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SOIL MECHANICS

Numerical Solution

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TUTORIAL-1

For a soil in natural state, given $e = 0.8$, $\omega = 24\%$ and $G_s = 2.68$, Determine the moist unit weight, dry unit weight and degree of saturation. If the soil is made completely saturated by adding water, what would its moisture content be at that time? Also find saturated unit weight.

→ Solution:-

Given that:-

$e = 0.8$

$\omega = 24\%$

$G_s = 2.68$

Moist unit weight (γ) = ?

dry unit weight (γ_d) = ?

Degree of saturation (S) = ?

We know that,

$$\gamma = \frac{(G + Se)\gamma_\omega}{1+e} \quad \ldots\ldots (1)$$

$\because Se = \omega G$

$$\therefore S = \frac{\omega G}{e} = \frac{\frac{24}{100} * 2.68}{0.8} = 0.804$$

Now, eqn (1) becomes,

$$\gamma = \left(\frac{2.68 + 0.804 * 0.8}{1 + 0.8}\right) * 9.81 * 1000$$

$$= 18.11 \text{ kN/m}^3.$$

Also,

$$\gamma_d = \frac{G\gamma_\omega}{1+e} = \frac{2.68 * 9.81 * 1000}{1 + 0.8} = 14.606 \text{ kN/m}^3$$

And,

Degree of saturation (s) = $\frac{wG}{e} * 100\%$

$= 0.804 * 100\%$

$= 80.4\%$

Case II:-
If the soil is made completely saturated by adding water, then,
moisture content (w) = ?
Saturated unit weight (γ_{sat}) = ?
We have,

$w = \frac{Se}{G} = \frac{1*0.8}{2.68} = 0.2985 = 29.85\%$

[∵ S=1 for saturated soil]

and,

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

$= \frac{(2.68+0.8) * 9.81 * 1000}{1+0.8}$ [∵ S=1]

$= 18.97 \text{ kN/m}^3$

Hence, the moisture content is 29.85% at that time and saturated unit weight is 18.97 kN/m³.

08(F) PH.

2. A soil sample in its natural state has a mass of 2.29kg and volume of $1.15*10^{-3} m^3$. Under an oven-dried state, the dry mass of the sample is 2.035kg. The specific gravity of the solids is 2.68. Determine the total density, water content, void ratio, porosity, degree of saturation and air void ratio.

Solution:-
Given that,

Mass of soil sample (m_s) = 2.29kg.
Volume of soil sample (V) = $1.15 * 10^{-3} m^3$
Dry mass of the sample (m_d) = 2.035kg.
Specific gravity of soil (G_s) = 2.68
Total density (ρ) = ?
Water content (w) = ?
Void ratio (e) = ?
Porosity (n) = ?
Degree of saturation (s) = ?
Air void ratio (n_a) = ?
We know that,

$\rho = \frac{m_s}{V} = \frac{2.29}{1.15*10^{-3}} = 1991.30 \text{ kg/m}^3$

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

$= \frac{2.29 - 2.035}{2.035} * 100\%$

$= 12.53\%$

Now,

$e = \frac{V_v}{V_s}$ ..... (1)

But $G = \frac{\gamma_s}{\gamma_w} \Rightarrow \gamma_s = G\gamma_w = 2.68 * 1000$
$= 2680 \text{ kg/m}^3$

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

or, $V_s = \frac{W_s}{\gamma_s} = \frac{2.035}{2680} = 7.593 * 10^{-4} m^3$

∴ $V_v = V - V_s$

$= 1.15*10^{-3} - 7.593*10^{-4} = 3.907*10^{-4} m^3$

from eqn. (1),

$$e = \frac{3.907 * 10^{-4}}{7.593 * 10^{-4}} = 0.514$$

$$n = \frac{e}{1+e} = \frac{0.514}{1+0.514} = 0.3398 * 100\% = 33.98\% \simeq 34\%$$

Also,

$$S = \frac{wG}{e} = \frac{12.53 * 2.68}{0.514} = 65.33\%$$

$$n_a = n_{ac} = n(1-s) = 0.3398(1-0.6533) = 0.1178 = 11.78\%$$

011(F) Additional practice

Q. A $5*10^{-5}$ m^3 specimen of a moist soil weigh 0.932 N, its dry weight is 0.735 N and the sp. gravity of soil solid is 2.68. Compute water content, void ratio, porosity and degree of saturation.

→ Solution: -

Given that,

Volume of specimen (v) = $5*10^{-5}$ m^3.

Weight of moist soil (W_m) = 0.932 N

Weight of dry soil (W_d) = 0.735 N

Sp. gravity of soil (G_s) = 2.68

Water content (w) = ?

Void ratio (e) = ?

Porosity (n) = ?

Degree of saturation (s) = ?

We know that,

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

$$= \frac{(0.932 - 0.735)}{0.735} * 100\%$$

=

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

$$Se = Gw$$

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

$$\rho_d = M/V = ?$$

$$\rho = \frac{M}{V} = \frac{0.932}{5 \times 10^{-5}} \ N/m^3$$

$$\rho_d = \frac{\rho}{1+w}$$

3. The wet density of soil is 2gm/cc. The specific gravity of solids is 2.7 and the moisture content of the soil is 15%. Calculate a) Dry density b) Porosity c) Void ratio d) Degree of saturation e) % of air voids.

→ Solution:-

Given that,

Wet density of soil (ρ) = 2 gm/cm³

Specific gravity of solids (G_s) = 2.7

Moisture content of the soil (w) = 15%

a) Dry density (ρ_d) = ?

b) Porosity (n) = ?

c) void ratio (e) = ?

d) Degree of saturation (S) = ?

e) % of air voids (n_a) = ?

We know that,

a) $\rho_d = \frac{\rho}{1+w} = \frac{2}{1+0.15} = 1.74$ gm/cm³.

We have,

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

or, $1.74 = \frac{2.7*1}{1+e}$

or, $1+e = \frac{2.7}{1.74} \Rightarrow e = 0.5517$

b) $n = \frac{e}{1+e} = \frac{0.5517}{1+0.5517} = 0.3555$

$= 35.55\%$

c) $e = 0.5517$

d) $Se = wG$

or, $S = \frac{wG}{e} = \frac{0.15*2.7}{0.5517} = 0.734$

e) $n_a = na_c$

$= n(1-S)$

$= 0.3555(1-0.734)$

$= 0.0945$

$\approx 9.45\%$

A pycnometer weighing 620 gm was used in the following measurements on samples 'A' and 'B' of the same soil. Sample A was oven dried and 'B' was completely saturated. Weight of pycnometer when filled with water only is 1495 gm. Determine:

	A	B
Sample		
Weight of sample only (gm)	980	1020
Weight of Pycnometer full of sample & water	2112	2030

i) The specific gravity of the soil.

ii) The water content & void ratio of sample B.

→ Solution:-

Given that,

Mass of pycnometer (m_1) = 620 gm

$m_2 = (980+620)$ gm = 1600 gm

$m_3 = 2112$ gm

$m_4 = 1495$ gm

i) Sp. gravity of soil (G) = ?

ii) w = ? & e = ?

We know that,

Mass of dry soil $(m_s) = m_2 - m_1$
$= 1600 - 620$
$= 980$ gm.

And,

i) $G = \frac{m_s}{m_s - (m_3 - m_4)} = \frac{980}{980 - (2112 - 1495)} = 2.7$

ii) For sample B,

$Se = wG$

or, $e = \frac{wG}{S} =$

$w = \frac{m_w}{m_s} =$

5. A partially saturated soil sample from earth fill has a natural moisture content of 20% and bulk unit weight of 1.8 gm/cm³. The specific gravity of solid is 2.7. Determine the degree of saturation and void ratio. If the sample gets saturated subsequently, determine its unit weight.

→ Solution:-

Given that,

Natural moisture content $(w) = 20\%$

Bulk unit weight $(\gamma_t) = 1.8$ gm/cm³

Specific gravity of solid $(G) = 2.7$

Degree of saturation $(S) = ?$

Void ratio $(e) = ?$

We know that,

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

or, $1.8 = \gamma_d (1 + 0.2)$

or, $\gamma_d = 1.5$

And,

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

or, $1 + e = \frac{2.7 * 1}{1.5}$

or, $e = 0.8$.

Now,

$Se = wG$

or, $S = \frac{20 * 2.7}{80} = 0.675$

$= 67.5\%$.

Case II:-

When the sample gets saturated subsequently, $S = 1$.

~~or, $1*e = 0.2*2.7$~~

~~or, $e = 5.4$~~

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

$= \frac{(2.7 + 0.8) * 1}{(1 + 0.8)}$

$= 1.94$ gms/cc.

The unit weight is 1.94 gms/cc when the sample gets saturated subsequently.

6. A 50cc volume of moist soil weights 95gm. Its dry weight is 75gm and specific gravity of solids is 2.68. Compute the water content, void ratio, porosity and unit weight.

→ Solution:-

Given that,

Volume (V) = 50cc

Weight of moist solid (m) = 95gm

Weight of dry solid (m_s) = 75gm

Specific gravity (G_s) = 2.68

Water content (ω) = ?

Void ratio (e) = ?

Porosity (n) = ?

Unit weight (γ) = ?

We know that,

i) $\omega = \frac{m_\omega}{m_s} = \frac{m - m_s}{m_s} = \frac{95-75}{75} * 100\% = 26.67\%$

ii) $\gamma = \frac{m}{V} = \frac{95}{50} = 1.9$ gm/cc.

We have,

$\gamma_d = \frac{\gamma}{1+\omega} = \frac{1.9}{1+26.67} = 1.5$ gm/cc.

We have,

$\gamma_d = \frac{G\gamma_\omega}{1+e}$

or, $1.5 = \frac{2.68 * 1}{1+e} \Rightarrow e = 0.786$

and, $n = \frac{e}{1+e} = \frac{0.786}{1+0.786} = 0.44$ Ans.

Rakesh Sah

8. A cube of dried clay having sides 4cm long has a mass of 110gm. The same cube of soil, when saturated at unchanged volume has a mass of 135gm. Draw a phase diagram showing the consistents of volumes and weights and then determine the specific gravity of soil solids and the void ratio.

→ Solution:-

Given that -

Mass of dry clay (m_s) = 110gm.

Mass of saturated clay (m_{sat}) = 135gm.

Specific gravity of soil (G_s) = ?

Void ratio (e) = ?

Side of cube (l) = 4cm

∴ Volume of cube (V) = $4^3 = 64cm^3$

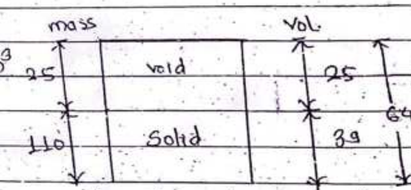


fig. Phase diagram.

Since the volume remains constant before and after saturation,

mass of water = 135 - 110 = 25 gm.

Volume of water in void = $\frac{25 gm}{1 gm/cc}$ = 25cc.

∴ Volume of void = 25cc.

$G = \frac{\gamma_s}{\gamma_\omega} = \frac{2.82}{1} = 2.82$

$\gamma_s = \frac{m_s}{V_s} = \frac{110}{39} = 2.82$ gm/cc.

and,

$e = \frac{V_v}{V_s} = \frac{25}{39} = 0.64$ Ans.

8. A sample of sand has porosity of 40% and specific gravity of its solid is 2.75. Find out the dry unit weight, saturated unit weight, when fully saturated. {Submerged unit weight and saturated unit weight, when the degree of saturation is 50%.

→ Solution:-

Given that,

Porosity of sand (n) = 40%

Specific gravity (Gs) = 2.75

Dry unit weight (Wd) = ? } S = 100%

Saturated unit weight (Wsat) = ? }

Submerged unit weight (Wsub) = ? } S = 50%

Saturated unit weight (Wsat) = ? }

We know that,

$\gamma_d =$

γ_d

γ_{sat}

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

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

9.81 kN/m³ = 10 gm/cc

γ_{sat}

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

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

or, $0.40 = \frac{e}{1+e}$ or, $0.40 + 0.40e = e$ or, $e = 0.67$

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

or, $0.40 = \frac{e}{1+e}$ ⇒ $e = 0.67$

Now,

$\gamma_d = \frac{G\gamma_w}{1+e} = \frac{2.675*1}{1+0.67} = 1.65$ gm/cc.

Also,

$\gamma_{sat} = \frac{(G+e)\gamma_w}{(1+e)} = \frac{(2.75+0.67)*1}{1+0.67} = 2.05$ gm/cc.

Case II

$\gamma_{sat} = \frac{(G+Se)\gamma_w}{1+e} = \frac{(2.75+0.5*0.67)*1}{1+0.67} = 1.85$ gm/cc

and

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

$= 1.85 - 1$

$= 0.85$ gm/cc.

9. Prove that:-

i) $S = \frac{w}{\frac{\gamma_w(1+w)}{\gamma} - \frac{1}{G_s}}$

> Solution:-

We have,

$\gamma_d = \frac{\gamma}{1+w}$ .... (1)

$\gamma_d = \frac{G\gamma_w}{1+e}$ .... (2)

Equating (1) & (2), we get,

$$\frac{\gamma}{1+w} = \frac{G\gamma_w}{1+e}$$

or, $\gamma(1+e) = G\gamma_w(1+w)$ $\quad \left[\because e = \frac{Gw}{S}\right]$

or, $1 + \frac{wG}{S} = \frac{G\gamma_w(1+w)}{\gamma}$

or, $\frac{wG}{S} = \frac{G\gamma_w(1+w)}{\gamma} - 1$

or, $\frac{w}{S} = \frac{\gamma_w(1+w)}{\gamma} - \frac{1}{G}$

or, $S = \dfrac{w}{\frac{\gamma_w(1+w)}{\gamma} - \frac{1}{G_s}}$ proved.

11) $\gamma_d = (1-n)G\gamma_w$

→ Solution:-

We have,

$V = V_a + V_w + V_s$

or, $V = V_v + V_s$ $\quad [\because V_v = V_a + V_w]$

or, $1 = \frac{V_v}{V} + \frac{V_s}{V}$

or, $1 = n + \frac{V_s}{V}$

or, $\frac{V_s}{V} = 1 - n$

or, $\frac{M_s}{\gamma_s} \cdot \frac{1}{V} = 1 - n$ $\quad \left[\because \gamma_d = \frac{m_s}{V}\right]$

or, $\gamma_d \times \frac{1}{G\gamma_w} = 1 - n$

or, $\gamma_d = (1-n)G\gamma_w$ proved.

10. In a field density, the mass of soil removed from a small hole was 1.59 kg. The volume of the hole is 900cm³. The field water content was found to be 25%. If the density of the soil is 2700kg/m³, determine the dry density and degree of saturation of the fill.

→ Solution:-

Given that,

mass of soil (M) = 1.59 kg

Volume of hole (V) = $900\,cm^3 = 900 \times 100^{-3} = 9\times10^{-4}\,m^3$

water content (w) = 25% = 0.25

Density of soil (ρ) = 2700 kg/m³

Dry density (ρ_d) = ?

We have,

$$M_s = \frac{M}{1+w} = \frac{1.59}{1+0.25} = 1.272$$

And,

$$\rho_d = \frac{M_s}{V} = \frac{1.272}{9\times10^{-4}} = 1413.33\,kg/m^3.$$

Degree of saturation (S) = ?

$$V_w = \frac{W_w}{\rho_w} = \frac{M - M_s}{1000} = \frac{1.59 - 1.272}{1000} = 3.18\times10^{-4}$$

& $V_v = V - V_s$

$= 9\times10^{-4} - \frac{1.272}{2700} = 4.288\times10^{-4}$

$$\therefore S = \frac{V_w}{V_v}\times100 = \frac{3.18\times10^{-4}}{4.288\times10^{-4}}\times100\% = 74.16\%$$

Rakesh Sah

11 A sample of wet soil has a volume of $0.0192\ m^3$ and a mass of 32 kg. When the sample is dried out in an oven, its mass reduces to 28.5 kg. Determine i) dry density ii) void ratio iii) porosity iv) Degree of saturation. Take specific gravity of solids as 2.65.

⇒ Solution:-

Given that:-

mass of wet soil (M) = 32 kg.

Volume of wet soil (V) = $0.0192\ m^3$.

Mass of dry soil (M_d) = 28.5 kg.

i) Dry density (ρ_d) = ?

ii) Void ratio (e) = ?

iii) Porosity (n) = ?

iv) Degree of saturation (S) = ?

We have,

$M_s + M_w = M$

or, $M_s + wM_s = M$

or, $M = M_s(1+w)$ ....(1)

∴ $M_w = M - M_s = 32 - 28.5 = 3.5$ kg.

Now,

$V_w = \frac{M_w}{\rho_w} = \frac{3.5}{1000} = 3.5 \times 10^{-3}\ m^3$

and, $V_s = \frac{M_s}{G\rho_w} = \frac{28.5}{2.65 \times 1000} = 0.01075$

∴ $V_v = V - V_s$

$= 0.0192 - 0.01075$

$= 8.445 \times 10^{-3}\ m^3$

Now,

i) $\rho_d = \frac{M_s}{V} = \frac{28.5}{0.0192} = 1484.375\ kg/m^3$

ii) $e = \frac{V_v}{V_s} = \frac{8.445 \times 10^{-3}}{0.01075} = 0.78$

iii) $n = \frac{V_v}{V} \times 100 = \frac{8.445 \times 10^{-3}}{0.0192} \times 100 = 43.98\%$

iv) $S = \frac{V_w}{V_v} \times 100 = \frac{3.5 \times 10^{-3}}{8.445 \times 10^{-3}} \times 100 = 41.44\%$

12). A soil sample with specific gravity of solids 2.65 has a bulk (mass) specific gravity of 1.8. Determine the void ratio assuming the soil to be perfectly dry.

⇒ Solution:-

Given that,

Sp. gravity of solids (G_s) = 2.65

Bulk sp. gravity ( ) = 1.8

Void ratio (e) = ?

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

or, $1.8 = \left(\frac{2.65}{1+e}\right) \times 1$

or, $1.8 + 1.8e = 2.65$

or, $e = \frac{2.65 - 1.8}{1.8}$

$= 0.4722$

Hence, the void ratio was found to be 0.47.

13) Two soil C and D are mixed dry in proportion of 35% : 65% by mass. The specific gravity of soil C and D are 2.65 : 2.75 respectively. If the bulk density of mixed soil is 1.7 gm/cc at 15% water content, determine void ratio and degree of saturation.

→ Solution:-

Given that,

$$\frac{\text{Mass of C}}{\text{Mass of D}} = \frac{35}{65}$$

$$\frac{G_C}{G_D} = \frac{2.65}{2.75}$$

Bulk density of mixed soil (ρ_{mix}) = 1.7 gm/cc

water content (w) = 15%

void ratio (e) = ?

Degree of saturation (S) = ?

We know that,

$$G_{mix} = \frac{m_C G_C + m_D G_D}{m_C + m_D}$$

$$= \frac{0.35 \times 2.65 + 0.65 \times 2.75}{0.35 + 0.65} = 2.715$$

Also,

$$(\rho_{mix})_d = \frac{\rho_{mix}}{1+w} = \frac{1.7}{1+0.15} = 1.478$$

And, $(\rho_{mix})_d = \frac{G \rho_w}{1+e}$

or, $1.478 = \frac{2.715 * 1}{1+e}$

$\therefore e = 0.83$

And, $Se = Gw$

or, $S * 0.83 = 2.715 * 0.15$

$\therefore S = 0.4906 \simeq 49.06\%$

14. A natural soil deposit has a bulk unit weight of 16 kN/m³ and water content of 11%. Determine the amount of water required to be added to 1 m³ of soil to raise the water content to 18%. Assume void ratio to remain constant.

→ Solution:-

Given that,

Bulk unit weight (γ) = 16 kN/m³

water content (w) = 11%

Volume of soil (V) = 1 m³

final water content (w_f) = 18%

Amount of water required to be added (V_w) = ?

We have,

$$\gamma_d = \frac{\gamma}{1+w} = \frac{16}{1+0.11} = 14.41 \text{ kN/m}^3.$$

$\therefore$ Weight of dry soil $(W_s)_1 = \gamma_d * V = 14.41 * 1 = 14.41$ kN

And,

$W_w = w W_s = 0.11 * 14.41 = 1.58$ kN.

But,

$\gamma_w = \frac{W_w}{V_w} \Rightarrow (V_w)_1 = \frac{W_w}{\gamma_w} = \frac{1.58}{9.81} = 0.161 \text{ m}^3$

Similarly,

for w = 18%,

$$\gamma_d = \frac{\gamma}{1+w} = \frac{16}{1+0.18} = 13.56 \text{ kN/m}^3.$$

$\therefore (W_s)_2 = 13.56$ kN

$W_\omega = \omega W_s = 0.18 \times 13.56 = 2.44\,KN.$

and,

$(V_\omega)_2 = \frac{W_\omega}{\gamma_\omega} = \frac{2.44}{9.81} = 0.248\,m^3.$

Now,

amount of water to be added $(V_{\omega}) = (V_\omega)_2 - (V_\omega)_1$

$= 0.248 - 0.161$

$= 0.087\,m^3$

Hence, $0.087m^3$ of water is required to be added to $1\,m^3$ of soil to raise the water content to 18%.

15. A soil has a liquid limit 25%, flow index 12.5% and the plastic limit 15%. Determine the plasticity index and toughness index if water content of the soil in the natural condition in the field is 20%. find the liquidity index and relative consistency.

