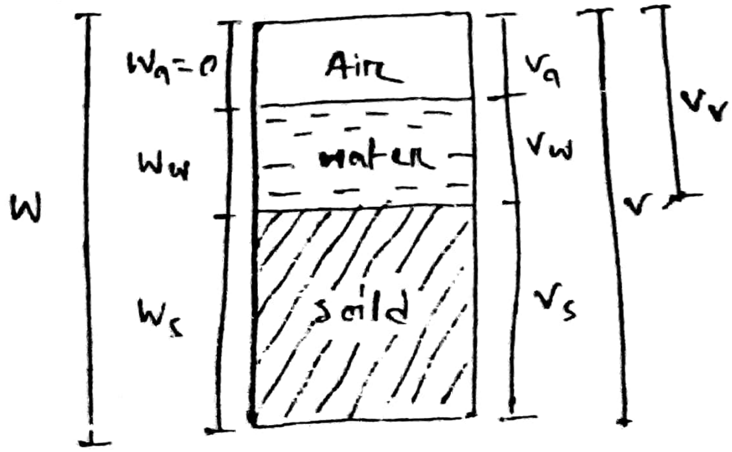


partially saturated soil



Fully dry Soil

* Draw the phase relation of soil.



Here,

$V_v = V_a + V_w$

$V = V_v + V_s$

Fig: phase Diagram of a Soil

Necessary Equations:

① Moisture (water) content, $w = \frac{W_w \text{ (wt of water)}}{W_s \text{ (wt of soil solids)}} * 100\%$.

② Void Ratio, $e = \frac{V_v}{V_s}$; expressed in decimal.

③ porosity, $n = \frac{V_v}{V} * 100\%$.

④ Degree of saturation, $S = \frac{V_w}{V_v} * 100\% = \frac{wG_s}{e} = \frac{W_w/\gamma_w}{V_v}$

⑤ $e = \frac{n}{1-n}$ ⑥ $n = \frac{e}{1+e}$

⑦ Unit wt, $\gamma = \frac{W}{V}$ ⑧ ~~Dry unit wt, $\gamma_d = \frac{W_s}{V}$~~

⑨ $\gamma_d = \frac{\gamma}{1+w} = \frac{G_s\gamma_w}{1+e}$ ⑩ $\gamma_{sat} = \frac{W_{sat}}{V} = \frac{(G_s+e)\gamma_w}{1+e}$

⑩ Unit wt, $\gamma = \frac{W}{V} = \frac{W_s + W_w}{V} = \frac{W_s(1+w)}{V} = \gamma_d(1+w)$

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

⑪ Density

$\rho = \frac{G_s\rho_w(1+w)}{1+e}$

Dry Density.

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

saturated density

$\rho_{sat} = \frac{(G_s+e)\rho_w}{1+e}$

⑫ $e = (wG_s)/S$; For completely saturated soil, $S = 1$

([illegible]) $\therefore e = wG_s = \frac{G_s\gamma_w}{\gamma_d} - 1$

⑬ Submerged Unit wt, $\gamma' = \frac{(G_s\gamma_w - \gamma_w)}{1+e} = \gamma_{sat} - \gamma_w$

⑭ Relative Density, $D_r = \frac{e_{max} - e}{e_{max} - e_{min}}$ (%).

⑮ Density Index, $I_d = \frac{\rho_{max} - \rho}{\rho_{max} - \rho_{min}}$

⑯ $\gamma_d = \frac{W_s}{V}$

* Unit weight of water, $\gamma_w = 9.81 kN/m^3$
$= 1000 kg/m^3$
$= 62.4 lb/ft^3$.

* Relation between void Ratio & porosity:

$$\text{void ratio, } e = \frac{V_v}{V_s} = \frac{V_v}{V-V_v} = \frac{V_v/V}{1-V_v/V} = \frac{n}{1-n}$$

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

* Relation between moist unit wt, γ & dry unit wt, γ_d:

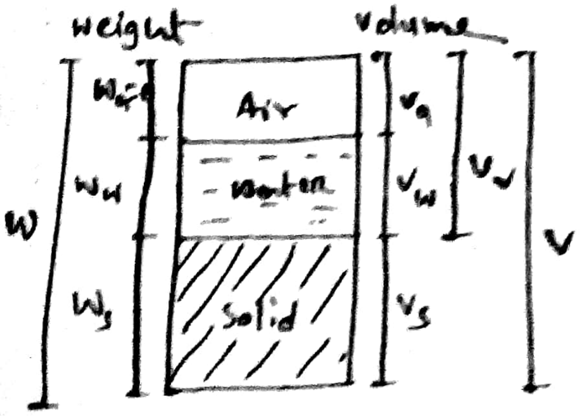
we know, $\gamma = \frac{W}{V}$, $\gamma_d = \frac{W_s}{V}$

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

$$= \frac{W_s}{V}\left(1+\frac{W_w}{W_s}\right) = \gamma_d(1+w)$$

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

* Relationships among Unit wt, void ratio, moisture content & specific gravity:



Assume, $V_s = 1$

void ratio, $e = \frac{V_v}{V_s}$

$\Rightarrow V_v = e$

$\therefore$ Total volume, $V = V_v + V_s$
$= e+1$

weight of soil solids,
$W_s = V_s G_s \gamma_w$
$= G_s \gamma_w \; [V_s = 1]$

Water content, $w = \frac{W_w}{W_s} \Rightarrow W_w = wW_s = wG_s\gamma_w$.

Unit weight, $\gamma = \frac{W}{V} = \frac{W_w + W_s}{1+e} = \frac{wG_s\gamma_w + G_s\gamma_w}{1+e}$

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

$$\Rightarrow \frac{\gamma}{1+w} = \frac{G_s\gamma_w}{1+e} \Rightarrow \boxed{\gamma_d = \frac{G_s\gamma_w}{1+e}} \checkmark$$

$$\therefore \boxed{e = \frac{G_s\gamma_w}{\gamma_d} - 1} \checkmark$$

* prove that, For completely saturated soil, $e = wG_s$.

Proof: We know, $\gamma_w = \frac{W_w}{V_w}$.

$\Rightarrow V_w = \frac{W_w}{\gamma_w} \Rightarrow V_w = \frac{wG_s\gamma_w}{\gamma_w}$ $\left[w = \frac{W_w}{W_s} \Rightarrow W_w = wG_s\gamma_w\right]$

$\Rightarrow V_w = wG_s$

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

$\Rightarrow S = \frac{wG_s}{e}$ $\left[\text{when } V_s = 1,\ e = \frac{V_v}{V_s} \Rightarrow V_v = e\right]$

$\Rightarrow Se = wG_s$

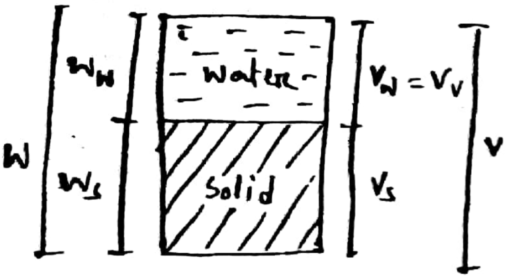
For completely saturated soil, $S = 1$.

$\therefore e = wG_s$

(proved).

* Relationship among saturated unit wt, void ratio, specific gravity or prove that, $\gamma_{sat} = \frac{(G_s+e)\gamma_w}{1+e}$

proof: For fully saturated soil,



Assume, $V_s = 1$ $\therefore V_v = e$

$\therefore$ Total volume, $V = 1+e$

Again, $V_W = \frac{W_W}{\gamma_W}$

$\Rightarrow e = \frac{W_W}{\gamma_W}$ $[V_v = V_W = e]$

$\Rightarrow W_W = e\gamma_W$

Also, $V_s = \frac{W_s}{\gamma_s}$ $\Rightarrow W_s = V_s\gamma_s \Rightarrow W_s = V_s G_s \gamma_W$

$\Rightarrow W_s = G_s\gamma_W$ $[V_s = 1]$

$$\therefore \gamma_{sat} = \frac{W}{V} = \frac{W_W + W_s}{1+e} = \frac{e\gamma_w + G_s\gamma_W}{1+e}$$

$$\therefore \boxed{\gamma_{sat} = \frac{(G_s+e)\gamma_W}{1+e}}$$

* Submerged Unit wt or Buoyant unit wt: Submerged unit weight is the weight of soil solids in air minus the weight of water displaced by soil solids per unit vol^m of soil. It is denoted by γ'. $\gamma' = \frac{G_s\gamma_W - \gamma_W}{1+e}$

* prove that, $\gamma' = \gamma_{sat} - \gamma_W$

proof: $\gamma_{sat} = \frac{(G_s+e)\gamma_W}{1+e}$

$$\therefore \gamma_{sat} - \gamma_W = \frac{G_s\gamma_W + \cancel{e\gamma_W} - \gamma_W - \cancel{e\gamma_W}}{1+e} = \frac{G_s\gamma_W - \gamma_W}{1+e} = \gamma'$$

$\therefore \gamma_{sat} - \gamma_W = \gamma'$ (proved)

* **Relative Density:** The term relative density is commonly used to indicate the in situ denseness or looseness of granular soil. Denoted by Dr.

$$D_r = \frac{e_{max} - e}{e_{max} - e_{min}} \quad , \quad D_r \rightarrow 1 \sim 100\%.$$

we know,

$$e = \frac{G_s \gamma_w}{\gamma_d} - 1 \quad , \quad e_{max} = \frac{G_s \gamma_w}{\gamma_{d\,min}} - 1$$

$$e_{min} = \frac{G_s \gamma_w}{\gamma_{d\,max}} - 1 .$$

$$\therefore D_r = \left[\frac{\gamma_d - \gamma_{d\,min}}{\gamma_{d\,max} - \gamma_{d\,min}}\right]\left[\frac{\gamma_{d\,max}}{\gamma_d}\right]$$

* For a soil, $e = 0.75$, $w = 22\%$, $G_s = 2.66$. calculate – n, γ, γ_d, S

soln:

$$n = \frac{e}{1+e} = \frac{0.75}{1+0.75} = 0 \text{ [illegible] } 8$$

$$\gamma = \frac{G_s \gamma_w (1+w)}{1+e} = \frac{2.61 * 62.4 * (1+.22)}{1+.75} = 115.71 \text{ lb/ft}^3$$

$$\gamma_d = \frac{G_s \gamma_w}{1+e} = \frac{2.66 * 62.4}{1+.75} = 99.85 \text{ lb/ft}^3$$

$$S = \frac{w G_s}{e} = \frac{.22 * 2.66}{.75} * 100 = 78.03\%$$

Ans

2> $w = \frac{W_W}{W_S}$ => $0.11 = \frac{W_W}{W_S}$ => $W_W = 0.11 W_S$

We get, $W = W_W + W_s + \cancel{W_a}^{0}$

=> $23 = 0.11 W_s + W_s$ => $W_s = 20.72$ lb.

$\therefore W_W = 23 - 20.72 = 2.28$ lb

Again, $W_s = V_s G_s \gamma_w$

=> $V_s = \frac{W_s}{G_s \gamma_w} = \frac{20.72}{2.7 * 62.4} = 0.123$ ft^3.

$\therefore$ vol^m of water, $V_W = \frac{W_W}{\gamma_W} = \frac{2.28}{62.4} = 0.0365$ ft^3.

$\therefore$ vol^m of air, $V_a = 0.2 - 0.123 - 0.0365$

$= 0.0405$ ft^3.

$\therefore \gamma_d = \frac{W_s}{V} = \frac{20.72}{0.2} = 103.6$ lb/ft^3

Dry unit wt, $\gamma_d = \frac{\gamma}{1+w} = \frac{115}{1+.11} = 103.6$ lb/ft^3

void ratio, $e = \frac{G_s \gamma_w}{\gamma_d} - 1 = \frac{2.7 * 62.4}{103.6} - 1$

$= 0.626$

4) porosity, $n = \frac{e}{1+e} = \frac{0.626}{1+0.626} = 0.385 = 38.5\%$

5) Degree of saturation, $Se = wG_s$

$$\Rightarrow S = \frac{0.11 * 2.7}{0.626} = 0.474 = 47.4\%$$

Ans

* A moist soil has a volume of 0.33 ft^3 and weighs 39.93 lb. The oven-dried weight of the soil is 34.59 lb. If $G_s = 2.67$, calculate -

1) Moisture content. 2) Moist Unit wt 3) Dry unit wt 4) void ratio 5) porosity 6) Degree of saturation.

Soln:

1) Moisture content, $w = \frac{W_w}{W_s} * 100 = \frac{39.93 - 34.59}{34.54} * 100$

$= 15.6\%$

GTEL-36

(bulk)

2) Moist unit wt, $\gamma = \frac{W}{V} = \frac{39.93}{0.33} = 121 \text{ lb}/ft^3$

3) Dry unit wt, $\gamma_d = \frac{W_s}{V} = \frac{34.59}{0.33} = 104.7 \text{ lb}/ft^3$

4) void ratio, $e = \frac{G_s \gamma_w}{\gamma_d} - 1 = \frac{2.67 * 62.4}{104.7} - 1$

$= 0.591$

5) porosity, $n = \frac{e}{1+e} * 100 = 37.1\%$

6) Degree of saturation, $S = \frac{w G_s}{e} * 100$

$= \frac{0.152 * 2.67}{0.591} * 100$

$= 70.48\%$

Ans

Scanned by CamScanner

* The dry density of a sand with a porosity of 0.387 is 1600 kg/m^3. Find, the void ratio of the soil and the specific gravity of the soil solids.

Sol^n: Here, $n = 0.387$, $\gamma_d = 1600$ kg/m^3

$\therefore$ void ratio, $e = \frac{n}{1-n} = \frac{0.387}{1-0.387} = 0.631$ Ans.

Again, $\gamma_d = \frac{G_s \gamma_w}{1+e} \Rightarrow G_s = 2.61$ Ans.

or, Assume, $V = 1$ m3

$$\gamma_d = \frac{W_s}{V} \Rightarrow W_s = \gamma_d * V$$
$$= 1600 * 1 = 1600 \text{ kg}$$

Again,
$$n = \frac{V_v}{V} \Rightarrow 0.387 = \frac{V_v}{1}$$
$$\Rightarrow V_v = 0.387 \text{ m}^3$$

$\therefore V_s = 1 - 0.387 = 0.613$ m^3.

we know,
$$W_s = V_s G_s \gamma_W$$
$$\Rightarrow G_s = \frac{W_s}{V_s \gamma_w} = \frac{1600}{0.613 * 1000}$$
$$\Rightarrow G_s = 2.62$$

Ans

* A dry soil unit weight 112 lb/ft^3 when specific gravity 2.7. Then compute saturated unit weight of the soil?

Sol^n:
$$\gamma_d = \frac{G_s \gamma_w}{1+e}$$
$$\Rightarrow 112 = \frac{2.7 * 62.4}{1+e}$$
$$\Rightarrow e = 0.504$$

Again,
$$\gamma_{sat} = \frac{(G_s + e)\gamma_w}{1+e}$$
$$= \frac{(2.7 + 0.504) * 62.4}{1 + 0.504}$$
$$= 132.71 \text{ lb/ft}^3$$

Ans

* The moist unit weight of a soil is 16.5 kN/m^3. Given that, $w = 15\%$ and $G_s = 2.7$. Determine –

a) Dry unit weight b) porosity c) Degree of saturation

d) Mass of water, in kg/m^3, to be added to reach full saturation.

Soln: Here, $\gamma = 16.5\ kN/m^3$, $w = 0.15$, $G_s = 2.7$

a) $\gamma_d = \dfrac{\gamma}{1+w} = \dfrac{16.5}{1+0.15} = 14.35\ kN/m^3$

b) $\gamma_d = \dfrac{G_s \gamma_w}{1+e} \Rightarrow e = \dfrac{G_s \gamma_w}{\gamma_d} - 1 \Rightarrow e = 0.846$

$\therefore n = \dfrac{e}{1+e} = 0.458$

c) $Se = wG_s \Rightarrow S = \dfrac{wG_s}{e} \Rightarrow S = \dfrac{0.15 * 2.7}{0.846}$

$= 47.9\%$

d) $\gamma_{sat} = \dfrac{(G_s+e)\gamma_w}{1+e} = \dfrac{(2.7+0.846)*9.81}{1+0.846} = 18.84\ kN/m^3$

Mass of water (kg/m^3) to be added,

$= \dfrac{(\gamma_{sat} - \gamma)*1000}{9.81}$ [KN-কে নিউটন, আর N-কে 9.81 দ্বারা ভাগ করে kg.]

$= 238.5\ kg/m^3$

Ans

* Find void ratio, dry density, unit weight of sand if $S = 0.46$, ~~w = 14%~~. Also find degree of saturation and Submerged unit wt of sand if $w = 14\%$, $n = 30\%$ and $G_s = 2.7$?

Sol^n^: • void ratio, $e = \frac{n}{1-n} = \frac{0.3}{1-0.3} = 0.429$

• Dry density, $\gamma_d = \frac{G_s \gamma_w}{1+e} = \frac{2.7 * 62.4}{1+0.429} = 117.9 \text{ lb/ft}^3$

• Unit wt of sand, $\gamma_s = \frac{(G_s + \underline{S}e)\gamma_w}{1+e}$

$$= \frac{(2.7 + 0.49 * 0.429) * 62.4}{1+0.429}$$

$$= 127.1 \text{ lb/ft}^3$$

• Degree of saturation, $S = \frac{wG_s}{e}$

$$= \frac{0.14 * 2.7}{0.429} = 0.881$$

* Submerged unit wt, $\gamma' = \frac{G_s\gamma_w - \gamma_w}{1+e}$

$$= \frac{2.7 * 62.4 - 62.4}{1+0.429} = 74.23 \text{ lb/ft}^3$$

Ans

* The dry unit weight of a soil having 15% moisture content is 17.5 KN/m³. Find the Bulk unit weight, saturated unit weight & submerged unit weight. $G_s = 2.7$

sol^n: Bulk unit weight, γ $\left[\gamma_d = \frac{\gamma}{1+w}\right]$

$$\gamma = \gamma_d(1+w)$$
$$= 17.5(1+.15) = 20.13 \text{ KN/m}^3.$$

Again, $\gamma_d = \frac{G_s\gamma_w}{1+e} \Rightarrow e = \frac{G_s\gamma_w}{\gamma_d} - 1$

$$\Rightarrow e = \frac{2.7 * 9.8}{17.5} - 1 = 0.512$$

$\therefore$ Saturated unit wt, $\gamma_{sat} = \frac{(G_s+e)\gamma_w}{1+e}$

$$= \frac{(2.7+.512)*9.8}{1+.512} = 20.82 \text{ KN/m}^3$$

$\therefore$ submerged unit weight, $\gamma' = \gamma_{sat} - \gamma_w$

$$= 20.82 - 9.8$$
$$= 11.02 \text{ KN/m}^3$$

Ans

* A 100% saturated soil has a wet unit weight of 120 lb/ft³ and water content of 36%. Determine void ratio & specific gravity.

sol^n: $Se = wG_s \Rightarrow e = 0.36\,G_s$ —— (i) $[S=1]$

$$\gamma_{sat} = \frac{(G_s+e)\gamma_w}{1+e} \Rightarrow 120 = \frac{(G_s+0.36G_s)*62.4}{1+0.36G_s} \quad [\text{From (i)}]$$

$\Rightarrow G_s = 2.87$ Ans.

$\therefore$ void ratio, $e = 0.36 * 2.87 = 1.03$

Ans

* The dry mass of a sample of aggregate is 1982 gm. The mass is a Saturated surface dry condition is 2006.7 gm. The net volume of aggregate is 734.4 cm^3. Find the apparent specific gravity, Bulk specific gravity and the percentage absorption?

Soln: Here, volume of water, $V_w = \frac{2006.7 - 1982}{1}$ $\left[v = \frac{M}{\rho} \right]$

$= 24.7\ cm^3$.

Net volume, $V_D = 734.4\ cm^3$

Bulk volume, $V_B = V_D + V_W$

$= 734.4 + 24.7 = 759.1\ cm^3$.

∴ Apparent specific gravity, $G_A = \frac{W_D}{V_D} = \frac{1982}{734.4}$

$= 2.699$ Ans

∴ Bulk specific gravity, $G_B = \frac{W_D}{V_B} = \frac{1982}{759.1}$

$= 2.61$ Ans

∴ % of absorption $= \frac{2006.7 - 1982}{1982} \times 100$

$= 1.25\%$ Ans

* For a given Sandy soil, $e_{max} = 0.75$, $e_{min} = 0.46$ and $G_s = 2.68$. What is the moist unit wt of compaction in the field if $D_r = 78\%$ & $w = 9\%$?

Soln:

$D_r = \frac{e_{max} - e}{e_{max} - e_{min}} \Rightarrow e = e_{max} - D_r (e_{max} - e_{min})$

$= 0.75 - 0.78 (0.75 - 0.46) = 0.529$.

∴ Moist unit wt, $\gamma = \frac{G_s \gamma_w (1 + w)}{1 + e}$

$= \frac{2.68 \times 9.81 \times (1 + .09)}{1 + .529} = 18.8\ KN/m^3$ Ans

* A loose uncompacted sand fill 6 ft in depth has a relative density (Dr) of 40%. Laboratory tests indicated that -

$e_{max} = 0.90$, $e_{min} = 0.46$, $G_s = 2.65$

a) What is the dry unit weight of the sand?

b) If the sand is compacted to a relative density of 75%, what is the decrease in thickness of the 6ft fill.

Soln:

Let,
For Uncompacted – D_{r_1}, e_1, γ_d,
compacted – $D_{r_2}, e_2, \gamma_{d_2}$

a) $D_{r_1} = \frac{e_{max}-e_1}{e_{max}-e_{min}}$

$\Rightarrow e_1 = e_{max} - D_{r_1}(e_{max}-e_{min}) = 0.90-0.4(0.90-0.46)$

$\therefore e_1 = 0.724$.

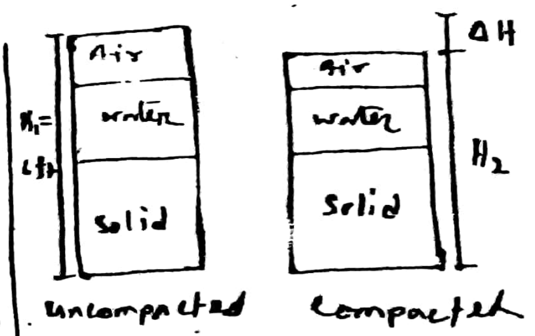
$\therefore$ Dry unit wt, $\gamma_{d_1} = \frac{\gamma_w}{1+e_1} = \frac{2.65*62.4}{1+.724} = 95.9 \, lb/ft^3$

Ans

b) $D_{r_2} = \frac{e_{max}-e_2}{e_{max}-e_{min}}$

$\Rightarrow e_2 = 0.9-0.75(0.9-0.46)$
$= 0.57$

$\therefore \gamma_{d_2} = \frac{2.65*62.4}{1+.57} = 105.32 \, lb/ft^3$



$AV = AH_1$ $\quad$ $V = AH_2$

A → Same for both.

$\therefore \gamma_{d_1} = \frac{W_s}{V_1} \Rightarrow W_s = \gamma_{d_1} A H_1$

$\gamma_{d_2} = \frac{W_s}{V_2} \Rightarrow W_s = \gamma_{d_2} A H_2$

$\therefore \gamma_{d_1} A H_1 = \gamma_{d_2} A H_2$

$\Rightarrow H_2 = \frac{\gamma_{d_1} H_1}{\gamma_{d_2}}$

$= \frac{95.9*6}{105.32} = 5.463 \, ft$

$\therefore \Delta H = 6 - 5.463$
$= 0.537 \, ft.$
$= 6.44 \, in$

Ans

$W = 15\%$, Unit weight $120\ lb/ft^3$, $e_{min} = 0.50$, $e_{max} = 0.85$ for densest and loosest state. compute S, relative density (D_r / I_d) when specific gravity 2.65.

Sol^n: Let, $V = 1\ ft^3$

$$\gamma = \frac{W}{V} \Rightarrow \gamma = \frac{W_W + W_S}{1} \Rightarrow \gamma = W_W + W_S \quad \text{---} ①$$

$$w = \frac{W_W}{W_S} \Rightarrow W_W = 0.15 W_S$$

$$\text{from} ① \Rightarrow 120 = 0.15 W_S + W_S \Rightarrow W_S = 104.3\ lb.$$

$$\therefore W_W = 0.15 \times 104.3 = 15.7\ lb.$$

$$\therefore V_s = \frac{W_s}{\gamma_s} \Rightarrow V_s = \frac{W_s}{G_s \gamma_W} \Rightarrow V_s = \frac{104.3}{2.65 \times 62.4} = 0.63\ ft^3$$

$$V_W = \frac{W_W}{\gamma_W} \Rightarrow V_W = \frac{15.7}{62.4} = 0.25\ ft^3$$

$$V_a = 1 - (0.63 + 0.25) = 0.12\ ft^3$$

$$\therefore S = \frac{V_W}{V_v} = \frac{0.25}{0.25 + 0.12} = 0.6757 = 67.57\%$$

Ans

$$D_r = \frac{e_{max} - e}{e_{max} - e_{min}} = \frac{.85 - .59}{.85 - .50} = 0.74$$

Ans

$$e = \frac{V_v}{V_s} = \frac{.25 + .12}{.63} = 0.59$$

OR

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

$$120 = \frac{2.65 \times 62.4 (1 + .15)}{1 + e}$$

$$e = 0.5897$$

$$S = \frac{wG_s}{e} = .6798 = 67.98\%$$

$$D_r = \frac{e_{max} - e}{e_{max} - e_{min}} \times 100 = 75.8\%$$

Ans.