→ Solution:-

Given that,

Liquid limit $(LL) = 25\%$

Flow index $(I_f) = 12.5\%$

Plastic limit $(P.L) = 15\%$

Plasticity Index $(P.I) = ?$

Toughness Index $(I_T) = ?$

water content $(\omega_n) = 20\%$

Liquidity index $(LI) = ?$

Relative consistency $(I_C) = ?$

We have,

$P.I = LL - P.L$

$= 25\% - 15\%$

$= 10\%$

$I_t = \frac{P.I}{I_f} = \frac{10}{12.5} = 0.8$

$I_C = \frac{L.L - \omega_n}{P.I} = \frac{25\% - 20\%}{10\%} = 0.5$

and, $L.I = \frac{\omega_n - P.L}{P.I} = \frac{20\% - 15\%}{10\%} = 0.5$

16. Soil A and B were saturated in their natural condition and they have the same specific gravity. The water content and index properties of the soils are in the following table:

Soil	A	B
Liquid limit(%) (L.L)	62	62
Plastic limit (%) (P.L)	26	40
Natural water content (%)	27	60

Which of these soils:

a) Has greater liquidity index?

→ Solution:- $P.I = L.L - P.L =$

For soil A, $(P.I)_A = 36\%$ $(P.I)_B = 22$

$(L.I)_A = \frac{\omega_n - P.L}{P.I} = \frac{27 - 26}{36} = 0.0278$

$(L.I)_B = \frac{60 - 40}{22} = 0.90$

$\therefore (L.I)_B > (L.I)_A$

b) Is softer in its natural condition. (1.c) (L-I)

Here,

~~$(I_C)_A = 26$~~

Since,
$(L.I)_B > (L.I)_A$
So, soil B is softer in its natural condition.

iii) Has greater plasticity index?
→ Here,
$(P.I)_A = 36\%$
$(P.I)_B = 22\%$
$\therefore (P.I)_A > (P.I)_B$

iv) Is more cohesive? (P.I)
$(P.I)_A = 36\%$
$(P.I)_B = 22\%$
$\therefore (P.I)_A > (P.I)_B$.
So, soil A is more cohesive than soil B

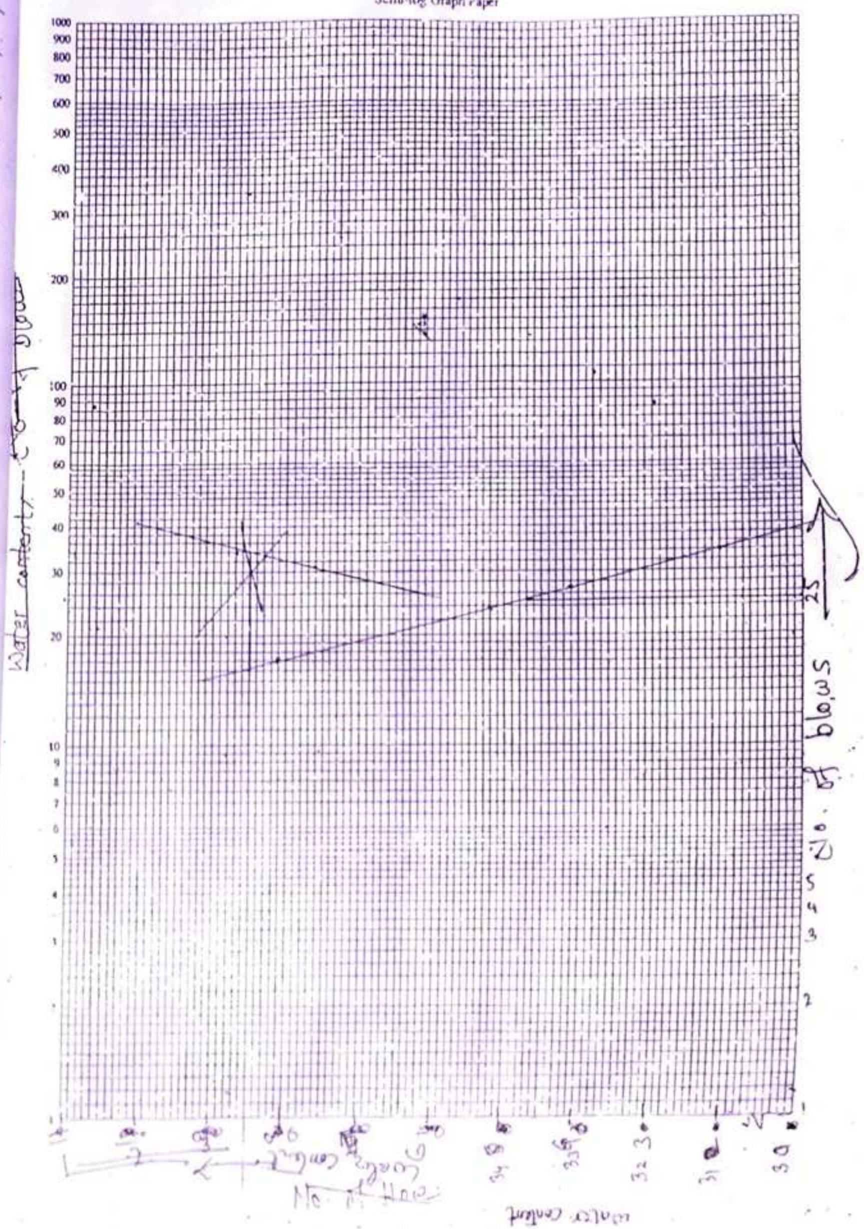
(c)

v) Has greater void ratio in its natural ~~st~~ conditions? Give reasons to your answers
→ $e_A = \frac{wG}{S_r} = \frac{27 \times 2.72}{1} = 0.734$

$e_B = \frac{wG}{S_r} = \frac{60 \times 2.72}{1} = 1.63$

So, the soil B has greater void ratio in its natural condition

Semi-log Graph Paper



17. The following data has been obtained from liquid and plastic limit tests on the soil in the laboratory.

Details	Liquid limit test data				Plastic limit test data	
Mass of container with wet soil	43.39	47.62	45.24	45.58	43.62	40.38
" " " " dry "	40.18	43.61	41.87	41.5	41.94	38.85
Mass of container	29.86	31.5	32.01	30.51	33.1	30.9
No. of blows (N)	34	27	23	17		

Determine:-

a) Liquid limit, plastic limit and flow index.

b) Classify the soil as per USCS

c) Liquidity index if the natural water content of soil is 28.9%

⇒ Solution:-

Details	L.L test limit				P.L. test limit	
Mass of wet soil (gm)	43.39	47.62	45.24	45.58	43.62	40.38
dry	40.18	43.61	41.87	41.5	41.94	38.85
Mass of container	29.86	31.5	32.01	30.51	33.1	30.9
No. of blows (N)	34	27	23	17		
Mass of water (m_w)	3.21	4.01	3.37	4.08	1.68	1.53
Dry mass of soil (m_s)	10.32	12.11	9.86	10.99	8.84	7.95
water content, $w = \frac{m_w}{m_s} \times 100\%$	31.1%	33.1	34.2%	37.1	19%	19.2

Now,

L.L = 33.5% (from Graph) (water content corresponding to 25 blows)

Now,

S.No	Mass of container (g)	Mass of container with wet soil (g)	Mass of container with dry soil (g)	M_w (g)	M_s (g)	w (%)
1	33.1	43.62	41.94	1.68	8.84	19
2.	30.9	40.38	38.85	1.53	7.95	19.25

And,

Plastic limit = $\frac{19+19.25}{2}$ = 19.13%

Plasticity index = (34-19.13)% = 14.87

→ Flow index (I_f) = $\frac{w_2 - w_1}{\log(N_2/N_1)}$ = ~~$\frac{19.25-19}{\log}$~~ $\frac{33.1 - 34.1}{\log(27/34)}$ =

For liquid limit of 34%, the plasticity index from the A line equation is

P.I. = 0.73 (34-20)

P.I. = 10.22%

Since $I_{P.I}$ < 50% and I_p = 14.87% the point lies above A line. From USCS classification, the soil is classified as clay of low plasticity (CL).

Liquidity index = $\frac{w_n - P.L}{P.I} \times 100\%$ = $\frac{0.289 - 0.1913}{0.148} \times 100\%$ = 66.01%

Additional Practise (2016 Fall) :- Solution same as Q.N. 17.

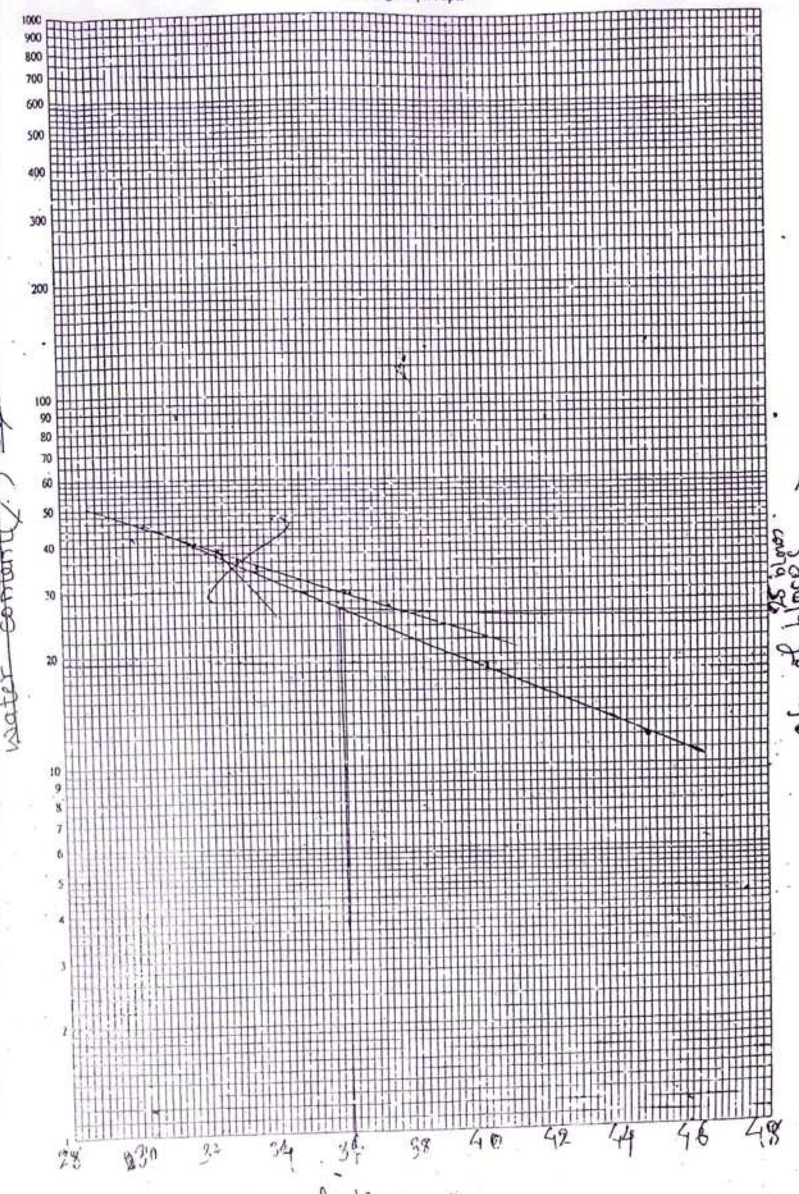
Additional practise (2008 Fall, 2015 Fall)

In order to determine the liquid limit of silty clay, the following results were recorded.

Mass of container with wet soil, (gm)	19.62	21.26	19.5	21.28
" " " dry soil, (gm)	16.74	18.39	17.24	19.08
Mass of container (gm)	10.46	11.71	10.87	11.62
No. of blows (N)	12	19	28	41

Plot the result in graph paper and determine the liquid limit for

Semi-log Graph Paper



the soil. If the plastic limit for the soil was 22% and natural water content 35%, find the plasticity index, liquidity index and consistency index.

> Solution:-

Given that:-

Plastic limit (PL) = 22%.

Natural water content (ω) = 35%

Now,

water Content (%)	$\frac{(19.62-16.74)}{(16.74-10.46)}$ = 45.86	$\left(\frac{21.26-18.39}{18.39-11.21}\right)$ = 39.97	$\left(\frac{19.5-17.24}{17.24-10.87}\right)$ = 35.48	$\left(\frac{21.28-19.08}{19.08-11.62}\right)$ = 29.49
No. of blows	12	19	28	41

Now, plotting the graph between no. of blows Vs water content, we have,

From graph, water content at 25 blows = 37%.

∴ Liquid limit = 37%

Now,

Plasticity index (PI) = L.L - PL = (37 - 22) = 15%.

Liquidity index (LI) = $\frac{\omega_n - PL}{PI} = \frac{35-22}{15}$ = 86.67%

and Consistency index (CI) = $\frac{LL - \omega_n}{PI} = \frac{37-35}{15}$ = 13.33%

Additional practice (2009 Spring)

The results of a liquid limit test are given below:

No. of blows (N)	48	38	29	20	14
water content (%)	32.1	35.9	40.7	46.1	52.8

i) Determine the liquid limit for the soil

ii) If the plastic limit for the soil be 23%, find the plasticity index, flow index and toughness index. Hence comment the nature of the soil.

→ Solution -

a) From graph, we have,
water content corresponding to 25 blows = 43%

∴ Liquid limit (LL) = 43%

Plastic limit (PL) = 23%

Now,

b) Plasticity index (PI) = LL − PL = (43 − 23)% = 20%

Toughness index $(I_t) = \frac{PI}{I_f}$

flow index $(I_f) = \frac{\omega_2 - \omega_1}{\log(N_2/N_1)} = \frac{\omega_{10} - \omega_{100}}{\log(100/10)}$

from graph,

$\omega_{10} = 54\%$ & $\omega_{100} = 16\%$

$\therefore I_f = \frac{54-16}{1} = 38\%$

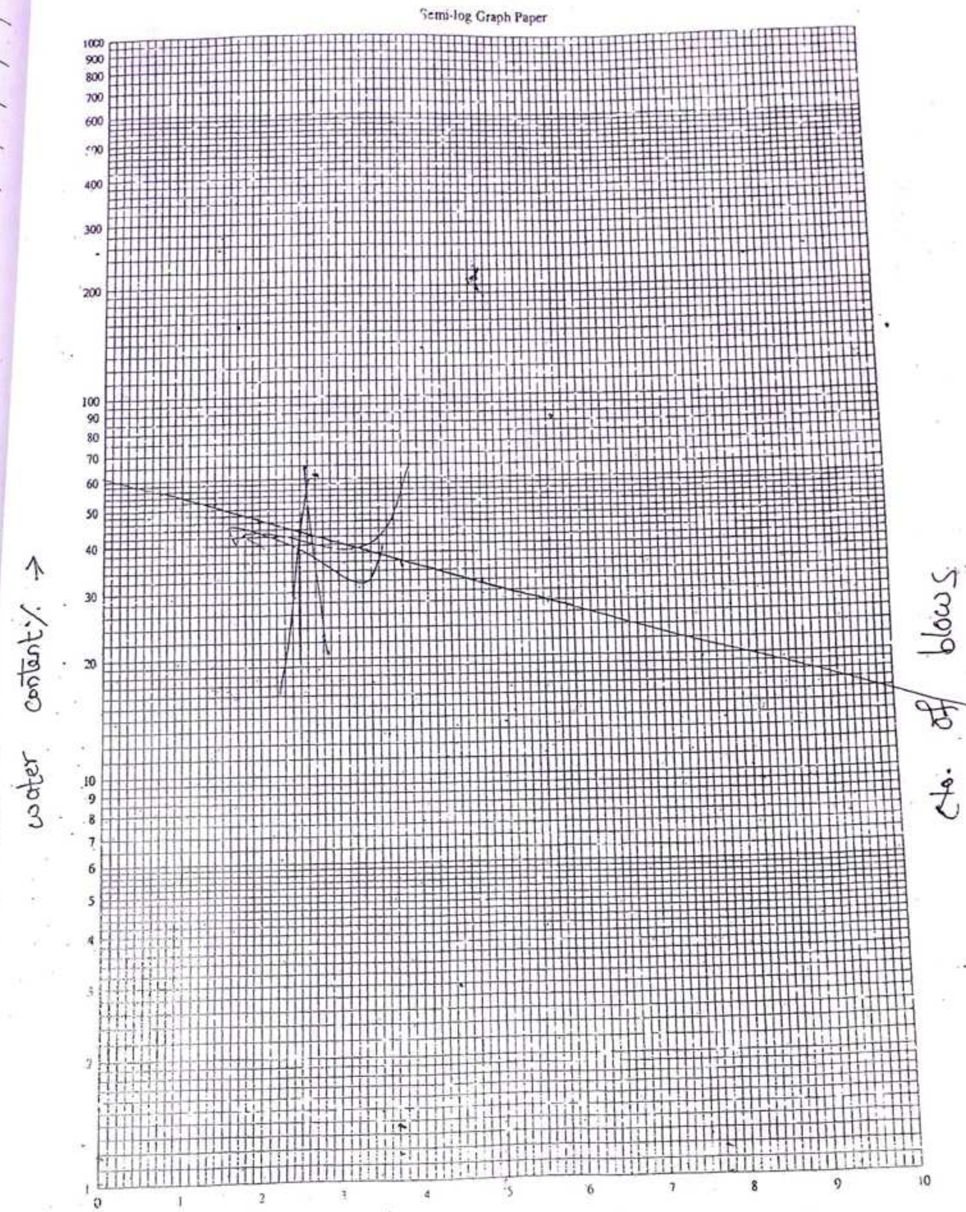
& $I_t = \frac{20}{38} = 0.53$

18. Two soils were tested for their consistency in the lab. following data were obtained.

Soil A		Soil B	
No. of blows (N)	water content (%)	No. of blows (N)	water content w%
8	43	5	65
20	39	15	61
30	37	30	59
45	25	40	58
water content at which soil crumbles at 3mm ϕ	25	water content at which soil crumbled at 3mm diameter	30%
Natural water content % (40%)		Natural 50%	

Determine: ..

R.K. SHAH.



R.K. SAH

19. For the sieve analysis of following grain size distribution has been obtained:

Sieve No.	4	10	20	40	100	200	Pan
Sieve opening (mm)	4.75	2	0.85	0.425	0.15	0.075	
Soil retained (gm)	100	180	120	140	160	200	100

a) Determine the percent finer in each sieve.
b) Without drawing the grain size distribution curve, determine the coefficient of uniformity and coefficient of curvature.
c) According to the USCS should the soil be classified as sand or Gravel?

→ Solution:-

a)

Sieve no.	Sieve Opening (mm)	Soil retained (gm)	Percent retained	Comm. % retained	% Passing
4	4.75	100	10	10	90
10	2	180	18	28	72
20	0.85	120	12	40	60
40	0.425	140	14	54	46
100	0.15	160	16	70	30
200	0.075	200	20	90	10

From the above table, we have $D_{10} = 0.075$, $D_{30} = 0.15$mm and $D_{60} = 0.85$mm.

b) $\therefore C_u = \frac{D_{60}}{D_{10}} = \frac{0.85}{0.075} = 11.33$

$$C_c = \frac{D_{30}^2}{D_{60} * D_{10}} = \frac{(0.15)^2}{0.85 * 0.075} = 0.353.$$

c) As more than 50% is retained on 0.075mm sieve, the soil is coarse grained. Coarse fraction = 90%, Gravel fraction = 10% and sand fraction = 80%. As more than half of the coarse fraction is smaller than 4.75mm sieve, the soil is sand.

20. The sieve analysis of a given sample of soil gave information that 57% of the particle pass through 75 micron sieve. The liquid and plastic limit of the soil were 62% and 28% respectively. Classify the soil as per the USCS.

→ Solution:-

Given that:-

% of particle passing through 0.075mm sieve = 57%.

Liquid limit (L.L) = 62%

Plastic limit (PL) = 28%. ~~P.I. = L.L − P.L.~~ = 62 − 28 = ~~43.6~~ (34)

Now,

$P.I = 0.73(L.L - 20)$

$= 0.73(62 - 20) = 30.66$

Since P.I > ~~25%~~ and L.L = 62% > 50%

So the soil is CH

Hence, the given soil is CH - Inorganic clay of high plasticity

21. 500gm of dry soil was subjected to a sieve analysis. The weight of soil retained on each sieve is as follows:-

Sieve Size	4.75mm	2mm	1mm	425µ	212µ	150µ	75µ
Wt. of soil retained on sieve (g)	10	165	100	85	40	30	50

Plot the grain size distribution curves and determine the following:

a) Effective size, D_{10}
b) Coefficient of uniformity, C_u
c) Coefficient of curvature C_c
d) The graduation of the soil.

→ Solution:-

The calculations have been calculated in the table below:-

Sieve Size (mm)	Soil retained (g)	Percentage retained	Cum % retained	% Passing
4.75	10	2 $(\frac{10}{500} \times 100\%)$	2	98
2	165	33	35	65
1	100	20	55	45
0.425	85	17	72	28
0.212	40	8	80	20
0.150	30	6	86	14
0.075	50	10	96	4

From the particle size distribution curve, we have,

$D_{10} =$ ~~0.13~~ 0.13

$D_{30} =$ ~~130~~ ~~0.43~~ 0.90

$D_{60} =$ ~~0.50~~ 1.80

a) $D_{10} = 0.13$

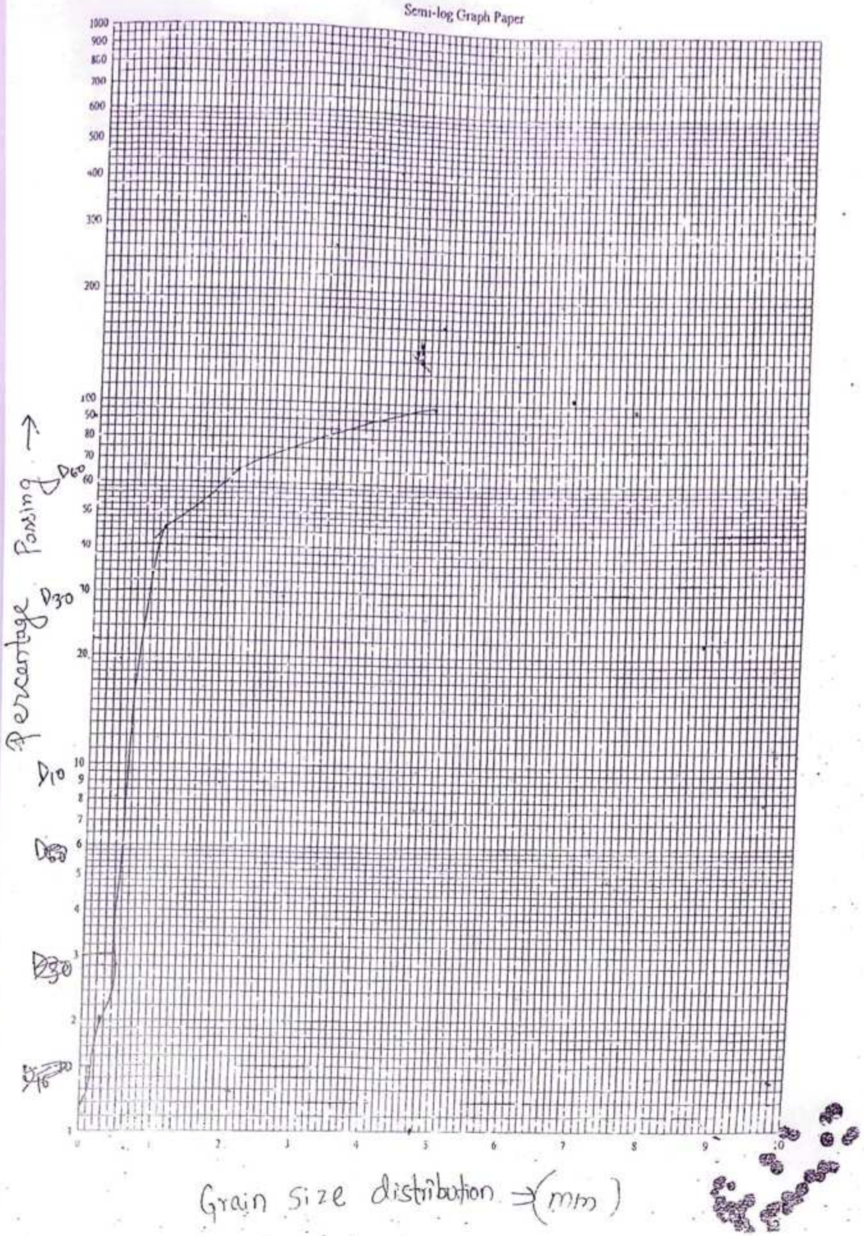
b) $C_u = \frac{D_{60}}{D_{10}} = $ ~~$\frac{0.50}{0.13}$~~ $=$ ~~3.8~~ $\frac{1.80}{0.13} = 13.84$

c) $C_c = \frac{D_{30}^2}{D_{60} \times D_{10}} = $ ~~$\frac{0.43^2}{0.50 \times 0.13}$~~ $=$ ~~2.84~~ $= \frac{0.90^2}{1.80 \times 0.13} = 3.46$

And
The gradation of the soil is sand, soil is well graded.

Additional practise (2016 Spring).
> Solution same as above expect for classification:
above classification is
According to IS: 1498-1959

Clay Size	Silt size (mm)			Sand size (mm)			Gravel (mm)
	Fine	Medium	Coarse	Fine	Medium	Coarse	
	0.002	0.006	0.02	0.06	0.2	0.6	2



22. A 1000cc core cutter weighing 946.80gm was used to find out the in-situ weight of an embankment. The weight of core cutter filled with soil was noted to be 2770.60gm. Laboratory test on the sample indicates a water content of 10.45% and specific gravity of solids is 2.65. Determine the bulk unit weight, dry unit weight, void ratio and degree of saturation.

→ Solution:-

Given that,

Volume of cutter = 1000cc

Empty weight of cutter = 946.80gm

Weight of cutter + soil = 2770.60gm

Sp. gravity (G) = 2.65

Now,

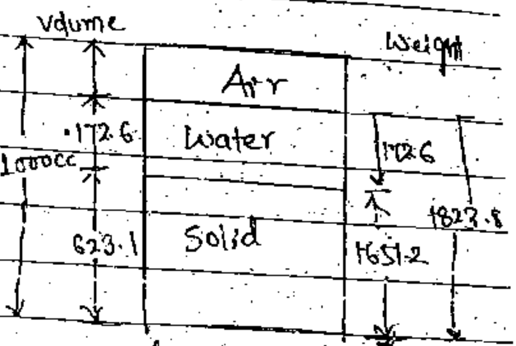


fig. Phase diagram

Weight of soil solid $(W_s) = \frac{W}{1+w} = \frac{2770.6-946.80}{1+0.1045} = 1651.2\,gm$

$W_w = W - W_s = 1823.8 - 1651.2 = 172.6\,gm.$

$V_s = \frac{W_s}{\gamma_s} = \frac{W_s}{G*\gamma_s} = \frac{1651.2}{2.65*1} = 623.1cc.$

$V_w = \frac{W_w}{\gamma_w} = \frac{172.6}{1} = 172.6cc.$

and, $V_v = V - V_s = 1000 - 623.1 = 376.9\,cc.$

Now,

Bulk unit weight $(\gamma) = \frac{W}{V} = \frac{1823.8}{1000} = 1.82\,g/cc.$

Dry unit weight $(\gamma_d) = \frac{W_s}{V} = \frac{1651.2}{1000} = 1.65\,g/cc.$

Void ratio $(e) = \frac{V_v}{V_s} = \frac{376.9}{628.1} = 0.61$

Degree of saturation $(S_r) = \frac{V_w}{V_v} = \frac{172.6}{376.3} = 0.458 = 45.8\%$

Date ____
Page ____

23. In a proctor test, for one observation, mass of wet sample is missing. The oven dry mass of this sample was 1800 gm. The volume of the mould used is 900 cc. If the saturation of this sample was 80%, determine the moisture content and bulk density of the sample assuming specific gravity = 2.7.

→ Solution:-

Given that:-

Oven dry mass (Md) = 1800 gm.

Volume of mould (V) = 900 cc.

Saturation (S) = 80% = 0.8

Moisture content (ω) = ?

bulk density (γ) = ?

Sp. Gravity (G) = 2.7

We have,

~~bulk density =~~

$$V_s = \frac{W_s}{\gamma_s} = \frac{W_s}{G * \gamma_w} = \frac{1800}{2.7 * 1} = 666.67$$

And,

$V_v = V - V_s = 900 - 666.67 = 233.33$ cc.

$W_w = V_w * \gamma_w = 233.33 * 1 = 233.33$ gm.

$$\text{water content } (\omega) = \frac{W_w}{W_s} * 100\%$$

$$= \frac{233.33}{1800} * 100\%$$

$= 12.96\%$.

And, $Se = \omega G \Rightarrow e = \frac{12.96 * 2.7}{80} = 0.437$

$$\text{And, } \gamma = \frac{(G + S_r e)\gamma_w}{1+e} = \frac{(2.7 + 0.8 * 0.437) * 1}{1 + 0.437}$$

$= 2.12$ g/cc

Date ____
Page ____

24. A sample of clay with a mass of 680 gm was coated with paraffin wax. The combine mass of clay and wax was 692.5 gm. The volume of clay and wax was found by immersion in water to be 345 ml. The soil sample was then broken open. The moisture content and specific gravity of soil solid particle test gave 17% and 2.67 respectively. The specific gravity of paraffin wax was 0.88. determine: (i) bulk density (ii) degree of saturation (iii) void ratio (iv) dry density of the soil mass.

→ Solution:-

Given that,

Weight of clay = 680 gm.

Weight of clay + wax = 692.5 gm.

∴ Weight of wax = 692.5 - 680 = 12.5 gm

Displaced volume of water = 345 ml.

∴ Volume of clay and wax = 345 ml.

Sp. gr. of parafin wax (G_{wax}) = 0.88

$$G_{wax} = \frac{\gamma_{wax}}{\gamma_{water}} \Rightarrow \gamma_{wax} = \gamma_w * G_{wax} = 1 * 0.88 = 0.88 \text{ gm/cc.}$$

$$\text{Volume of wax} = \frac{W_{wax}}{\gamma_{wax}} = \frac{12.5}{0.88} = 14.2 \text{ cm}^3$$

Volume of clay = 345 - 14.2 = ~~330~~ 330.8 ml.

$$\therefore \gamma_{clay} = \frac{W_{clay}}{V_{clay}} = \frac{680}{330.8} = 2.056 \text{ gm/cc.}$$

$$\gamma_{d(clay)} = \frac{\gamma_{clay}}{1+\omega} = \frac{2.056}{1+0.17} = 1.75 \text{ gm/cc}$$

$$\gamma_d = \frac{G\gamma_w}{1+e} \Rightarrow 1.75 = \frac{2.67 * 1}{1+e} \Rightarrow e = 0.52$$

& $Se = G\omega$

$$\therefore S = \frac{0.17 * 2.67}{0.52} = 0.874$$

RAKESH KUMAR SAH

25. Classify the soil A and B with the following properties according to USCS.

Soil	L.L %	P.I %	% passing # 200 sieve (0.075mm)
A	62	30	60
B	38	17	82

-> Solution:-

For soil A

$P.I = 0.73(L.L-20)$

$= 0.73(62-20)$

$= 30.66\%$

For soil B

$PI = 0.73(LL-20)$

$= 0.73(38-20)$

$= 13.14\%$

Since, P.I of A = 30.66% > 25% and L.L > 50%,

so the soil is

Additional practise (2014 fall)

Solution same as above.

26. The following data refers to a light silty clay that was assumed to be saturated in the undisturbed condition. On the basis of these data, determine the liquidity index, sensitivity and void ratio of the saturated soil. Classify the soil according to the unified soil classification.

Index properties ->	Unconfined Compressive Strength kN/m²	Water content %	Liquid Limit, %	Plastic Limit %	Shrinkage Limit %	% passing no. 200 sieve
Undisturbed	244	22				
Remoulded	144	22	45	20	12	90

As per above data, soil is classified as A-7-6 according to AASHTO classification. Find the index of the soil.

-> Solution:-

We know that,

$P.I = L.L - P.L = 45\% - 20\% = 25\%$

Now;

i) Liquid index $(L.I) = \frac{w_n - P.L}{P.I} = \frac{22-20}{25} = 0.08$

ii) Sensitivity $(S_t) = \frac{q_{u,undisturbed}}{q_u', remoulded} = \frac{244}{144} = 1.69$

iii) Void ratio(e) = ?

$Se = Gw$

or, $1 * e = 2.68 * 0.22$ $[\because S=1]$

$\therefore e = 0.5896$

To classify according to USCS,

from formula, $P.I. = 0.73(L.L-20) = 18.25\%$

Since, 25% > 18.25 and L.L = 45% < 50%, so the soil is CL.

To classify according to AASHTO,

$GI = 0.2a + 0.005ac + 0.01bd$ .... (1)

$a = 90-35 = 55 \Rightarrow 40$

$c = 45-40 = 5$

$b = 40\%$

$d = 25-10 = 15$

Then eqn (1) becomes,

$G.I = 0.2*40 + 0.005*40*5 + 0.01*40*15$

$= 15$

Hence, the soil is A-7-6(15) according to AASHTO.

— X — X — X —

Rakesh Kr. Sah

Tutorial-2

Ch.5 - Soil Compaction:-

1. The maximum dry density of a sample by light compaction test is 1.78 gm/cm³ at optimum moisture content of 15%. Find the air voids and degree of saturation. What would be the corresponding value of dry density on zero air void line at OMC? Assumed G = 2.67.

→ Solution:-

Given that:

Max. dry density (ρ_d) = 1.78 gm/cm³.

Moisture content (ω) = 15%.

$G = 2.67$

Air voids (n_a) = ?

degree of saturation (S) = ?

We have,

$$\rho_d = \frac{G\rho_\omega(1-n_a)}{1+\omega G}$$

$$\text{or, } 1.78 = \frac{2.67 \times 1(1-n_a)}{1+0.15 \times 2.67}$$

$\therefore n_a = 6.63\%$

and, $\rho_d = \frac{G\rho_\omega}{1+\frac{\omega G}{S}} \Rightarrow 1.78 = \frac{2.67 \times 1}{1+\frac{0.15 \times 2.67}{S}}$

or, $1.78 = Se = G\omega$; $1.78 = \frac{2.67S}{S+0.4005}$

or, $\gamma_d = \frac{G\gamma_\omega(1+\omega)}{1+e}$; $1.78S + 0.71289 = 2.67S$

or, $1.78 = \frac{2.67 \times 1(1+0.15)}{1+e} \Rightarrow e = 0.725$

& $S = \frac{G\omega}{e} =$

And, $\rho_d = \frac{G\rho_\omega}{1+\frac{\omega G}{S}} \Rightarrow 1.78 = \frac{2.67 \times 1}{1+\frac{0.15 \times 2.67}{S}}$

$$\text{or, } 1.78 = \frac{2.67S}{S+0.4005}$$

$\therefore S = 80.5\%$.

And, for corresponding value of dry density on zero air void line at OMC, degree of saturation, $S = 1$.

$$\therefore \rho_d = \frac{G\rho_\omega}{1+\omega G} = \frac{2.67 \times 1}{1+0.15 \times 2.67} = 1.906 \text{ gm/cm}^3.$$

2. Earth is required to be excavated from pits for building an embankment. The unit weight of soil in wet condition is 18 kN/m³ and its water content is 8%. In order to build a 4m high embankment with top width 2m and side slope 1:1, estimate the quantity of earth required to be excavated per meter length of embankment. The dry density required in the embankment is 15 kN/m³ with the moisture content 10%. Take sp. gr. of solids as 2.67.

→ Solution:-

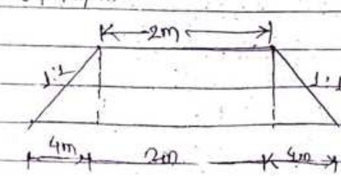
Given that:-

$\gamma = 18 \text{ kN/m}^3 = \frac{W}{V}$

$\omega = 8\%$

$\rho_d = \frac{18}{1+0.18} = 16.67 \text{ kN/m}^3$

Required dry density of embankment $(\rho_d)_{emb} = 15 \text{ kN/m}^3$.



The volume of embankment soil per unit length $= \left(\frac{10+2}{2}\right) \times 4 \times 1$

$= 24 \text{ m}^3$

Dry wt. of $1m^3$ excavated soil = 16.67KN

Dry wt. of $1m^3$ embankment soil = 15KN.

Dry wt. of soil required for $1m^3$ of embankment

∴ Volume of soil required $= \frac{15}{16.67} = 0.899$ m

∴ Total quantity of earth/w required excavated per meter length of embankment

$= 0.899 \times 24$

$= 21.58 m^3$

3. In a standard proctor test on a soil having specific gravity as 2.7, the following results were obtained:

water content (%)	5	8	10	12	15	20
Bulk unit wt, kg/m³	1890	2130	2200	2210	2160	2080

Plot the dry density versus water content curve. Show zero air void line on the same curve. Determine the values of void ratio, porosity and degree of saturation for the soil at optimum moisture content condition.

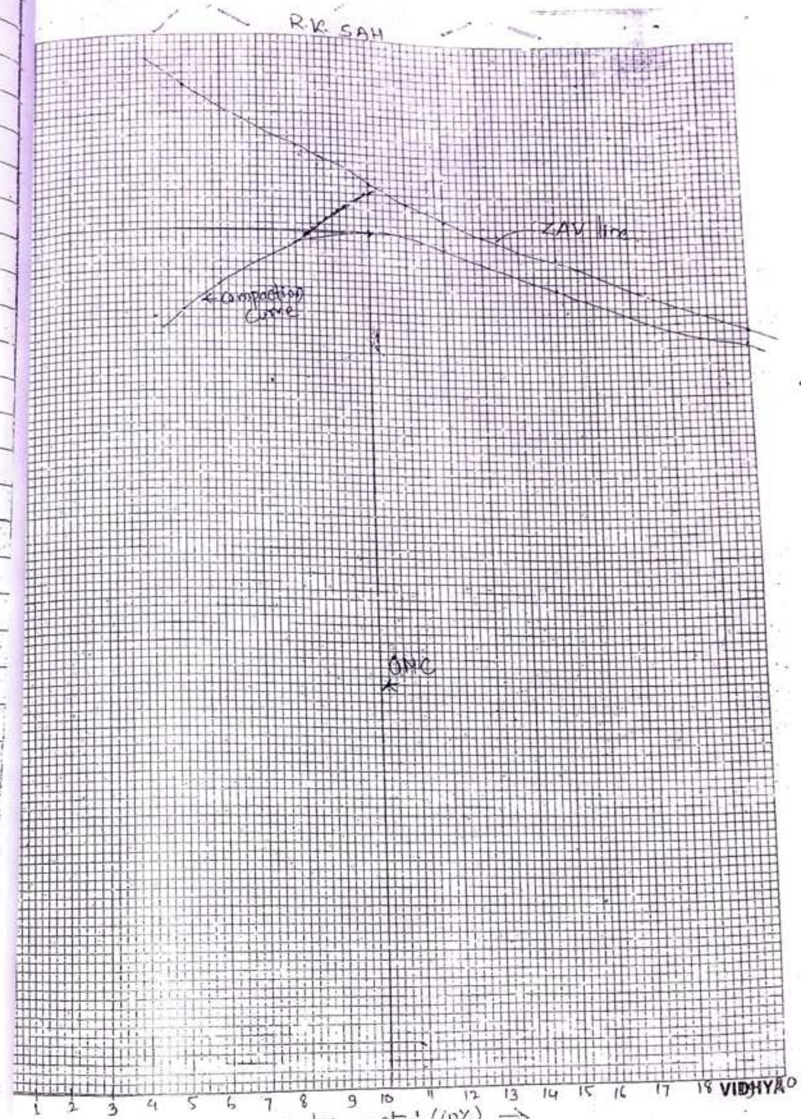
→ Solution:-

Given that;

Sp. gr. of soil (G) = 2.7.

Now,

Water Content (%)	Bulk unit weight (γ) (Kg/m³)	Dry unit weight (Kg/m³) $\gamma_d = \frac{\gamma}{1+w}$	for zero air void line, $\gamma_d = \frac{G\gamma_w}{1+wG}$
5	1890	1800	2378.86
8	2130	1972.22	2220.39



10	2200	2000	2125.98
12	2210	1973.21	2039.27
15	2160	1878.26	1921.71
20	2080	1733.33	1753.25

From the graph, we have,

OMC = 10%.

and Max. dry unit weight $(\gamma_d)_{max} = 2000\ kg/m^3$.

Also,

$\gamma_d = \frac{G\gamma_w}{1+e} \Rightarrow 2000 = \frac{2.7 \times 1000}{1+e} \Rightarrow e = 0.35$

$n = \frac{e}{1+e} = \frac{0.35}{1+0.35} = 25.92$

And,

$Se = Gw$

$\Rightarrow S = \frac{0.1 \times 2.7}{0.35} = 77.14\%$ &

Additional practice (2009 Spring):

The result of a laboratory Proctor test were shown below:

No. of test	1	2	3	4	5	6
Wt of mould + soil	3.526	3.711	3.797	3.906	3.924	3.882
Water content	7.33	9.40	11.23	15.20	16.92	19.39

The mould is 12.7cm high and has an internal diameter of 10cm. The weight of the empty mould is 1.89 kg. Plot the moisture content dry density curve and determine the optimum moisture content and maximum dry density. Given, G = 2.68.

→ Solution:-

We have,

Volume of mould (V) = $\frac{\pi r^2 h}{4} = \frac{3989.82\ cm^3}{4}$

No. of Test	Water Content	Wt. of mould + soil	Wt. of soil only	$\rho = \frac{m}{V}$ kg/cm³	$\rho_d = \frac{\rho}{1+w}$
1	7.33	3.526	1.636	4.1×10^{-4}	3.81×10^{-4}
2	9.40	3.711	1.821	4.56×10^{-4}	4.16×10^{-4}
3	11.23	3.797	1.907	4.77×10^{-4}	4.28×10^{-4}
4	15.20	3.906	2.016	5.05×10^{-4}	4.38×10^{-4}
5	16.92	3.924	2.034	5.097×10^{-4}	4.36×10^{-4}
6	19.39	3.882	1.992	4.99×10^{-4}	4.18×10^{-4}

Now,
from Graph we have,
OMC = 15.2% and,
Max. dry density = 4.38

4. An earthen embankment of 1×10^6 m³ volume is to be constructed with a soil having a void ratio of 0.8 after compaction. There are three borrow pit marked by A, B and C having soils with void ratios of 0.90, 1.50 and 1.80 respectively. The cost of excavation & transporting the soil is Rs. 0.25, 0.23 and 0.18 per m³ resp. Calculate the volume of soil to be excavated from each pit. Which borrow pit is most economical? Take sp. gr. of soil as 2.65.