Unit weight $= 131\ lb/ft^3$, $w = 14\%$, $G_s = 2.67$. Find γ_d, γ_d at zero air voids, γ_{sat} if the voids are filled with water.

Sol^n:

$$w = \frac{W_w}{W_s} \Rightarrow 0.14 = \frac{W_w}{W_s} \Rightarrow W_w = .14 W_s$$

Let, volume, $V = 1\ ft^3$

$$\gamma = \frac{W}{V} \Rightarrow 131 = \frac{W_s + W_w}{1} \Rightarrow W_s = 115\ lb$$

$$\therefore W_W = 16\ lb.$$

$\therefore \gamma_d = \frac{W_s}{V} = \frac{115}{1} = 115 \text{ lb/ft}^3 \quad \therefore \cancel{\gamma_d = \frac{\gamma_{bulk}}{1+w}} = \cancel{\frac{115}{1+.14} = 100.88 \text{ lb/ft}^3}$

$V_w = \frac{W_w}{\gamma_w} = \frac{16}{62.4} = 0.26 \text{ ft}^3$

$V_s = \frac{W_s}{\gamma_s} = \frac{W_s}{G_s \gamma_w} = \frac{115}{2.67 \times 62.4} = 0.69 \text{ ft}^3$

$V_a = 1 - (V_w + V_s) = 0.06 \text{ ft}^3.$

$V_v = 1 - V_s = 0.31 \text{ ft}^3$

$\therefore \gamma_d$ at zero air void $= \frac{W_s}{V_s + V_w} = \frac{115}{0.69 + .26}$

$= 121.1 \text{ lb/ft}^3$

γ_{sat} - (voids are filled with water ~~[illegible]~~ (no air))

$= \frac{115 + V_v \gamma_w}{V} = \frac{115 + .31 \times 62.4}{1} = 134.4 \text{ lb/ft}^3$

Ans

A 27.50 lb soil sample has a volume of 0.320 ft³. Moisture content of 15.20% and specific gravity of soil solids 2.67. Compute bulk density, dry der degree of saturation, void ratio.

Soln -

$\gamma_{bulk} = \frac{W}{V} = \frac{27.50}{.320} = 85.94 \text{ lb/ft}^3.$

$\gamma_d = \frac{\gamma_{bulk}}{1+w} = \frac{85.94}{1 + .152} = 74.6 \text{ lb/ft}^3$

Again, $\gamma_d = \frac{G_s \gamma_w}{1+e} \Rightarrow e = \frac{G_s \gamma_w}{\gamma_d} - 1$

$= \frac{2.67 \times 62.4}{74.6} - 1 = 1.24$

$\therefore S = \frac{w G_s}{e} = \frac{.152 \times 2.67}{1.24} = 32.73\%$

Ans

* Determine the i) water content ii) Dry density iii) Bulk density iv) void ratio v) Degree of saturation. Given sample dia 3.81cm, sample height = 7.62cm, wet wt = 166.8 gm, oven dry wt = 140 gm, Sp. Gr. = 2.70.

Soln:

BUET

① $w = \frac{166.8-140}{140} \times 100 = 19.14\%$

② $\rho_d = \frac{140}{\frac{\pi}{4}(3.81)^2 \times 7.62} = 1.61 \text{ gm/cm}^3$

③ $\rho = \frac{166.8}{\frac{\pi}{4}(3.81)^2 \times 7.62} = 1.92 \text{ gm/cm}^3$

(IV) $e = \frac{G_s \gamma_w}{\rho_d} - 1 = \frac{2.7 \times 1}{1.61} - 1 = 0.68$

(V) $S = \frac{G_s w}{e} = \frac{2.70 \times .1914}{.68} = .76 = 76\%$

Ans.

* The difference between max^m & min^m void ratios is 0.3 & field void ratio is 0.4. If relative density is 66.6%, find out the density (saturated) at its loosest condition of sand.

Soln: Assume, $G_s = 2.7$

Buet

BPDB-16

$D_r = \frac{e_{max} - e_f}{e_{max} - e_{min}} = \frac{e_{max} - 0.4}{0.3}$

$\Rightarrow 66.6/100 = \frac{e_{max} - 0.4}{0.3} \Rightarrow e_{max} = 0.60$

$\therefore \gamma_{sat} = \left(\frac{G_s + e}{1+e}\right) * \gamma_w = \frac{2.7 + 0.6}{1+0.6} \times 9.81$

$= 20.23 \text{ KN/m}^3$

Ans.

* In a field hole is cut off volume 1.1 ft³. The wet mass of the hole is 130 lb and dry mass is 119 lb. Determine the degree of saturation if sp gravity is 2.7.

B-16 Soln:

$w = \frac{130 - 119}{119} \times 100 = 9.24\%$

$\gamma_d = \frac{119}{1.1} = 108.18 \text{ lb/ft}^3$

$e = \frac{G_s \gamma_w}{\gamma_d} - 1 = \frac{2.7 \times 62.4}{108.18} - 1 = 0.56$

$S = \frac{w G_s}{e}$

$= \frac{.0924 \times 2.7}{.56}$

$= 44.55\%$ Ans.

"Compaction"

* Define compaction. Write down the basic difference between compaction & consolidation.

Ans:

Compaction: Compaction is a kind of densification that is realized by rearrangement of soil particle without outflow of water. It is realized by the application of mechanical energy. It is not involved in fluid flow but with moisture changing.

Basic Difference:

- In case of compaction, only air is to be removed i.e. volume reduction due to expulsion of air but no out flow of water is occured. Dynamic Load is applied.
- In case of consolidation, both air & water is to be removed i.e. volume reduction due to expulsion of air & water and so [illegible] flow of water is occured. static Load is applied.

* **Objectives of Compaction:**

- increases the strength of soil which increases the bearing capacity of foundations constructed over them.
- decreases the amount of undesirable settlement of structures.
- Increases the stability of slopes of embankment.
- To decrease the permeability and compressibility

* prove that, Modified Energy is 5 times greater than compaction energy or effort.

proof:

$$\text{Energy}, E = \frac{\left(\text{wt of hammer}\right) * \left(\text{Height of drop}\right) * \left(\text{No. of layer}\right) * \left(\text{No. of blows per layer}\right)}{\text{volume of mold}}$$

For standard proctor test,

BPDB-16

$$E_s = \frac{\frac{2.5 * 9.81}{1000} * \frac{305}{1000} * 3 * 25}{\frac{944}{1*10^6}}$$

$$= 594 \text{ kN-m/m}^3$$

For modified proctor test

$$E_M = \frac{\frac{4.59 * 9.81}{1000} * \frac{457}{1000} * 5 * 25}{\frac{944}{1*10^6}}$$

$$= 2695 \text{ kN-m/m}^3$$

$$\therefore \frac{E_M}{E_s} = \frac{2695}{594} \simeq 4.54 \simeq 5$$

$$\therefore E_M = 5\, E_s$$ (proved)

* Draw Compaction curve. Why dry density increases with increase of water content but after sudden times it falls.

Ans:

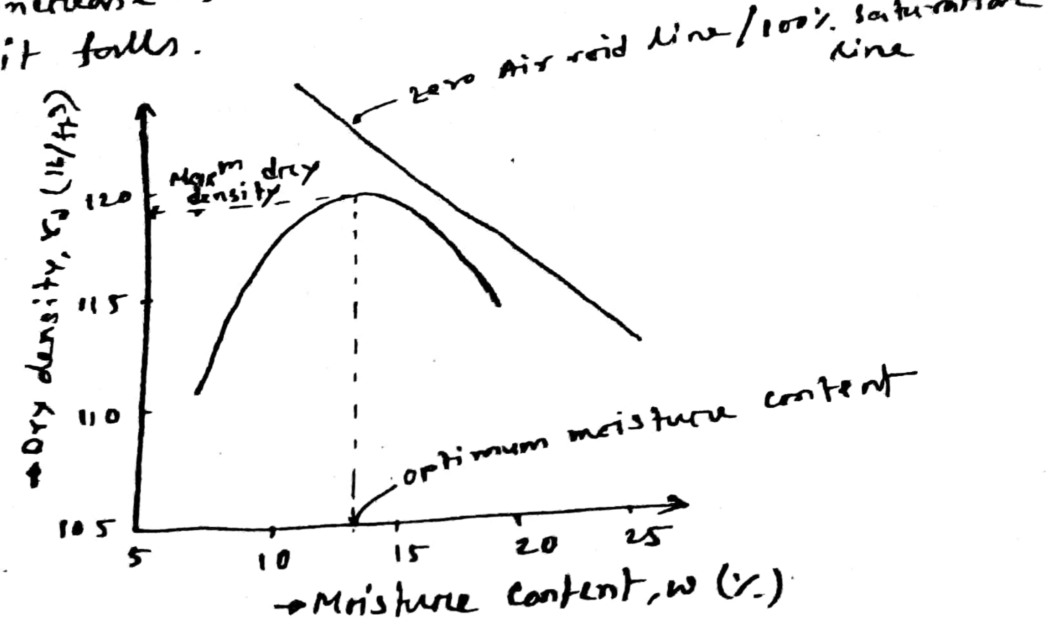


Fig: Compaction curve.

During compaction, water is added to the soil which acts as a lubricating agent on the soil particles. The soil particles slip on each other and move into densely packed positions.

For similar compacting efforts, the dry unit weight will increase with the increase of moisture content. However, beyond a certain point, additional moisture tends to reduce the dry unit wt because water takes up spaces that would have been occupied by the soil solids.

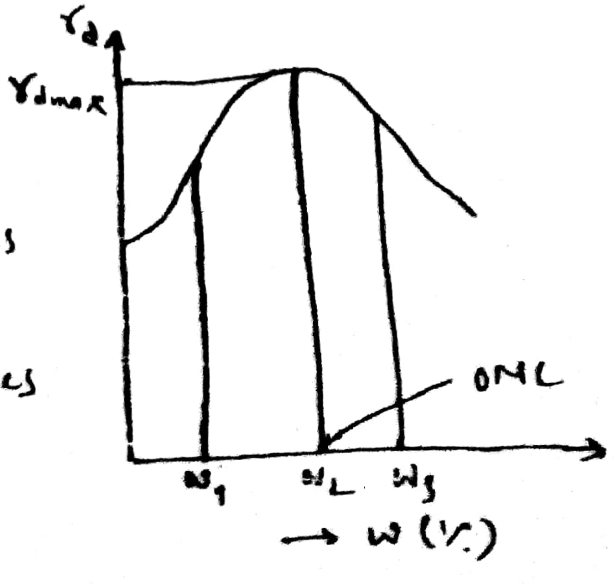
From Figure,

When w = 0, Dry unit wt γ_d

$w = w_1$, " " " increases

$w = w_2$, " " " maxm

$w = w_3$, " " " decreases



* $w = 18.5\%$, $\gamma_d = 100\ lb/ft^3$, $G_s = 2.65$. Find.

① Degree of saturation

② Maximum dry unit weight to which this soil can be compacted with 20% moisture content.

Soln:

① $\gamma_d = \frac{G_s \gamma_w}{1+e} \Rightarrow e = \frac{2.65 * 62.4}{100} - 1 = 0.654$

Again, $Se = wG_s$

$\Rightarrow S = \frac{0.185 * 2.65}{0.659} = 0.75 = 75\%$

② This is when $S = 100\%$.

$\therefore Se = wG_s \Rightarrow e = \frac{0.2 * 2.65}{1} = 0.53$

$\therefore \gamma_{d\,max} = \frac{G_s \gamma_w}{1+e} = \frac{2.65 * 62.4}{1+0.53} = 108.1\ lb/ft^3$

Ans

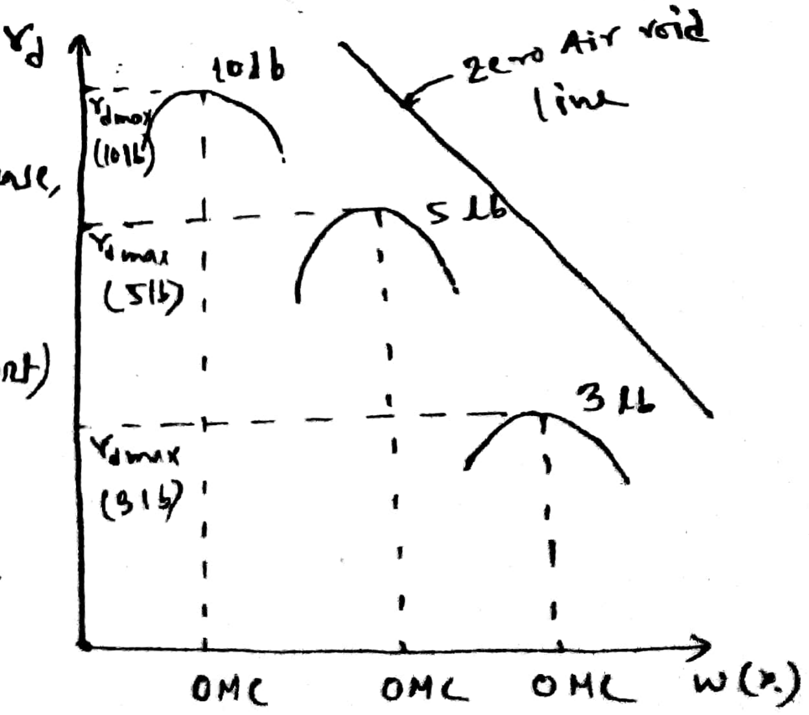
* The compaction curve moves upward left with the increase of energy - Explain.

or

By the way of neat sketches, explain the effect of compactive effort on $\gamma_d \rightarrow w(\%)$ relationship,

Ans:

- If the hammer weight or nos. of blows per layer increase, the energy per unit volume increases.
- If compaction energy (effort) increases, the max^m dry density increases but OMC decrease to some extent.



- The increase of maxm dry density moves the curve upward.
- The decrease in OMC moves the curve left.

* Factors on which compaction depends.

Ans: ① Moisture (water) content.

② Compaction effort.

③ The soil type.

* sketch the effect of soil type on the compaction characteristics of soil.

Ans:

* Factors affecting compaction in the field.

Ans: ① Soil type ② Moisture content ③ Types of compaction equipment (roller) ④ No. of roller passes.

* Type of compaction mold:

parameter	Mold-1	Mold-2
Dia of mold	4"	6"
Volm of mold	$\frac{1}{3}$ ft³	$\frac{1}{1333}$ ft³
No of blow per layer	25	56

* **Air void Line:** A line which shows the relationship between dry density and moisture content for a compacted soil containing constant percentage of air void.

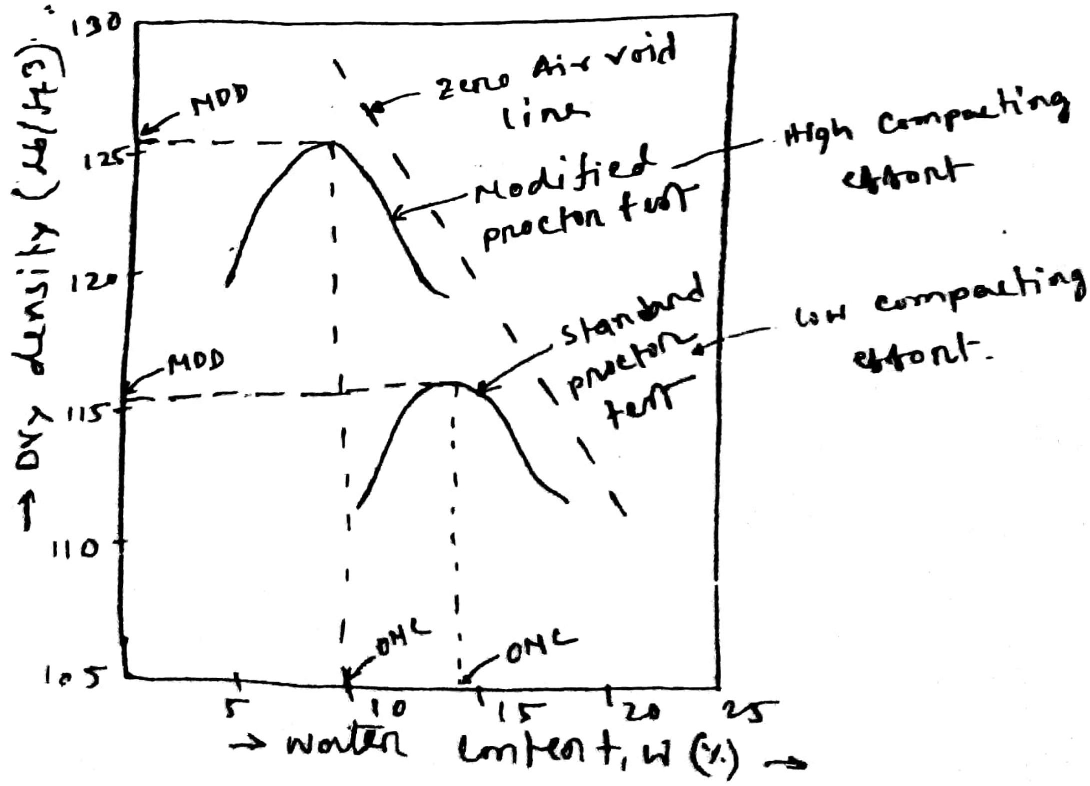
$$\gamma_d = \frac{(1-V_a)G_s\gamma_w}{1+wG_s}$$

* **Zero Air void Line:** A line which shows the relationship between dry density and moisture content for a compacted soil containing 0% air void or 100% saturation.

$$\gamma_d = \frac{G_s\gamma_w}{1+wG_s}$$

* Sketch typical moisture-density relationship for a cohesive soil under two different level of compaction effort and show the related features (OMC, MDD, Zero Air void Line). What is the use of standard proctor test?

Ans:



Use of standard proctor test: standard proctor test is used where lower loads are expected and lower dry unit weights are required.

* **Relative Compaction:** In most specification for earthwork, the contractor is instructed to achieve a compacted field dry unit wt of 90 to 95% of the maxm dry unit wt determined in the laboratory. This specification for field compaction is termed as relative compaction, R.

$$R = \frac{\gamma_d \text{ (field)}}{\gamma_d \text{ (max-lab)}} * 100 \quad ; \text{ expressed in } \%$$

* **Compaction Method:**

① pressure → contact pressure betn equipment & ground.

② Impact → series of blows to the soil

③ Vibration → Vibratory compaction equipment is used to vibrate the soil.

④ kneading / Manipulation → impart some shearing forces to soil mass.

* **Compaction Equipment:**

① Smooth wheel roller → pressure

② pneumatic rubber tired roller → pressure + kneading

③ Sheeps foot roller → P + K

④ vibratory roller → P+K+V

⑤ Cater pillar tamping foot → P+K+V+I

Roll: 1201032

'Classification of Soil"

means passin se retained.

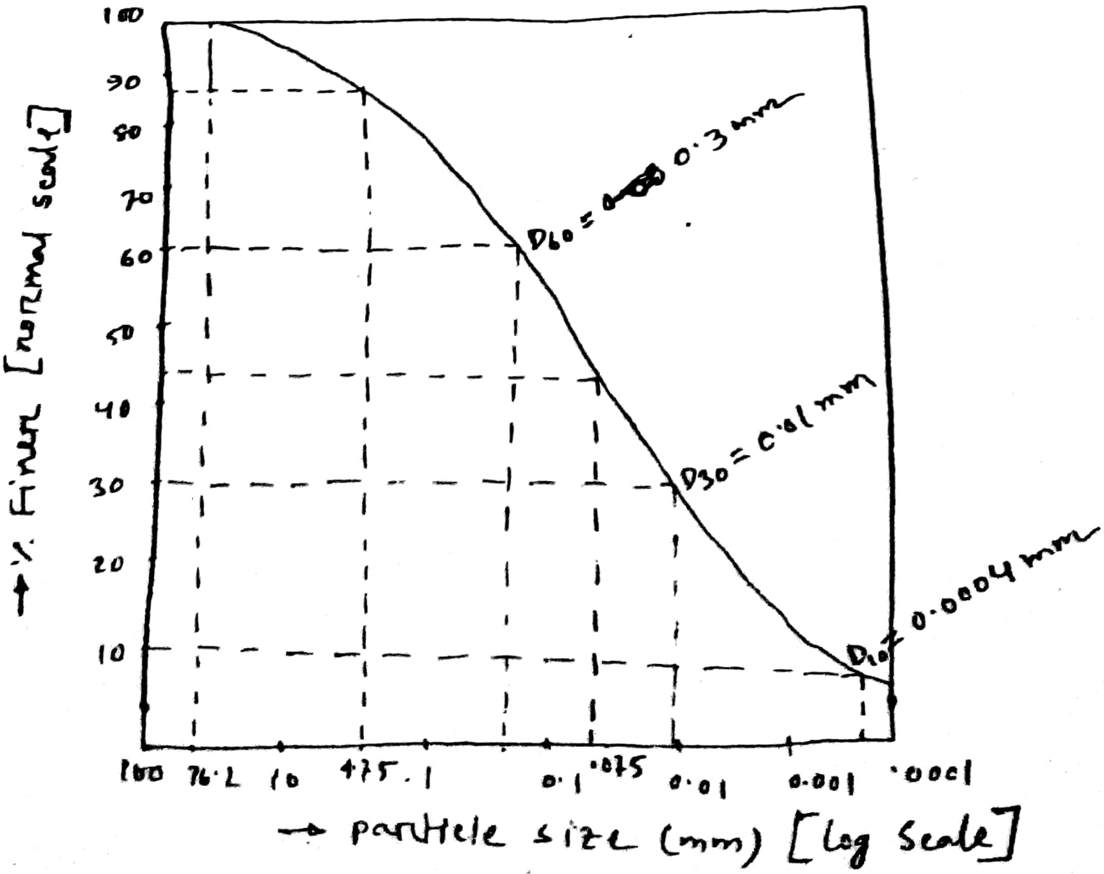
* Classify soil based on particle size.

Name of organization	Grain size (mm)			
	Gravel	Sand	Silt	Clay
MIT	> 2	2 to 0·06	0·06 to 0·002	< 0·002
U.S. Department of Agriculture (USDA)	> 2	2 to 0·05	0·05 to 0·002	< 0·002
AASHTO	76·2 to 2	2 to 0·075	0·075 to 0·002	< 0·002
Unified soil classification system (USCS), (ASTM)	76·2 to 4·75	4·75 to 0·075	< 0·075	< 0·075

Coarse (4·75 to 2·0) Medium (2 to 0·425) Fine (0·425 to 0·075)

* Draw particle size distribution curve. Find (%) of gravel, sand & fines, D_{10}, D_{30}, D_{60}, Cu, Cc for ASTM.

Ans:



Gravel(76.2 to 4.75mm):

$= 100 - 87 = 13\%$

Sand (4.75 to 0.075mm)

$= 87 - 44 = 43\%$

Fines or silt & clay (<0.075mm)

$= 44\%$

$D_{10} = 0.0004mm$, $D_{30} = 0.01mm$, $D_{60} = 0.30mm$

Shape parameter:

co-efficient of uniformity, $C_u = \frac{D_{60}}{D_{10}}$ $(7, 1)$

$= 30$

co-efficient of curvature/gradation, $C_c = \frac{D_{30}^2}{D_{10} * D_{60}}$

$= 0.83$

Ans

* Unified soil classification system (USCS) for Coarse-grained & fine-grained soil.

Ans:

For Fine Grained Soils

	First letter		Second letter
M	predominantly silt	L	Low plasticity.
C	predominantly clay	H	High plasticity.
O	organic		

uk-14

For Coarse grained soils:

	First letter		Second letter
S	predominantly sand	P	poorly graded
G	predominantly gravel	W	well graded
		M	silty
		C	clayey

Sand $\geq 50\%$ Coarse fraction passes #4 sieve	clean sand	SW
		SP
	Sand $> 12\%$ fines	SM
		SC
silt & clay Liquid limit < 50	inorganic	ML
		CL
	organic	OL
silt & clay Liquid limit ≥ 50	Inorganic	MH
		CH
	organic	OH

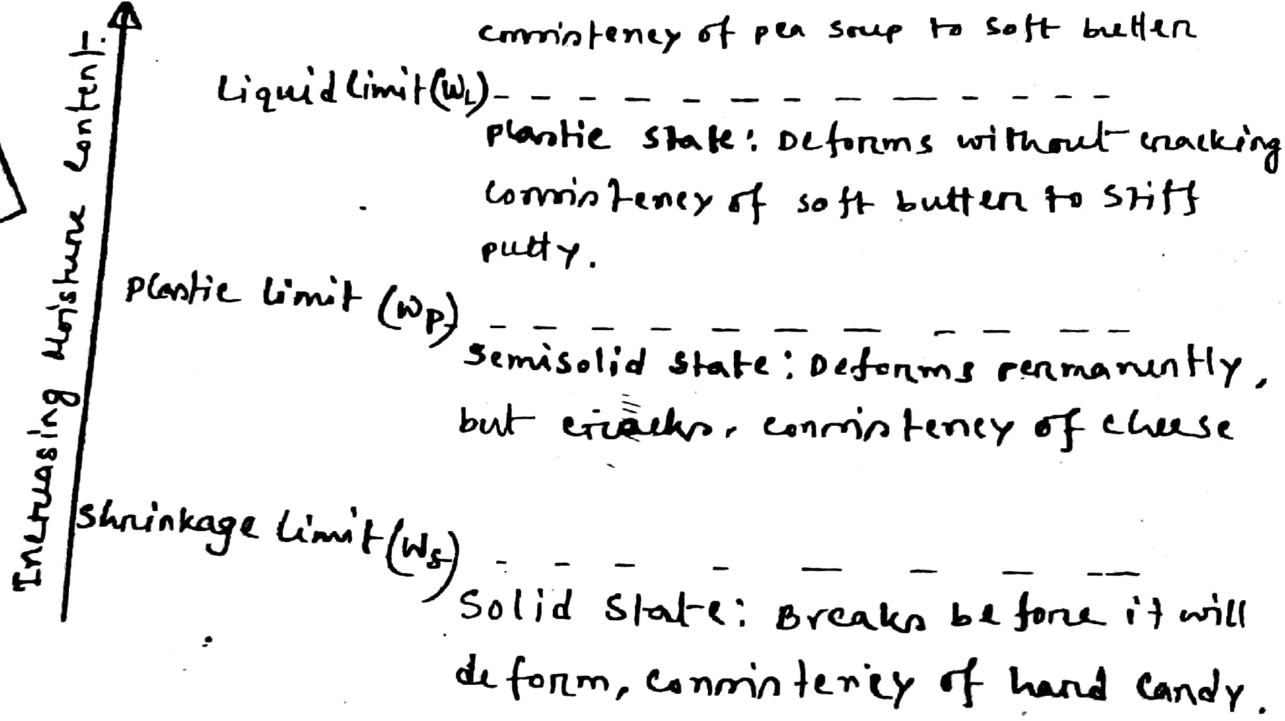
pea – মটর

"Consistency of soil"

* **Consistency:** consistency, in general, is that property of materials which is expressed by it's resistance to flow.

* Show with sketch, consistency of fine-grained soils at different moisture contents.

Ans:



Liquid state: Deforms easily, consistency of pea soup to soft butter

Liquid Limit (W_L)

Plastic state: Deforms without cracking, consistency of soft butter to stiff putty.

Plastic limit (W_P)

Semisolid state: Deforms permanently, but cracks, consistency of cheese

Shrinkage Limit (W_S)

Solid State: Breaks before it will deform, consistency of hard candy.

pea – মটর
utty – পুটি
eese – পনির

* What is consistency limit or Atterberg Limit?

Ans: The water content at which the soil changes from one state to another state is called consistency or Atterberg Limit. This limits are –

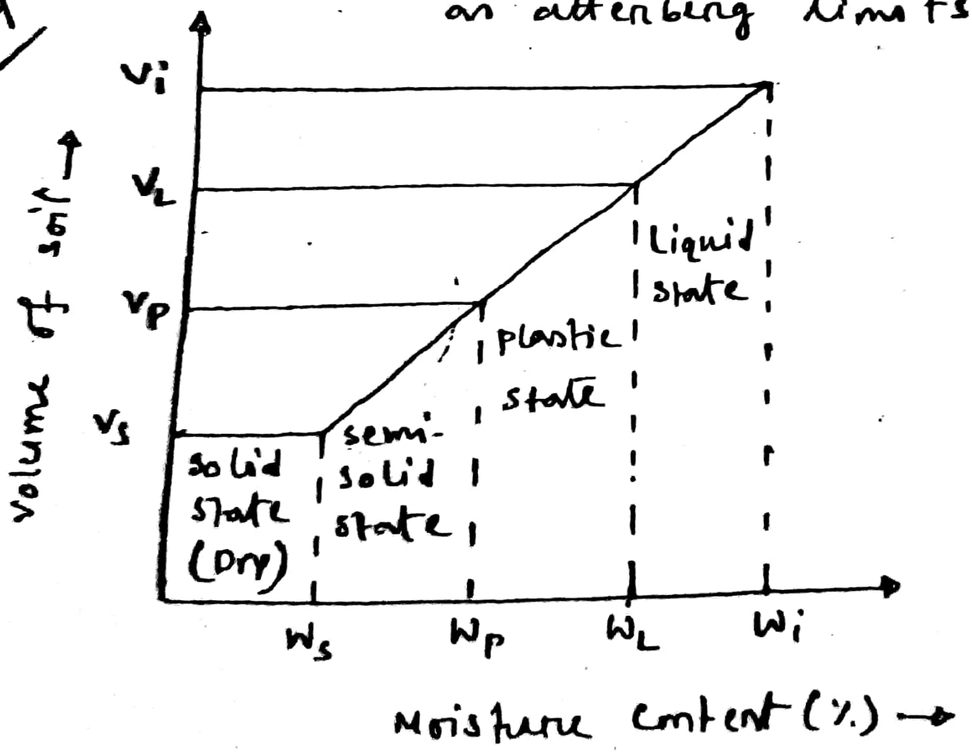
1) Liquid Limit (W_L)
2) plastic Limit (W_P)
3) shrinkage limit (W_S)

* **Liquid Limit:** The water content at which the soil changes from plastic state to liquid state.

Plastic limit: The water content at which the soil changes from semisolid state to plastic state.

Shrinkage limit: The maximum water content at which a reduction in water content will not cause a decrease in volume of the soil mass. - This ~~two~~ three limits are known as atterberg limits.

SA-14



* Draw the consistency limit / Atterberg limit.

OR

Draw a curve relation with W_L, W_P & W_S.

Ans: Figure above.

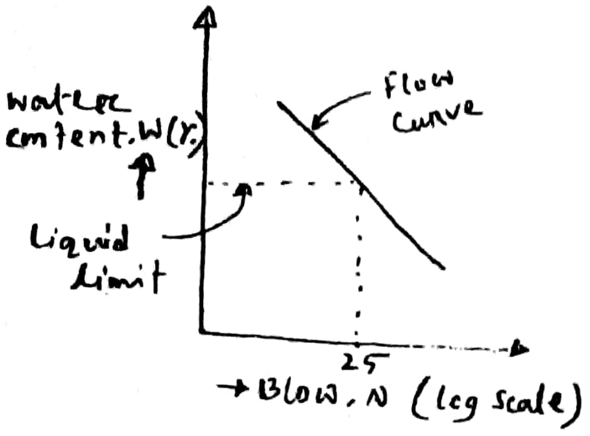
"Geotechnical Engineering lab - I"

* Exp-1: Determination of Liquid & plastic limit.

Liquid Limit:

① Cassagrande's Method:

- The soil is sieved using #40 sieve.
- Blow range 15 ~ 35
- A semilog graph is plotted to find flow index.

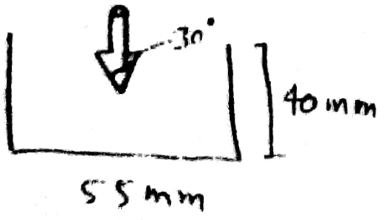


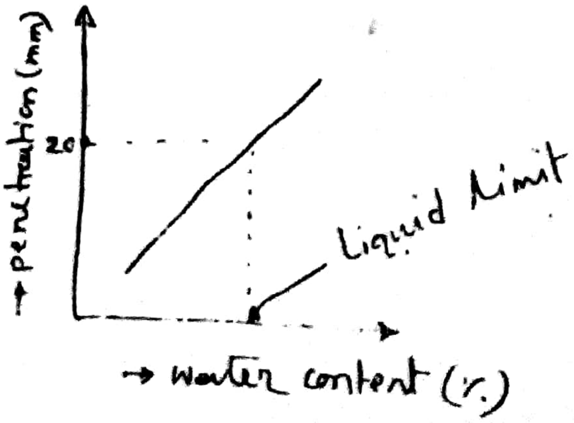
$$\text{Flow Index, } I_F = \frac{W_1 - W_2}{\log\left(\frac{N_1}{N_2}\right)}$$

Fig: Cassagrande grooving tool.

② Fallcone / cone-penetrometer method:

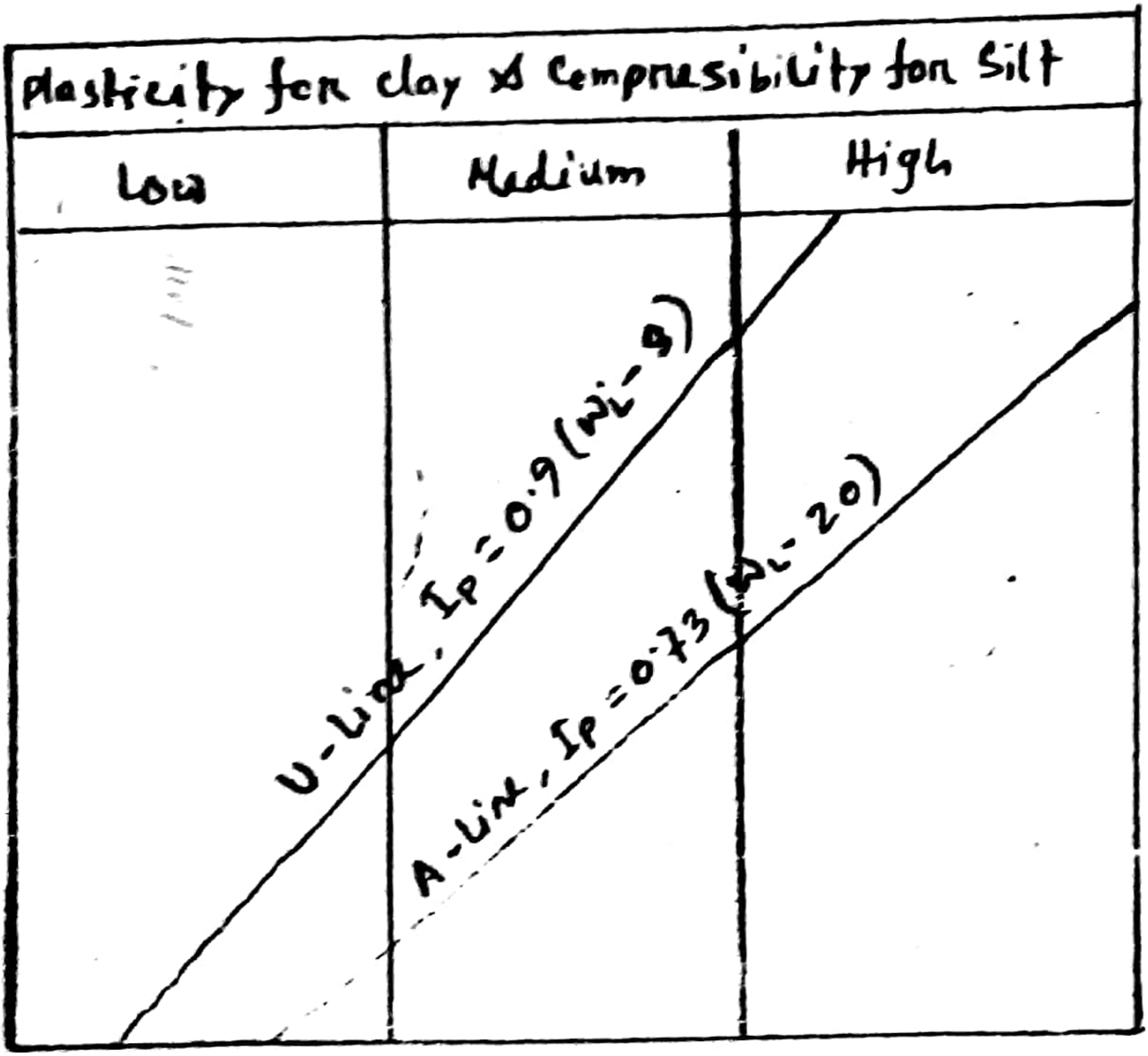
- Consists of a stainless steel cone 35 mm long with an apex angle of 30°.
- A cylindrical metal cup, 55 mm dia, 40 mm deep used to contain soil.
- The cone is released to penetrate the soil paste for exactly 5 sec.
- A graph is plotted to find water content for 20 mm penetration.





0 – 3	Non - plastic
3 – 15	slightly plastic
15 – 30	Medium plastic
> 30	High plastic .

ity Chart : (Developed by Cassagrande)



Scanned by CamScanner

In a liquid limit & plastic limit test, $w_L = 45$, $w_p = 23$, classify the soil.

Soln: $I_p = w_L - w_p = 45 - 23 = 22$

A line, $I_p = 0.73(w_L - 20) = 0.73(45 - 20) = 18.25$

$\therefore I_p = 22$ is above the A- line.

So, the soil is inorganic clay with medium plasticity.

Ans

In a Liquid limit test, the moisture content at 10 blows was 70% and that for 100 blows was 20%. What is the liquid limit of the soil?

Soln: For 10 blows $\rightarrow w_{L_1} = w_1 \left(\frac{N_1}{25}\right)^{.121} = 70 \times \left(\frac{10}{25}\right)^{.121} = 62.7\%$

For 100 blows $\rightarrow w_{L_2} = w_2 \left(\frac{N_2}{25}\right)^{.121} = 20 \times \left(\frac{100}{25}\right)^{.121} = 23.7\%$

$\therefore$ Required liquid limit $= 62.7 - 23.7 = 39\%$

Ans

 # Liquidity Index, I_L: represents the current state of soil.

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

Where

W_N = Moisture content of soil in its natural state.

$I_L = 0$ means the soil is currently at plastic state

$I_L = 1$ means the soil is currently at liquid state

$I_L < 0$ means the soil is at semisolid state.

 # Toughness Index:

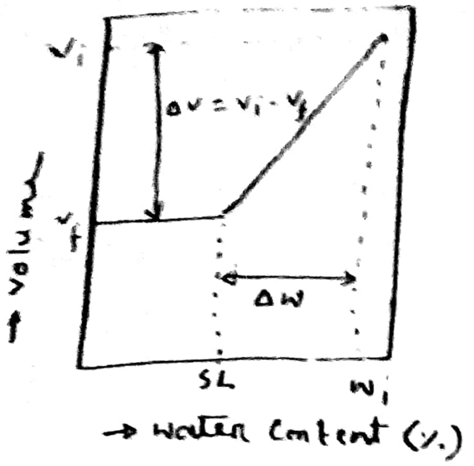
$$I_T = \frac{I_P \rightarrow \text{plasticity Index}}{I_F \rightarrow \text{Flow Index}}$$

*** Activity (A):** The plasticity index (I_P) of a soil increase linearly with the percentage of clay-size fraction present.

$$\text{Activity, } A = \frac{I_P}{\text{\% of clay size fraction by weight}}$$

Activity	Clay
<0.75	Inactive clay
0.75 to 1.25	Normal clay
>1.25	Active clay.

Determination of shrinkage limit :



Shrinkage limit

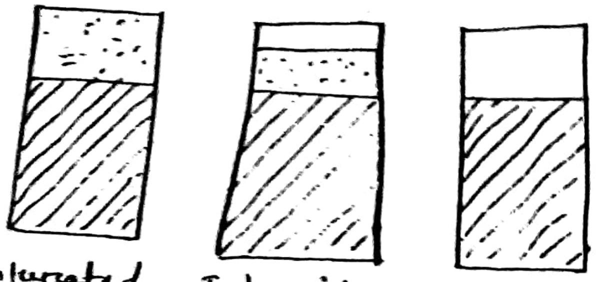
$$W_s = W_i - \Delta W$$

$$w_1 = \frac{M_1 - M_2}{M_2} \times 100 \quad \left[w = \frac{M_w}{M_s} \times 100 \right]$$

$$\Delta W = \frac{(V_i - V_f)\rho_w}{M_2} \times 100$$

* $M_2 = M_s$

* Phase Diagram of shrinkage limit :



Saturated State   Intermidiate State   Dry State

* Expression of Shrinkage limit with known specific gravity, $SL = \left(\frac{V_f \rho_w}{M_2} - \frac{1}{G_s} \right) \times 100$ → in percentage.

* Shrinkage ratio: It may be defined as the ratio of a given volume change, expressed as (%) of the dry volume to the corresponding change in water content above the shrinkage limit.

$$SR = \frac{\frac{\Delta V}{V_f}}{\frac{\Delta M}{M_2}} = \frac{\frac{\Delta V}{V_f}}{\frac{\Delta V \rho_w}{M_2}} = \frac{M_2}{V_f \rho_w}$$

* volumetric shrinkage, $V_s = SR(W_i - SL)$

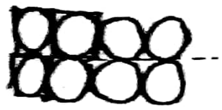
* Linear shrinkage, $L_s = 100\left[1-\left(\frac{100}{V_s+100}\right)^{1/3}\right]$

* Expression for finding G_s.

$$G_s = \frac{1}{\frac{1}{SR} - \frac{SL}{100}}$$

* prove that, in case of very loose state of packing and very dense state of packing, the void ratios are about 0·91 & 0.35 respectively when all the grains of soil mass assumed to be spheres of uniform size.

proof: **Loose condition:**



→ Volume of cube (total volᵐ)

$V = D^3$

→ volᵐ of sphere (soil solid only)

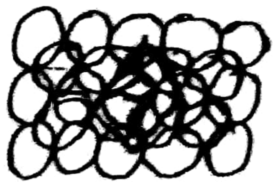
$V_s = \frac{4}{3}\pi\left(\frac{D}{2}\right)^3$

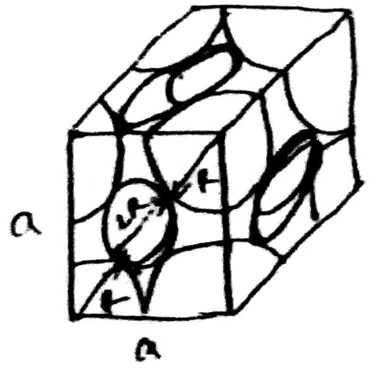
∴ volume of void, $V_v = V - V_s$

$= D^3 - \frac{4}{3}\pi\left(\frac{D}{2}\right)^3$

∴ void ratio, $e = \frac{V_v}{V_s} = 0{\cdot}91$

Dense condition:





∴ $a^2 + a^2 = (4R)^2$

$\Rightarrow a = 2\sqrt{2}R = \sqrt{2}D$

$$V = (\sqrt{2}\, D)^3$$

$$V_s = 6 \times \frac{1}{2} \times \frac{4}{3} \pi \left(\frac{D}{2}\right)^3 + 8 \times \frac{1}{8} \times \frac{4}{3} \pi \left(\frac{D}{2}\right)^3$$

$$= 4 \times \frac{4}{3} \pi \left(\frac{D}{2}\right)^3$$

$$V_v = V - V_s$$

$$= D^3\, 2^{3/2} - \frac{16}{3} \pi \frac{D^3}{8} = D^3 \left(2^{3/2} - \frac{2\pi}{3}\right)$$

$$\therefore e = \frac{V_v}{V_s} = 0.35$$

Ans

* A saturated soil with a volume of 19·65 m^3 has a mass of 36 gm. When the soil was dried, its volume and mass were 13·5 cm^3 & 25 gm respectively. Determine the shrinkage limit of the soil.

Solⁿ: Shrinkage Limit, $W_s = W_i - \Delta W$

$$W_i = \frac{M_1 - M_2}{M_2} * 100 = \frac{36 - 25}{25} * 100 = 44\%.$$

$$\Delta W = \frac{(V_i - V_f)\rho_w}{M_2} * 100 = \frac{(19.65 - 13.5) * 1}{25} * 100 = 24.6\%.$$

$$\therefore W_s \text{ or } SL = 44 - 24.6 = 19.4\%$$

Ans

1) Remove and replace the poor ground.

2) Compaction by surcharge load.

3) vibration of ground surface.

4) Dynamic Compaction of soil.

"Bearing Capacity"

April-14 The supporting power of a soil or rock is reffered to as the bearing capacity of the soil.

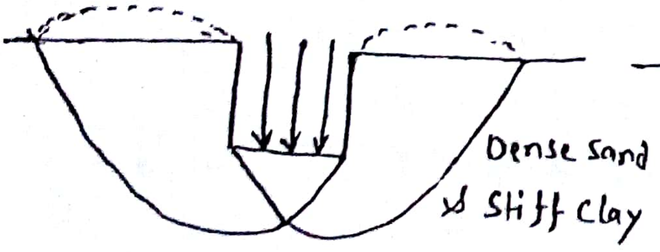
* **Ultimate bearing Capacity:** The ultimate bearing capacity is the minimum gross pressure intensity at the base of the foundation at which the soil fails in shear. (q_u)

possible modes of bearing capacity failure:

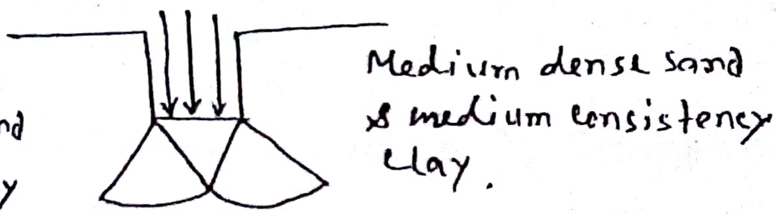
a) **General shear failure:**
- Heaving occurs on both sides.
- slight tilting of the footing.
- occur for low Compressible soil.

b) **Local shear failure:**
- Significant compression under the footing
- Slight heaving occurs.
- No tilting of the footing.
- High compressible soil.

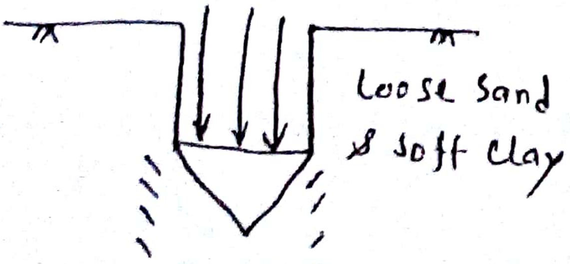
c) **punching shear failure:**
- Compression of soil under the footing.
- No heaving occurs.
- No tilting of the footing.
- low Compressible soil.



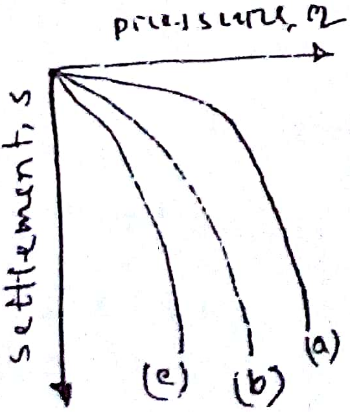
(a)



(b)



(c)



Net ultimate bearing capacity (q_{nf}): It is the minimum net pressure intensity causing shear failure of the soil.

$$q_{nf} = q_{ult} - q_o$$

$q_o \rightarrow$ overburden pressure $= \gamma D$

Net safe bearing capacity (q_{ns}): It is the ratio of net ultimate bearing capacity to the factor of safety.

$$q_{ns} = \frac{q_{nf}}{F.S}$$

Safe bearing Capacity (q_s): It is the maximum pressure which the soil can carry safely without risk of shear failure.

$$q_s = q_{ns} + \gamma D$$

Allowable bearing Capacity; It is the maximum pressure which may be applied to the soil such that the soil is satisfied by providing adequate factor of safety as well as the tolerable settlement of the foundation.

$$q_{all} = \frac{q_u}{FS}$$

Effect of ground water table on bearing capacity of a soil.

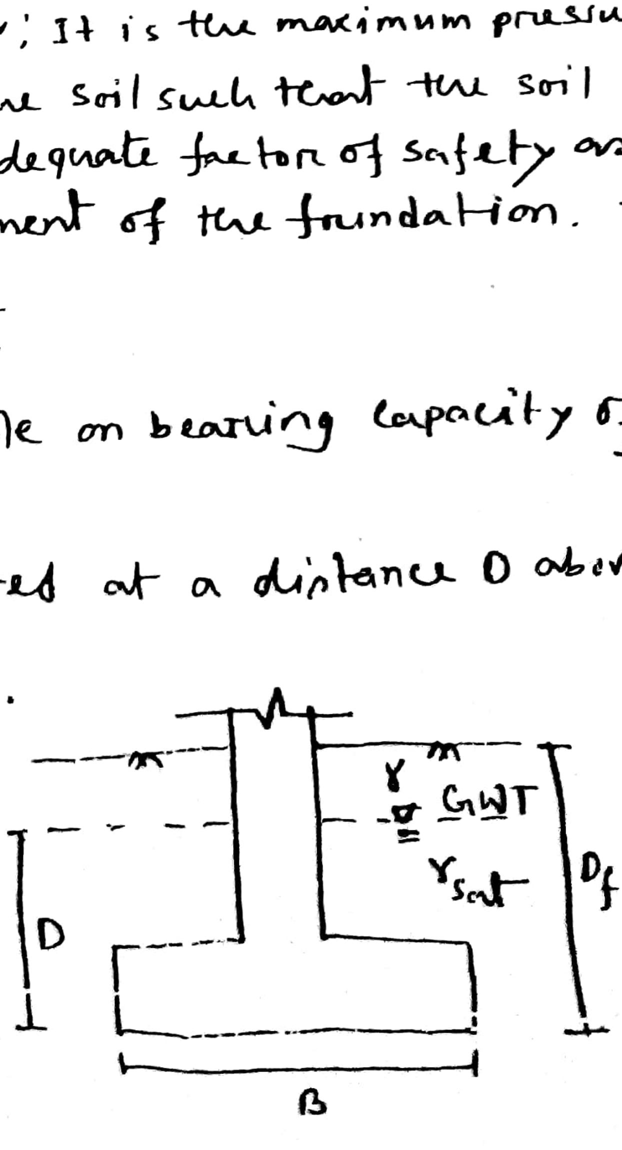
Soln: (a) The G.W.T. is located at a distance D above the bottom of the foundation.