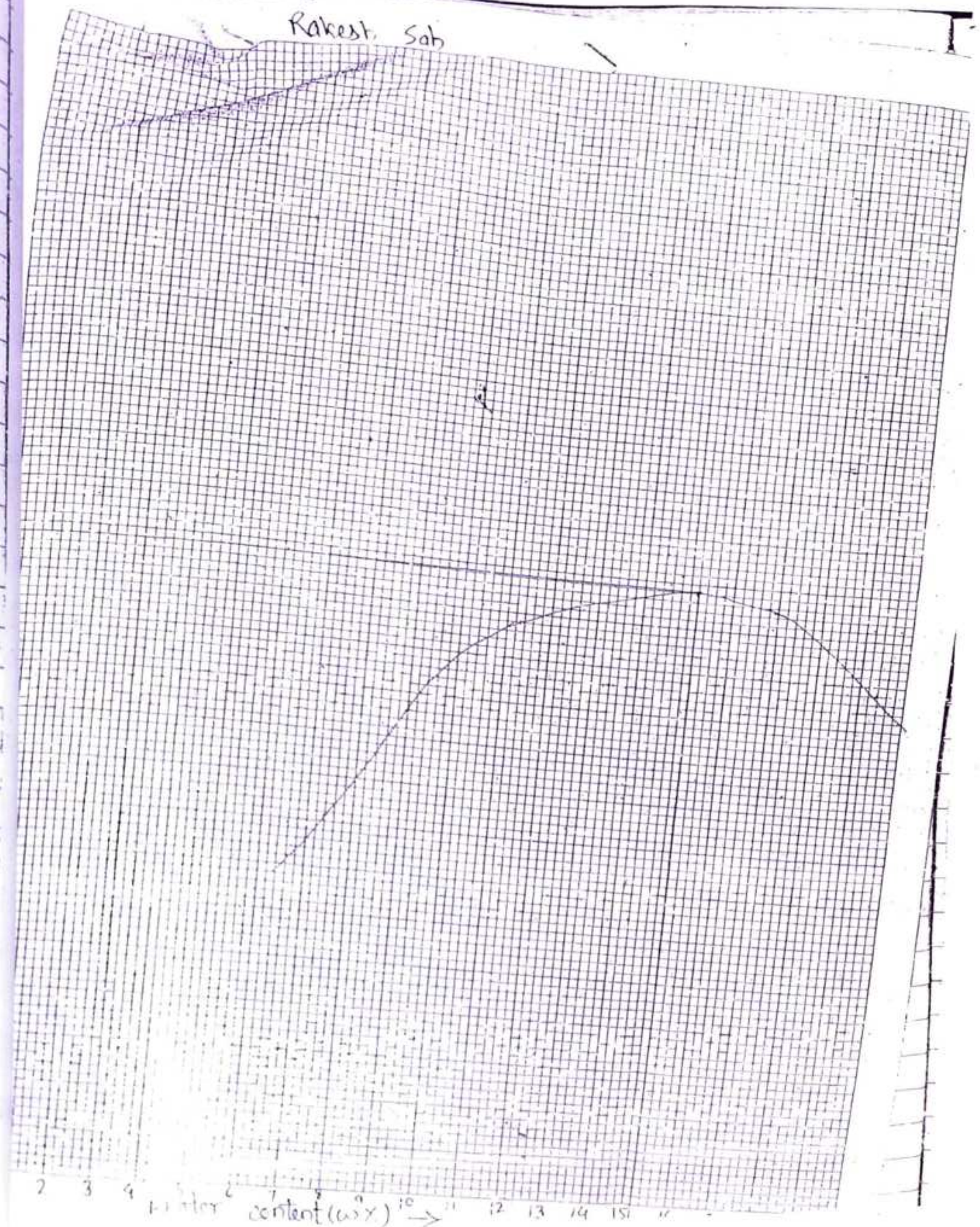
→ Solution:-

We have, dry unit wt: $\gamma_d = \frac{G\gamma_w}{1+e} = \frac{2.65 \times 9.81}{1+0.8} = 14.44$ kN/m³

$\therefore (\gamma_d)_A = \frac{2.65 \times 9.81}{1+0.90} = 13.68$ kN/m³.

$(\gamma_d)_B = \frac{2.65 \times 9.81}{1+1.5} = 10.39$ kN/m³

$(\gamma_d)_C = \frac{2.65 \times 9.81}{1+1.8} = 9.28$ kN/m³



Taking $1 m^3$ volume of soil,
dry weight = $14.44 KN/m^3 * 1 m^3 = 14.44 KN$.

$(\text{dry weight})_A = 13.68$ KN
$(\text{dry weight})_B = 10.39$ KN
$(\text{dry weight})_C = 9.28$ KN

Now,
Vol. of soil A req. to make an embankment of $1 m^3$ is $\frac{14.44}{13.68} = 1.06 m^3$

" " " B " " " " " " $= \frac{14.49}{10.39} = 1.39 m^3$

" " " C " " " " $= \frac{14.44}{9.28} = 1.56 m^3$

And,
cost of soil A = Rs. $0.25/m^3 * 1.06$ = Rs. 0.265
" " " B = Rs. $0.23/m^3 * 1.39$ = Rs. 0.320
" " " C = Rs. $0.18/m^3 * 1.56$ = Rs. 0.281

Hence,
Volume of soil to be excavated from pit A $= 1.06*1*10^6$
$= 1.06*10^6 m^3$
" " " " " " " " B $= 1.39*1*10^6 = 1.39*10^6 m^3$
" " " " " " C $= 1.56*10^6*1 = 1.56*10^6 m^3$

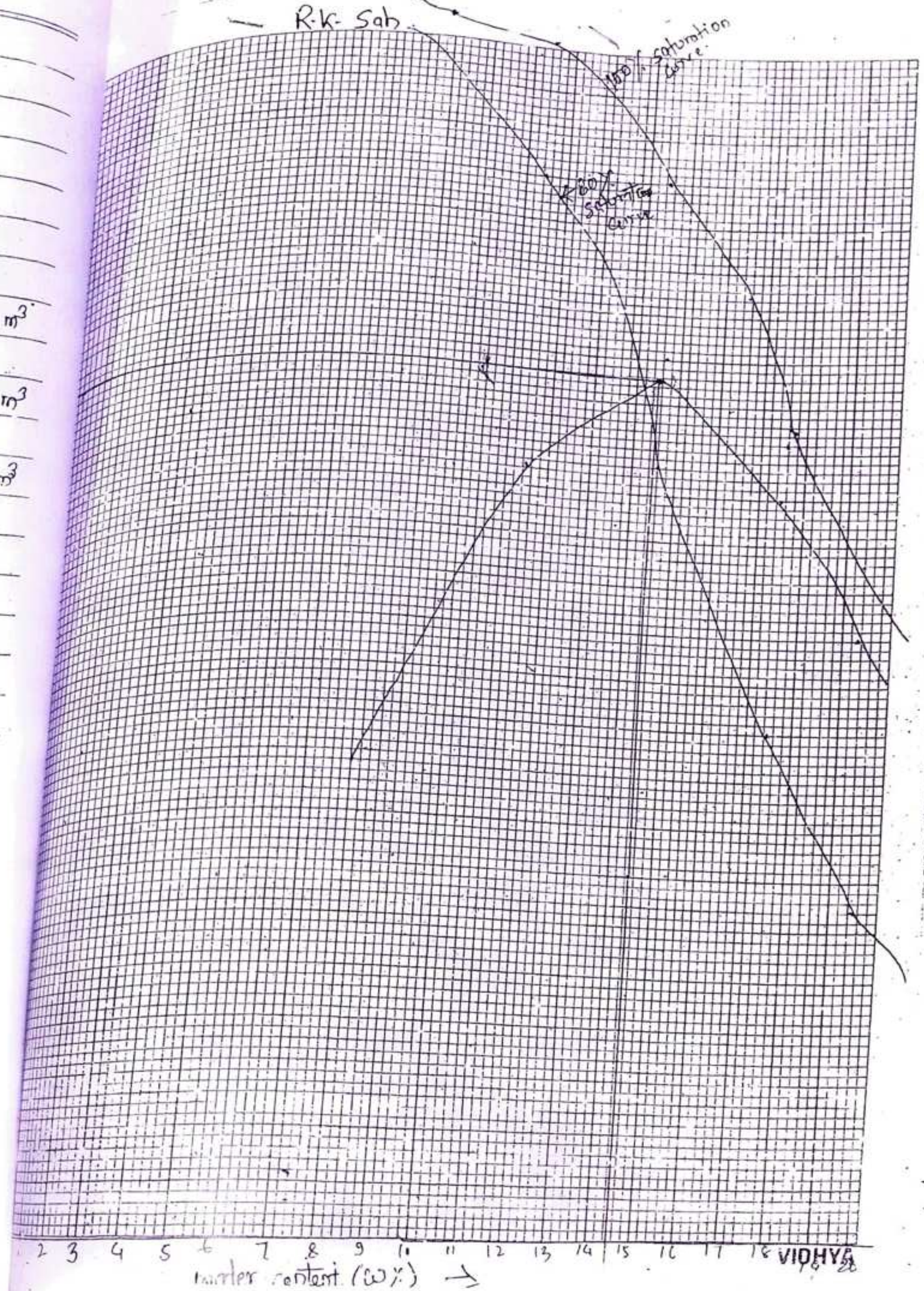
So, Borrow pit A is most economical.

5). A proctor compaction test was conducted on a soil sample and the following observations were made.

water content, w%	8	11.5	14.5	17.5	19.5	21.5
Mass of wet soil, m kg	1.70	1.90	2.0	1.98	1.95	1.92

If the volume of mould used was 950cc and specific gravity of soil was 2.65, draw dry density versus water content curve and (i) plot the 100% and 80% saturation curve (ii) Find maxi. dry

R.K. Sah



density and optimum moisture content. (III) Find the degree of saturation at OMC (IV) If it is proposed to secure a relative compaction of 95% in the field, what is the range of water content that can be allowed?

→ Solution: -

Volume of mould (V) = 950 cc

Sp. gr. of soil (G) = 2.65

w%	Mass of wet sample (gm)	Density of dry sample $\rho_d = (\rho/(1+w))$	100% saturation $\rho_d = G\rho_w/(1+wG)$	80% saturation $\rho_d = (G\rho_w)/(1+\frac{wG}{0.8})$	wet density (ρ)
8	1700	1.66	2.18	2.094	1.79
11.5	1900	1.79	2.03	1.92	2
14.5	2000	1.83	1.92	1.79	2.10
17.5	1980	1.78	1.81	1.68	2.089
19.5	1950	1.72	1.75	1.61	2.05
21.5	1920	1.66	1.688	1.55	2.02

I) Plotted in Graph paper.

II) From Graph,

Max. dry density = 1.83

OMC = 14.5%

III) Degree of saturation at OMC =

$(\rho_d)_{max} = \frac{G\rho_w}{1+e}$

$\therefore e = \frac{G\rho_w}{(\rho_d)_{max}} - 1 = \frac{2.65 \times 1}{1.84} - 1 = 0.44$

Now, $S = \frac{wG}{e} = \frac{0.145 \times 2.65}{0.44} = 0.8733 = 87.33\%$

IV) $R.C = 95\% = \frac{\rho_d, field}{\rho_d, lab} \Rightarrow \rho_d(field) = 1.84 \times 0.95$

$= 1.784$ gm/cc.

ρ_d field $= \frac{G\rho_w}{1+wG}$

or, $1.784 = \frac{2.65 \times 1}{1+w \times 2.65}$

$\therefore w = 18.32\%$

for ρ_d lab $w = 16.61\%$

6. A sample of soil compacted according to standard proctor test has a unit weight of 20 kN/m³ at 100% compaction and at optimum water content of 15%. What is the dry unit weight? What is the dry unit weight at zero air voids? If the voids becomes filled with water, what would be the saturated unit weight? Assume G = 2.67.

→ Solution:-

Given that:-

Unit weight (ρ) = 20 kN/m³ at 100% compaction

ω = 15%

ρ_d = ?

ρ_d at zero air voids = ?

ρ_{sat} = ?

We have,

$$\rho_d = \frac{\rho}{1+\omega} = \frac{20}{1+0.15} = 17.39\ kN/m^3$$

At zero air voids. (ZAV),

$$\rho_d = \frac{G\rho_w}{1+\omega G} = \frac{2.67*9.81}{1+0.15*2.67} = 18.7\ kN/m^3.$$

When voids filled with water,

$$\rho_d = \frac{G\rho_w}{1+e} \Rightarrow 17.39 = \frac{2.67*9.81}{1+e} \Rightarrow e = 0.506,$$

Now,

$$\rho_{sat} = \frac{(G+e)\rho_w}{1+e} = \frac{(2.67+0.506)*9.81}{1+0.506}$$

$= 20.69\ kN/m^3$.

Additional practice (2012 Fall)

Solution same as above.

Rakesh Shah

7. A sample of soil is prepared by mixing a quantity of dry soil with 11% by weight of water. Find the weight of this wet mixture, which will be required to produce by static compaction, a cylindrical specimen 150mm diameter and 125mm deep with 5% air void. Take specific gravity of solids are 2.7. Also find the dry density and void ratio of soil.

→ Solution:-

Given that:-

Water content (ω) = 11% = 0.11

Sp. gr. of solid (G) = 2.7

Degree of saturation (S_r) = 100% - 5% = 95% = 0.95

Now,

$\omega G = S_r e$

$$\therefore e = \frac{\omega G}{S_r} = \frac{0.11*2.7}{0.95} = 0.313$$

Again,

$$\gamma_d = \frac{G\rho_w}{1+e} = \frac{2.7*1}{1+0.313} = 2.06\ gm/cm^3$$

And,

$$\gamma_d = \frac{\gamma_t}{1+\omega}$$

$$\therefore \gamma_t = \gamma_d(1+\omega) = 2.06(1+0.11) = 2.28\ gm/cm^3$$

$$\text{Volume of soil} = \frac{\pi d^2 h}{4} = \frac{\pi * 15^2 * 12.5}{4} = 2208.93cm^3$$

Now, $\gamma = \frac{W}{V} \Rightarrow W = 2.28 * 2208.93$

$= 5036.37\ gm.$

∴ Weight of the wet mixture = 5.036 kg.

8. The following data are available in connection with the construction of an embankment.

i) Soil from borrow pit: natural density = 1.65 Mg/m^3 & ω = 12%.
ii) Soil after compaction: density = 2 mg/m^3 & ω = 18%.
Hence for every 50 m^3 of compacted soil of embankment estimate
i) the quantity of soil to be excavated from the borrow pit.
ii) The amount of water to be added.

→ Solution:-
We have,

i) $\rho_{d,nat} = \frac{\rho}{1+\omega} = \frac{1.65}{1+0.12} = 1.473\ mg/cm^3$

$\rho_{d,after} = \frac{2}{1+0.18} = 1.695\ mg/cm^3$.

∴ Volume of soil required for 1 m^3 compacted soil = $\frac{1.695}{1.473}$ = 1.15 m^3

∴ for 50 m^3 compacted soil, volume of soil required
= 50 * 1.15
= 57.53 m^3.

ii) Now, in natural state, the moisture present in 1.695 mg of dry soil would be
= 1.695 * 0.12 = 0.2034 mg/cm^3.
And the moisture which the soil passes during compaction is 1.695 * 0.18 = 0.3051 mg/m^3.
Hence,
Mass of water to be added = 0.3051 − 0.2034
= 0.1017 mg/m^3

∴ for 50 m^3 soil, water to be added = 50 * 0.1017
= 5.085 mg
= 5085 kg

Rakesh Kumar Sah

9 An earth embankment is to be compacted to a dry density of 1.84 gm/cc at moisture content of 15%. The bulk density and water content in the borrow pit are 1.77 and 8% respectively. How much excavation should be carried out in the borrow pit for each cubic meter of embankment?

→ Solution:-
Given that:
Dry density (ρ_d) = 1.84 gm/cc.
water content (ω) = 15%.
Bulk density of borrow pit (ρ_b) = 1.77
water content of " " (ω_b) = 8%
we have,

$\gamma_{d,b} = \frac{\gamma_b}{1+\omega} = \frac{1.77}{1+0.08} = 1.64\ gm/cc.$

Here,
for 1 cm^3, dry unit wt = 1.64 gm.

∴ Volume of borrow pit = $\frac{1.84}{1.64}$ = 1.12 m^3

—— X —— X ——

Rakesh Kumar Sah

Tutorial - 3

Ch. 6 - Principle of effective stress, capalarity and Permeability on soil:-

1. A sample of coarse sand is 15cm high and 5.5cm diameter. It was tested in a constant head permeameter. Water permeated through the soil under a hydrostatic head of 50cm for 6 seconds. The water was collected and it was found to weigh 500 gms. find the coefficient of permeability in cm/sec.

→ Solution:-

Given that:-

Height of sample (L) = 15cm.

Diameter of sample (d) = 5.5cm

Time (t) = 6sec.

Weight of water (W) = 500 gms.

Hydrostatic head (h) = 50cm.

Coefficient of permeability (K) = ?

We have,

$$K = \frac{qL}{Ah}$$

$$q = \frac{V}{t} = \frac{W}{\gamma * t} = \frac{500}{1*6} = 83.33\ cm^3/s.$$

$$\therefore K = \frac{83.33*15*4}{\pi*5.5^2*50} = 1.052\ cm/sec.$$

Additional Practise (2016 Spring)

Calculate the coefficient of permeability of a soil sample 8cm height and C/s area 60cm². It is observed that in 12 minutes, 600ml of water passed down under constant head of 40cm.

The test specimen weigh 750 gm in dry condition. Take sp. Gr. of soil solid as 2.7. Calculate the seepage velocity.

→ Solution:-

Given that:-

Length of specimen, L = 80mm

Area of X-section, A = 6000 mm²

Quantity of flow, Q = 600 mL = 600cc.

Time of flow, t = 12 min = 12*60 = 720 sec.

Constant head, H = 400 mm.

Now, $q = KiA$

or, $\frac{Q}{t} = K \cdot \frac{H}{L} \cdot A$

$$\therefore K = \frac{Q \cdot L}{A \cdot H \cdot t} = \frac{600*80*1000}{6000*400*720} = 0.028\ mm/s.$$

Discharge velocity, $v = \frac{q}{A} = \frac{Q}{At} = \frac{600*1000}{6000*720} = 0.139\ mm/s.$

Dry weight of specimen, W_d = 750gm.

∴ Volume of specimen, $V = 6000*80 = 4.8*10^5\ mm^3$.

$$\gamma_d = \frac{W_d}{V} = \frac{750*9.81}{4.8*10^5*1000} = 1.5328*10^{-5}\ gm/mm^3.$$

$$e = \frac{G\gamma_w}{\gamma_d} - 1 = \frac{2.7*9.8*10^{-6}}{1.53*10^{-5}} - 1 = 0.726$$

$$n = \frac{e}{1+e} = 0.42$$

∴ Seepage velocity, $V_s = \frac{v}{n} = \frac{0.139}{0.42} = 0.33\ mm/sec.$

2. To find out the coefficient of permeability of clay the permeability test was carried out in a variable head permeameter having the diameter of 10cm. The initial head of water in the standpipe was

found to be 45cm and it was observed to drop to 30cm in 3.5 minutes. If the height of the sample was 15cm and the diameter of the standpipe was 1.9cm, determine the coefficient of permeability

→ Solution:-

Given that:-

Diameter of pipe (D) = 10 cm

Length of sample (l) = 15 cm

Diameter of standpipe (d) = 1.9 cm

Initial height (h_1) = 45 cm

Final height (h_2) = 30 cm

Time (t) = 3.5 min = 210 sec.

Area of pipe (a) = $\frac{\pi d^2}{4}$ = 2.835 cm².

Coefficient of permeability (K) = ?

We know that,

$$K = \frac{al}{At} \ln\left(\frac{h_1}{h_2}\right)$$

$$= \frac{2.835 * 15 * 4}{\pi * 10^2 * 210} \ln\left(\frac{45}{30}\right)$$

= 0.00105 cm/s

≃ 0.907 m/day &

Additional Practice (2013 Spring)

In falling head permeameter test, the initial head ($t=0$) is 40cm. The head drops by 5cm in 10 minutes. Calculate the time required to run the test for the final head to be at 20cm. If the sample is 6cm in height and 50 cm² in X-sectional area, calculate the coefficient of permeability, taking area of standpipe as 0.5cm².

→ Solution:-

Given that:

Initial head (h_1) = 40 cm

Final head (h_2) = 5 cm

time (t) = 10 min = 600 sec.

Area of stand pipe (a) = ~~$\frac{\pi * 0.5^2}{4}$ = 0.1963 cm²~~ 0.5 cm²

Length of sample (L) = 6 cm

Area of sample (A) = 50 cm².

We have,

$$K = \frac{al}{At} \log_e \frac{h_1}{h_2}$$

$$= \frac{0.5 * 6}{50 * 600} \log_e \frac{40}{5}$$

= ~~8.16 * 10⁻⁵ cm/sec.~~ 2.079 * 10⁻⁴ cm/sec.

Now, For final head to be 20 cm,

~~$K = \frac{al}{At} \log_e \frac{h_1}{h_2}$~~

or, ~~8.16 * 10⁻⁵ =~~

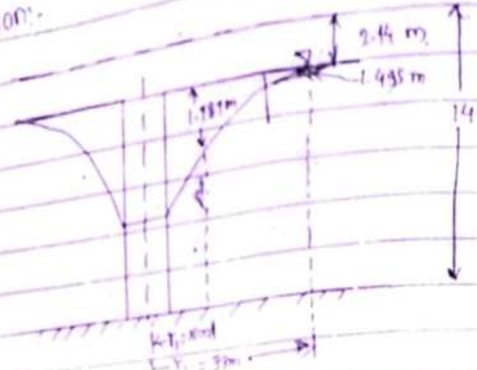
$$2.079 * 10^{-4} = \frac{0.5 * 6}{50 * t} \ln\left(\frac{40}{20}\right)$$

or, t = 200 sec. &

3. In order to determine the avg coefficient of permeability of a bed of sand 14 meters thick, overlying on impermeable stratum, a well was sunk through the sand and a pumping test was carried out. After a certain interval, the discharge was 12.4 litres per second and draw down in the observation wells at 16 meters and 33 meters resp. from the pumping well were found to be 1.787 and 1.495m resp. If the GWT was originally at 2.14m below the ground surface,

find (a) the permeability of sand layers and (b) an approximate value for the effective grain sizes.

Solution:-



Discharge (q) = 12.4 litre/sec = $12.4*10^{-3}\ m^3/sec$.

$h_1 = (14-2.14-1.787)m = 10.073\ m$

$h_2 = (14-2.14-1.495)m = 10.365\ m$

$r_1 = 16m$

$r_2 = 33m$

We have,

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

$$= \frac{12.4*10^{-3}}{\pi(10.365^2-10.073^2)} * \log\left(\frac{33}{16}\right)$$

$$= 4.78*10^{-4}\ m/sec$$

Again we have,

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

$$\therefore D_{10} = \sqrt{\frac{k}{c}} = \sqrt{\frac{4.78*10^{-4}}{125}} \quad [\because C = 125]$$

$$= 0.02\ mm$$

Rakesh Kumar Sah

4. A horizontal stratified deposit consists of three layers with equal permeability values in horizontal as well as vertical direction. The permeability of the layers is $8*10^{-4}$ cm/sec, $50*10^{-4}$ cm/sec and $15*10^{-4}$ cm/sec and their thickness are 6m, 3m and 18m respectively. Find the effective average permeability of the deposits in horizontal & vertical direction.