Then, second term

$$q = \gamma(D_f - D) + \gamma' D$$

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

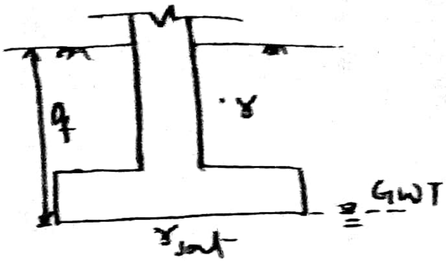
$\hookrightarrow$ third term $\left(\frac{1}{2}\gamma' B N_\gamma\right)$



(b) If GWT coincides with the bottom of the foundation, then

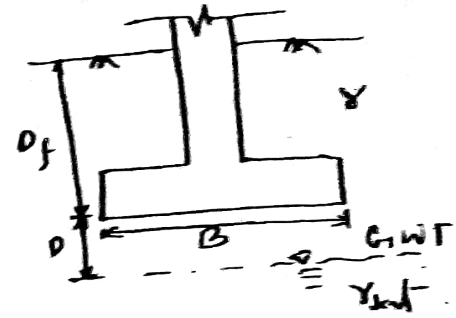
$q = \gamma D_f$ ← second term

$\gamma' = \gamma_{sat} - \gamma_w$
↳ 3rd term



(c) When GWT is at a depth D bel: the bottom of the foundation. Then,

$q = \gamma D_f$ ← second term



$\gamma_{av} = \frac{1}{B}\left[\gamma D + \gamma'(B-D)\right]$ for $D \leq B$

$v = \gamma$ for $D > B$. → γ_{av} → 3rd term.

* Write down the assumptions of Tarzaghi's soil bearing capacity theory.

Ans:
- Footing base rough
- Footing lay at shallow depth, $D_f \leq B$
- The soil is homogenous & isotropic and its shear strength is represented by Coulomb's equation, $\tau = \sigma \tan\alpha + c$
- Footing load is vertical & UDL.
- Footing is long (L/B ratio is infinite)

* Write down the factors affecting the bearing capacity of shallow foundation resting on sand.

Ans:
- Nature of soil
- Nature of the foundation
- Size, shape & depth of foundation
- Rigidity of structure.
- Total & differential settlement of that structure.
- Location of Ground water table.

Bearing capacity equation:

① Tarzaghi's equation; (For general shear failure)

i) For continuous/ strip footing

$$q_{ult} = cN_c + qN_q + \frac{1}{2}\gamma B N_\gamma$$

ii) for square footing

$$q_{ult} = 1.3\,cN_c + qN_q + 0.4\,\gamma B N_\gamma$$

iii) for circular footing

$$q_{ult} = 1.3\,cN_c + qN_q + 0.3\,\gamma B N_\gamma$$

For cohesive soil ($\phi = 0$), the 3rd term ($\frac{1}{2}\gamma B N_\gamma$, $.4\gamma B N_\gamma$, $.3\gamma B N_\gamma$) will be zero, because it depends on ϕ.

where,

$$N_q = \frac{q^2}{2\cos^2(45+\phi/2)} \; ; \; q = \gamma D_f.$$

$$N_c = (N_q - 1)\cot\phi$$

$$N_\gamma = \frac{\tan\phi}{2}\left(\frac{K_{p\gamma}}{\cot^2\phi} - 1\right)$$

② Meyerhof equation:

$$q_{ult} = cN_c s_c d_c + qN_q s_q d_q + \frac{1}{2}\gamma B N_\gamma s_\gamma d_\gamma$$

where, s = shape factor, d = depth factor.

$$N_q = \tan^2(45+\phi/2)\, e^{\pi\tan\phi} = K_p e^{\pi\tan\phi}$$

$$N_c = (N_q - 1)\cot\phi$$

$$N_\gamma = (N_q - 1)\tan(1.4\phi)$$

③ Hansen & vesic:

$$q_{ult} = cN_c s_c d_c i_c g_c b_c + qN_q s_q d_q i_q g_q b_q + \frac{1}{2}\gamma B N_\gamma s_\gamma d_\gamma i_\gamma g_\gamma b_\gamma$$

where, $N_q = K_p e^{\pi\tan\phi}$

$$N_c = (N_q - 1)\cot\phi$$

$N_\gamma = 1.5\,(N_q - 1)\tan\phi$ → Hansen

$N_\gamma = 2\,(N_q - 1)\tan\phi$ → vesic

* A square footing 2m x 2m is built in a homogeneous bed of sand. $\gamma = 19\ KN/m^3$, $\phi = 36°$. The depth of bone of the footing below the ground surface is 1.2m. Calculate the safe load carried by footing with a F.S = 3. Assume $N_c = 65.4$, $N_q = 49.4$, $N_\gamma = 54$

Soln: For square footing

$q_{ult} =$ ~~cNc~~ $1.3cN_c + qN_q + .4\gamma BN_\gamma$

For sand, $c = 0$

$q = \gamma H = 19 \times 1.2 = 22.8\ KN/m^2$

$\therefore q_{ult} = 1.3 \times 0 \times 65.4 + 22.8 \times 49.4 + .4 \times 19 \times 2 \times 54$

$= 1947\ Kpa.$

$\therefore q_{all} = \frac{q_{ult}}{F.S.} = \frac{1947}{3} = 649\ Kpa$

Safe load $= q_{all} \times B^2$

$= 649 \times 2^2$

$= 2596\ KN$

Ans

* Calculate the ~~allowable~~ ultimate bearing capacity of soil per unit area of -

i) A strip footing of 1m wide

ii) A square footing of 3m x 3m

iii) A circular footing of 3m dia

$\gamma = 17.66\ KN/m^3$, $c = 19.62\ KN/m^2$, $\phi = 25°$, $N_c = 17.5$, $N_q = 7.5$

$N_\gamma = 5$, Footing 1m below the ground level.

Soln: ① $q_{ult} = cN_c + qN_q + \frac{1}{2}\gamma BN_\gamma$

$= 19.62 \times 17.5 + 17.66 \times 1 \times 7.5 + \frac{1}{2} \times 17.66 \times 1 \times 5$

$= 520\ Kpa$

② $q_{ult} = 1.3cN_c + qN_q + .4\gamma BN_\gamma$

$= 685\ Kpa$

③ $q_{ult} = 1.3cN_c + qN_q + .3\gamma \overset{Dia}{B} N_\gamma$

$= 658\ Kpa$

Ans

* Determine the net ultimate bearing capacity of a strip footing 1·2m wide and having the depth of foundation of 1m. $\phi = 35°$, $\gamma = 18\ KN/m^3$, $c = 15\ KN/m^2$

Sol^n:

$$q_{ult} = cN_c + qN_q + \frac{1}{2}\gamma B N_\gamma$$

$$= 15 \times 57{\cdot}8 + (18\times1) * 41{\cdot}4 + \frac{1}{2} \times 18 \times 1{\cdot}2 * 42{\cdot}4$$

$$= 2070\ KN/m^2$$

$N_c = 57{\cdot}8$
$N_q = 41{\cdot}4$
$N_\gamma = 42{\cdot}4$

Net ultimate bearing capacity

$$q_{nf} = q_{ult} - \gamma D$$

$$= 2070 - 18 * 1$$

$$= 2052\ KN/m^2$$

Ans.

* Determine the allowable gross load and net allowable load for a square footing of 2m side and with a depth of foundation 1m. $\phi = 25°$, $\gamma = 18\ KN/m^3$, $c = 15\ KN/m^2$, F.S = 3. $N_c = 57{\cdot}8$, $N_q = 41{\cdot}4$, $N_\gamma = 42{\cdot}4$.

Sol^n.

$$q_{ult} = 1{\cdot}3\, cN_c + qN_q + {\cdot}4\gamma B N_\gamma$$

$$= 1{\cdot}3 \times 15 \times 57{\cdot}8 + (18\times1) * 41{\cdot}4 + {\cdot}4 \times 18 \times 2 \times 42{\cdot}4$$

$$= 2483\ KN/m^2$$

$$q_{all} = \frac{q_{ult}}{F.S} = 827{\cdot}67\ KN/m^2$$

$\therefore$ Allowable gross load $= 827{\cdot}67 * 2^2 = 3311\ KN$

Ans

$$q_{nf} = q_{ult} - \gamma D$$

$$= 2483 - 18*1 = 2965\ KN/m^2$$

$\therefore$ Net allowable load $= \frac{2465}{F.S.} * 4 = \frac{2465}{3} * 4$

~~= [illegible] KN~~ = 3287 KN

Ans

Tarzaghi's equation (For local shear failure)

i) For Continuous/strip footing

$$q_{ult} = c'N_c' + qN_q' + \frac{1}{2}\gamma B N_\gamma'$$

ii) For Square footing

$$q_{ult} = 1.3\,c'N_c' + qN_q' + 0.4\gamma B N_\gamma'$$

iii) For Circular footing

$$q_{ult} = 1.3\,c'N_c' + qN_q' + 0.3\gamma B N_\gamma'$$

For cohesive soil the 3rd term ($\frac{1}{2}\gamma B N_\gamma'$, $.4\gamma B N_\gamma'$, $.3\gamma B N_\gamma'$) will be zero, because it depends on φ.

Where, $c' = \frac{2}{3}c$, $\tan\varphi' = \frac{2}{3}\tan\varphi$, ~~$N_q' = \frac{2}{3}N_q$, $N_c' = \frac{2}{3}N_c$, $N_\gamma' = \frac{2}{3}N_\gamma$~~

* Determine the ultimate bearing capacity of a Strip footing 1.2m wide and having the depth of foundation is 1m. if take for c-φ Soil, $c = 18\ \text{kN/m}^2$, $\gamma = 18\ \text{kN/m}^3$, $\varphi = 35°$, $N_c = 57.7$, $N_q = 14.4$, $N_\gamma = 42.4$ and take for c-soil, $c = 30\ \text{kN/m}^2$, $\gamma = 20\ \text{kN/m}^3$, $\varphi = 0$ then $N_c = 5.14$, $N_q = 1$, $N_\gamma = 2.4$.

i) General shear failure

ii) Local shear failure.

Soln: For c-φ Soil:

i) General shear failure:

$$q_{ult} = cN_c + qN_q + \frac{1}{2}\gamma B N_\gamma$$

$$= 18 \times 57.7 + (18\times1)\times14.4 + \frac{1}{2}\times18\times1.2\times42.4$$

$$= 1755.7\ \text{kN/m}^2$$

ii) Local shear failure:

$$q_{ult} = c'N_c' + qN_q' + \frac{1}{2}\gamma B N_\gamma'$$

$$= 12\times25.1 + (18\times1)\times12.7 + \frac{1}{2}\times18\times1.2\times9.7$$

$$= 634.6\ \text{kN/m}^2$$

General shear failure:

$c = 18\ \text{kN/m}^2$, $\gamma = 18\ \text{kN/m}^3$

$\varphi = 35°$, $N_c = 57.7$, $N_q = 14.4$

$N_\gamma = 42.4$

Local shear failure:

$c' = \frac{2}{3}c = \frac{2}{3}*18 = 12\ \text{kN/m}^2$

$\gamma = 18\ \text{kN/m}^3$.

$\tan\varphi' = \frac{2}{3}\tan\varphi$

$\Rightarrow \varphi' = \tan^{-1}(\frac{2}{3}\tan35) = 25°$

$N_c' = 25.1$, $N_q' = 12.7$, $N_\gamma' = 9.7$

For c-soil:

i) General shear failure:

$q_{ult} = cN_c + qN_q + \frac{1}{2}\gamma B N_\gamma$

$= 30 \times 5.14 + (20 \times 1) \times 1 + 0$

$= 174.1 \text{ KN/m}^2$

ii) Local shear failure:

$q_{ult} = c'N_c' + qN_q' + \frac{1}{2}\gamma B N_\gamma'$

$= 20 \times 5.14 + (20 \times 1) \times 1 + 0$

$= 122.8 \text{ KN/m}^2$

Ans

general shear failure:

$\phi = 0°,\ c = 30 \text{ KN/m}^2$

$\gamma = 20 \text{ KN/m}^3$

$N_c = 5.14,\ N_q = 1,\ N_\gamma = 2.4$

Local shear failure

$c' = \frac{2}{3} * 30 = 20 \text{ KN/m}^2$

$\gamma = 20 \text{ KN/m}^3$

$N_c' = 5.14,\ N_q' = 1,\ N_\gamma' = 2.4$

* A 2m x 2m footing is laid at a depth of 1.3m below the ground surface. Determine the ultimate bearing capacity if i) water table [illegible]es to the level of the base, ii) water table rise t[illegible] [illegible]round surface, iii) water table is 1m below the [illegible]e. iv) water table is [illegible] the ground su[illegible]ce. Given that, $\gamma = 20$ k[illegible], [illegible], c[illegible] KN/m^2, $\phi = 30°$, $N_c = 37.2$ $N_q = 22.5$, $N_\gamma = 19.7$.

Important

Effect of WT

Sol^n: (i) Water table rises to t[illegible] [illegible]el of the base:

$q = \gamma D_f = 20 \times 1.3 = 26 \text{ KN/m}^2,\ \gamma' = 20 - 9.8 = 10.2 \text{ KN/m}^3$

$q_{ult} = 1.3\,cN_c + qN_q + .4\,\gamma' B N_\gamma$

$= 1.3 \times 24 \times 37.2 + 26 \times 22.5 + .4 \times \overset{10.2}{[illegible]} \times 2 \times 19.7$

$= 1906.39 \text{ KN/m}^2$

ii) water table rise the ground surface:

$q = \gamma' D_f = 10.2 * 1.3 = 13.26 \text{ KN/m}^2$

$q_{ult} = 1.3\,cN_c + qN_q + .4\,\gamma' B N_\gamma$

$= 1.3 * 24 * 37.2 + 13.26 * 22.5 + 4 * 10.2 * 2 * 19.7$

$= 1619.74 \text{ KN/m}^2$

iii) water table is 1m below the base:

$D < B$, $q = \gamma D_f = 20 * 1.3 = 26 \ KN/m^2$

$\therefore \gamma_{av} = \frac{1}{B}[\gamma D + \gamma'(B-D)]$

$= \frac{1}{2}[20 \times 1 + 10.2(2-1)]$

$= 15.1 \ KN/m^2$

$\therefore q_{ult} = 1.3 cN_c + qN_q + .4 \gamma_{av} B N_\gamma$

$= 1.3 * 24 * 37.2 + 26 * 22.5 + .4 * 15.1 * 2 * 19.7$

$= 1983.62 \ KN/m^2$

(iv) water table 0.5m below ground surface:

$q = \gamma(D_f - D) + \gamma' D$

$= 20(1.3 - 0.8) + 10.2 * 0.8$

$= 18.16 \ KN/m^2$

$q_{ult} = 1.3 cN_c + qN_q + .4 \gamma' B N_\gamma$

$= 1.3 * 24 * 37.2 + 18.16 * 22.5 + .4 * 10.2 * 2 * 19.7$

$= 1730 \ KN/m^2$

Ans

* Footing size = 2m×2m, $\gamma_{sat} = 20.4 \ KN/m^3$, $\phi = 20°$. $N_c = 37.2$, $N_q = 22.5$, $N_\gamma = 19.7$, $c = 20kpa$, Depth of foundation = 5m, F.S = 3 calculate the allowable bearing capacity.

soln:

$q = \gamma' D_f = (\gamma_{sat} - \gamma_w) D_f = (20.4 - 9.81) * 5 = 52.95 kpa.$

$q_{ult} = 1.3 cN_c + qN_q + .4 \gamma' B N_\gamma$

$= 1.3 \times 20 \times 37.2 + 52.95 \times 22.5 + .4 \times (20.4 - 9.81) \times 2 \times 19.7$

$= 2325.5 \ kpa.$

$\therefore q_{all} = \frac{q_{ult}}{FS} = \frac{2325.5}{3} = 775.16 \ kpa$

Ans

Md. Miraz Gazi
B.sc. in CE, KUET

Settlement

 # Settlement: Settlement of a soil mass may be defined as the vertical downward displacement of soil mass due to the change of volume of void is a mass.

 # Types of settlement:

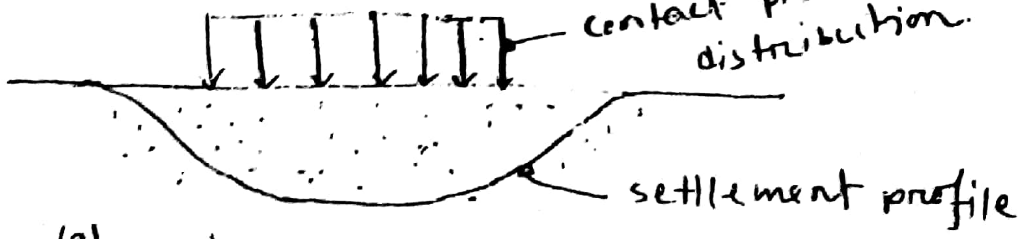
① **Immediate settlement:** When the load is applied on the soil solid, the soil solids are deformed by air compression.

② **Primary Consolid settlement:** The reduction in volume of soil due t ueering out of water from the void. (When soil ubjected to an increase in σ_z').

③ **Secondary consolidation** nt: The reduction in volume of soil due to reduction of all excess pore w ure tends to zero. It is due to particle o reo e and decomposition of organic materials.

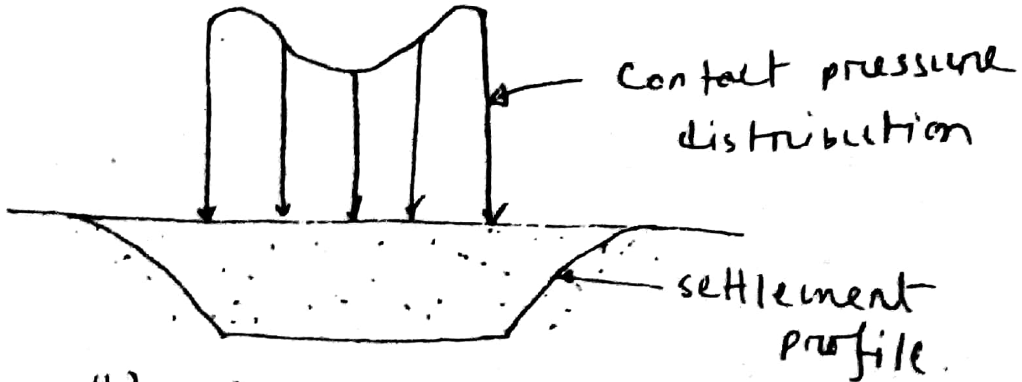
* Draw the contac re and settlement profile of rigid & flexible foundation rest on clay and sand.

Ans: 1. Contact pressure & settlement profile on clay



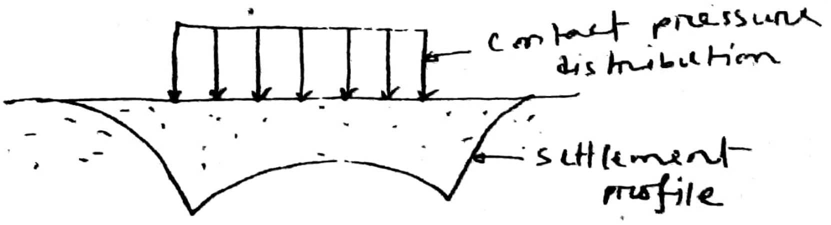
(a) Flexible Foundation.

8.

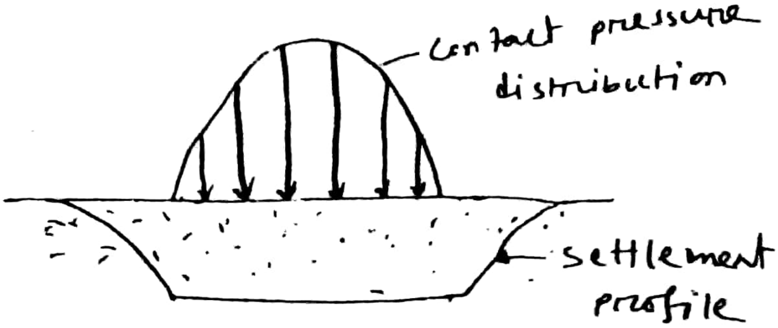


(b) Ri

2. Contact pressure and settlement profile on sand



(a) Flexible Foundation.



(b) Rigid Foundation.

* **Causes of settlement:**

- Deformation of soil mineral grains
- Compression of water.
- Expulsion of water from void spaces.
- Relocation of soil particles.

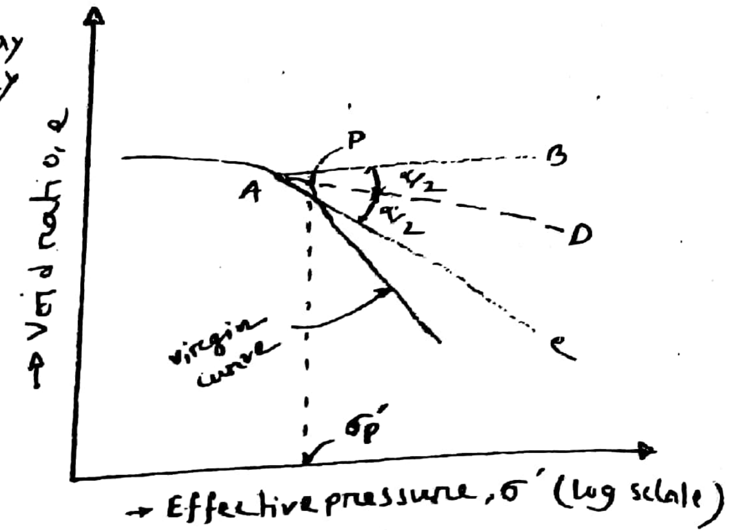
* **Pre-consolidation pressure:** The temporary overburden pressure to which a soil has been subjected and under which it got consolidated is known as pre-consolidation pressure.

* **Normally Consolidated soil:** The soil which has never been subjected to an effective pressure greater than the existing overburden pressure and which is completely consolidated by the existing overburden pressure.

* **Over consolidated soil:** It is the soil which is subjected to a pressure in excess of its present overburden pressure

Determination of preconsolidation pressure:

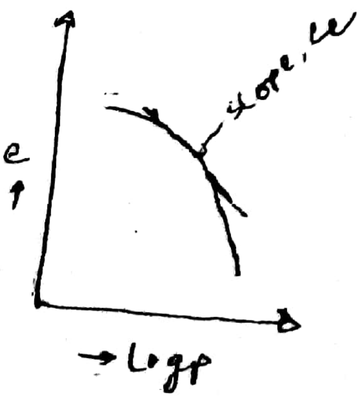
An undisturbed sample of clay is consolidated in the laboratory and the pressure, void ratio relationship is plotted on a semilog paper as shown in figure.



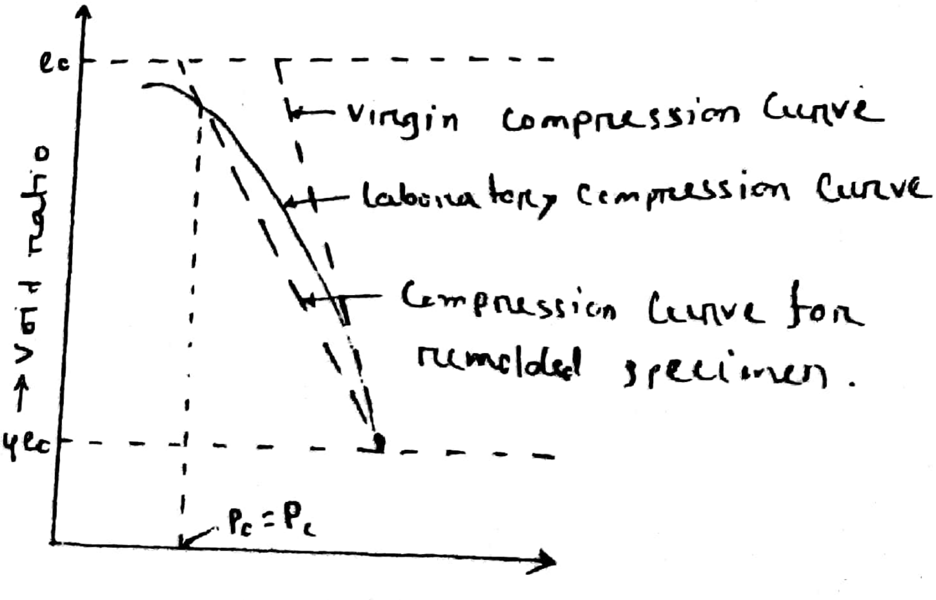
The approximate point A of maximum curvature is selected and the horizontal line AB is drawn. A tangent Ac is also drawn to the curve and the bisector AD is drawn. The straight portion of virgin curve is extended behind to meet the bisector AD in P. The point P corresponds to the pre-consolidation pressure σp'.

Draw e-logp diagram for NCC & OCC soil.