Solution:-

Equivalent permeability in horizontal direction is,

$$K_x = \frac{K_1H_1 + K_2H_2 + K_3H_3}{H_1+H_2+H_3} = \frac{(8*6+3*50+15*18)*10^{-4}}{6+3+18}$$

$$= 17.33*10^{-4}\ cm/sec.$$

Again, equivalent permeability in vertical direction is,

$$K_z = \frac{H_1+H_2+H_3}{\frac{H_1}{K_1}+\frac{H_2}{K_2}+\frac{H_3}{K_3}} = \frac{6+3+18}{\left(\frac{6}{8}+\frac{3}{50}+\frac{18}{15}\right)*10^{-4}} = 13.43*10^{-4}\ cm/sec$$

Additional practice (2008 Fall):

Same as above.

5. A layer of sand 6m thick lies beneath a clay stratum 5m thick and above a thick scale. In order to determine the permeability of sand a well was driven to the top of shale and water pumped out at the rate of $10*10^{-3}\ m^3/sec$. Two observation wells were driven through the clay at 15m and 30m from the pump well and the water was found to rise to levels of 3m and 2.5m below the ground surface respectively. Calculate the coefficient of permeability of the soil.

Solution:-

Given that

Thickness of sand (H) = 6 m

Discharge (q) = $10*10^{-3}\ m^3/sec$.

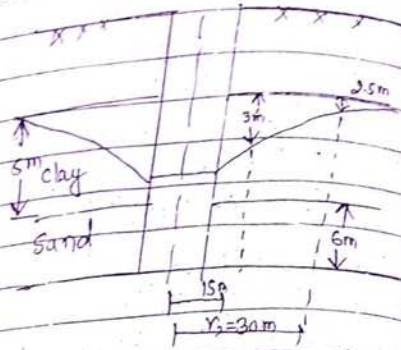
$r_1 = 15m$, $r_2 = 30m$

$H_1 = (11-3)m = 8m$

$H_2 = (11-2.5)m = 8.5m$

We have,

$$K = \frac{2.3 * q * \log_{10} \frac{r_2}{r_1}}{2\pi H (H_2 - H_1)}$$



$$= \frac{2.3 * 10 * 10^{-3} * \log_{10} \frac{30}{15}}{2 * \pi * 6 * (8.5-8)}$$

$= 3.67 * 10^{-4}$ m/sec

$\therefore K = 0.367$ mm/sec.

6. During a pumping test, a well was sunk through a stratum of dense sand 10m deep overlying on impervious stratum. Observation wells were drilled at 6m and 12m from the well. Initially, the water level in the well was 2.5m below the ground surface. After pumping until steady conditions had been ~~reached~~ acheived, the water level in observation wells had dropped 1.95m and 0.5m respectively. If the steady discharge was 5 litres/sec, determine the coefficient of permeability.

> Solution:-

Given that:

$q = 5$ litres/sec $= 5 * 10^{-3}$ m³/s.

$r_1 = 6m$ $r_2 = 12m$

$S_1 = 1.95m$ $S_2 = 0.5m$

Initial position of water table is 2.5m below ground level.

$h_1 = 10 - 2.5 - 1.95 = 5.55m$

$h_2 = 10 - 2.5 - 0.5 = 7m$

We have,

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

$$= \frac{5 * 10^{-3} \ln(12/6)}{\pi(7^2 - 5.55^2)}$$

$= 6.06 * 10^{-5}$ m/s.

Additional practise (2011 F)

> Solution: Same as above.

7. A sand layer 10m thick overlies on impervious stratum. The water table is in the sandy layer at a depth of 1.5m below the ground surface. Water is pumped out from a well at the rate of 100 litre per second and drawdown the water table at radial distance of 3.0m and 25m is 3.0m and 0.50m respectively. Determine the coefficient of permeability.

> Solution:

$q = 100$ litres/sec $= 100 * 10^{-3}$ m³/s.

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

$S_1 = 3.0m$ $S_2 = 0.5m$.

$H_1 = 10 - 1.5 - 3 = 5.5$

$H_2 = 10 - 1.5 - 0.5 = 8$

Now,

$$K = \frac{q \ln(r_2/r_1)}{\pi(h_2^2 - h_1^2)} = \frac{100 * 10^{-3} \ln(25/3)}{\pi(8^2 - 5.5^2)}$$

$= 1.99 * 10^{-3}$ m/sec.

$\simeq 1.99$ mm/sec.

Rakesh Kumar Sah.

8. What is the height of capillary rise and the capillary pressure in fine grained soil with effective size of 0.002mm?

→ Solution:-

Given that:

Effective size $(D_{10}) = 0.002mm$

Equivalent diameter of the tube $(d) = 0.2 * D_{10}$
$= 0.2 * 0.002 = 4*10^{-4} mm$

Again, $h_c = \frac{0.3}{d} = \frac{0.3}{4*10^{-4}} = 75 m$

And, Capillary pressure $= -h_c * \gamma_w = -7500*1 \ gm/cm^2$

$\therefore P_c = 735.75 \ kN/m^2$

9. Water table was found 1.6m below the ground surface in a deep deposit of very fine sand. Above water table, the sand was found saturated by capillary water. The unit weight of saturated sand was 2.2 tons/m³. What is the effective vertical pressure on a horizontal plane at a depth of 4.5m below the ground surface?

→ Solution:-

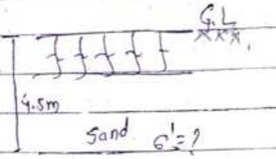
Given that:-

$\gamma_{sat} = 2.2 \ tons/m^3$

Effective vertical pressure $(\sigma') = ?$

Total pressure $(\sigma) = \sigma' + u$

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



At 4.5 m,

$\sigma = 1.6*2.2 + (4.5-1.6)*2.2$
$= 9.9 \ tons/m^2$

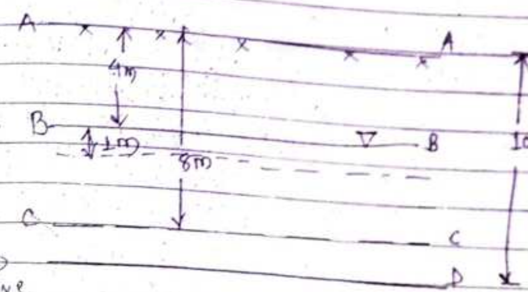
Also,

$u = (4.5-1.6)*9.81$
$= 28.449 \ kN/m^2$
$= 2.9 \ tons/m^2$

$\therefore \bar{\sigma} = 9.9 - 2.9$
$= 7.0 \ tons/m^2$

10. A fine sand deposit is located between the ground surface to a depth of 10m. The soil has an avg void ratio of 0.70 and specific gravity of 2.65. The water table is found at a depth of 4m below the ground surface. Above the water table the degree of saturation of sand is 55%. Determine the total stress, pore water pressure and effective stress at a depth of 8m below the ground surface. Calculate also the change in the effective stress if the soil gets saturated upto a height of 1m above the water table due to capillary.

→ Solution:-



We have,

$\gamma_t = \frac{(G + S_r e)\gamma_w}{1+e} = \frac{(2.65+0.55*0.7)*10}{1+0.7} = 17.85 \ kN/m^3$

Similarly, $\gamma_{sat} = \frac{(G+e)\gamma_w}{1+e} = \frac{(2.65+0.7)*10}{1.7} = 19.71 \ kN/m^3$

Now,

Case I:

At C-C,

$\sigma = (17.85*4 + 19.71*4) = 150.24\ KN/m^2$

$u = 10*4 = 40 KN/m^2$

$\therefore \bar{\sigma} = \sigma - u = (150.24 - 40) = 110.24\ KN/m^2$.

Case II

At C-C,

$\sigma = (17.85*3 + 19.71*5) = 152.10 KN/m^2$

$u = 10*4 = 40\ KN/m^2$

$\therefore \bar{\sigma} = \sigma - u = (152.10 - 40) = 112.10\ KN/m^2$.

And,
change in effective stress at 0.8 m = $(112.10 - 110.24)$
$= 1.86 KN/m^2$

Additional Practise (2012 Spring and 2011 Spring)

A 10m thick bed of sand is underlain by a layer of clay 6.5m thickness. The water table which was originally at the ground surface is lowered by drainage to a depth of 4.2m where the degree of saturation above the lowered water table reduces to 50%. Determine the increase in magnitude of effective vertical pressure at the mid of clay layer due to lowering of water table. The γ_{sat} of sand is 20.6, γ_{sat} for clay = 17.6 & γ_d of sand is 16.7 KN/m³.

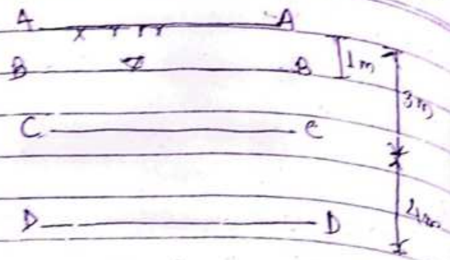
→ Solution:

11. A soil deposit consists of an upper layer of 3m thick having a unit weight of 18 kN/m³ and lower layer of 4m thick having a unit weight of 22 kN/m³. Determine the total stress, pore pressure and effective stress at the bottom of lower layer (i.e. at 7m below ground surface) if the water table is

a) 1m below the ground surface.
b) Exactly at the ground surface
c) 1m above the ground surface.

→ Solution:

Case - I : at D-D.

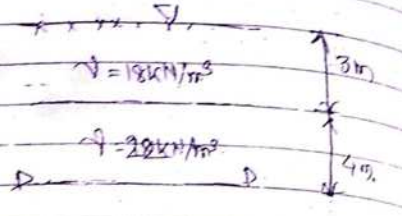


$\sigma = (18*3 + 22*4) = 142\ kN/m^2$

$u = 10*6 = 60\ kN/m^2$

$\therefore \bar{\sigma} = \sigma - u$
$= (142-60)\ kN/m^2$
$= 82\ kN/m^2$.

Case II: At D-D

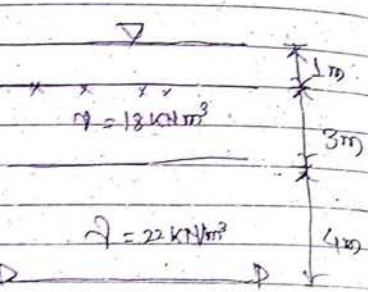


$\sigma = (18*3+22*4) = 142\ kN/m^2$

$u = 10*7 = 70\ kN/m^2$

$\therefore \bar{\sigma} = \sigma - u$
$= 142-70 = 72\ kN/m^2$

Case III At D-D.



$\sigma = (10*1 + 18*3 + 22*4)$
$= 152\ kN/m^2$

$u = 10*8 = 80\ kN/m^2$

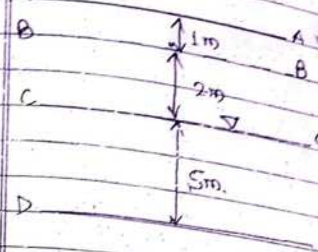
$\therefore \bar{\sigma} = \sigma - u$
$= 152-80$
$= 72\ kN/m^2$

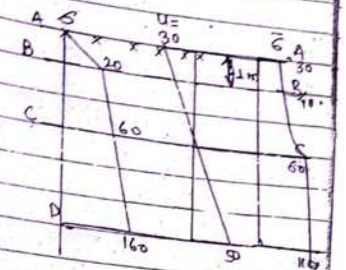
Additional Practice (2011 Fall)

The following informations were obtained from borehole test. ~~0 to 3.5~~ 0-3.5m depth - fine sand with saturated unit weight = 1620 kg/m³, 3.5 to 8.0 m depth - clay with saturated unit weight = 2000 kg/m³ and below 8.0m - medium sand. The water table was at the ground level, calculate and draw diagrams showing the

12. The water table in a deposit of sand 8m thick is at a depth of 3m below the surface. The sand is saturated above the water table. The bulk density of sand is 20 kN/m³. Calculate the effective pressure at 1m, 3m and 8m below the surface. Also plot the variation of total pressure, neutral pressure and effective pressure over the depth of 8m.

→ Solution:-





At A-A

$\sigma = 0 \quad u = -10*3 = -30\ kN/m^2$

$\therefore \bar{\sigma} = \sigma - u = 30\ kN/m^2$.

At B-B

$\sigma = 20*1 = 20\ kN/m^2$

$u = -10*2 = -20\ kN/m^2$.

$\therefore \bar{\sigma} = \sigma - u = 40\ kN/m^2$

At C-C:- $\sigma = 20*3 = 60\ kN/m^2$ & $u = 0$.

$\therefore \bar{\sigma} = 60\ kN/m^2$.

At D-D

$\sigma = 20*3 + 20*5 = 160\ kN/m^2$

$u = 10*5 = 50\ kN/m^2$

$\sigma - u = \bar{\sigma} = 110\ kN/m^2$

14. Calculate the critical hydraulic gradient in a saturated soil with a specific gravity of 2.68 and water content of 50%.

→ Solution:-

Given that:

Specific gravity (G) = 2.68

water content (ω) = 50% = 0.5

We have,

$$\omega G = S_r e$$

$$\therefore e = \frac{\omega G}{S_r} = \frac{0.5 \times 2.68}{1} = 1.34.$$

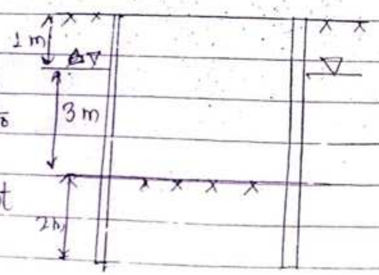
Now, Critical hydraulic gradient is

$$i_c = \frac{\gamma'}{\gamma_w} = \frac{G-1}{1+e} = \frac{2.68-1}{1+1.34} = 0.72$$

15. A trench is excavated in fine sand up to a depth of 4m. The excavation is carried out by providing necessary side supports and pumping water. The position of water table is 1m below the ground surface at the sides of excavation and there is no water inside the excavation. Examine the possibility of quick sand condition if G = 2.64 and e = 0.7.

→ Solution:-

Given that:-



Sheeting is taken at 2m depth below the bottom of the trench to increase the seepage path.

The equation for critical gradient is

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

If the trench is to be stable, the hydraulic gradient i is

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

There will be no quick condition if,

$$\frac{h}{L} < \frac{G-1}{1+e}$$

For the given data,

$$i_c = \frac{G-1}{1+e} = \frac{2.64-1}{1+0.7} = 0.96$$

$$\therefore \frac{h}{L} = \frac{3}{2} = 1.5$$

Here, $\frac{h}{L} > i_c$, hence quick sand condition occurs.

If, L is more than 4m, then $i = h/L = 3/4 = 0.75 < 0.96$,

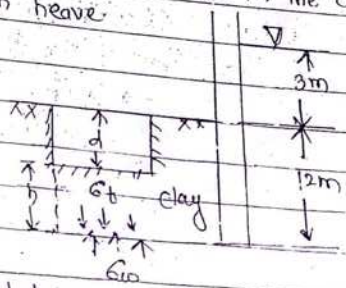
So, no quick sand condition occurs.

16). A 12m thick layer of relatively impervious stratum of clay lies over a bed of gravel aquifer. Piezometer tubes introduced to the gravel layer level standing in the tubes 3m above the top surface of the clay strata. The properties of the clay are e = 1.2 and G = 2.7. Determine

a) the effective stress at the top level of gravel strata.

b) The depth of excavation that can be made in the clay strata without the bottom heave.

→ Solution:-



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

$$= \frac{(2.7+1.2)\times 10}{1+1.2}$$

$$= 17.73 \text{ kN/m}^3$$

i) At the top level of gravel strata,

Gravel

$\sigma = 12 * 17.73 = 212.76 \, kN/m^2$

$u = 15 * 10 = 150 \, kN/m^2$.

$\therefore \bar{\sigma} = \sigma - u = (212.76 - 150) = 62.76 \, kN/m^2$.

ii) Depth of excavation must be such that,

$\sigma = u_w$ ∵ Heave occurs only when $\bar{\sigma} = 0$ i.e. $\sigma = u_w$

Let h be the height of the bottom of excavation from gravel layer

$\therefore \sigma = \gamma_{sat} * h = 17.73h \, kN/m^2$.

Again,

$u_w = 150 \, kN/m^2$.

$\therefore 17.73h = 150$.

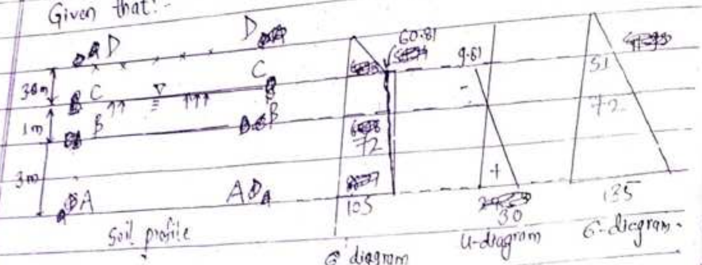
$\therefore h = 8.5 \, m$.

$\therefore$ Depth of excavation $= (12 - 8.5) \, m = 3.5 \, m$

17. A sand deposit consists of two layers. The top layer is 3.0 m thick ($\gamma = 17 \, kN/m^3$) and the bottom layer is 4m thick ($\gamma_{sat} = 21 \, kN/m^3$). The water table is at a depth of 4m from the surface and the zone of capillary saturation is 1m above the water table. Draw the diagrams, showing the variation of total stress, neutral stress and effective stress.

> Solution:-

Given that:-



$\gamma = 1709.67 * 9.81 * 10^{-3} = 16.77 \, kN/m^3$.

~~$\gamma_{sat} = 2064.52 * 9.81 * 10^{-3} = 20.25 \, kN/m^3$.~~

~~At AA~~

~~$\sigma = 0, \; u = 0 \quad \sigma' = \sigma - u = 0$.~~

~~At BB (immediately above)~~

Solution:-

At AA:-

$\sigma = 3.0 * 17 + 4 * 21 = 135 \, kN/m^3$

$u = 3 * 10 = 30 \, kN/m^2$

$\therefore \sigma' = (\sigma - u) = 135 - 30 = 105 \, kN/m^3$.

At BB:-

$\sigma = 3.0 * 17 + 1 * 21 = 72 \, kN/m^3$

$u = 0$.

$\therefore \sigma' = \sigma - u = 72 \, kN/m^3$

At CC (immediately above)

$\sigma = 3 * 17 = 51 \, kN/m^2$

$u = 0$,

$\therefore \sigma' = 51 \, kN/m^2$.

At CC (immediately below)

$\sigma = 3.0 * 17 = 51$

$u = -1 * 9.81 = -9.81 \, kN/m^2$.

$\therefore \sigma' = (\sigma - u) = 51 + 9.81 = 60.81 \, kN/m^3$.

At D-D

$\sigma = 0, \quad u = 0 \quad \therefore \sigma' = 0$.

18. A sand deposit is 12m thick and overlies a bed of soft clay. The ground water table is 4m below the ground surface. If the sand above the water table has a degree of saturation of 40%, plot the diagram showing

the variation of total stress, pore pressure and effective stress. The void ratio of sand is 0.8 and G = 2.7.

> Solution:-

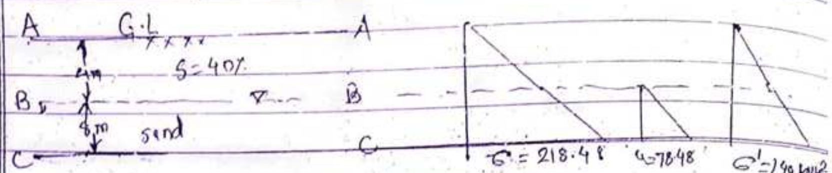
Given that:-

$G = 2.7$, $e = 0.8$ $S = 40\%$

Now,

$\gamma = \frac{(G+Se)\gamma_w}{1+e} = \frac{(2.7+0.4*0.8)9.81}{1+0.8} = 16.46\ kN/m^3$

$\gamma_{sat} = \frac{(G+e)\gamma_w}{1+e} = \frac{(2.7+0.8)*9.81}{1+0.8} = 19.08\ kN/m^3$



~~Total stress(σ) :-~~

At point A-A,

$\sigma = u = \sigma' = 0$,

At point B-B (4m below G.L):-

$\sigma = 16.46*4 = 65.84\ kN/m^2$

$u = 0$

$\sigma' = \sigma - u = 65.84\ kN/m^2$.

At C-C (12m below G.L)

$\sigma = 16.46*4 + 19.08*8 = 218.48\ kN/m^2$.

$u = 9.81*8 = 78.48\ kN/m^2$.

$\therefore \bar{\sigma} = \sigma - u$

$= 218.48 - 78.48$

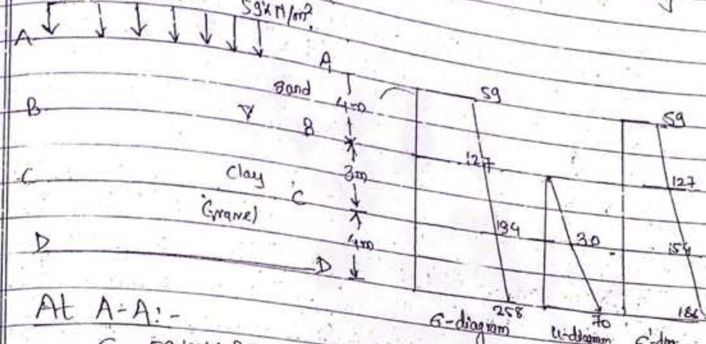
$= 140\ kN/m^2$

Additional Practise (2015 Spring) (Same as above)

19. A soil profile consists of surface layer of sand 4.0m thick ($\gamma = 17\ kN/m^3$) and intermediate layer of clay 3.0m thick ($\gamma = 19\ kN/m^3$), and the bottom layer of gravel 4.0m thick ($\gamma = 18.5\ kN/m^3$). The water level is at the upper surface of the clay layer. Determine the effective pressure at various levels immediately after the placement of a surcharge load of 59 kN/m² to the ground surface.