① **NCC soil:**



① Normally consolidated clay



Case-1: $(\sigma_0' < \sigma_0' + \Delta\sigma' \leq \sigma_c')$

$$S_c = \frac{C_s H}{1+e_0} \log \frac{\sigma_0' + \Delta\sigma'}{\sigma_0'}$$

C_s = Swell index ≈ $\frac{1}{5}$ to $\frac{1}{10}$ of C_c

σ_c' = preconsolidation pressure

Case-2: $(\sigma_0' < \sigma_c' < \sigma_0' + \Delta\sigma')$

$$S_c = \frac{C_s H}{1+e_0} \log \frac{\sigma_c'}{\sigma_0'} + \frac{C_c H}{1+e_0} \log \frac{\sigma_c' + \Delta\sigma'}{\sigma_c'}$$

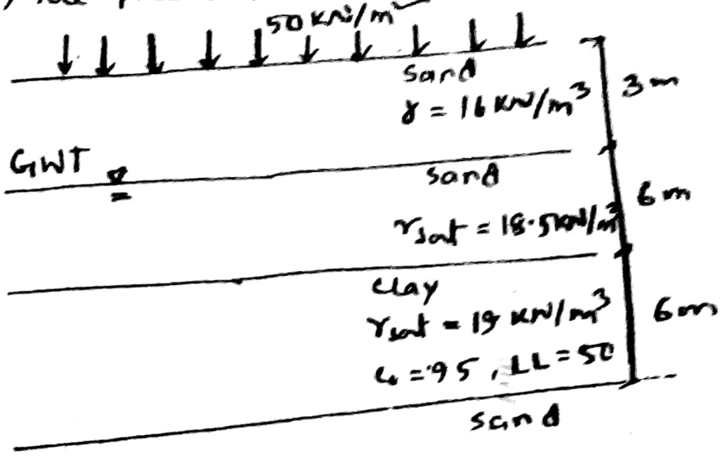
tio) - preconsolidation pressure

* A soil profile shown in figure. If a uniformly distributed Load is applied at the ground surface, what is the settlement of the clay layer caused by primary consolidation if $\rightarrow C_s = \frac{1}{5}$ of C_c

i) The clay is normally consolidated.

ii) The preconsolidation pressure is 190 kN/m²

iii) The preconsolidation pressure is 170 kN/m²



Solⁿ: (i) NCC soil:

$$\sigma_0' = 16*3 + (18.5-9.81)*6 + (19-9.81)*3$$

$$= 127.71 \text{ kN/m}^2$$ (at mid point of clay layer)

$$\Delta\sigma' = 50 \text{ kN/m}^2$$

$$C_c = 0.009(LL-10) = 0.009(50-10) = 0.36$$

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

$$= \frac{.36*6}{1+.95} * \log\left(\frac{127.71+50}{127.71}\right) = 0.159 \text{ m}$$ Ans

(ii) $\sigma_c' = 190 \text{ kN/m}^2 > \sigma_0'+\Delta\sigma' = 127.71+50 = 177.71 \text{ kN/m}^2$

$$C_s = \frac{1}{5} C_c = 0.072$$

$$S_c = \frac{C_s H}{1+e_0} \log\left(\frac{\sigma_0'+\Delta\sigma'}{\sigma_0'}\right) = \frac{.072*6}{1+.95} \log\left(\frac{177.71}{127.71}\right)$$

$$= 0.032 \text{ m}$$ Ans

(iii) $\sigma_c' = 170 \, kN/m^2 < \sigma_0' + \Delta\sigma' = 177.71 \, kN/m^2$

$$S_c = \frac{C_s H}{1+e_0} \log \frac{\sigma_c'}{\sigma_0'} + \frac{C_c H}{1+e_0} \log \frac{\sigma_0' + \Delta\sigma'}{\sigma_c'}$$

$$= \frac{.072 \times 6}{1+.75} \log \frac{170}{127.71} + \frac{.36 \times 6}{1+.95} \log \frac{177.71}{170}$$

$$= 0.049 m$$

Ans

* Calculate the settlement of a 2.5m deep clay layer due to increase of 30 KN/m² pressure at mid height of clay layer. Given effective vertical stress at mid height of layer 130 KN/m². $e = 0.80$, $C_c = 0.28$.

Soln: For NCC:

$$S_c = \frac{C_c H}{1+e} \log \frac{\sigma_0' + \Delta\sigma'}{\sigma_0'} = \frac{.28 \times 2.5}{1+.8} \log \frac{130+30}{130}$$

$$= 0.035 m$$

Ans

* The average natural water content of the normal clay deposit is 40%, unit weight is 2.8 gm/cc and compressive index is 0.36. If $C_v = 6 \times 10^{-5}$ sft/min and clay deposits 20 ft thick. Find settlement if drained top and bottom the existing effective overburden pressure at the centre of clay layer is 2 ton/sft and the increase of pressure causing the expected settlement is 0.29 ton/sft ?

Soln:

$$\gamma_s = G_s \gamma_w \Rightarrow G_s = \frac{\gamma_s}{\gamma_w} = \frac{2.8}{1}$$

$$\Rightarrow G_s = 2.8$$

$$\therefore e = w G_s = 0.4 \times 2.8 = 1.12$$

$$S_c = \frac{C_c H}{1+e} \log \frac{\sigma_0' + \Delta\sigma'}{\sigma_0'}$$

$$= \frac{.36 \times 20}{1+1.12} \log \frac{2.29}{2} = .1991 \text{ ft}$$

Ans

$C_v = 6 \times 10^{-5}$ sft/min

$\gamma_s = 2.8$ gm/cc

$w = 40\% = 0.40$

$C_c = 0.36$

$\sigma_0' = 2$ ton/sft

$\Delta\sigma' = 0.29$ ton/sft.

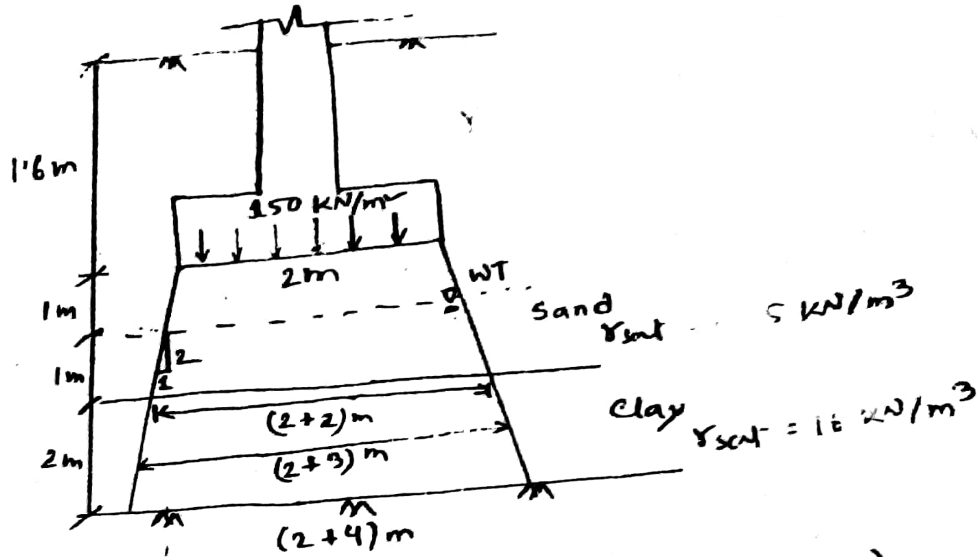
$H = 20$ ft.

$\gamma_w = 1$ gm/cc

* A building column has a footing area of 2m x 3m and transmits a pressure increment of 150 KN/m² as it's base embedment 1.6m below ground level as shown in figure. Assuming a pressure distribution of 2 vertical to 1 horizontal, determine the consolidation settlement at the middle of clay layer. Consider the pressure variation across the thickness of clay layer.

(i) For Sand, $\gamma = 16.5\ KN/m^3$, $\gamma_{sat} = 18.5\ KN/m^3$

(ii) For clay, $\gamma_{sat} = 16\ KN/m^3$, $e = 0.95$, $C_c = 0.26$



Solⁿ;

$(16-9.81)$

$\sigma_0' = 16.5*2.6 + 1*(18.5-9.8$ [illegible]

$= 57.78\ KN/m^2$ (at mid point of clay layer)

$\Delta\sigma'$ at top, middle & bottom of clay layer.

$$(\Delta\sigma')_{top} = \frac{150*2*3}{4*5} = 45\ KN/m^2$$

$$(\Delta\sigma')_{middle} = \frac{150*2*3}{5*6} = 30\ KN/m^2$$

$$(\Delta\sigma')_{bottom} = \frac{150*2*3}{6*7} = 21.43\ KN/m^2$$

$$\therefore \Delta\sigma' = \frac{1}{6}\left[(\Delta\sigma')_{top} + 4(\Delta\sigma')_{middle} + (\Delta\sigma')_{bottom}\right]$$

$$= 31.07\ KN/m^2$$

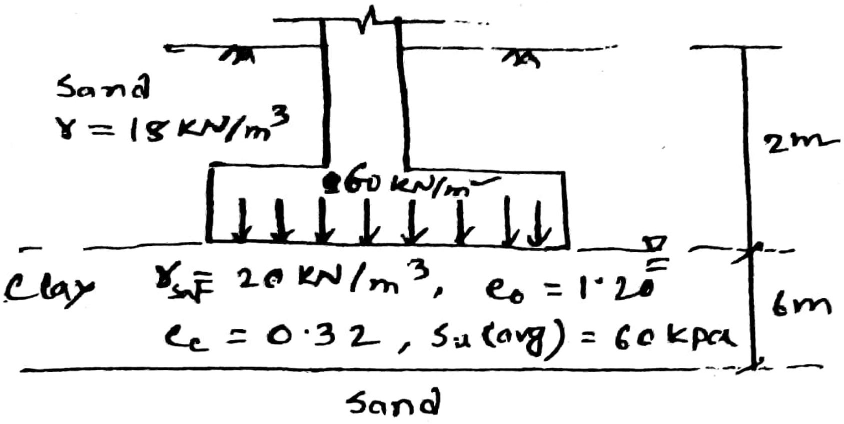
settlement, $S_c = \frac{C_c H}{1+e} \log\left(\frac{\sigma_c' + \Delta\sigma'}{\sigma_c'}\right)$

$= \frac{0{\cdot}26 * 2}{1 + {\cdot}95} \log\left(\frac{57{\cdot}78 + 31{\cdot}07}{57{\cdot}78}\right)$

$= 0{\cdot}0498$ m

Ans

* A circular foundation of 2m dia directly resting on a 6m thick normally consolidated clay layer as shown in figure. Determine the consolidation settlement of the clay layer if the footing subjected to 60 KN/m² uniform load.



Ans:

$\sigma_c' = 18 * 2 + (20 - 9{\cdot}81) * 3$

$= 66{\cdot}57$ KN/m²

$\Delta\sigma' = 60$ KN/m²

$\therefore$ settlement, $S_c = \frac{C_c H}{1+e_o} \log \frac{\sigma_o' + \Delta\sigma'}{\sigma_c'}$

$= \frac{{\cdot}32 * 6}{1 + 1{\cdot}2} \log \frac{66{\cdot}57 + 60}{66{\cdot}57}$

$= 0{\cdot}244$ m

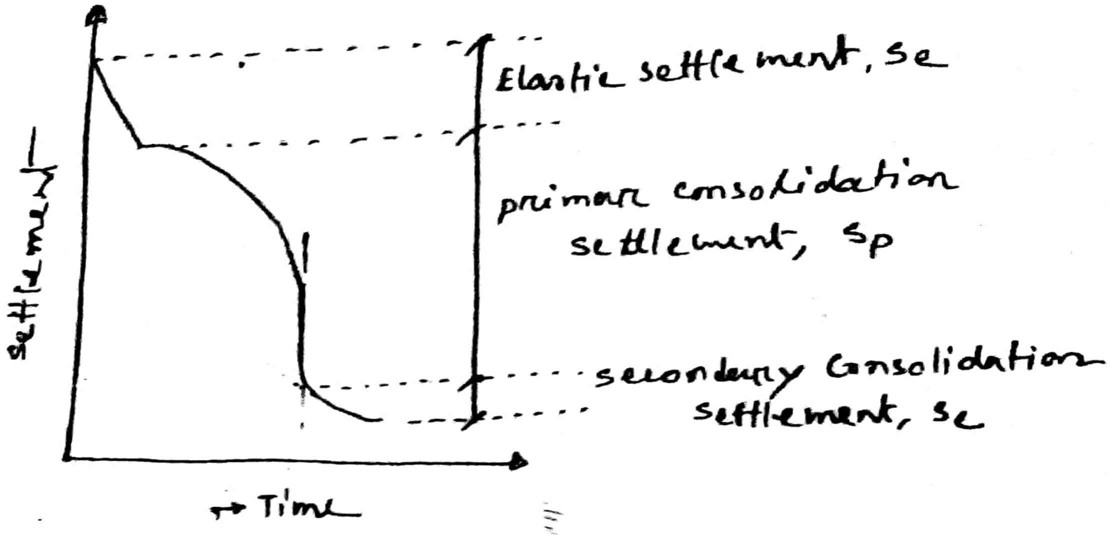
Ans.

#Write down the type of settlement with sketch.

Ans: ① Elastic/Immediate settlement

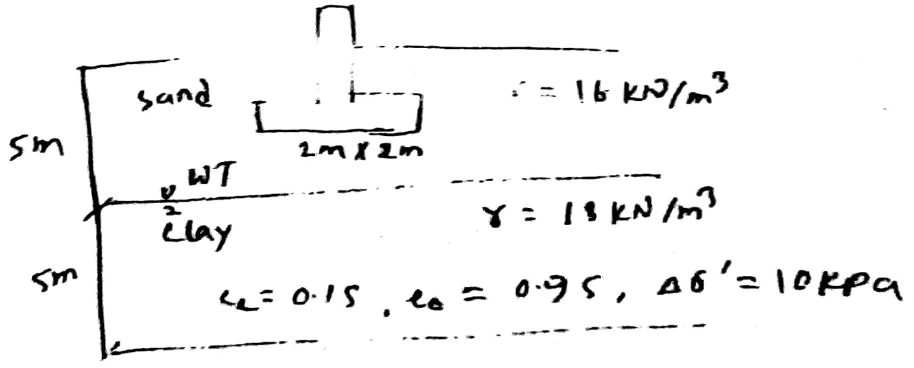
② primary consolidation settlement.

③ secondary consolidation settlement.



∴ Total settlement, $S_T = S_e + S_p + S_c$

* Calculate the consolidation settlement of the clay layer under this foundation.



Titas-14

Soln:

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

$$= \frac{.15 \times 5}{1+.95} \log \frac{100.48+10}{100.48}$$

$$= 0.0158 \text{ m}$$

Ans.

$\sigma_0' = 16 \times 5 + (18-9.81) \times 2.5$

$= 100.48 \text{ KN/m}^2$

* The thickness of clay layer is 2.5m. The value of void ratio is 0.7 and compression index is 0.28. The existing pressure at the mid of the clay layer is 40 kN/m² and a 30 kN/m² pressure is applied to the mid height of the layer. Find the consolidation settlement of the layer.

Soln: $\sigma' = 40$ kN/m², $\Delta\sigma' = 30$ kN/m², $C_c = \cdot 28$, $e_o = 0.7$, $H = 2.5$m

$$S = \frac{C_c H}{1+e_o} \log \frac{\sigma_o' + \Delta\sigma'}{\sigma_o'}$$

$$= \frac{\cdot 28 \times 2.5}{1+\cdot 7} \log \frac{40+30}{40} = 0.10007 \text{ m}$$

Ans.

Md. Miraz Gazi
B.Sc. in CE, KUET

'consolidation'

* $S = \frac{a_v}{1+e_o} \Delta P H = m_v \Delta P H$ $\boxed{P \text{ means } \sigma}$

Where,
a_v = coefficient of compressibility $= \frac{\Delta e}{\Delta P}$

m_v = coefficient of volume compressibility

$= \frac{a_v}{1+e_o}$ $\frac{\Delta e}{\Delta P(1+e_o)}$

* Compression index, $C_c = 0.009(LL-10)$ [Undisturbed]

$= 0.007(LL-10)$ [Disturbed]

* Compression index, $C_c = \frac{e_1 - e_2}{\log(P_1/P_2)}$ বড়টা always

* Swell index, $C_s = \frac{1}{5} C_c$ to $\frac{1}{10} C_c = 0.1 C_c$ to $0.2 C_c$

* Co-efficient of conson. $\frac{T_v d^2}{t}$

\# single drainage হলে ২ দিয়ে ভাগ হবে না,
\# Double drainage → 2 দিয়ে ভাগ

Where, T_v = Time factor
t = time
d = (thickness of sample)/2

* Time factor, $T_v = \frac{\pi}{4}\left(\frac{U}{100}\right)^2$ $[0 < U \leq 60\%]$

$= 1.781 - 0.933 \log(100 - U)$ $[U > 60\%]$

Where, U = Degree of consolidation

* $k = C_v m_v \gamma_w$, $k \rightarrow$ Hydraulic conductivity.

* $T_v \propto U^2$

* The laboratory consolidation data on undisturbed clay sample are as follows $e_0 = 1·1$, $P_0 = 95$ kN/m² $e_1 = 0·90$ & $P_1 = 475$ kPa. Calculate the co-efficient of volume compressibility and what will be the void ratio for a pressure of 600 kPa and effective pressure is 95 kPa?

Sol^n:

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

$$= \frac{0·2}{380(1+1·1)}$$

$$= 2·51*10^{-4} \ m^2/KN$$

Again,

$$C_c = \frac{e_0 - e_1}{\log\left(\frac{P_1}{P_2}\right)}$$

$$= \frac{1·1 - ·9}{\log\left(\frac{475}{95}\right)} = 0·286$$

$e_0 = 1·1$, $e_1 = 0·9$

$\Delta e = 1·1 - 0·9 = 0·2$

$P_2 = 95$ KPa, $P_1 = 475$ KPa

$\Delta P = P_1 - P_2$

$= 380$ KPa

$m_v = ?$

Again

$e_0 = 1·1$

$P_2 = 95$ KPa

$P_1 = 600$ KPa

$e_1 = ?$

$$\therefore C_c = \frac{e_0 - e_1}{\log\left(\frac{P_1}{P_2}\right)}$$

$$\Rightarrow e_1 = -0·286 * \log\left(\frac{600}{95}\right) + 1·1 = 0·871$$

Ans.

* Determine the co-efficient of vol^n compressibility if the settlement is 5 cm for 7·5 m clay layer having increasing pressure of 80 kg/cm².

Sol^n:

$$S = m_v \Delta P H$$

$$\Rightarrow m_v = \frac{S}{\Delta P H}$$

$$= \frac{5}{80 \times (7·5 \times 100)} = 8·33 \times 10^{-5} \ cm^2/kg$$

Ans.

* Calculate the total settlement of compressible soil stratum 2m deep and co-efficient of volume compressibility 0.2 cm²/kg under a pressure increment of 2 kg/cm²?

Soln:

B-13

$S = m_v \Delta P H$

$= 0.2 \times 2 \times 200$

$= 80$ cm

Ans

$m_v = 0.2$ cm²/kg

$\Delta = 2$ kg/cm²

$H = 2$ m $= 200$ cm

* A clay stratum 5m thick has the initial void ratio of 1.5 and the effective overburden pressure of 120 kN/m² and when the sample is subjected to an increased pressure of 120 kN/m² the void ratio reduces to 1.44. Determine the co-efficient of volume compressibility and final settlement of the stratum

Soln:

$m_v = \dfrac{\Delta e}{\Delta P(1+e_0)}$

$= \dfrac{0.06}{120(1+1.5)} = 2 \times 10^{-4}$ m²/kN

$\therefore S = m_v \Delta P H$

$= 2 \times 10^{-4} \times 120 \times 5$

$= 0.12$ m

Ans

$e_0 = 1.5,\ e_1 = 1.44$

$\Delta e = 1.5 - 1.44 = 0.06$

$P_0 = 120$ kN/m²

$P_1 = (120 + 120)$ kN/m²

$= 240$ kN/m²

$\Delta P = 240 - 120 = 120$ kN/m²

Note: Single drainage-এ ২ দ্বারা ভাগ করতে হয় না, Double drainage-এ ২ দ্বারা ভাগ.

* The time required to reach 50% consolidation for a soil specimen of 3 cm thick tested in a consolidometer under single drainage condition was 30 min. Determine the time required for the same soil of 4m thick to reach the same degree of consolidation, if it has double drainage path.

Soln:

$$C_v = \frac{T_{50} d_1^2}{t_1} \Rightarrow T_{50} = \frac{c_v t_1}{d_1^2}$$

$$c_v = \frac{T_{50} d_2^2}{t_2} \Rightarrow T_{50} = \frac{c_v t_2}{d_2^2}$$

$$\because T_{50} = T_{50} \Rightarrow \frac{c_v t_1}{d_1^2} = \frac{c_v t_2}{d_2^2}$$

$$\Rightarrow \frac{30 \times 60}{(.03)^2} = \frac{t_2}{\left(\frac{4}{2}\right)^2}$$

$d_1 = (3/100)$
$= 0.03\,m \rightarrow$ single drainage
$d_2 = 4/2 = 2m \rightarrow$ Double drainage

$$\Rightarrow t_2 = 8000000\,sec = 92.59 \text{ days}$$

Ans

* The time required for 50% consolidation of a 25mm thick clay layer (drained at both top and bottom) in the laboratory is 2 min 20 sec. How long (in days) it will take for a 3m thick clay layer of the same clay in the field under pressure increment to reach 50% consolidation? In the field, there is a rock layer at the bottom of the clay.

Soln:

$$T_{50} = \frac{c_v t_{lab}}{d_{lab}^2} = \frac{c_v t_{field}}{d_{field}^2}$$

$$\Rightarrow \frac{140}{(.0125)^2} = \frac{t_{field}}{(3)^2}$$

$$\Rightarrow t_{field} = 8064000\,s = 93.3d$$

Ans

$t_{lab} = 140\,sec$
$d_{lab} = \frac{25}{2} = 12.5\,mm$
$= 0.0125\,m$
$d_{field} = 3m$
↳ only bottom আছে ২ দ্বারা ভাগ হয়নি

* Again, how long (days) will it take in the field for 30% primary consolidation to occur?

Sol^n: we know, $T_v \propto U^2$

$$\Rightarrow \frac{t_1}{t_2} = \frac{U_1^2}{U_2^2} \Rightarrow t_2 = \left(\frac{U_2}{U_1}\right)^2 * t_1$$

$$= \left(\frac{30}{50}\right)^2 * \underline{93.33}$$

$$= 33.6 \text{ days}$$

Ans

* A 3m thick layer (double drainage) of saturated clay under surcharge loading under 90% consolidation in 75 days. Find the co-efficient of consolidation of clay for the pressure range.

Sol^n: $T_{90} = \frac{c_v t}{d^2}$

U = 90%
t = 75 days
d = $\frac{3}{2}$ = 1.5m

PGCL-17 $T_{90} = 1.781 - .933 \log(100-U)$

$= 1.781 - .933 \log(100-90)$

$= 0.848$

$$\therefore C_v = \frac{T_{90} d^2}{t} = \frac{.848 * 1.5^2}{75} = 0.0254 \text{ m}^2/\text{day}$$

Ans

* For a normally consolidated laboratory clay specimen drained on both sides, The following are given,

$\sigma_0' = 3000$ lb/ft^2, $\sigma_0' + \Delta\sigma' = 6.000$ lb/ft^2, $e_0 = 1.1$, $e = 0.9$

Thickness of clay specimen = 1 in

Time for 50% consolidation = 2 min

a) Determine the hydraulic conductivity (ft/min) of the clay for the loading range.

b) How long (days) will it take for a 6ft clay layer in the field (drained on one side) to reach 60% consolidation?

Sol^n:

a) Co-efficient of volume compressibility,

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

$$= \frac{1.1 - .9}{3000(1+1.1)} = 3.17 \times 10^{-5} \, ft^2/lb$$

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

$$= \frac{.197 \times (.0417)^2}{2}$$

$$= 1.71 \times 10^{-4} \, ft^2/min$$

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

$= \frac{\pi}{4} \left(\frac{50}{100}\right)^2$

$= 0.197$

$d = \frac{1}{2} = .5 in = .0417 ft$

$t = 2$ min

$\therefore$ Hydraulic conductivity, $K = C_v m_v \gamma_w$

$$= 1.71 \times 10^{-4} \times 3.17 \times 10^{-5} \times 62.4$$

$$= 3.39 \times 10^{-7} \, ft/min$$

b)

$$T_v = \frac{\pi}{4} \left(\frac{60}{100}\right)^2 = 0.2827$$

$d = 6$ ft

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

$$= \frac{.2827 \times 6^2}{1.71 \times 10^{-4}} = 59929.4 \, min$$

$$= 41.6 \text{ days}$$

Ans.

Scanned by CamScanner

* A clay layer of 15″ thick and is drained at the top only under the given surcharge the settlement is 12·21″.

a) What is the avg. degree of consolidation for clay layer when settlement is 3″.

b) $C_v = 0.003\ cm^2/sec$, how long will take for 50% consolidation.

c) If the thickness of clay layer of 15′ is drained on both sides how ~~log~~ long will take for 50% Consolidation?

Soln.:

a) ~~a~~ $U(\%) = \frac{3}{12.21} * 100 = 24.57\%$

b) $T_v = \frac{\pi}{4}\left(\frac{50}{100}\right)^2 = 0.197$ | $d = 15 * 2.54 = 38.1\ cm$

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

$\Rightarrow t = \frac{T_v d^2}{C_v} = \frac{.197 * (38.1)^2}{.003} = 95322.39\ sec$

$= 1.103$ days

Ans

c)

$t = \frac{T_v d^2}{C_v} = \frac{.197 * (228.6)^2}{.003}$ | $d = \frac{15 * 12 * 2.54}{2} = 228.6\ cm$

$= 3431606$ sec

$= 39.72$ days

Ans

* In a laboratory soil sample 50% consolidation is done in 3 min. Thickness of soil sample is 2.5 cm. What is the time required for 6m soil sample with same rate of consolidation.

Sol^n:

$$T_v = \frac{C_v t_1}{d_1^2}$$

$$T_v = \frac{C_v t_2}{d_2^2}$$

$t_1 = 3$ min

$d_1 = \frac{2.5}{2} = 1.25$ cm

$d_2 = \frac{6}{2} = 3m = 300$ cm

$$\therefore \frac{C_v t_1}{d_1^2} = \frac{C_v t_2}{d_2^2}$$

$$\Rightarrow t_2 = \frac{t_1 d_2^2}{d_1^2} = \frac{3 \times 300^2}{1.25^2} = 172800 \text{ min}$$

$= 120$ days

Ans

"permeability"

* **permeability:** It is the property of soil which allows the seepage of fluids through the interconnected void space.

* **Darcy law:** The discharge velocity of water through saturated soil is proportional to the hydraulic gradient.

$$V \propto i$$

$$\Rightarrow v = k i$$

↳ co-efficient of permeability.

* **Co-efficient of permeability:** It can be referred to as the velocity of water under unit hydraulic gradient. It is expressed as m/day.

$$K = C D_{10}^2$$

where, C = Alen-Hezen Constant = 100

D_{10} = particle size having 10% smaller size particle than given size.