> Solution:-

Given that:-



At A-A:-

$\sigma = 59\ kN/m^2$, $u = 0$ $\therefore \bar{\sigma} = 59\ kN/m^2$.

At BB

$\sigma = (59 + 17*4) = 127\ kN/m^2$.

$u = 0$

$\therefore \bar{\sigma} = \sigma - u = 127\ kN/m^2$

At C-C:-

$\sigma = 59 + 17*4 + 3*19 = 184\ kN/m^2$

$u = 3*10 = 30\ kN/m^2$.

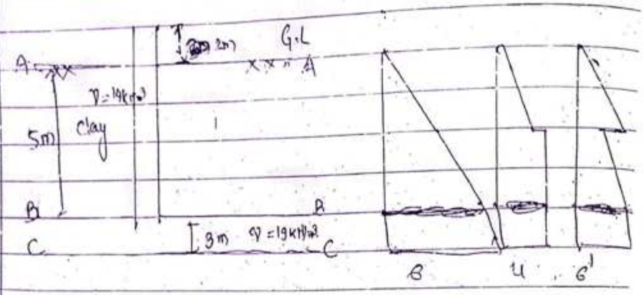
$\therefore \bar{\sigma} = \sigma - u = 154\ kN/m^2$.

At D-D

$\sigma = (59 + 17*4 + 19*3 + 18.5*4) = 258\ kN/m^2$ $u = 10*7 = 70\ kN/m^2$

20. A soil profile consists of a surface layer of clay 5m thick with unit weight 14 KN/m³ and a sand layer 3 m thick with unit weight 19 KN/m³ overlying an impermeable rock. The water table is at the ground surface. If the water table in a standpipe driven into the sand layers rises 2m above the ground surface then, draw the plot showing the variation of total stress, pore water pressure and effective stress.

→ Solution:-



At A-A $\sigma = \bar{\sigma} = u = 0$.

At B-B:- 5m below G.L

$\sigma = 5*14 = 70$ KN/m²

$u = 9.81*5 = 49.05$ KN/m².

$\sigma' = 70 - 49.05 = 20.95$ KN/m²

At B-B (5m below G.L) for sand layer.

$\sigma = 14*5 = 70$ KN/m²

$u = 9.81*(5+2) = 68.48$ KN/m².

$\therefore \bar{\sigma} = 70 - 68.48 = 1.52$ KN/m².

At C-C (8m below G.L)

$\sigma = 14*5 + 19*3 = 127$ KN/m².

$u = 9.81*(8+2) = 98.1$ KN/m².

$\bar{\sigma} = \sigma - u = 28.9$ KN/m²

23. A masonry dam has pervious sand as foundation. Determine the maximum permissible upward gradient if a factor of safety of 5 against quick sand condition, take porosity, n=45% and $G_s = 2.65$.

→ Solution:-

Critical hydraulic gradient, $i_c = \frac{G-1}{1+e}$

$\because e = \frac{n}{1-n} = \frac{0.45}{1-0.45} = 0.82$.

$\therefore i_c = \frac{2.65-1}{1+0.82} = 0.9$.

Hence, permissible hydraulic gradient $= \frac{i_c}{FS} = \frac{0.9}{5} = 0.18$

22. In a site reclamation project, 2.5m of graded fill ($\gamma = 22 kN/m^3$) were laid in the compacted layers over on existing layer of silty clay ($\gamma = 18 KN/m^3$) which has 3m thick. This was underlain by a 2m thick layer of gravel ($\gamma = 20 KN/m^3$). Assuming the water table remains at the surface of silty clay, draw the effective stress profiles for case after the fill has been placed.

→ Solution:-

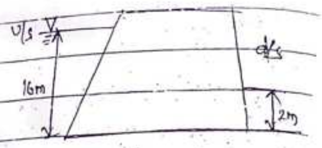
RAKESH KUMAR SAH

Tutorial- 4

Ch-7: Seepage Analysis through Soils.

1. A concrete dam is constructed across a river over permeable strata of the soil of limited thickness. The water head on u/s side is 16m and on the d/s side is 2m. The flow net constructed under the dam gives $N_f = 7$, $N_d = 21$. Calculate the seepage loss through the subsoil if the average value of coefficient of permeability is $6*10^{-3}$ cm/sec horizontally and $3*10^{-4}$ cm/sec vertically. Calculate the exit gradient if the average length of the last field is 0.9m.

→ Solution:-



Given that:-

No. of flow channels (N_f) = 7

No. of equipotential drops (N_d) = 21

Water level on U/s side (h_1) = 16m

water level on d/s side (h_2) = 2m

∴ Total hydraulic head causing flow (h) = $h_1 - h_2 = 16 - 2 = 14m$

Permeability in horz. direction (K_H) = $6*10^{-3}$ cm/s.

Permeability in vertical direction (K_V) = $3*10^{-3}$ cm/s

$$\therefore K_e = \sqrt{K_V \cdot K_H} = \sqrt{6*10^{-3} * 3*10^{-3}} = 1.34*10^{-3} \text{ cm/sec}$$

Again,

$$q = k_e h \frac{N_f}{N_d} = 1.34*10^{-3} * 14 * \frac{7}{21} = 6.26*10^{-3} \text{ cm/s}$$

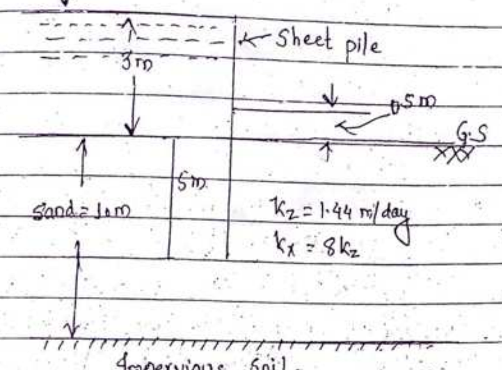
Head loss per potential drop $= \Delta H = \frac{h}{N_d} = \frac{14}{21} = 0.67$

$$\therefore \text{The exit gradient } (i) = \frac{\Delta h}{l_e} = \frac{0.67}{0.9} = 0.74$$

2). A sand deposit 10m thick overlies an impervious soil. A vertical sheet pile penetrates halfway into the sand deposit. The water level on one side of the wall is 3.0m and on the other side is 0.5m above the ground level. The sand stratum has a vertical permeability of 1.44m per day, and the horizontal permeability equals 8 times the permeability in the vertical direction. A flow net construction reveals that there are 12 flow channels and 26 potential drops. Determine the seepage flow per day.

→ Solution:-

Given that:



$K_x = 8k_z$.

$k_z = 1.44$ m/day.

The equivalent coefficient of permeability,

$$k_e = \sqrt{k_x \cdot k_z} = \sqrt{8k_z \cdot k_z} = \sqrt{8}\, k_z = \sqrt{8} * 1.44$$

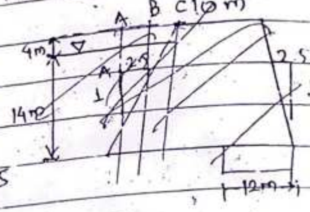
$\therefore k_e = 4.07$ m/day.

The seepage is given by

$$q = k_e h \frac{N_f}{N_d}$$

$$= 4.07\,(3 - 0.5) * \frac{12}{26} = 4.7 \text{ m}^3/\text{s}/\text{m}$$

3. The section of homogeneous earth dam is shown in fig. 1. The coefficient of permeability in the horizontal and vertical directions are $8*10^{-7}$ cm/sec and $3.6*10^{-7}$ cm/sec respectively. Estimate the seepage through the dam section.



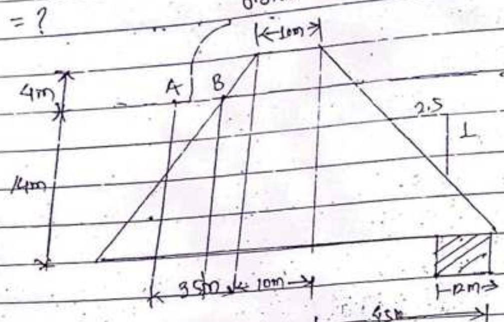
→ Solution:-

Given that:

$K_x = 8*10^{-7}$ cm/s

$k_z = 3.6*10^{-7}$ cm/s

$q = ?$



$K = \sqrt{k_x \cdot k_z} = \sqrt{8*10^{-7} * 3.6*10^{-7}} = 5.36*10^{-7}$ cm/s $\simeq 5.36*10^{-9}$ cm/s.

$b = (45-12)+10+10+10.5 = 63.5$ m

$S = \sqrt{b^2+h^2} - b = \sqrt{63.5^2+14^2} - 63.5$

$= 1.52$

Now,

$q = kS$

$= 5.36*10^{-9} * 1.52$

$= 8.14*10^{-9}$ m³/s/m.

Additional Practice (2008 Spring, 2011 Spring)
Solution same as above question.

4. A homogeneous earth dam 30m high has free board of 1.5m. a flat flow net was constructed and the following results were noted: nos of potential drop-12, nos. of flow channels-3. The dam has a 18m long horizontal filter at its downstream end. Calculate the seepage loss across the dam per day, if the width of dam is 200m and the coefficient of permeability of the soil is $3.35*10^{-4}$ cm/sec.

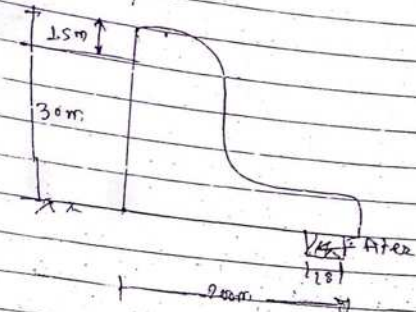
→ Solution:-

Given that,

$N_d = 12$

$N_f = 3$

$k = 3.35*10^{-4}$ cm/sec



∴ Head loss,

$H = 30 - 1.5$

$= 28.5$ m.

Now,

We know that,

Discharge $(Q) = kH \frac{N_f}{N_d}$

$= 3.35*10^{-6} * 28.5 * \frac{3}{12}$

$= 2.38*10^{-3}$ m³/sec

$\simeq 2.06$ m³/day

5. A 30m high dam has a top width of 10m and a free board of 5m. The dam is made of coarse sand having coefficient of permeability 4.5×10^{-4} cm/sec. A 60m long horizontal drainage blanket is placed near the d/s end of the dam. Determine the quantity of seepage loss if the bottom width of the dam be 185m.

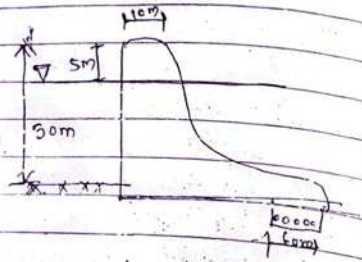
→ Solution:-

Given that,

$K = 4.5 \times 10^{-4}$ cm/sec.

$H = 30-5 = 25$ m

Length of dam (L) = 185 m.



Here,

$\frac{x}{1} = \frac{87.5}{30}$

$\therefore x : 1 = H : V = 2.91 : 1 = \frac{35}{12} : 1$

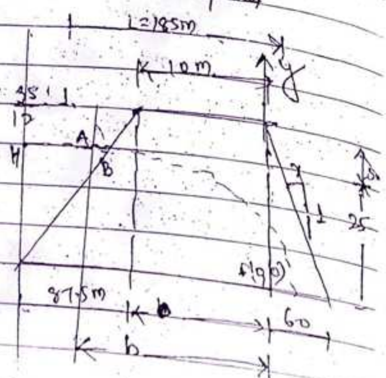
$\therefore HB = \frac{25 \times 35}{12} = 72.917$ m

$AB = 0.3 HB = 0.3 \times 72.917$

$= 21.875$ m

$b = 185 - 60 - 0.7 \times 72.917$

$= 73.958$



&

$S = \sqrt{b^2 + h^2} - b$

$= \sqrt{73.958^2 + 25^2} - 73.958$

$= 4.111$ m

Now, Seepage discharge $q = KS$

$= 4.5 \times 10^{-6} \times 4.111$

$= 1.85 \times 10^{-5}$ m³/sec/m

$\simeq 18.5$ cm³/sec/m &

6. A flow net for flow around a single row of sheet pile in a permeable soil is shown in fig. below. If $K_x = K_z = 5 \times 10^{-2}$ mm/sec, determine:

i) How high (above ground surface) will the water rise, if measured by piezometers placed at A, B, C and D?

ii) What will be the rate of seepage through the flow channel-II per unit length of the sheet pile?

iii) What is total seepage per unit length through permeable layer?

→ Solution:

Date ____ Page ____

Rakesh Kumar Sah

Tutorial - 5

Ch-8- Vertical Stresses below applied load.

1. A water tower has a circular foundation of 10m diameter. If the total weight of the tower, including the foundation is $2*10^5$ KN. Calculate the vertical stress at a depth of 3m below the centre of foundation.

→ Solution:-

Given that;

Radius (R) = 10/2 = 5m.

Total weight = $2*10^5$ KN

depth (z) = 3m

Vertical stress (σ_z) = ?

We have,

$$I_c = 1 - \frac{1}{\left[1+\left(\frac{R}{z}\right)^2\right]^{3/2}} = 1 - \frac{1}{\left\{1+\left(\frac{5}{3}\right)^2\right\}^{3/2}} = 0.8638$$

$$q = \frac{load}{area} = \frac{2*10^5}{\pi/4*10^2} = 2546.48 \text{ KN/m}^2.$$

$$\therefore \sigma_z = I_c . q = 0.8638 * 2546.48$$
$$= 2199.67 \text{ KN/m}^2$$

Additional Practice (2012 Fall)

A Raft foundation of 8m dia is placed on the ground surface and carries a load of 10,000 KN including self-weight. Calculate the vertical stress increment at the centre of clay layer using Boussinesq's theory. The clay layer is 2m thick and it exist 5m below ground surface.

→ Solution:-

Given that,

Diameter of foundation (d) = 8m

Load (Q) = 10,000 KN.

Date ____ Page ____

Now,

$$q = \frac{Q}{A} = \frac{10,000}{\pi/4*8^2} = 198.94 \text{ KN/m}^2$$

Now, vertical stress increment is given by

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

$$= 198.94\left[1 - \left\{\frac{1}{1+\left(\frac{4}{6}\right)^2}\right\}^{3/2}\right] \quad \because z = 5+1 = 6$$

$$= 84.34 \text{ KN/m}^2$$

2. A water tower weighing 15,000 KN is to be considered as concentrated load acting on the ground surface. Compute the vertical stress at a depth of 8meters below the surface. Also compute the vertical stress at a distance of 7m away from the center of the water tower.

→ Solution:-

Given that:-

load of water tower (Q) = 15,000 KN.

z = 8m r = 0

σ_z = ?

We have,

Vertical stress at a depth 8m,

$$\sigma_z = I_B . \frac{Q}{z^2} \quad \ldots\ldots (1)$$

$$I_B = \frac{3}{2\pi} . \frac{1}{\left[1+\left(\frac{r}{z}\right)^2\right]^{5/2}} = \frac{3}{2\pi} . \frac{1}{[1+0]^{5/2}} = 0.477$$

from eqn (1),

$$\sigma_z = \frac{0.477 * 15000}{8^2} = 111.90 \text{ kN/m}^2$$

Case-II:-

$r = 7m$, $z = 8m$, $Q = 15,000$ kN.

$$I_B = \frac{3}{2\pi} * \left[\frac{1}{\left[1+\left(\frac{7}{8}\right)^2\right]}\right]^{5/2} = 0.115.$$

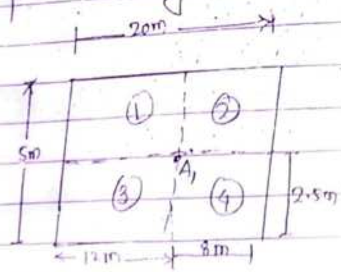
∴ Vertical stress at a distance 7m from centre of the water tower,

$$\sigma_z = \frac{0.115 * 15000}{8^2} = 27.015 \text{ kN/m}^2$$

Additional Practice [2015 Spring]:-
Solution same as above.

3. A strip footing is given in plan as shown in fig. 1. The load per unit area is 300 kN/m². Determine the intensity of vertical stress at a point 5m directly below point A_1.

→ Solution:-



Given that,
$q = 300$ kN/m²
$B = 20$m
$z = 5$m

Now,
$Q = q * A$
$= 300 * 20 * 5 = 30,000$ kN.

For 1, $m = L/z = \frac{12}{5} = 2.4$ ⇒ $I_1 = 0.13$

$n = B/z = \frac{2.5}{5} = 0.5$ ~~⇒ $I_1 = 0.125$~~

for (2),
$m = 8/5 = 1.6$, $n = \frac{2.5}{5} = 0.5$ ⇒ $I_2 = 0.125$

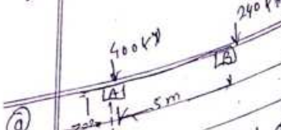
for (3),
$m = 12/5 = 2.4$ & $n = 2.5/5 = 0.5$ ⇒ $I_3 = 0.13$

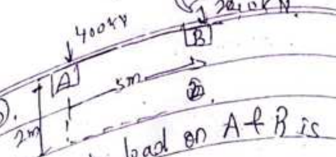
for (4),
$m = 8/5 = 1.6$, $n = 2.5/5 = 0.5$ ⇒ $I_4 = 0.125$.

∴ $I = I_1 + I_2 + I_3 + I_4 = 0.51$.

∴ Vertical stress $(\sigma_z) = q.I$
$= 300 * 0.51$
$= 153$ kN/m²

9 – 2. Two columns A and B are placed at 5meter center to center. Through point A, a load of 400kN is acting and from point B a load of 240kN. Calculate the vertical stress due to these loads on a horizontal plane 2m below the ground surfaces at points: (a) vertically below the points A and B (b) 10m horizontally away from point A.

(a) 

(b) 

Now, vertical stress at point ① due to load on A & B is

Due to A, $I_B = \frac{3}{2\pi}$

$(\sigma_z)_A = \frac{I_B \cdot Q}{z^2} = \frac{3}{2\pi}\cdot\frac{400}{2^2} = 47.746\ kN/m^2$
$= 4.86\ t/m^2$

$\Rightarrow R = 5m$

Due to B, $I_B = \frac{3}{2\pi}\frac{1}{\{1+(5/2)^2\}^{5/2}} = 3.373\times10^{-3}$

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

$(\sigma_z)_B = \frac{I_B\cdot Q}{z^2} = 3.373\times10^{-3}\times\frac{240}{2^2} = 0.202\ kN/m^2$

∴ Total stress vertically below point A is,
$(\sigma_z)_1 = (\sigma_z)_A + (\sigma_z)_B = 47.948\ kN/m^2 = 4.887\ t/m^2$

Again,
Vertical stress at point ② due to load A & B,

Due to load B,
$(\sigma_z)_B = \frac{I_B\cdot Q}{z^2} = \frac{3}{2\pi}\times\frac{240}{2^2} = 28.647\ kN/m^2$

Due to load A,
$I_B = \frac{3}{2\pi[1+(R/2)^2]^{5/2}} = \frac{3}{2\pi[1+(5/2)^2]^{5/2}} = 3.373\times10^{-3}$

$(\sigma_z)_A = \frac{I_B\cdot Q}{z^2} = 3.373\times10^{-3}\times\frac{400}{2^2} = 0.337\ kN/m^2$

∴ Total stress vertically below point B is
$(\sigma_z)_2 = (\sigma_z)_B + (\sigma_z)_A$
$= 28.984\ kN/m^2$
$\approx 2.955\ t/m^2$

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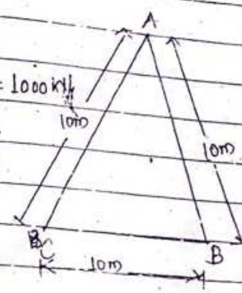
5. A water tank in three pillars is in the form of an equilateral triangle in plan and the sides of the triangle are 10 meters. The total weight of water tower is 1000 kN. Find the vertical stress 10m below the ground surface under any one of the legs.

Solution:-

Given that:-

Total weight of tower = 1000 kN



Since, load is coming equally on three legs of the tower,

∴ Load on each leg $= \frac{1000}{3}$

$= 333.33\ kN$.

Now, vertical stress at 10m below one of the leg (say below A) is caused by three point loads, coming through the three A, B, C legs.

i) Vertical stress caused by load on leg A, below which the stress is computed, $Q = 333.33\ kN$, $z = 10$ & $r = 0$

∴ $\sigma_{z1} = \frac{3Q}{2\pi z^2}\left[\frac{1}{1+(r/z)^2}\right]^{5/2}$

$= \frac{3*333.33}{2\pi*10^2}\left[\frac{1}{1+0^2}\right]^{5/2} = 1.59\ kN/m^2$

ii) Vertical stress caused by load on either of the other two legs B and C, ∵ $Q = 333.33$, $z = 10m$ $r = 10m$

∴ $\sigma_{z2} = 0.2813\ kN/m^2 = \sigma_{z3}$

∴ $\sigma_z = \sigma_{z1} + \sigma_{z2} + \sigma_{z3} = (1.59 + 0.28 + 0.28)\ kN/m^2$
$= 2.15\ kN/m^2$

6. A point load of 132 kN is applied at the ground surface. Construct a pressure bulb when the stress imposed becomes 20% of the applied load.

→ Solution:-

Given that:-

Load $(Q) = 132$ kN.