* Factors affecting permeability of soil / co-efficient of permeability.

Ans:
- The size of soil grains
- The properties of pore fluid.
- The void ratio of soil.
- viscosity.
- The shape & arrangements of pores.
- The degree of saturation.

Formula for determining 'K' (Laboratory)

Constant Head Method

- $K = \frac{QL}{Aht}$ Designation / উপাধি
- For highly permeable materials (coarse grained i)
- Not precise method.
  ↳ [illegible]

Falling Head Method

- $K = 2.303 \frac{aL}{At_0} \log \frac{h_1}{h_2}$

Q = vol^m of water collected

A = x-sectional area of soil

t_0 = Time of collection of water.

h = Constant head difference

h_1 = initial head

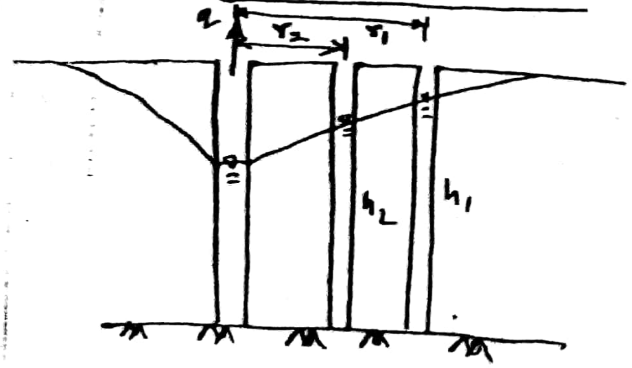
h_2 = final head after time t

a = x-sectional area of stand pipe

- for low permeable materials.
- precise method

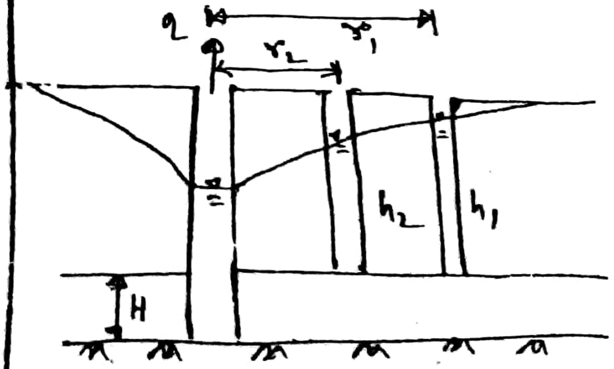
Formula for determining 'K' (field):

Unconfined Aquifer



- $K = \frac{2.303 q \log\left(\frac{r_1}{r_2}\right)}{\pi (h_1^2 - h_2^2)}$

confined Aquifer



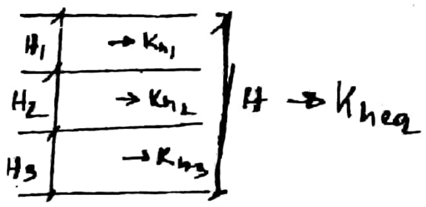
- $K = \frac{q \log\left(\frac{r_1}{r_2}\right)}{2.727 H (h_1 - h_2)}$

Ratio of equivalent permeability = $\frac{K_{Hor}}{K_{ver}}$

Discharge by permeability, $Q = KiA$

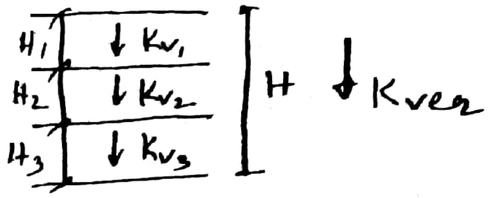
Equivalent co-efficient of permeability in stratified soils:

Horizontal flow



$$K_{heq} = \frac{1}{H}\left(K_{h_1}H_1 + K_{h_2}H_2 + \cdots + K_{h_n}H_n\right)$$

vertical flow



$$K_{veq} = \frac{H}{\frac{H_1}{K_{v_1}} + \frac{H_2}{K_{v_2}} + \cdots + \frac{H_n}{K_{v_n}}}$$

* Determine permeability of soil, which effective size of the particle is 0.008 mm & co-efficient of curvature 3.5.

Soln.

$$K = C D_{10}^2$$
$$= 100 * (.0008)^2$$
$$= 6.4 \times 10^{-5} \text{ cm/sec}$$

Ans

$D_{10} = 0.008$ mm
$= .0008$ cm
$C = 100$ [fixed if not given]

* The result of a constant head permeability test for a fine sand sample having a dia of 150 mm & a length of 300 mm are as follows –

i) Constant head difference = 500 mm
ii) Time of collection of water = 5 min
iii) volume of water collected = 350 cm^3
iv) Temperature of water = 24°c

Soln:

$$K = \frac{QL}{hAt} = \frac{350 \times 30}{50 \times 176.63 \times 300}$$
$$= 3.96 * 10^{-3} \text{ cm/sec}$$

Ans

$Q = 350$ cc
$L = 300$ mm $= 30$ cm
$A = \frac{\pi}{4}(15)^2 = 176.63$ cm^2
$t = 5$ min $= 300$ sec
$h = 500$ mm $= 50$ cm.

* In a constant head permeability test a sample 8 cm long was tested. The inside dia of the sample is 5 cm. After a state of steady flow was established under a head of 50 cm, discharge of 120 cc was collected is 30 sec. Compute the value of K.

Soln:

$$K = \frac{QL}{hAt} = \frac{120\times8}{50\times\frac{\pi}{4}\times5^2\times30} = 3.3\times10^{-2}\ \text{cm/sec}$$

Ans

* A falling head permeability test was performed in a permeameter with an inside dia of 5 cm. The inside dia of stand pipe was 2mm. The sample had a length of 8 cm. During a period of 6 min, the head on the sample decreased from 100 to 50 cm. Compute the value of K ~~& seepage velocity~~.

Soln:

$$K = 2.303\frac{aL}{At}\log\frac{h_1}{h_2}$$

$$= 2.303 * \frac{\frac{\pi}{4}*(.2)^2*8}{\frac{\pi}{4}*(5)^2*6\times60}\log\frac{100}{50}$$

$$= 2.5*10^{-5}\ \text{cm/sec}$$

Ans

* For a variable head permeability test, the following are given, length of specimen is in, area of specimen 3 in² and K = 0.0688 in/min. What should be the area of stand pipe for head to drop from 25 to 12 in in 8 min?

Soln:

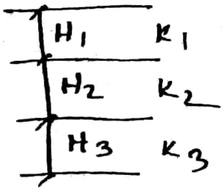
$$K = 2.303\frac{aL}{At}\log\frac{h_1}{h_2}$$

$$\Rightarrow 0.0688 = 2.303\frac{a*15}{3*8}\log\left(\frac{25}{12}\right)$$

$$\Rightarrow a = 0.15\ \text{in}^2$$

Ans

* A layered soil shown in figure, where $H_1 = 3$ ft, $k_1 = 10^{-4}$ cm/sec, $H_2 = 4$ ft, $K_2 = 3.2 * 10^{-2}$ cm/s, $H_3 = 6$ ft $K_3 = 4.1 * 10^{-5}$ cm/sec. Determine the ratio of equivalent permeability.



soln.

$$K_{heq} = \frac{1}{H}(K_1H_1 + K_2H_2 + K_3H_3)$$

$$= \frac{1}{3+4+6}(10^{-4}*3 + 3.2*10^{-2}*4 + 4.1*10^{-5}*6)$$

$$= 9.89*10^{-3} \text{ cm/sec}$$

$$K_{veq} = \frac{H}{\frac{H_1}{K_1} + \frac{H_2}{K_2} + \frac{H_3}{K_3}}$$

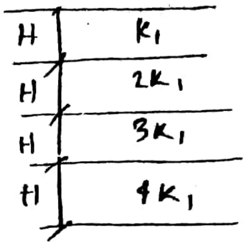
$$= \frac{3+4+6}{\frac{3}{10^{-4}} + \frac{4}{3.2*10^{-2}} + \frac{6}{4.1*10^{-5}}}$$

$$= 7.37*10^{-5} \text{ cm/sec}$$

$\therefore$ Ratio of eq. permeability $= \frac{K_{heq}}{K_{veq}} = 134.19$ Ans

* A stratified soil profile consists of 4 layers of equal thickness. The co-efficient of permeability of the 2nd, 3rd & 4th layers are 2, 3 and 4 times than that of the first layer respectively. Determine the effective co-efficient of permeability of the soil deposit for horizontal (parallel) and vertical (per-pendicular) direction of soil in terms of the co-efficient of permeability of the 1st layer.

soln:



$$K_{heq} = \frac{1}{4H}(K_1H + 2K_1H + 3K_1H + 4K_1H)$$

$$= 2.5\,K_1$$ Ans

$$K_{veq} = \frac{4H}{\frac{H}{K_1} + \frac{H}{2K_1} + \frac{H}{3K_1} + \frac{H}{4K_1}}$$

$$= \frac{4}{\frac{1}{K_1}\left(1 + \frac{1}{2} + \frac{1}{3} + \frac{1}{4}\right)}$$

$$= \frac{4K_1}{\frac{12+6+4+3}{12}} = \frac{48K_1}{25k.}$$ Ans

* A soil layer having 100 mm x 100 mm cross-section. Water is supplied to maintain a constant head difference 300 mm across the sample. The hydraulic conductivities of soil in the direction of flow through $h_1 = 150$ mm, $k_1 = 10^{-2}$ cm/sec, $h_2 = 150$ mm, $k_2 = 3\times10^{-3}$ cm/sec $h_3 = 150$ mm, $k_3 = 4.9\times10^{-4}$ cm/sec. Find the rate of water supply in cm³/hr?

Note: water flow 90' in sample → Kveq (perpendicular)
water flow same of sample → KHeq (parallel)

soln:

$$K_{veq} = \frac{150+150+150}{\frac{150}{10^{-2}} + \frac{150}{3\times10^{-3}} + \frac{150}{4.9\times10^{-3}}} = 0.001213 \text{ cm/sec}$$

$$Q = K_{veq}\,i\,A = .001213 * \frac{h}{L} * A$$

$$= .001213 * \frac{300}{450} * \left(\frac{100}{10} \times \frac{100}{10}\right) = .0809 \text{ cm}^3/\text{sec}$$

$$= 291.12 \text{ cm}^3/\text{hr}$$ Ans

Hydraulic conductivity of clay soil is $3 * 10^{-7}$ cm/sec. The viscosity of water at 25°C is $0.0911 * 10^{-4}$ g.sec/cm². Calculate the absolute permeability of soil.

Sol^n:

$$K = \bar{K}\,\frac{\gamma_w}{\eta}$$

$$\Rightarrow \bar{K} = \frac{K\eta}{\gamma_w} = \frac{3*10^{-7} * .0911 * 10^{-4}}{1\ gm/cm^3} = 0.2733*10^{-11}\ cm^2$$

Ans

* A horizontal stratified soil deposit of three layers each uniform itself. The permeability of these three layers is 8×10^{-4} cm/s, 52×10^{-4} cm/s and 6×10^{-4} cm/s and their thickness are 7, 3 & 10m respectively. Find the avg. permeability of the deposit in vertical direction.

Buet

Sol^n:

$$K_v = \frac{H_1+H_2+H_3}{\frac{H_1}{K_1}+\frac{H_2}{K_2}+\frac{H_3}{K_3}} = \frac{7+3+10}{\frac{7}{8\times10^{-4}}+\frac{3}{52\times10^{-4}}+\frac{10}{6\times10^{-4}}}$$

$$= 7.69\times10^{-4}\ cm/sec$$

Ans

"Seepage & Flow Net"

* **Seepage**: It is the slow movement of water through the continuous void space of the soil sample.

* **Flow net**: A series of flow lines and equipotential lines under any hydraulic structure is called flow net.

* **Equipotential line**: The line which connects the points that show the same piezometric elevation. It is orthogonal to flow line.

* **Flow line**: It is the line along which water particle travel from U/s to d/s.

* Seepage velocity, $V_s = \frac{V}{n} = \frac{V}{\frac{e}{1+e}} = V * \left(\frac{1+e}{e}\right)$

* **Formula for seepage quantities**:

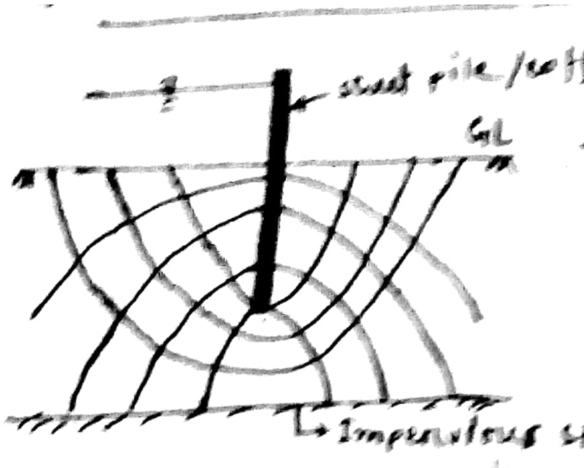
$$q = K \frac{h_L}{N_d} N_f = K \Delta H N_f \quad \left[\Delta H = \frac{h_L}{N_d}\right]$$

where, K = Co-efficient of permeability

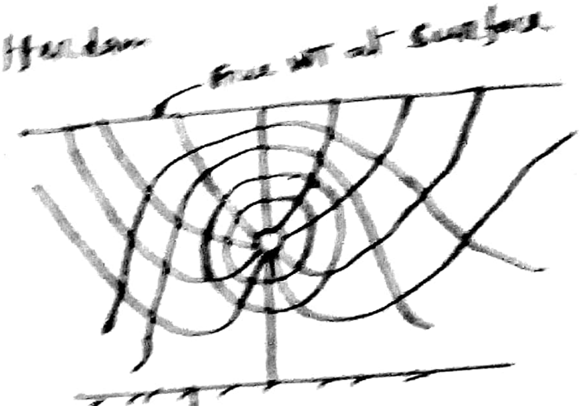
h_L = total hydraulic head.

N_d = No. of drop.

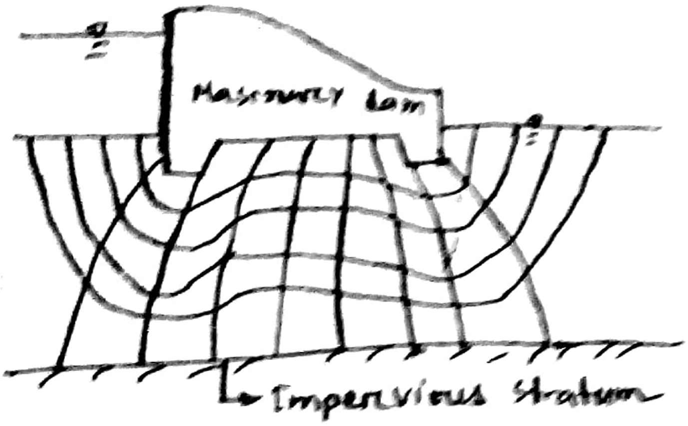
N_f = No. of channel.



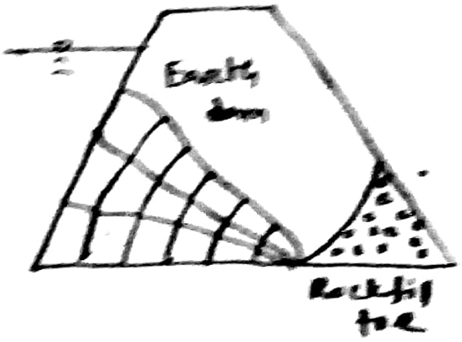
a) Flow net for sheet pile, coffer dam.



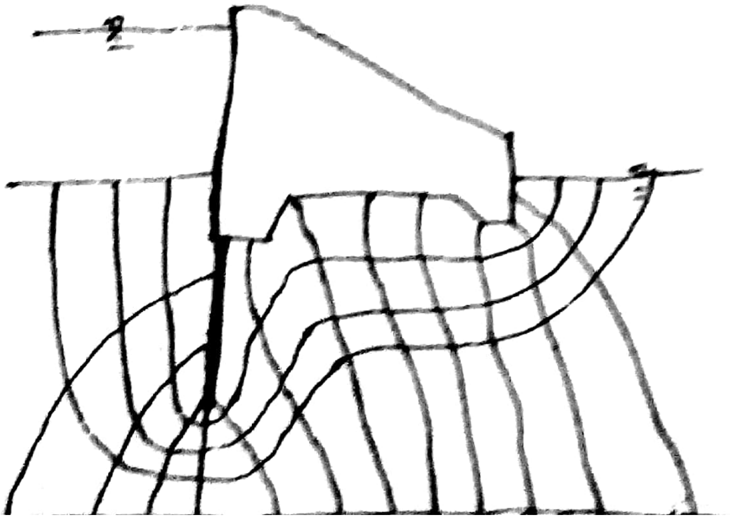
b) Flow net for drain



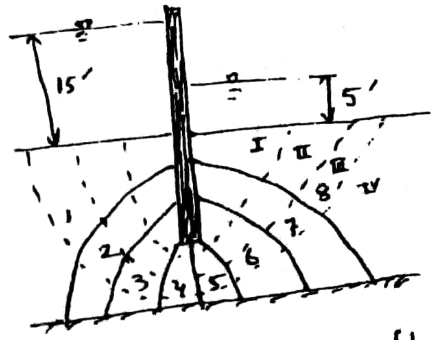
c) Flow net for masonry dam



d) Flow net for earth dam with rockfill toe



* A flow net for flow around a single row of sheet piles in a permeable soil layer is shown in figure. Given $K_x = K_y = K = 5*10^{-3}$ m/sec. Determine the total rate of seepage through the permeable layer per unit width.



Soln:

$$q = K \frac{h_L}{N_d} N_f$$

$$= 5*10^{-3} * \frac{3.05}{8} * 4$$

$$= 7.625*10^{-3} \text{ m}^3/\text{s/m}$$

Ans

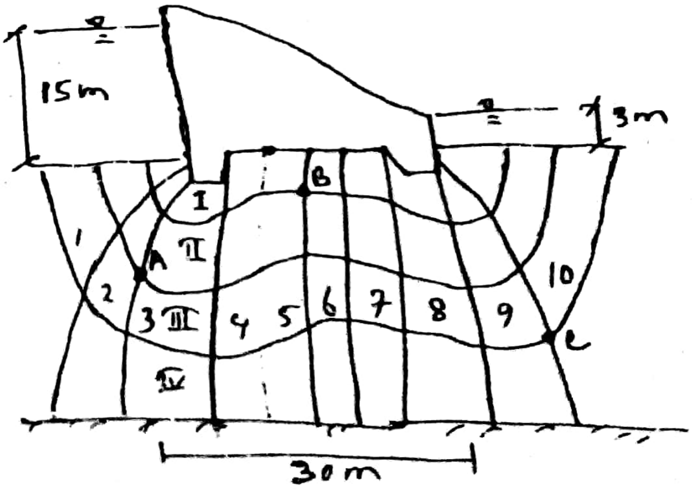
$h_L = 15 - 5 = 10 \text{ ft} = 3.05\text{m}$

$N_d = 8, N_f = 4$

* For the flow net and other specification as shown in the following figure, determine the followings -

a) How high water rise if a piezometer is placed at A, B & C.

b) if K = 0.01 mm/sec, calculate seepage loss.



Soln:

a) $h_A = h_L - \frac{\Delta h}{N_d} * 2$

$= 15 - \frac{(15-3)}{10} * 2$

$= 9.6$ m above ground surface

$h_B = h_L - \frac{\Delta h}{N_d} * 5$

$= 6$ m above ground surface

$h_L = 15 - 3 = 12$m

$N_d = 10, N_f = 4$

$h_C = 15 - \frac{15-3}{10} * 9$

$= 1.2$m above ground surface

b) Seepage loss, $q = K \frac{h_L}{N_d} N_f$ | $K = 0.01$ mm/sec $= 0.864$ m/day

$= 0.864 * \frac{(15-3)}{10} *$

$= 4.15$ m^3/day/m

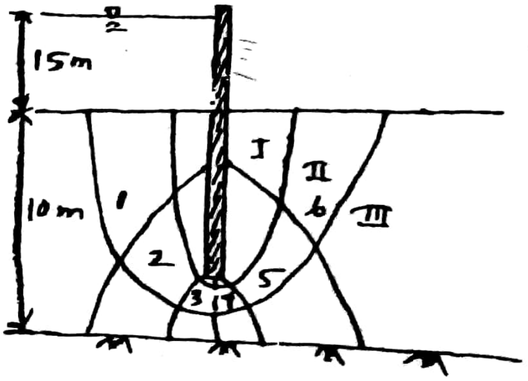
∴ Total loss $= 4.15 * 30$

$= 124.411$ m^3/day

Ans

* A deposit of cohesionless soil with a permeability of $9*10^{-2}$ cm/sec has a depth of 10m with impervious layer below. A sheet pile wall is driven into the deposit to a depth of 7.5m. The wall extends above the surface of the soil & a 2.5m depth of water acts on the side. Sketch the flow net & determine the seepage quantities per meter length of the wall.

Soln:



Here, $N_f = 3$, $N_d = 6$, $h_L = 15$m

$K = 9 * 10^{-2}$ cm/sec

$= 9 * 10^{-4}$ m/sec

∴ $q = K \frac{h_L}{N_d} N_f$

$= 9*10^{-4} * \frac{15}{6} * 3 = 3.75 * 10^{-4}$ m^3/s/m

Ans

"Stress in soil"

* **Geostatic stress:** The stresses caused by gravity acting on the soil or rock is called geostatic stress, σ.

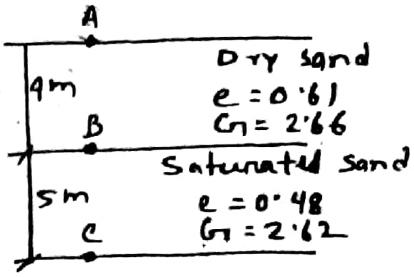
* **Induced stress:** The stress produced by the external loads onto the ground, is called induced stress.

* **Effective stress, σ':** It is the stress carried by the soil particles.

* **Total stress (σ):** It is the stress carried by the soil particles and the liquids and gases in the voids.

* **pore water pressure (u):** It is the pressure of the water held in the soil pores.

* A soil profile is shown in figure. Calculate the total stress, pore water pressure and effective stress at A, B & C ?



Soln: **At point A:**

Total stress, $\sigma = 0$

pore water pressure, $u = 0$

Effective stress, $\sigma' = 0$

At point B:

Total stress, $\sigma = \gamma_d H_1$

$$= \frac{G_s \gamma_w}{1+e} * H = \frac{2{\cdot}66 * 9{\cdot}8}{1+{\cdot}61} * 4 = 64{\cdot}76 \text{ KN/m}^2$$

pore water pressure, $u = 0$

Effective stress, $\sigma' = 64{\cdot}76 \text{ KN/m}^2$

At point C:

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

$$= 64{\cdot}76 + \frac{(G_s + e)\gamma_w}{1+e} * 5$$

$$= 64{\cdot}76 + \frac{(2{\cdot}62 + {\cdot}48) * 9{\cdot}8}{1+{\cdot}48} * 5$$

$$= 167{\cdot}16 \text{ KN/m}^2$$

pore water pressure, $U_c = H_2 \gamma_W = 5 \times 9.81$
$= 49.05\ KN/m^2$

Effective stress, $\sigma_0' = \sigma - u_c$
$= (169.16 - 49.05)\ KN/m^2$
$= 120.11\ KN/m^2$
Ans

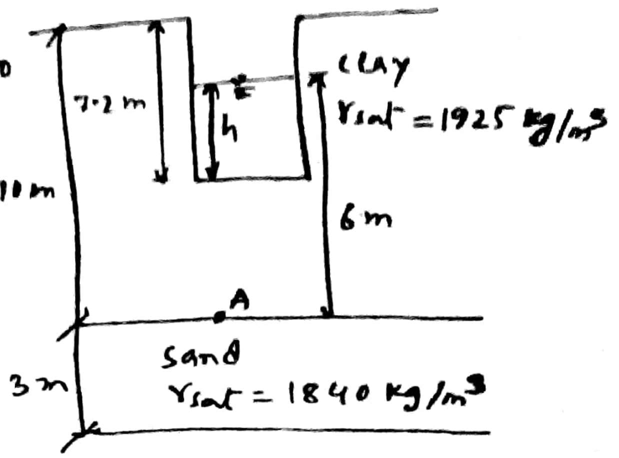
What would be the maximum height of water 'h' in the cut. so that the stability of the saturated clay is not lost?

sol^n:

$\sigma_A = (10-7.2)\gamma_{sat} + h\gamma_W$

⇒ ~~$\sigma_A = (10-7.2)*1925 + h*1000$~~

~~$= 15390\ kg/m^2$~~



⇒ $\sigma_A = (10-7.2) \times 1925 + h\gamma_W$

$u_A = 6\gamma_W$

For maximum cut,
$\sigma' = 0$

⇒ $\sigma_A - u_A = 0$

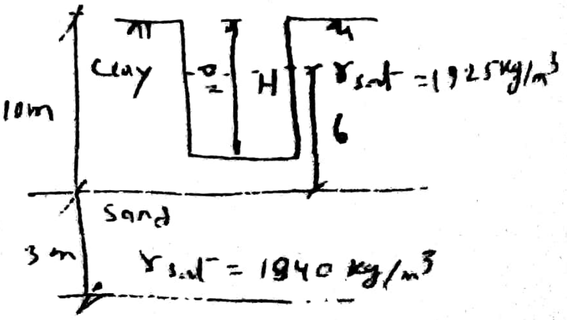
⇒ $(10-7.2)*1925 + h*1000 - 6*1000 = 0$

⇒ $h = 0.61\ m$

Ans

* 10m Clay underline by sand layer. γ_{sat} for sand is $1840\ kg/m^3$, γ_{sat} for clay $= 1925\ kg/m^3$. Determine the height of cut in the clay layer.

sol^n.



$\sigma' = (10-H)\gamma_{sat} - 6\gamma_W$

For max^m stable cut
$\sigma' = 0$

⇒ $(10-H) \times 1925 - 6 \times 1000 = 0$

$\therefore H = 6.88\ m$ Ans:

* Determine the total stress & effective stress of a swimming pool having 5m of water level from ground.

Soln:

Total stress, $\sigma = \sigma' + u$

$= 0 + \gamma_w h = 0 + 9.81 \times 5$

BUET

$= 49 \ KN/m^2$

Effective stress, $\sigma' = 0$

Ans.

* A sand layer has 8 ft depth, WT is at 2 ft depth. The sand layer is overlying on a clay layer of large depth. The moist (above WT) and saturated unit wt of of sand are 18 KN/m³, 20 KN/m³. The saturated unit wt of clay is 21 KN/m³. Find the total stress and effective stress (pressure) at 15 ft depth.

-16

Soln.

Total pressure, $\sigma = 18 \times 2 + 20 \times 6 + 21 \times 7$

$= 303 \ KN/m^2$

Effective pressure, $\sigma' = 18 \times 2 + (20 - 9.81) \times 6 + (21 - 9.81) \times 7$

$= 175.47 \ KN/m^2$

Ans.

"Soil Exploration"

Write down the laboratory and field test of soil.

Ans: Laboratory test of soil:

① Determination of Atterberg limit.
② Grain size Analysis.
③ compaction test.
④ permeability test.
⑤ consolidation test.
⑥ Unconfined consolidation test.
⑦ Direct shear test.
⑧ Unconfined compression test.

In-situ (Field) test of soil:

① Standard penetration Test (SPT)
② Cone [illegible] n test (CPT)
③ vane shear test (VST)
④ plate bearing test
⑤ pile load test.
⑥ permeability test.

purpose of soil exploration: The purposes for soil exploration are to obtain information as bases for:

- New structures
- Existing structures
- Highway & Airfields
-

SPT Test

 # N-value: The SPT sampler is driven into the sand by using 63.5 kg (140 lb) hammer with a fall distance of 0.76 m (30 in). The SPT hammer is driven at a total of 450 mm with the nos. of blows recorded for each 150 mm interval. 'N' value is defined as the penetration resistance of the sand which is equal to the sum of the number of blows required to drive the SPT sampler over the depth interval of 150 mm to 450 mm. No. of blows for first 150 mm is neglected to avoid sealing errors.

Why field N-value is to be corrected?

Ans: For standard penetration tests made at shallow depth, the number of blows is too low. But at a greater depth of the same soil with same relative density would give higher penetration resistance. Thus a correction is needed to find actual N-value from field N-value.

① **Submergence correction / Dilatency correction:** For very fine & silty sand situated below the water table -

$$N_{corrected} = 15 + \frac{1}{2}(N_f - 15) \text{ ; when } N > 15$$

② **Correction for overburden pressure:** The influence of overburden pressure on the standard penetration resistance is shown by the following equation -

$$N_{corrected} = N\left(\frac{50}{P+10}\right) \rightarrow \text{when p is in psi}$$

where, p = effective overburden pressure ≤ 40 psi

$$N_{corr} = N * 0.77 \log \frac{2000}{\sigma_v'} \rightarrow \sigma_v' \text{ is in } kN/m^2$$

write down the methods of soil exploration?

Solⁿ: - Trial pit method
- Boring method.
  - i) Hand Auger method.
  - ii) Mechanical Auger boring
  - iii) Wash boring.
  - iv) Rotary drilling method
  - v) percussion drilling method.

Undisturbed sample: The sample in which the natural property of soil doesn't change during sampling is known undisturbed sample. This sample used for determination of compressibility, permeability & shear stress, etc.

Disturbed sample: The sample in which the natural property of soil gets distured during sampling is known as disturbed sample. This sample used for determination of liquid limit, plastic limit, grain size distribution and specific gravity, etc.

Area Ratio: The degree of disturbance of sample collected by various method can be expressed by a term called the area ratio.

$$A_r = \frac{D_o^2 - D_i^2}{D_i^2} * 100$$

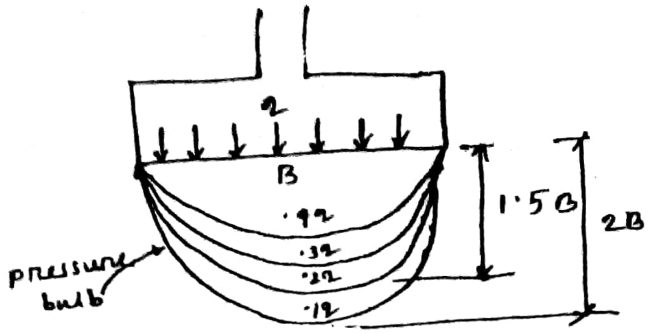
where,

D_o = outside dia of sampler

D_i = Inside dia of sampler.

A soil sample can generally be considered undisturbed if the area ratio is $\leq 10\%$.

 # Significant depth: The depth upto which the net loading intensity on the stress increment due to structural loading can produce perceptible contribution to settlement or shear failure may be called significant depth.



#

Sand (cohesionless)

N-value (No of blows/ft)	Relative Density
0 – 4	very loose
4 – 10	loose
10 – 30	Medium
30 - 50	Dense
>50	very dense

Clay (cohesive)

N-value	Consistency
0 - 2	very soft
2 – 4	soft
4 – 8	Medium
8 – 15	stiff
15–30	very stiff
>30	Hard.

#Vane shear test: This test is used to determine the shearing resistance of cohesive soil by applying direct shear. The apparatus consists of four-bladed vane fastened to the bottom of a vertical rod. The vane and the rod can be pushed into the soil without appreciably disturbing the materials. The assembly in then rotated and the torque required to turn the vane is measured. Since the soil fails along a cylindrical surface passing through the outer edges of the vane, the shearing resistance can be computed if the dimension of the vane and the torque are known. If the vane is rotated rapidly through several revolutions, the soil become remolded and the shearing stress can again be deter-mind. Not only shearing resistance but also the sensitivity of the clay.

Sensitivity of clay: sensitivity is the property of clay which is measured in terms of loss of shear strength due to remolding. It is measured as-

$$S_t = \frac{q_u\,(\text{undisturbed})}{q_u\,(\text{remolded/disturbed})}$$

where, q_u = unconfined compressive strength

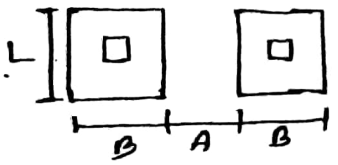
⇒ sensitivity of normal clay - 2 to 4.

Cone penetration test: One of the static penetro-meter is the Dutch cone in which 60° cone is used. The test is generally taken every 20cm of depth in soil strata. This is performed by pushing the outer tube connected to the cone-penetrometer to the desired testing depth. The inner rod is pushed then at 2cm during which the maximum force required is read from the gauges on the hydraulic load cell. Full extension of the cone required 7cm of movements. If a friction jacket cone is utilised the cone extends 3.5cm after which the friction jacket as well as the cone moves down-ward for another 3.5 cm. The readings for the friction jacket cone consists of first the cone resistance reading and then the cone resistance and friction jacket resistance reading. By subtracting cone resistance reading from cone & friction jacket

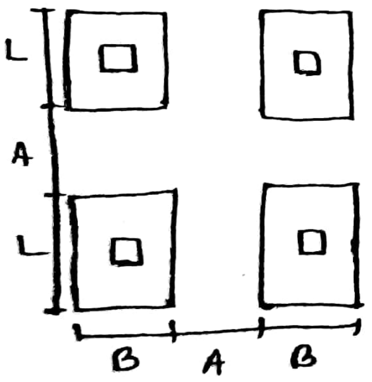
reading, a value of resistance is obtained for local friction. After that, the outer tube is then pushed to the next test point during which the cone and the friction are collapsed in a preparation of the next test.

Guide line for depth of boring :

① For isolated column footings $D = 1·5B$ if $A > 4B$



② For two adjacent footings –



Min^m $D = 1·5B$; $L \geq B$ & $A \geq 4B$

Min^m $D = 1·5L$; $L > B$ & $A < 2B$

③ For adjacent rows off footing

$D = 4·5B$ if $A \leq 2B$

$D = 3B$ if $A > 2B$

$D = 1·5B$ if $A \geq 4B$

④ $D = 1·5B$ for square footing

⑤ $D = 3B$ for strip footing.

⑥ For pile foundation –

$D = 1·5 \times$ the width of pile group.

Formula to calculate N'_{70} (standard N-value):

$$C_N = \left(\frac{95.76}{P_0}\right)^{1/2}$$

$$N'_{70} = C_N * N * \eta_1 * \eta_2 * \eta_3 * \eta_4$$

Where, $\eta_1 = \frac{E_r}{E_{rb}}$ $[E_{rb} = 70]$

Table for η_2:

Length	η_2
0~4 m	0.75
4~6 m	0.85
6~10 m	0.95
> 10 m	1.0

For η_3:

without liner → $\eta_3 = 1.0$

with liner:

i) Dense sand, clay , $\eta_3 = 0.80$

ii) loose sand , $\eta_3 = 0.90$

Table for η_4:

Hole dia	η_4
60~120 mm	1.0
150 mm	1.05
200 mm	1.15

* From a soil investigations, you have field N value of 25 at a level of 30′ depth below the existing ground surface. water table is at 15′ below the ground surface. Density of soil above the water table is 110 pcf and submerged density below the water table is 65 pcf. Determine the corrected N-value. The sub-soil is fine sand.

Soln:

$$N_{cor} = N * \frac{50}{P+10}$$

$$= 25 * \frac{50}{18.23+10}$$

$$= 44$$

Ans

$N = 25$

$$P = \frac{15*110+15*65}{144}$$

$$= 18.23 \text{ psi}$$

* The observed SPT value in a deposit of fully submerged fine silty sand was 45 at a depth of 6.5m. The average saturated unit weight of soil is 19.5 kN/m³. Find the corrected SPT value for dilateney and overburden effect.

Soln: $\sigma_v' = 19.5*6.5 - 9.81*6.5 = 62.9.85 \text{ kN/m}$

For overburden pressure:

$$N' = N * 0.77 \log \frac{2000}{\sigma_v'}$$

$$= 45 * 0.77 \log \frac{2000}{62.985}$$

$$= 52$$

Ans

For dilateney:

$$N' = 15 + \frac{1}{2}(N_f - 15)$$

$$= 15 + \frac{1}{2}(45 - 15)$$

$$= 30$$

Ans

Determine the standard N-value (N'_{70}) for the following data : N = 35, Rod length = 7.5 m, hole dia = 175 mm, $P_0' = 150$ kPa, use safety hammer with Er = 75, very stiff clay with liner.

Soln:

$$C_N = \left(\frac{95.76}{P_0'}\right)^{1/2} = \left(\frac{95.76}{150}\right)^{1/2} = 0.799$$

$$\eta_1 = \frac{E_r}{E_{rb}} = \frac{75}{70} = 1.07$$

$\eta_2 = 0.95$ for rod length 7.5 m

$\eta_3 = 0.80$ [very stiff clay with liner]

$\eta_4 = 1.10$ [for 175 mm dia]

$$N'_{70} = C_N * N * \eta_1 * \eta_2 * \eta_3 * \eta_4$$

$$= 0.799 * 35 * 1.07 * .95 * .80 * 1.10$$

$$= 25$$

Ans.

During the vane shear test, 20 N-m torque is applied by vane sh[illegible] apparatus. If the height of vane is 400 mm and a[illegible] f vane is 200 mm. Detn the undrained shear strength of the soil.

Ans: T = 20 N-m, H = 0.4 m, D = 0.2 m

$$\cancel{S_u = \frac{T}{\pi\left(D^2\frac{H}{2} + D^{1/2}\right)} = \frac{20}{\pi\left(0.2^2 * \frac{.4}{2} + \right.}}$$

$$S_u = \frac{T}{\pi\left(D^2 * \frac{H}{2} + D^3/6\right)} = \frac{20}{\pi\left(.2^2 * \frac{.4}{2} + \frac{(.2)^3}{6}\right)}$$

$$= 682 \text{ N/m}^2$$

Ans.

'Earth pressure'

#Earth pressure at rest: The lateral earth pressure is is called at-rest pressure when the soil mass is not subjected to any lateral yielding or movement. This case occurs when the retaining wall is firmly fixed at its top and is not allowed to rotate or move laterally

#Active earth pressure: A state of active pressure occurs when the soil mass yields in such way that it tends to stretch horizontally.

Ex → When retaining wall moves away from backfill.

#passive earth pressure: A state of passive pressure exists when the movement of the wall is such that the soil tends to compress horizontally.

Ex – When wall moves towards the soil

#Draw the pressure diagram for the following cases of cohesionless backfill and show the pressure intensity at the bottom.

a) Dry or moist backfill with no surcharge

b) submerged backfill

c) Backfill with uniform surcharge.

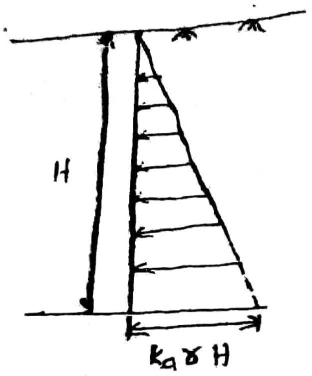
d) water stands to the both sides of the wall.

e) If the backfill is partially submerged.

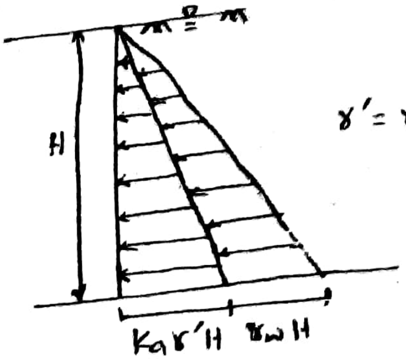
f) partially submerged backfill with ϕ_1 & ϕ_2.

g) partially submerged backfill with $\phi_2 > \phi_1$, $\gamma_1 < \gamma_2$

h)

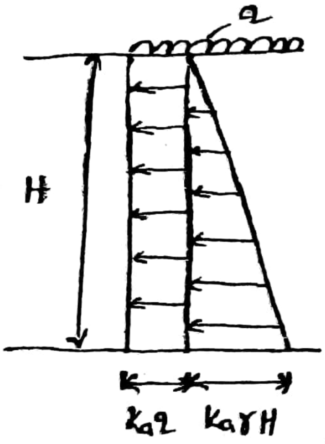


(a)

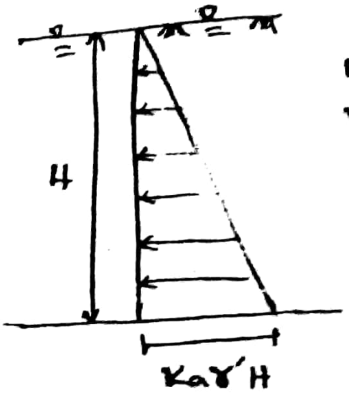


$\gamma' = \gamma_{sat} - \gamma_w$

(b)

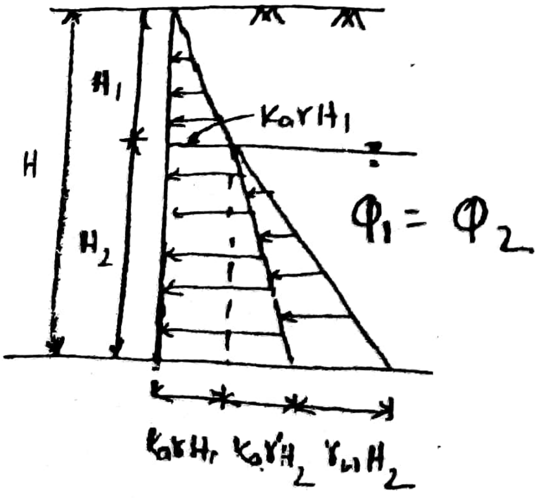


(c)

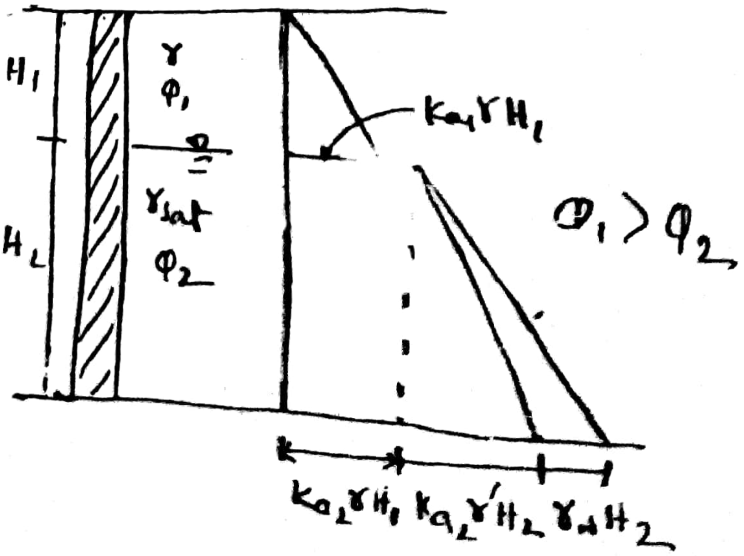


water effect vanished from both side.

(d)



(e)



(f)

- At rest condition, K

- At active condition, Ka

- At passive condition, Kp

For cohesive soil without surc

For active pressure:

$P_a = K_a \gamma H - 2c\sqrt{K_a}$

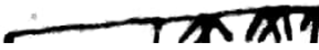
Depth of tensile crack,

$$z_c = \frac{2c}{\gamma\sqrt{K_a}} = \frac{2c}{\gamma} \, [\phi = 0]$$

Critical depth,

$$H_c = 2z_c = \frac{4c}{\gamma\sqrt{K_a}}$$ ← H_c পর্য without

For passive pressure:



For cohesive soil with surcharge:

$$P_a = Ka(\gamma H + q) - 2c\sqrt{Ka}$$

$$P_p = Kp(\gamma H + q) + 2c\sqrt{Kp}$$

Total active force before tension crack occur

$$Fa = \frac{1}{2} Ka \gamma H^2 - 2c\sqrt{Ka}\, H$$

Total active force after tension crack

$$Fa = \frac{1}{2}(Ka\gamma H - 2c\sqrt{Ka}) * (H - Zc)$$

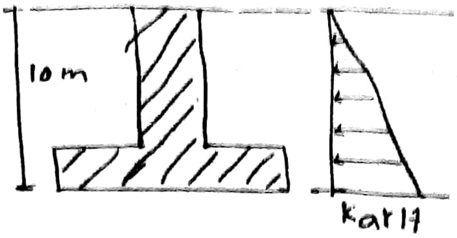
Total passive pressure/force

$$Fp = \frac{1}{2} Kp \gamma H^2 + 2c\sqrt{Kp}\, H$$

* Determine the total active thrust on a vertical retaining wall 10m height if the soil has following properties, $\phi = 35°$ and $\gamma = 19$ kN/m³

Sol^n:

$K_a = \tan^2(45-\phi/2) = 0.27$



Total active thrust.

$F_a = \frac{1}{2} K_a \gamma H * H$

$= \frac{1}{2} * 0.27 * 19 * 10 * 10$

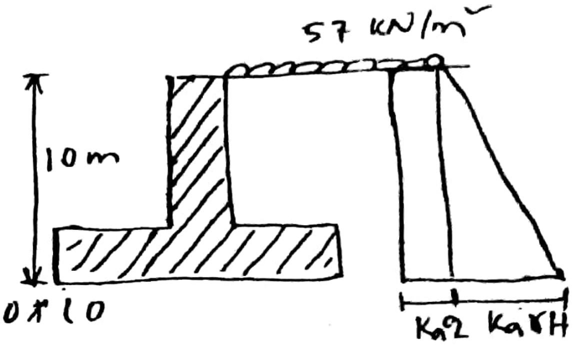
$= 256.5$ kN per m width.

Ans.

* Determine the total active thrust on a vertical retaining wall 10 m height, which is subjected to uniform surcharge 57 kN/m² on horizontal surface of soil and if the soil retaining has following properties, $\phi = 35°$ & $\gamma = 19$ kN/m³.

Sol^n:

$K_a = \tan^2(45 - \phi/2) = 0.27$



Total active thrust -

$F_a = K_a q H + \frac{1}{2} K_a \gamma H * H$

$= 0.27 * 57 * 10 + \frac{1}{2} * .27 * 19 * 10 * 10$

$= 410.4$ kN per m of width,

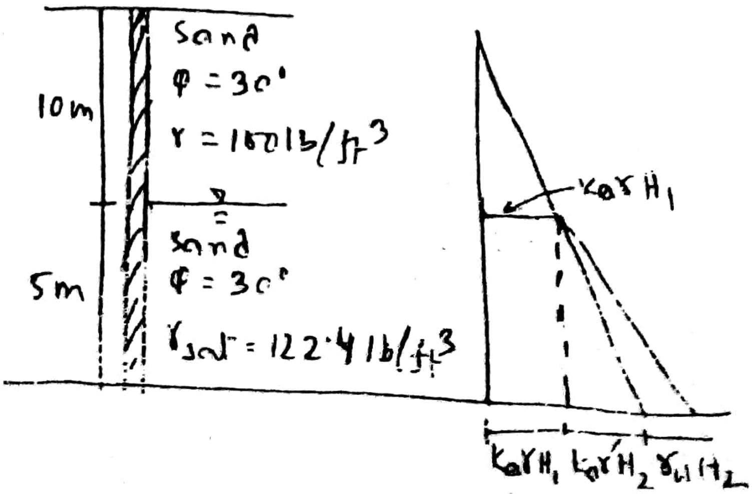
Ans

* Find the total thrust at rest for the following figure.

Sol^n:

~~$K_a = \tan^2(45 - \phi/2) =$~~

$K_0 = (1 - \sin\phi) = 0.5$



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11

$$\text{Total thrust} = \frac{1}{2} K_o \gamma H_1 * H_1 + K_o \gamma H_1 * H_2 + \frac{1}{2} K_o \gamma' H_2 * H_2 + \frac{1}{2} \gamma_w H_2 * H_2$$

$$= \frac{1}{2} * \frac{1}{2} * 100 * 10^2 + \frac{1}{2} * 100 * 10 * 5 + \frac{1}{2} * \frac{1}{2} * (122.4 - 62.4) * 5^2$$
$$+ \frac{1}{2} * 62.4 * 5^2$$

= 6155 lb per ~~foot~~ ft of width.

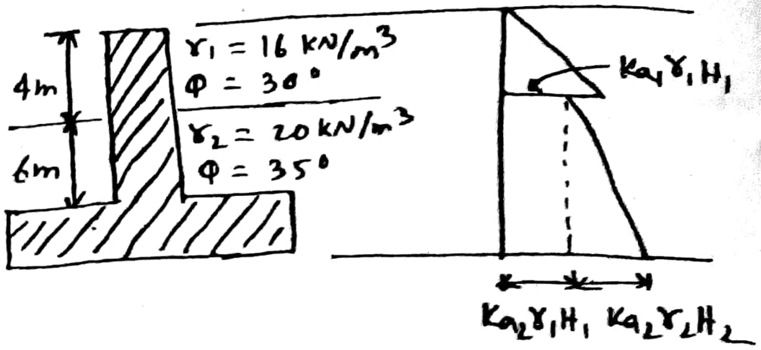
Ans

* Compute the total thrust and point of action on a retaining wall of 10 m height considering two layer of soil shown in figure?

soln:

$K_{a1} = \tan^2(45 - \frac{30}{2}) = .33$

$K_{a2} = \tan^2(45 - \frac{35}{2}) = .27$



$$\therefore \text{Total thrust} = \frac{1}{2} * K_a \gamma_1 H_1^2 + K_{a2}\gamma_1 H_1 H_2 + \frac{1}{2} * K_{a2}\gamma_2 H_2^2$$

$$= \frac{1}{2} * .33 * 16 * 4^2 + .27 * 16 * 4 * 6 + \frac{1}{2} * .27 * 20 * 6^2$$
$$= 42.24 + 103.68 + 97.2$$
$$= 243.12\ kN/m$$

let, total thrust acts at a distance x from bottom.

$\therefore$ we get,

$$\frac{1}{2} * K_{a1}\gamma_1 H_1^2 * (6 + 4/3) + K_{a2}\gamma_1 H_1 H_2 * \frac{6}{2} + \frac{1}{2} K_{a2}\gamma_2 H_2^2 * \frac{6}{3}$$
$$= 243.12 * x$$

$$\Rightarrow x = \frac{42.24 * (6 + 4/3) + 103.68 * 3 + 97.2 * 2}{243.12}$$

= 3.35 ft from the bottom.

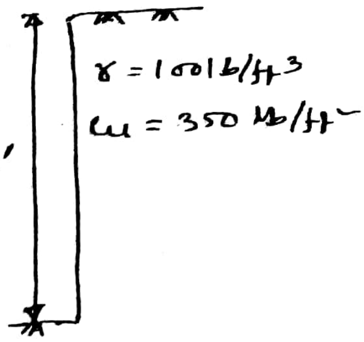
* A retaining wall that has a soft saturated clay backfill is shown in figure for the undrained condition $(\phi=0)$ of the backfill. Determine -

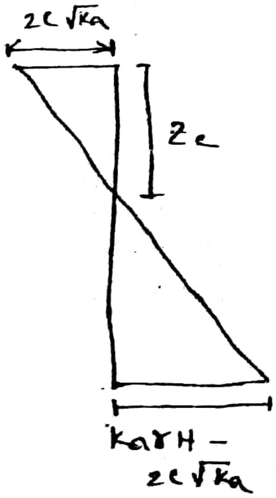
i) Maximum depth of tensile crack.

ii) Pa before the tensile crack occurs

iii) Pa after tensile crack occurs.