We have,

$$\sigma_z = \frac{I_B * Q}{Z^2}$$

or, $$0.2Q = \frac{I_B * Q}{Z^2}$$

$$\therefore I_B = 0.2 Z^2 = \frac{3}{2\pi\left[1+\left(\frac{r}{z}\right)^2\right]^{5/2}}$$

Putting $r=0$ in above eqn, $z = 1.545$. We can get the values of r for different values of z as tabulated below,

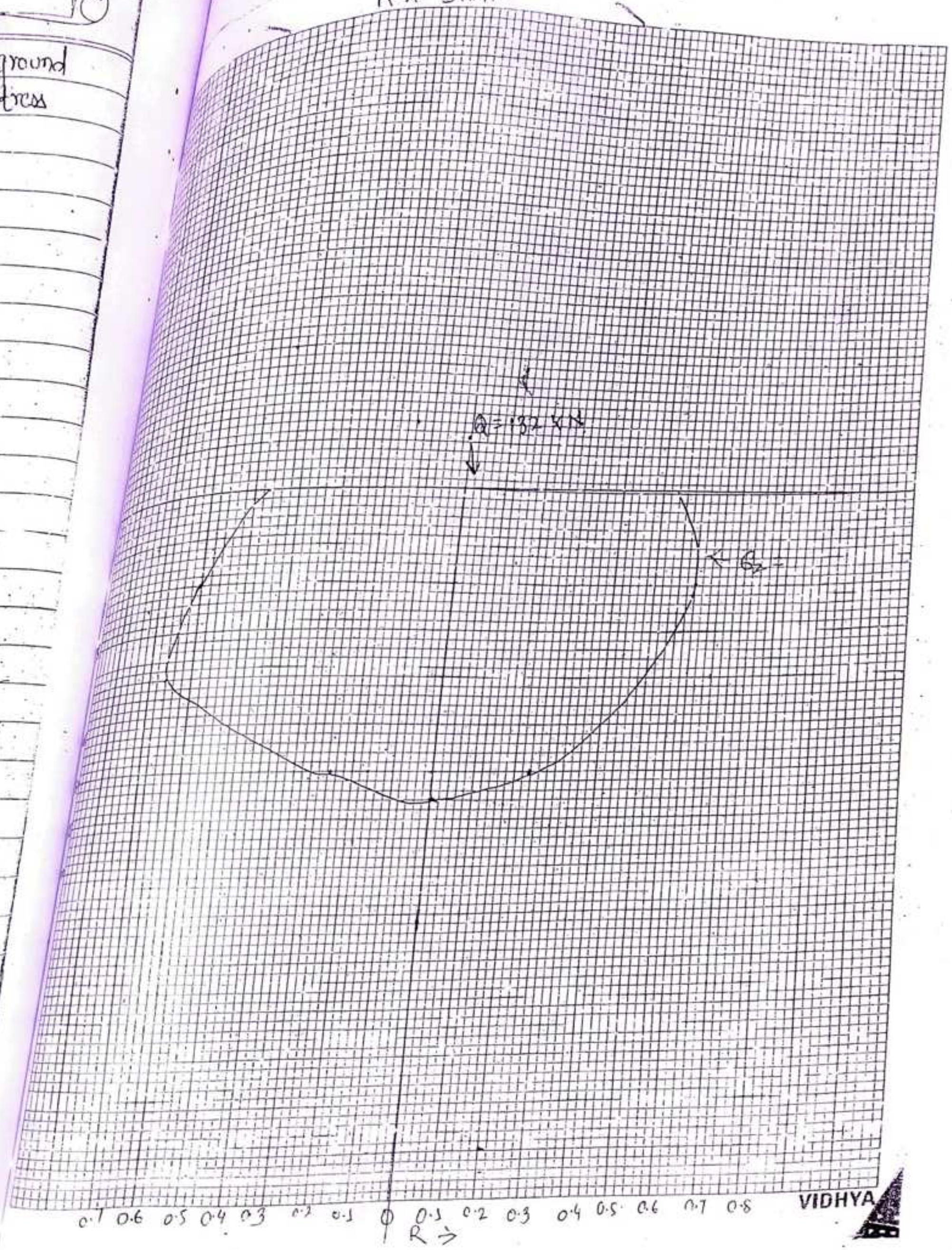
z	0.5	1	1.5	1.545
I_B	0.05	0.2	0.45	0.477
r/z	1.21	0.645	0.154	≈ 0.00
r	0.60	0.645	0.232	0.00

Now,

The above data is plotted in graph.

- Additional practise (2013 Spring)
- Additional practise (2016 spring)

Solution same as above.



VIDHYA

7. A water tower has circular foundation of diameter 9m. The total weight of the tower including the foundation is 17,500 kN. A very weak foundation capacity of 95 kN/m² lies 3m below the foundation level. Calculate the stress due to foundation at the top of the weak stratum. Give your comment with regard to the feasibility of the foundation construction at the top surface of the weak layer.

→ Solution:-

Given that,

Diameter of foundation (d) = 9m

Foundation Area (A) = $\frac{\pi d^2}{4} = \frac{\pi \times 9^2}{4} = 63.62 m^2$.

Load per unit area (q) = $\frac{17500}{63.62} = 275.07 KN/m^2$.

Again, $R/z = 4.5/3 = 1.5$

$\therefore \sigma_z = q \left[1 - \left\{\frac{1}{1+(R/z)^2}\right\}^{3/2}\right] = 275.07 * \left[1 - \left\{\frac{1}{1+1.5^2}\right\}^{1.5}\right]$

$= 228.12 KN/m^2$.

∴ Pressure increment at the top of the weak stratum due to construction of the water tower will be 228.12 kN/m². While the bearing capacity of the stratum is 95 kN/m² only. Hence, it is not safe to construct the water tower with the proposed foundation size.

8. A concentrated point load of 250 kN acts at the ground surface. Find the intensity of vertical pressure at depth of 5m below the ground surface at the point on the axis of loading using Boussinesq analysis. What will be the difference in the

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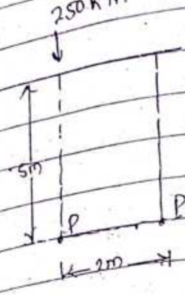
vertical pressure at the same point if the load shifted to a distance of 2m from its original position horizontally.



Solution:-

Case I:-

At point P,

$Q = 250\,kN$, $z = 5m$,

$r = 0$

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

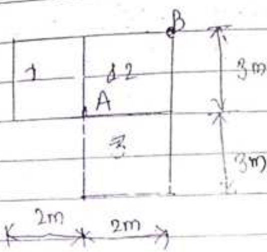
$$= \frac{3*250}{2\pi*5^2\left[1+0^2\right]^{5/2}} = 4.77\,kN/m^2.$$

Case II:- $z = 5m$, $r = 2m$.

$$\therefore \sigma_z = \frac{3Q}{2\pi z^2\left[1+\left(\frac{r}{z}\right)^2\right]^{5/2}} = \frac{3*250}{2\pi*5^2\left[1+\left(\frac{2}{5}\right)^2\right]^{5/2}} = 3.29\,kN/m^2$$

∴ Difference in vertical pressure $= 4.77 - 3.29 = 1.48\,kN/m^2$

9) Using Fadum's chart, determine the increase in vertical stress at 2m depth below A and B due to uniformly distributed load of $40\,kN/m^2$ on the ground surface shown by the area in the fig: 2.



Solution:-

Given that:-

for point A,

$z = 3m$, $q = 40\,kN/m$

for loaded area 1,

$m = \frac{B}{z} = \frac{2}{2} = 1$

$n = \frac{L}{z} = \frac{2}{2} = 1$

For m = 1 & n = 1, from Fadum chart, $I_N = 0.18$

$\therefore \sigma_z = 3*q*I_N = 3*40*0.18 =$ ~~26.4 kN/m²~~ $21.6\,kN/m^2$.

for the point B,

$A_1 = m = \frac{B}{z} = \frac{4}{2} = 2$ & $n = \frac{L}{z} = \frac{2}{2} = 1$, $I_N = 0.24$

$(Area)_2 = m = \frac{B}{z} = \frac{2}{2} = 1$ & $n = \frac{4}{2} = 2$, $I_N = 0.21$

$(Area)_3$ $m = \frac{B}{z} = \frac{2}{2} = 1$ & $n = \frac{L}{z} = \frac{2}{2} = 1$, $I_N = 0.21$

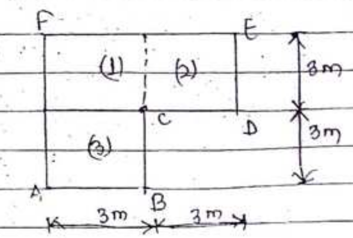
$\therefore \sigma_z = q\left((I_N)_1 + (I_N)_2 \mp (I_N)_3\right)$

$= 40(0.21 + 0.24 - 0.21)$

$= 9.6\,kN/m^2$

10) A uniformly distributed load ABCDEF having a plan area as shown in fig: 3 is placed on the ground surface. The intensity of the load is $20\,kN/m^2$. Determine the increase in the vertical stress due to the load at 6m depth below the corner points C and D.

Solution:-



Below corner point C,

For loaded area 1,

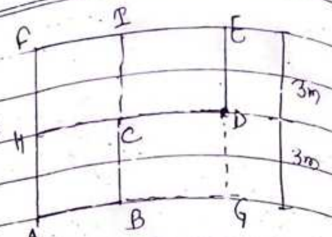
$m = \frac{B}{z} = \frac{3}{6} = 0.5$

$n = \frac{L}{z} = \frac{3}{6} = 0.5$

For m = 0.5 & n = 0.5, from Fadum chart, $I_N = 0.0818$.

$\therefore \sigma_z = 3*q*I_N = 3*20*0.0818 = 4.91\,kN/m^2$.

Below the corner point D,
Area 1 = Area of rect. FEDH,
$m = \frac{B}{z} = \frac{3}{6} = 0.5$ & $n = \frac{L}{z} = \frac{6}{6} = 1.0$



For $m = 0.5$ & $n = 1$, $I_{N1} = 0.1187$.

Area 2 = Area of rect HDGA,
$m = \frac{B}{z} = \frac{3}{6} = 0.5$ & $n = \frac{L}{z} = \frac{6}{6} = 1$.

∴ for $m = 0.5$ & $n = 1$, $I_{N2} = 0.1187$.

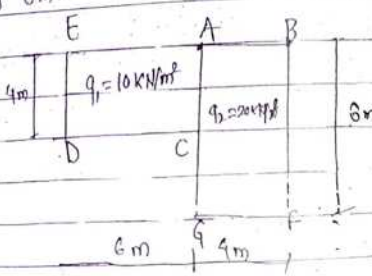
Area 3 = Area of rect. CDGB
$m = \frac{B}{z} = \frac{3}{6} = 0.5$ & $n = \frac{L}{z} = \frac{3}{6} = 0.5$

For $m = 0.5$ & $n = 0.5$, $I_{N3} = 0.0818$.

$\therefore \sigma_z = q[(I_N)_1 + (I_N)_2 - (I_N)_3]$
$= 20[0.1187 + 0.1187 - 0.0818]$
$= 3.112\ kN/m^2$

11). Two uniformly distributed loads of rectangular pattern with different load intensity are placed side by side on the ground surface as shown in the fig4. Determine the vertical ~~stress~~ pressure due to the load below point A and B at a depth of 6m.

→ Solution:-



Case I:- for point A,
Area1 = Rect. ACIE
$m = \frac{B}{z} = \frac{6}{6} = 1$

$n = \frac{L}{z} = \frac{4}{6} = 0.67$

we get,

$$I_N = \frac{1}{4\pi} * \left[\left\{ \frac{2mn * \sqrt{m^2+n^2+1}}{m^2+n^2+1+m^2n^2} \right\} * \frac{m^2+n^2+2}{m^2+n^2+1} \right\} + \tan^{-1}\left\{ \frac{2mn * \sqrt{m^2+n^2+1}}{m^2+n^2+1-m^2n^2} \right] \quad ...(1)$$

On substituting value of m & n, we get $(I_N)_1 = 0.145$

Area 2 = Rect. FBAG,
$m = \frac{B}{z} = \frac{4}{6} = 0.67$ & $n = \frac{L}{z} = \frac{6}{6} = 1$.

Putting these value in eqⁿ, we get, $(I_N)_2 = 0.145$.

$\therefore (\sigma_v)_A = (I_N)_1 * q_1 + (I_N)_2 * q_2$
$= 10 * 0.145 + 20 * 0.145$
$= 4.35\ kN/m^2$.

Case II:- for point B
Area 1 = Rect. BEDH,
$m = \frac{B}{z} = \frac{10}{6} = 1.67$ & $n = \frac{L}{z} = \frac{4}{6} = 0.67$

$\therefore (I_N)_1 = 0.159$ (from formula or we can use table).

Area 2: Rect. ABFG,
$m = \frac{B}{z} = \frac{4}{6} = 0.67$ & $n = \frac{L}{z} = \frac{6}{6} = 1 \Rightarrow (I_N)_2 = 0.145$

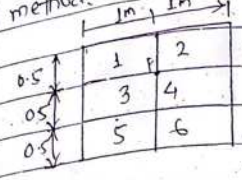
Area 3: Rectangle ABHC,
$m = \frac{B}{z} = \frac{4}{6} = 0.67$ & $n = \frac{L}{z} = \frac{4}{6} = 0.67$

$\therefore (I_N)_3 = 0.122$ (from eqⁿ. 1)

Thus,
$(\sigma_v)_B = (I_N)_1 * q_1 + (I_N)_2 * q_2 - (I_N)_3 * q_1$
$= 0.159 * 10 + 0.145 * 20 - 0.122 * 10$
$= 3.27\ kN/m^2$

R.K. SHAH

12) A rectangular foundation 2m * 1.5m carries a uniform load of 40 kN/m². Determine the vertical stress at P which is 3m below the ground surface as shown in fig. 5. Use equivalent point load method.



→ Solution:-

Given that

Rectangular area = $2*1.5 \, m^2$

Load intensity (q) = 40 kN/m²

Depth (z) = 3m.

The area is divided into 6 small area units as shown in the following figure.

∴ Load on each area = 40 * (1*0.5) = 20 kN/m².

The stress at point P are determined due to 6 points.

Using Boussinesq's sol^n.

for loads (1), (2), (3) & (4)

$$r = \sqrt{0.5^2 + 0.25^2} = 0.559 \, m.$$

$z = 3m.$

for loads (5) & (6),

$$r = \sqrt{0.5^2 + 0.75^2} = 0.901 \, m$$

$$\therefore \sigma_z = \sum_{i=1}^{6} \frac{3Q}{2\pi z^2} * \frac{1}{\left[1+\left(\frac{r_i}{z}\right)^2\right]^{5/2}}$$

$$= \frac{3*20}{2\pi*3^2} \left\{ \frac{4}{\left\{1+\left(\frac{0.559}{3}\right)^2\right\}^{5/2}} + \frac{2}{\left[1+\left(\frac{0.901}{3}\right)^2\right]^{5/2}} \right\}$$

$= 5.609 \, kN/m^2$

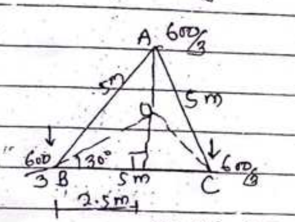
13) The plan of a three legged tower forms an equilateral triangle of side 5m. If the total weight of the tower is 600 kN and equally carried by all the legs, compute the vertical stress increase in the soil by the tower at a depth of 4m directly below the center of equilateral triangle.

→ Solution:-

Given that:-

Total weight of tower (Q) = 600 kN

height (z) = 4 m



$$\therefore \sigma_z = I_B \cdot \frac{Q}{z^2} \quad \ldots (1)$$

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

$\cos 30° = \frac{2.5}{BD}$

$\therefore r = 2.88 \, m$

$$= \frac{3}{2\pi\left[1+\left(\frac{2.88}{4}\right)^2\right]^{5/2}} = 0.168$$

From (1),

$$\sigma_z = \frac{0.168*600}{4^2} = 6.3 \, kN/m^2.$$

∴ Total stress = $3\sigma_z = 3*6.3 = 18.9 \, kN/m^2$.

Now,

Vertical stress (σ_z) due to load at A below 4m is

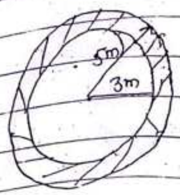
$(\sigma_z)_A =$

Total stress = $3*(\sigma_z)_A$

Additional practise (2012 Spring)

Solution same as above

14. A circular ring foundation for an over head tank transmits a constant pressure of 300 KN/m². Its internal diameter is 6m and external diameter is 10m. Compute the vertical stress on the central line of the footing at a depth of 6.5m below the ground level.



→ Solution:-

Given that:-

Pressure (q) = 300 KN/m²

Internal diameter = 6 m

External diameter = 10 m.

Vertical stress = ? for $z = 6.5m$

We have,

$\sigma_z = I_c \cdot q$.

$$I_c = 1 - \frac{1}{\{1+(\frac{R}{z})^2\}^{3/2}}$$

i) At $R = 5m$

$$I_{c1} = 1 - \frac{1}{\{1+(\frac{5}{6.5})^2\}^{3/2}} = 0.502.$$

$\therefore \sigma_{z1} = I_{c1} \cdot q = 0.502 * 300 = 150.6$

ii) At $R = 3m$,

$$I_{c2} = 1 - \frac{1}{\{1+(\frac{3}{6.5})^2\}^{3/2}} = 0.251.$$

$\therefore \sigma_{z2} = I_{c2} \cdot q = 0.251 * 300 = 75.45$

Hence,

Required stress (σ_z) = $\sigma_{z1} - \sigma_{z2}$

= 75.16 KN/m² //

R.K. SHAH

Find vertical stress at 5m below point P due to uniformly loaded hatched area of 300 KN/m² intensity as shown in fig. 6.

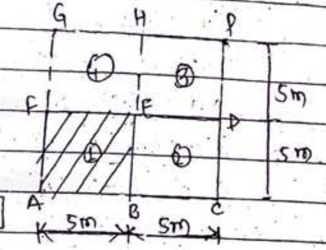
→ Solution:-

Given that,

$q = 300$ KN/m²

$z = 5m$

$\sigma_z = ?$



$\sigma_z = q\,[(I_N)_1 - (I_N)_2 - (I_N)_3 + (I_N)_4]$

I_{N1} [rect ACPG] : $m = 10/5 = 2$, $n = 10/5 = 2$ } from Fadum's chart $(I_N)_1 = 0.235$

I_{N2} [rect. GFDP] : $m = 10/5 = 2$, $n = 5/5 = 1$ ⇒ $(I_N)_2 = 0.2$

I_{N3} [rect BCHP] : $m = 10/5 = 2$, $n = 5/5 = 1$, $(I_N)_3 = 0.2$

$(I_N)_4$ [rect. HPDE] : $m = 5/5 = 1$, $n = 5/5 = 1$, $(I_N)_4 = 0.18$

$\therefore \sigma_z = q\,[(I_N)_1 - (I_N)_2 - (I_N)_3 + (I_N)_4]$

$= 300\,[0.235 - 0.2 - 0.2 + 0.18]$

$= 4.5$ KN/m² //

15. A ring footing of external diameter 8m and internal diameter 4m rests at a depth 2m below the ground surface. It carries a load intensity of 150 KN/m². Find the vertical stress at depth of 2m & 4m

along the axis of footing base. Neglect the effect of excavation on the stress.

→ Solution:-

$q = 150\ kN/m^2$.

$r_1 = 8/2 = 4m$ $r_2 = 4/2 = 2m$

$z_1 = 2m$ & $z_2 = 4m$

For circular area, $\sigma_z = q.I_c$

σ_z due to $r_1 = 4m$, & $z_1 = 2m$,

$$I_{c1} = 1 - \frac{1}{\{1+(4/2)^2\}^{3/2}} = 0.911$$

$\therefore \sigma_{z1} = 150 * 0.911 = 136.65\ kN/m^2$.

σ_z due to $r_2 = 2m$ & $z_2 = 2m$,

$$I_{c2} = 1 - \frac{1}{\{1+(2/2)^2\}^{3/2}} = 0.646$$

$\therefore \sigma_{z2} = 150 * 0.646 = 96.9\ kN/m^2$.

$\therefore$ Resultant vertical stress at $z = 2m$ is,

$\sigma_z = \sigma_{z1} - \sigma_{z2} = 136.65 - 96.9 = 39.75\ kN/m^2$ //

Another Case:-

At $z = 4m$,

σ_z due to $r_1 = 4m$, $I_{c1} = 1 - \frac{1}{\{1+(4/4)^2\}^{3/2}} = 0.646$

$\therefore \sigma_{z1} = 150 * 0.646 = 96.9\ kN/m^2$.

σ_z due to $r_2 = 2m$, $I_{c2} = 1 - \frac{1}{\{1+(2/4)^2\}^{3/2}} = 0.284$

$\therefore \sigma_{z2} = 150 * 0.284 = 42.6\ kN/m^2$.

$\therefore$ Resultant vertical stress at $z = 4m$ is

$\sigma_z = \sigma_{z1} - \sigma_{z2}$

$= 96.9 - 42.6$

$= 54.3\ kN/m^2$ //

17). A water tank is supported by a ring foundation of external diameter 10m and internal diameter of 7.5m. The ring foundation transmits uniform load intensity of 160 kN/m². Compute the vertical stress induced at a depth of 4m below the centre of ring foundation, using

i) Boussinsq analysis

ii) Westergaard's analysis.

→ Solution:-

Given that:

i) Using Boussinsq analysis,

$$\sigma_v = q\left[1 - \left\{\frac{1}{1+\left(\frac{R_o}{z}\right)^2}\right\}^{3/2}\right] - q\left[1 - \left\{\frac{1}{1+\left(\frac{R_i}{z}\right)^2}\right\}^{3/2}\right]$$

$$= 160\left[1 - \left\{\frac{1}{1+\left(\frac{5}{4}\right)^2}\right\}^{3/2}\right] - 160\left[1 - \left\{\frac{1}{1+\left(\frac{3.75}{4}\right)^2}\right\}^{3/2}\right]$$

$= 23.11\ kN/m^2$ //

ii) Using Westergaad's analysis

RAKESH KUMAR SHAH

Tutorial - 6

Ch - 9 Compressibility and Consolidation of Soil

1. Two clay samples A and B have initial void ratio of 0.55 and 0.632 respectively under a pressure 100 KN/m². The pressure increased to 150 KN/m². With the application of the pressure, the void ratio decreases to 0.495 and 0.616 respectively. The time taken by the specimen A to reach 50% consolidation is one-third of that required by the specimen B for the same consolidation. Find the ratio of the coefficient of the permeability of the two specimens A and B if their thickness was 40mm and 30mm resp.