Soln:





$$K_a = \tan^2\left(45-\frac{0}{2}\right)$$
$$= 1$$

(i) $$z_c = \frac{2c_u}{\gamma\sqrt{K_a}}$$
$$= \frac{2*350}{100\sqrt{1}} = 7\ ft$$

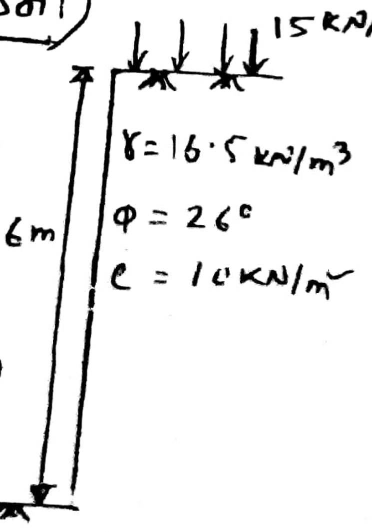
(ii)
$$P_a = \frac{1}{2}K_a\gamma H^2 - 2c\sqrt{K_a}\,H$$
$$= \frac{1}{2}*1*100*20^2 - 2*350*\sqrt{1}*20$$
$$= 6000\ lb/ft.$$

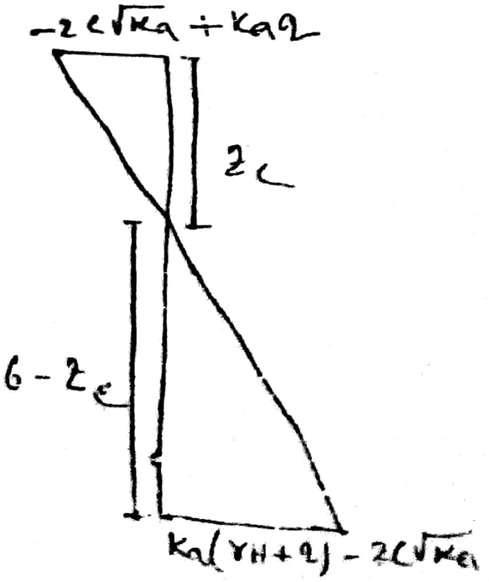
(iii) $$P_a = \frac{1}{2}\left(K_a\gamma H - 2c\sqrt{K_a}\right)*(H-z_c)$$
$$= \frac{1}{2}\left(1*100*20 - 2*350*\sqrt{1}\right)*(20-7)$$
$$= 8450\ lb/ft.$$

* A frictionless retaining wall is shown in figure. Determine the active force, Pa after the tensile crack occurs? (c−φ soil)

Important

Soln:





$$K_a = \tan^2\left(45-\frac{26}{2}\right) = 0.39$$

$$P_a = K_a(\gamma H + q) - 2c\sqrt{K_a}$$

at $h = 0$

$$P_a = \cdot 39(0+15) - 2*10\sqrt{\cdot 39}$$
$$= -6.64\ kN/m^2$$

at $h = 6$ m.

$P_a = .39\,(16.5*6+15) - 2*10*\sqrt{.39}$

$= 31.97$ KN/m^2

<u>find z_c:</u>

$$\frac{6.64}{z_c} = \frac{6.64+31.97}{6}$$

$\Rightarrow z_c = 1.03$

$\therefore$ The active force, $F_a = \frac{1}{2}\left(K_a(\gamma H+q) - 2c\sqrt{K_a}\right)*(H - z_c)$

$= \frac{1}{2}\left(.39\,(16.5*6+15) - 2*10\sqrt{.39}\right)*(6-1.03)$

$= 79.44$ KN/m

<u>Ans</u>.

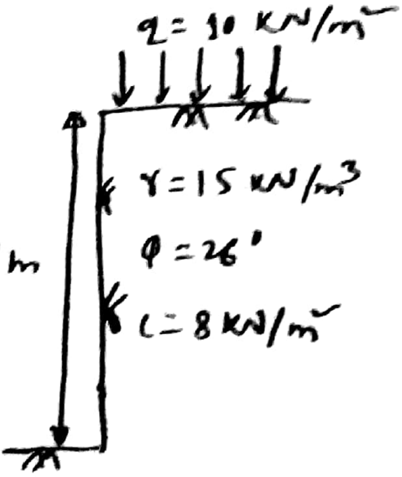
* A friction-less retaining wall is shown in figure. Find the passive resistance on the backfill and the location of the resultant passive force.

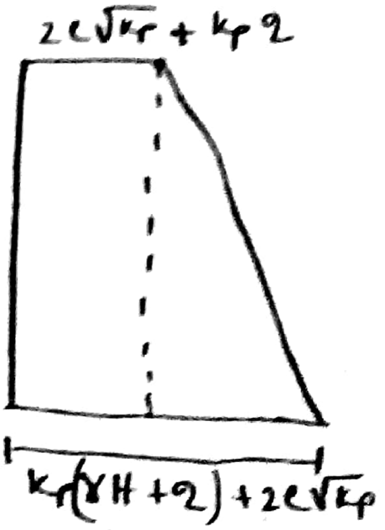
Important

<u>Soln:</u>

$K_p = \tan^2(45+\phi/2)$

$= 2.56$





$\therefore F_p = \frac{1}{2}\left(K_p(\gamma H+q) + 2c\sqrt{K_p}\right\} + 2c\sqrt{K_p} + K_p q)*H$

$= \frac{1}{2}\left(2.56\,(15*4+10) + 4*8*\sqrt{2.56}\right) + 2.56\times10)*4$

$= 512$ KN/m

Let, location of resultant is x m above bottom.

$\therefore 2c\sqrt{k_p} + k_p q = 51.2 \text{ kN/m}^2$

$k_p(\gamma H + q) + 2c\sqrt{k_p} = 204.5 \text{ kN/m}^2$

$\therefore 51.2 * 4 * 2 + \frac{1}{2}(204.5 - 51.2) * 4 * \frac{4}{3} = 512 * x$

$\Rightarrow x = 1.6$ m ~~from bottom~~ above bottom.

* A pipe is to be laid in a purely cohesion (cohesive) soil having undrained cohesion $c_u = 30$ kpa. Calculate the max^m^ depth upto which a vertical trench can be excavated in the soil without providing any lateral support?

Soln:

$$H = \frac{4c}{\gamma\sqrt{k_a}} = \frac{4\times30}{20\sqrt{1}} = 6\text{m}$$

Ans.

$\gamma = 20 \text{ kN/m}^2$, $c = 30 \text{ kN/m}^2$

$\varphi = 0$

$k_a = \tan^2(45 - \varphi/2)$

$= 1$

* Calculate the lateral force on a wall of 15' height with sand back. $\gamma = 115$ pcf, $\varphi = 30°$

Soln:

At rest, $k_o = 1 - \sin\varphi = 0.5$

$\therefore$ Total pressure $= \frac{1}{2}k_o\gamma H \times H$

$= \frac{1}{2}\times.5\times115\times15^2$

$= 6468.75$ lb/ft

confusion

$\therefore$ lateral force $= 6468.75\times15$

$= 97031.25$ lb

Ans.

* The unconfined compression of the soil is 50 kpa. Determine the depth of excavation without any lateral support.

Solⁿ: let, $\phi = 30°$, $\gamma = 16\ kN/m^3$

cohesion, $c = \frac{50}{2} = 25$ kpa.

Buet

$$\therefore H = \frac{4c}{\gamma\sqrt{Ka}} \qquad \Big|\ Ka = \tan^2(45 - \phi/2) = 0.33$$

$$= \frac{4\times25}{16\sqrt{.33}}$$

$$= 10.88\ m$$ Ans.

"pile Foundation"

Meyerhof's method for estimating Q_p:

① In sand: $(c=0)$

End bearing, $Q_p = A_p q' N_q^* \leq A_p q_1$

where,

$q_1 = 0.5 P_a N_q^* \tan\phi'$

$\hookrightarrow$ atmospheric pressure $= 100 \text{ kN/m}^2$

ϕ' = effective soil friction angle

Note: N_q^* is find with respect to ϕ from table.

② In clay: $(\phi=0)$

End bearing, $Q_p = c N_c A_p$

$= 9c A_p$

where, $N_c = 9$

c = Undrained cohesion of the soil below the tip of pile

A_p = Area of tip

$= \frac{\pi}{4} D^2$ (if circular)

Skin Friction (Q_s):

① Clay:

$Q_s = \alpha_2 c A_{surface}$

$= \alpha_2 c (\pi D L)$

where, Q_s = skin friction

α_2 = Reduction factor (~)

$c = \frac{q_u}{2}$

L = Length of embedment.

D = Dia of pile.

Ultimate Capacity in clay.

① $Q_u = Q_p + Q_s$

$= 9cA_p + \alpha_2 c(\pi DL)$

② $Q_u = Q_s$ [when clay soil is too soft]

$= \alpha_2 c \pi DL$

Capacity of pile Group & cluster:

The bearing capacity of a pile cluster can be determined based on two types of action-

① Individual Action } whichever is less
② Group Action

Individual Action:

Bearing capacity of pile Group = No. of piles × bearing Capacity of single pile.

Group Action:

$Q_u = Q_p + Q_s$

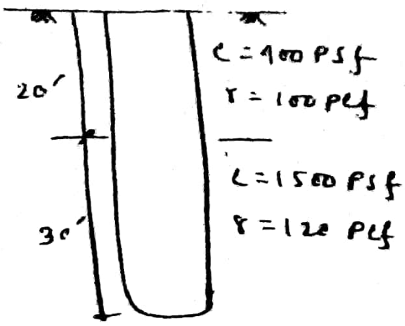
$= 9cA_p + \alpha_2 c A_{surface}$

Note: For group action $\alpha_2 = 1\cdot 0$

$A_{surface} = 4(\Sigma \text{ Pile Spacing c/c + pile size})L$

$A_p = (\text{pile end to end distance})^2$

* Determine the axial capacity of a drilled pier having length of 50', diameter of 2'.



soln:

let, $\alpha_1 = 0.55$ [AASHTO specification]

0' – 20'

$Qs_1 = \alpha_1 c \, A_{surface}$

$= 0.55 \times \frac{400}{1000} \times (\pi D L)$

$= 0.55 \times .4 \times (\pi \times 2 \times 20')$

$= 27.6 \, k$

20' – 50'

$Q_{s_2} = \alpha_1 c \, A_{surface}$

$= 0.55 \times \frac{1500}{1000} \times (\pi \times 2 \times 30)$

$= 155.5 \, K$

Note: pier-এর জন্য α_1 use করব।

$\alpha_1 = 0.3 \sim 0.5$

$= 0.55$ by AASHTO

Again,

$Q_p = 9 c \, A_p$

$= 9 \times \frac{1500}{1000} \times \frac{\pi}{4} \times (2)^2$

$= 42.4 \, K$

$\therefore Q_u = Q_{s_1} + Q_{s_2} + Q_p$

$= 225.5 \, K$

consider a F.S = 2.5

$\therefore Q_{all} = \frac{Q_u}{F.S.} = 90.2 \, K$ Ans

* Find the ultimate capacity of the 16"x16" square pile having a length of 50'

Soln:

0'-20':

$c = \frac{400}{1000} = 0.4$ ksf.

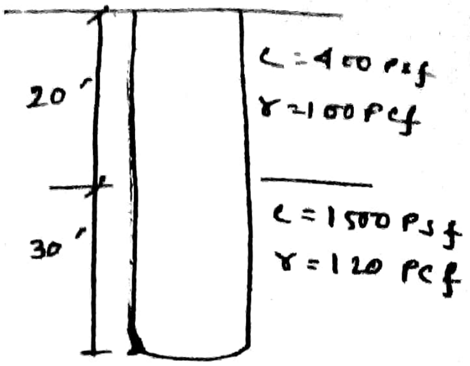
$q_u = 2c = 0.8$ KSf $= 0.4$ TSf

For $q_u = 0.4$ TSf ; $\alpha_2 = 0.96$

$Q_{s_1} = \alpha_2 c A_{surface}$

$= 0.96 \times .4 \times (4 \times \frac{16}{12}) \times 20$

$= 40.96$ K



20'-50'

$c = \frac{1500}{1000} = 1.5$ ksf

$q_u = 2c = 3$ ksf $= 1.5$ Tsf.

For, $q_u = 1.5$ Tsf, $\alpha_2 = 0.68$

$\therefore$ $Q_{s_2} = \alpha_2 c A_{surface}$

$= 0.68 \times 1.5 \times (4 \times \frac{16}{12}) \times 30$

$= 163.2$ K

$Q_p = 9cA_p$

$= 9 \times \frac{1500}{1000} \times (\frac{16}{12})^2$

$= 24$ K

$\therefore$ $Q_u = Q_{s_1} + Q_{s_2} + Q_p$

$= 228.16$ K

Ans.

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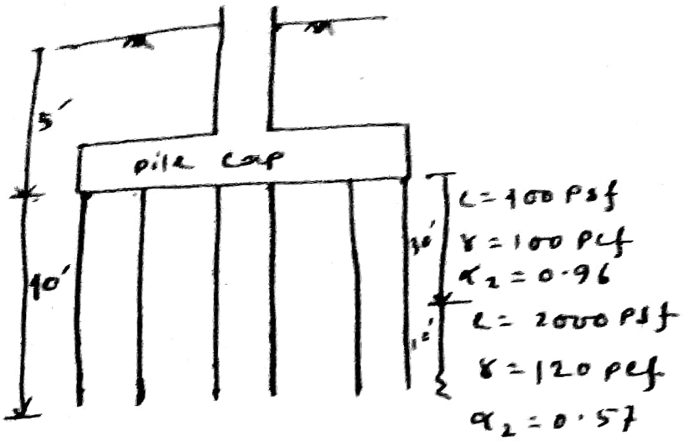
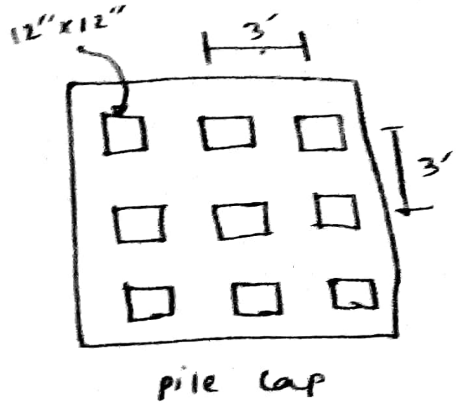
* Find the ultimate capacity of the group of piles shown in figure -





Soln:

Individual Action:

$$Q_s = (\alpha_2 C A_{surface})_{(5'\sim35')} + (\alpha_2 c A_{surface})_{(35'\sim45')}$$

$$= 0.96 \times \frac{400}{1000} \times \left(4 \times \frac{12}{12}\right) \times 30 + .57 \times \frac{2000}{1000} \times \left(4 \times \frac{12}{12}\right) 10$$

$$= 91\,68\ K$$

$$Q_p = 9 C A_p$$

$$= 9 \times \frac{2000}{1000} \times \left(\frac{12}{12}\right)^2$$

$$= 18 K.$$

$$\therefore Q_u = (91.68 + 18) * 9 = 987.12 K$$

Group Action: $\alpha_2 = 1.0$

$$A_{surface} = 4\left\{(3+3) + \frac{12}{12}\right\} * L$$

$$= 28L$$

$$\therefore Q_s = \alpha_2 C A_{surface}$$

$$= 1 * \frac{400}{1000} * 28 * 30 + 1 * \frac{2000}{1000} * 28 * 10$$

$$= 896 K$$

$Q_p = 9 c A_p$

$= 9 \times \frac{2000}{1000} \times (3+3+1)$

$= 882\ K$

$\therefore Q_u = Q_s + Q_p = 1778\ K$

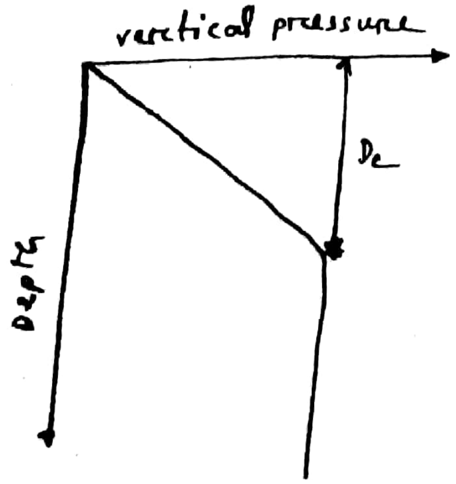
so, ultimate pile capacity, $Q_u = 987.12\ K$

Ans.

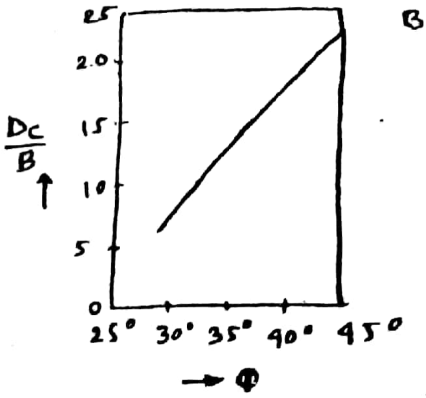
End bearing, $Q_p = q N_q A_p$

where, q = Effective vertical pressure at the pile tip for depth D_c

N_q = Bearing capacity factor for deep foundations.



(a)



B= pile tip width

(b)

Frictional resistance / skin friction,

$Q_s = K \tan\delta$ (area of σ_v diagram) x pile perimeter.

∴ Ultimate bearing capacity,

$$Q_u = Q_p + Q_s$$

Static methods for bored pile in sand:

$$Q_p = q N_q A_p$$

Q_s =

* A concrete pile, 30 cm diameter, is driven into a mediu dense sand ($\varphi = 35°$, $\gamma = 21\ KN/m^2$, $K = 1.0$, $\tan\delta = 0.70$) for a depth of 8 m. Estimate the safe load, F.S = 2.50, $N_q = 60$.

Soln:

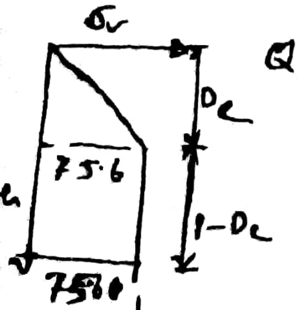
for $\varphi = 35°$, $\frac{D_c}{B} = 12$ [From fig (b)] | $B = \frac{30}{100} = 0.3\ m$

$\Rightarrow D_c = 12 \times 0.3 = 3.6m$

$\therefore$ Maximum value of $\sigma_v = 21 \times 3.6 = 75.6\ KN/m^2$

$\therefore Q_p = q N_q A_p$

$= 75.6 \times 60 \times \frac{\pi}{4} \times (.3)^2 = 320.5\ KN.$



$Q_s = K \tan\delta$ (Area of σ_v diagram) x pile perimeter

$= 1 \times 0.70 \left(\frac{1}{2} \times 75.6 \times 3.6 + 75.6 \times 4.4\right) \times (\pi \times 0.3)$

$= 309.2\ KN.$

$\therefore Q_u = Q_p + Q_s = 629.7\ KN.$

$\therefore$ safe load, $= \frac{Q_u}{F.S} = 251.9\ KN$

Ans.

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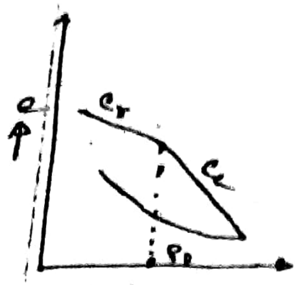
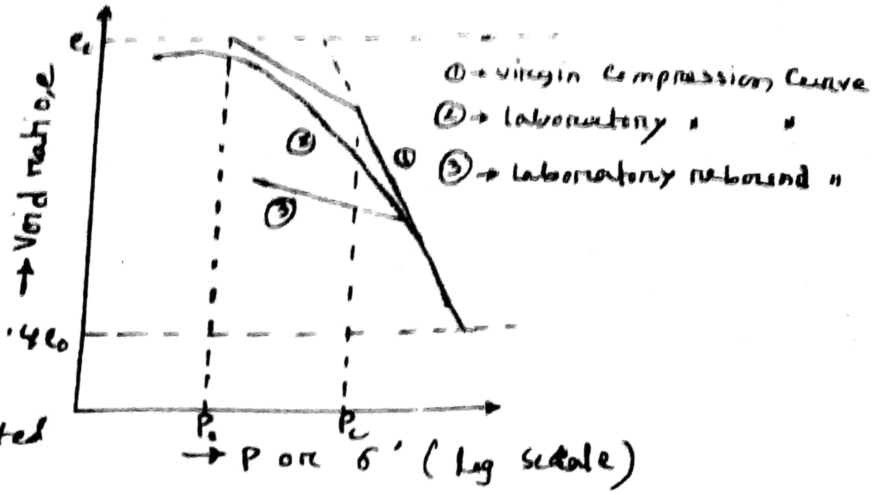
② OCC soil:



(ii) over consolidated clay



Settlement Equation:

NCC soil ($\sigma_0' = \sigma_c'$)	OCC soil
$S_c = \frac{C_c H}{1+e_0} \log \frac{\sigma_0' + \Delta\sigma'}{\sigma_0'}$ C_c = Compression index = 0.009 (LL − 10) [undisturbed] = 0.007 (LL − 10) [disturbed] H = clay layer thickness e_0 = Initial void ratio σ_0' = effective overburden pressure $\Delta\sigma'$ = Increase of effective pressure.	Case-1: $(\sigma_0' < \sigma_0' + \Delta\sigma' \leq \sigma_c')$ $S_c = \frac{C_s H}{1+e_0} \log \frac{\sigma_0' + \Delta\sigma'}{\sigma_0'}$ C_s = Swell index ≈ $\frac{1}{5}$ to $\frac{1}{10}$ of C_c σ_c' = preconsolidation pressure Case-2: $(\sigma_0' < \sigma_c' < \sigma_0' + \Delta\sigma')$ $S_c = \frac{C_s H}{1+e_0} \log \frac{\sigma_c'}{\sigma_c'} +$ $\frac{C_c H}{1+e_0} \log \frac{\sigma_c' + \Delta\sigma'}{\sigma_c'}$

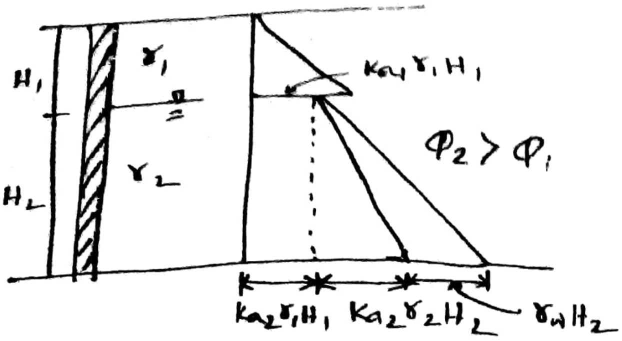
$C_c = \frac{e_1 - e_2}{\log \frac{\sigma_1'}{\sigma_2'}}$ when e_1, e_2 given

σ_1' → বড়টা always

* OCR (over Consolidation Ratio) = $\frac{\text{preconsolidation pressure}}{\text{Current pressure}}$

OCR > 1 or B. $\sigma_0' < \sigma_c'$, soil is OCC

OCR < 1 or $\sigma_0' \geq \sigma_c'$, soil is NCC



(8)

Earth pressure co-efficient:

- At rest condition, $K_o = 1-\sin\varphi$
- At active condition, $K_a = \frac{1-\sin\varphi}{1+\sin\varphi} = \tan^2(45^\circ - \frac{\varphi}{2})$
- At passive condition, $K_p = \frac{1+\sin\varphi}{1-\sin\varphi} = \tan^2(45^\circ + \frac{\varphi}{2})$

For cohesive soil without surcharge:

For active pressure:

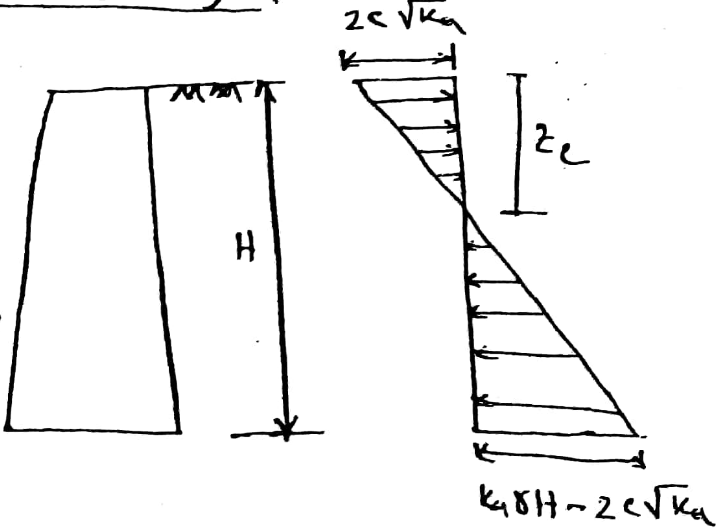
$P_a = K_a \gamma H - 2c\sqrt{K_a}$

Depth of tensile crack,

$$z_c = \frac{2c}{\gamma\sqrt{K_a}} = \frac{2c}{\gamma} \; [\varphi = 0]$$

Critical depth,

$$H_c = 2 z_c = \frac{4c}{\gamma\sqrt{K_a}}$$ ← H_c পর্যন্ত excavation করা যাবে without lateral support



For passive pressure:

$P_p = K_p \gamma H + 2c\sqrt{K_p}$