→ Solution:-

Given that:-

Sample A

$e_{0A} = 0.55$

initial, $P_{0A} = 100$ KN/m²

$P_f = 150$ KN/m²

final void ratio $(e_A) = 0.495$

degree of consolidation $(U) = 50\%$

time $(t_A) = t/3$

thickness/height of sample $(H_A) = 40$ mm

$\frac{K_A}{K_B} = ?$

Sample B

$e_{0B} = 0.632$

initial pressure $(P_{0B}) = 100$ KN/m²

final " $P_f = 150$ KN/m²

final void ratio $(e_B) = 0.616$

$U = 50\%$

time $(t_B) = t$

$H_B = 30$ mm

Since the degree of consolidation is same,

$T_{VA} = T_{VB}$ [∵ T_v = time factor]

or, $\frac{C_{VA} t_A}{d^2} = \frac{C_{VB} t_B}{d^2}$ ... (1)

$C_v = \frac{K}{m_v \gamma_w}$ $m_v = \frac{a_v}{1+e_0}$ $a_v = \frac{\Delta e}{\Delta P}$

Now,

1) $a_{VA} = -\frac{\Delta e_A}{\Delta P_A} = -\frac{(0.495-0.55)}{150-100} = 0.0011$

$a_{VB} = -\frac{\Delta e_B}{\Delta P_B} = -\frac{(0.616-0.632)}{150-100} = 0.00032$

2) $m_{VA} = \frac{0.0011}{1+0.55} = 7.09 * 10^{-4}$

$m_{VB} = \frac{0.00032}{1+0.632} = 1.96 * 10^{-4}$

3) $C_{VA} = \frac{K_A}{m_{VA}\gamma_w} = \frac{K_A}{7.09*10^{-4}*9.81} = \frac{K_A}{7.09*10^{-4}\gamma_w}$

$C_{VB} = \frac{K_B}{m_{VB}\gamma_w} = \frac{K_B}{1.96*10^{-4}\gamma_w}$

From eqn. (1), we have,

$\frac{K_A}{7.09*10^{-4}\gamma_w} * \frac{t/3}{40^2} = \frac{K_B}{1.96*10^{-4}\gamma_w} * \frac{t}{30^2}$

Or, $\frac{K_A}{K_B} = \frac{7.09*40^2*3}{1.96*30^2} = \frac{19.3}{1}$

∴ Req. ratio $= \frac{K_A}{K_B} = \frac{19.3}{1}$

2. The following information was obtained from the soil investigation work. Two layers of clay of thickness 2m each and sandwiched between two layers of dense sand was encountered at 5m and 14m depths respectively. The compression index, water content and specific gravity of both clays were 0.38, 30% and 2.7 respectively. The sand weighed 2.1 gm/cc and was completely submerged. Compute the total settlement under a uniformly distributed load of 3.5 kg/cm² on a large area over grd surface-

→ Solution:-

Given that:-

Case I:-

We will first calculate the settlement in the upper clay layer.

We will first calculate the Effective overburden pressure at section A-A i.e. centre of upper clay layer is, [$\because e = wG = 0.3*2.7 = 0.81$]

$\sigma'_v = \gamma_{sub.sand} * 5 + \gamma_{sub.clay} * 1$

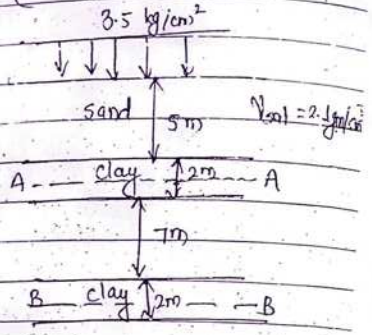
$\gamma_{sub.clay} = \frac{G_s - 1}{1+e} * \gamma_w = \frac{2.7-1}{1+0.81} * 10 = 9.39 \text{ kN/m}^3$

$\gamma_{sub.sand} = (\gamma_{sat} - \gamma_w) = (21-10) = 11 \text{ kN/m}^3$

Now,

$\sigma'_0 = 11*5 + 9.39*1$

$= 64.39 \text{ kN/m}^2$.



$\therefore \Delta H_1 = \frac{C_c}{1+e_0} \cdot H_0 \cdot \log_{10}\left(\frac{\sigma'_0 + \Delta\sigma'_v}{\sigma'_0}\right)$

$= \frac{0.38 * 200}{1+0.81} \log_{10}\left(\frac{64.39+350}{64.39}\right)$

$= 33.95 \text{cm}$.

Case II:- settlement in the lower clay layer.

Effective overburden pressure at section B-B i.e. centre of lower clay layer is

$\sigma'_v = \gamma_{sub.sand} * (5+7) + \gamma_{sub-clay} * (2+1)$

$= 11*12 + 9.39*3$

$= 160.17 \text{ kN/m}^2$.

$\therefore \Delta H_2 = \frac{C_c}{1+e_0} \cdot H_0 \log_{10}\left(\frac{\sigma'_0 + \Delta\sigma'_v}{\sigma'_0}\right)$

$= \frac{0.38}{1.81} * 200 \log_{10}\left(\frac{160.17+350}{160.17}\right) = 21.13 \text{cm}$.

$\therefore$ Total settlement $= \Delta H_1 + \Delta H_2$

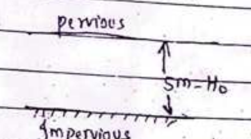
$= 33.95 + 21.13$

$= 55.1 \text{cm}$ Ans.

3\. A saturated soil stratum 5m thick lies above an impervious stratum and below a pervious stratum. It has a compression index of 0.25 and a coefficient of permeability of $3.2*10^{-4}$ cm/sec. It has a void ratio of at a stress of 1.5 kg/cm² is 1.9. Compute
(a) the change void ratio due to increase of stress of 2 kg/cm².
(b) settlement of soil stratum due to above increase of stress &
(c) time required for 50% consolidation, the time factor for 50% consolidation may be assumed to be 0.2.

→ Solution:-

Given that:-



Compression index $(C_c) = 0.25$

Coeff. of permeability $(K) = 3.2*10^{-4}$ cm/s

$e_0 = 1.9$

Stress $(\sigma_0) = 1.5 \text{ kg/cm}^2$.

a) $\Delta e = ?$ for $\Delta\sigma = 2 \text{ kg/cm}^2$

b) settlement = ?

c) $t_{50} = ?$ time factor $(T_v) = 0.2$

We have

a) $C_c = \frac{-\Delta e}{\log_{10}\left(\frac{\sigma}{\sigma_0}\right)}$ $\sigma_0 = 1.5 \text{ kg/cm}^2$, $\sigma = \sigma_0 + \Delta\sigma = 3.5 \text{ kg/cm}^2$

or, $0.25 = \frac{-\Delta e}{\log_{10}\left(\frac{3.5}{1.5}\right)}$

$\therefore \Delta e = 0.211$

Also,

b) $\Delta H = \frac{C_c}{1+e_0} H_0 \log\left(\frac{\sigma_0 + \Delta\sigma}{\sigma_0}\right)$

$= \frac{0.25}{1+1.9} * 5 \log\left(\frac{1.5+2}{1.5}\right)$

$= 0.365\ m$

c) $T_v = \frac{C_v t}{d^2}$ ∵ $d = 5m = 500cm$

$a_v = -\frac{\Delta e}{\Delta\sigma} = -\frac{(-0.211)}{2} = 0.1055\ cm^2/kg$

$m_v = \frac{a_v}{1+e_0} = \frac{0.1055}{1+1.9} = 0.0363\ cm^2/kg$

$C_v = \frac{k}{m_v \gamma_w} = \frac{3.2*10^{-4}}{0.0363*10^{-3}} = 8.815\ cm^2/s$

Now,

$T_v = \frac{C_v t_{50}}{d^2}$ $\left\{ \gamma_w = 9.81\ kN/m^3 = 1*10^{-3}\ kg/cm^3 \right.$

or, $0.2 = \frac{8.815 * t_{50}}{500^2}$ ⇒ $t_{50} = 5671.875\ sec$
$\simeq 94.53\ min.$

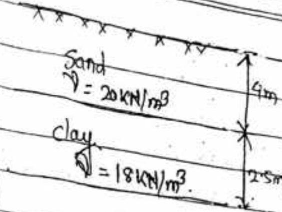
Additional practise (2015 fall)
Solution same as above process.

4. A clay layer 2.5m thick is overlain by a layer of sand 4m in thickness. The clay has a compression index of 0.22 and saturated unit weight of $18 kN/m^3$. The initial void ratio of clay is 1.30. Calculate the final settlement of the clay layer due to an increase in pressure of $30 kN/m^2$ at center of clay layer. Also, calculate the settlement when the table water rises to the ground surface. Unit weight of sand is $20 kN/m^3$.

→ Solution:-

Given that:-

Initial pressure at the center of clay layer,



$\sigma_0' = 4*20 + 1.25*18$
$= 102.5\ kN/m^2.$

$\therefore \Delta H = \frac{C_c}{1+e_0} \cdot H_0 \cdot \log_{10}\left(\frac{\sigma_0' + \Delta\sigma_v}{\sigma_0'}\right)$

$= \frac{0.22}{2.3} * 2.5 * \log_{10}\left(\frac{102.5+30}{102.5}\right)$

$= 0.0266m \simeq 2.67cm$

When water table rises to the ground surface,

$\sigma_0' = 4*20(20-10) + 1.25*18(18-10) = 50\ kN/m^2$

$\therefore \Delta H = \frac{0.22}{2.3} * 2.5 * \log_{10}\left(\frac{50+30}{50}\right) = 0.0488m$
$\simeq 4.88cm$ &

Additional Practice (2015 Spring)
Additional practice (2014 Spring)
Additional practice (2011 Fall)
} Soln same as above process.

5. A strata of normally consolidated clay of thickness 3m is drained on both sides. If it has a coefficient of permeability $K = 5*10^{-8}$ cm/sec and coefficient of volume compressibility $= 125*10^{-2}\ cm^2/kN$. Determine the total compression of the strata for a change in stress of $25\ t/m^2$. Also determine the time required for 80% consolidation.

→ Solution:-

Given that:-

Coefficient of permeability $(K) = 5*10^{-8}\ cm/s$

Coeff. of volume compressibility $(m_v) = 125*10^{-2}\ cm^2/kN$

Total compression $(\Delta H) = ?$

$\Delta\sigma = 25\ t/m^2 = 25*1000\ kg/m^2 = \frac{25000 * 9.81 * 10^{-3}}{1*10^4}\ kN/cm^2$

We have;

$\Delta H = m_v H_0 \Delta \sigma$

$= 125 \times 10^{-2} \times 300 \times 0.0245$

$= 9.2$ cm .... (1)

$\gamma_w = 9.81$ KN/m^3

$= \frac{9.81}{1 \times 10^6}$ KN/cm^3

$= 9.81 \times 10^{-6}$ KN/cm^3

$T_v = \frac{C_v t_{80}}{d^2}$

for $U = 80\% = 0.8$

$T_v = -0.933 \log_{10}(1-U) - 0.085$

$= -0.933 \log_{10}(1-0.8) - 0.085$

$= 0.567$

$C_v = \frac{K}{m_v \gamma_w} = \frac{5 \times 10^{-8}}{125 \times 10^{-2} \times 9.81 \times 10^{-6}} = 4.07 \times 10^{-3}$ cm^2/s.

Now, from (1),

$0.567 = \frac{4.07 \times 10^{-3} \times t_{80}}{150^2}$ $\left[\because d = \frac{H_0}{2} = \frac{300}{2} = 150\text{cm}\right]$

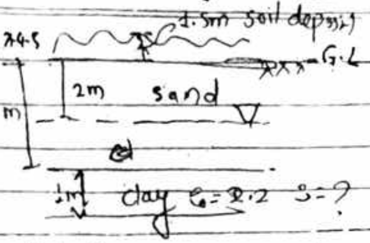
$\therefore t_{80} = 3134520.885$ sec

$\simeq 36.28$ days.

6.

$\bar{\sigma} = 18.5 \times 1.5 + 6 \times 18 + 17.5 \times 0.5 - 9.81 \times 4.5$

$= 100.36$ KN/m^2.



and initial $\bar{\sigma}_0 = 6 \times 18 + 17.5 \times 0.5 - 9.81 \times 4.5$

$= 72.61$ KN/m^2.

$\therefore \Delta\bar{\sigma} = 100.36 - 72.61 = 27.75$ KN/m^2.

Now,

$s = \frac{C_c}{1+e_0} H_0 \log_{10}\left(\frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_0}\right)$

$= \frac{0.4}{1+2.2} \times 1 \times \log_{10}\left(\frac{100.36}{72.61}\right)$

$= 0.01757\text{m} \simeq 17.57\text{mm}$.

7.

Here,

We have,

$$s = \frac{C_c}{1+e_0} H_0 \log_{10}\left(\frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_0}\right)$$

Now,

$$\bar{\sigma}_0 = 5*17.5 + 1.0*17.0 - 6*9.81$$
$$= 45.64\ kN/m^3$$ ← 5m लिदा s = 31.48 m

$$\Delta\bar{\sigma} = (2*19 + 45.64) - 45.64$$
$$= 38\ kN/m^3$$

$$\therefore s = \frac{0.5}{1.26} * 1 * \log_{10}\left(\frac{45.64+38}{45.64}\right)$$

$$= 0.03654\ m$$
$$= 36.54\ mm$$

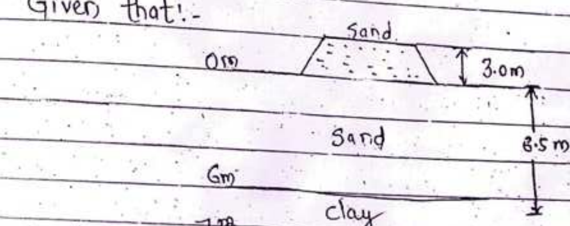
Rakesh Kumar Shah.

8. A clay layer of 1m thick is found between 6m and 7m below the ground surface. The clay layer is sandwiched between sand layers. The coefficient of volume change of the clay is $0.40*10^{-3}\ m^2/kN$. A 3m high sand embankment of circular plan area diameter 20m is placed on the ground surface. The unit weight of the sand is $20\ kN/m^3$. Determine the settlement of the clay layer under the center of the circular shape of radius r which can be calculated according to the given formula below.

$$\Delta\sigma' = \left[1 - \left(\frac{1}{1+(r/z)^2}\right)^{1.5}\right] * q.$$

→ Solution:-

Given that:-



Unit weight of sand (γ_{sand}) = $20\ kN/m^3$.

Height of embankment (H) = 3m.

∴ Stress (q) = 20*3 = $60\ kN/m^2$

Now,

$$\Delta\sigma'_v = q\left[1 - \left\{\frac{1}{1+(r/z)^2}\right\}^{1.5}\right] = 60\left[1 - \left\{\frac{1}{1+(\frac{2}{6.5})^2}\right\}^{1.5}\right]$$

$$= 7.6\ kN/m^2$$

We have,

$$\Delta H = m_v * \Delta\sigma'_v * H_0$$
$$= 0.4*10^{-3} * 7.6 * 1$$
$$= 3.05\ mm$$

9. A consolidation test was performed on a 25mm thick undisturbed clay sample, 50% consolidation occured in 5 minutes. In the field, a clay layer is 2m thick and is underlain by rock stratum. There is a sand layer above the clay layer. Determine the coefficient of consolidation. Also compute the number of days required for the field stratum to reach 50% consolidation.

→ Solution:-

Given that:-

In lab

Sample ht (h) = 25 mm

time = 5 m

C_v = ?

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

$= \frac{0.196 * 1.25^2}{5}$

$= 0.06125 \text{ cm}^2/\text{min}$

$= 61.25 \times 10^{-3} \text{ cm}^2/\text{min}$

$d = h/2 = 12.5 \text{ mm}$
$= 1.25 \text{ cm}$

for $U = 50\%$, $T_v = \frac{\pi}{4} U^2$

$= \pi/4 * 50^2 = 0.196$

In field

h = 2m = 200 cm.

U = 50%.

t_{50} = ?

for U = 50%,

$(T_v)_{lab} = (T_v)_{field}$.

$\frac{C_v t_{lab}}{d_{lab}^2} = \frac{C_v t_{field}}{(d_{field})^2}$

or, $\frac{5}{t_{field}} = \frac{1.25^2}{200^2}$

$\therefore t_{field} = 128000$ min

$\simeq 88.89$ days

10. Find the time required for 50% and 80% consolidation in a soil stratum, 10m thick with a pervious strata on top and bottom.

Also find the coefficient of consolidation if, $K = 1 * 10^{-9}$ m/s, $e_0 = 1.5$ and $a_v = 0.003 \text{ m}^2/\text{kN}$.

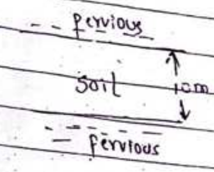
→ Solution:-

Given that:-

t_{50} = ?

t_{80} = ? $d = 10/2 = 5$m.

C_v = ?



$K = 1 * 10^{-9}$ m/s $e_0 = 1.5$ $a_v = 0.003 \text{ m}^2/\text{kN}$

We have,

$m_v = \frac{a_v}{1+e_0} = \frac{0.003}{1+1.5} = 0.0012 \text{ m}^2/\text{kN}$.

$\therefore C_v = \frac{K}{m_v \gamma_w} = \frac{1 * 10^{-9}}{0.0012 * 9.81} = 8.494 \times 10^{-8} \text{ m}^2/\text{s}$.

For $U = 50\%$, $T_v = \frac{\pi}{4} U^2 = \frac{\pi}{4} * 0.5^2 = 0.196$.

$\therefore C_v = \frac{T_v d^2}{t_{50}}$

or, $8.494 \times 10^{-8} = \frac{0.196 * 5^2}{t_{50}}$

$\therefore t_{50} = \frac{57687779.61}{60 * 60 * 24} = 667.68$ days

For $U = 80\%$, $T_v = -0.933 \log(1-U) - 0.085$

$= -0.933 \log(1-0.8) - 0.085$

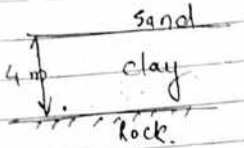
$= 0.567$.

$\therefore C_v = \frac{T_v d^2}{t_{80}}$

$\therefore$ or, $8.494 \times 10^{-8} = \frac{0.567 * 5^2}{t_{80}}$

$\therefore t_{80} = \frac{166882805.3}{60 * 60 * 24} = 1931.51$ days

11.



→ Given:

$C_v = 0.02\ cm^2/min = 3.33 \times 10^{-4}\ cm^2/sec$

$S_f = 15\ cm$

for settlement of 5m (S_5), time req. = ?

for 80% settlement, time t_{80} = ?

Here,

$S_5 = 5m$, $\frac{5}{15} \times 100 = 33.33\%$

for $U = 33.33\%$,

$T_v = \frac{\pi}{4} \times U^2 = \frac{\pi}{4} \times (0.3333)^2 = 0.0872$

$H_0 = d = 4m$ (one way down)

Now, we have,

$t_{33.33} = \frac{T_v d^2}{C_v} = \frac{0.0872 \times (4 \times 100)^2}{3.33 \times 10^{-4}}$

$= 41897897.9\ sec.$

$\simeq 484.93$ days.

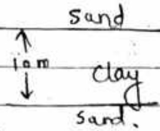
for $U = 80\%$,

$T_v = -0.933 \log_{10}(1-0.8) - 0.085 = 0.567.$

$\therefore t_{80} = \frac{0.567 \times 400^2}{3.33 \times 10^{-4}}$

$= 272432432.4\ sec$

$\simeq 3153.15$ days



→ Solution:

Given that:-

$H_0 = d = 10/2 = 5m$. (for two way drain)

$C_v = 0.025\ cm^2/min = 4.167 \times 10^{-4}\ cm^2/sec$

$S_f = 8cm.$

For 50% settlement, t_{50} = ?

Settlement in 6 months = ?

Here,

$T_v = \frac{\pi}{4} U^2$ for $U = 50\%$

$= \frac{\pi}{4} \times 0.5^2 = 0.196$

$\therefore t_{50} = \frac{T_v d^2}{C_v} = \frac{0.196 \times 500^2}{4.167 \times 10^{-4}} = 117590592.8\ sec.$

$\therefore t_{50} = 3.73$ yrs $\simeq 3.73 \times 12$ months.

Similarly,

T_v for 50% settlement = 0.196

For time (t_1) = 6months, $(T_v)_1$ is calculated as

We have,

$\frac{T_v}{(T_v)_1} = \frac{t}{t_1}$

$\Rightarrow T_{v_2} = \frac{0.196}{3.72712} * 6 = 0.0263$

For $(T_v)_1 = 0.0263$,

we have,

$U^2 = (T_v)_s * \frac{4}{\pi}$

$= (0.0263) * \frac{4}{\pi}$

$= 0.033486$

$\therefore U = 0.18299$

$\simeq 18.30\%$

In 6 months, there is 18.30% settlement.

Then total settlement in 6 months,

$S = U * S_f$

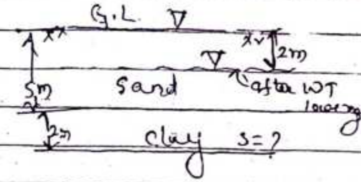
$= 0.183 * 8$

$= 1.464\,cm$

Hence, total settlement = 1.464 cm

→ Solution:

Given that,



$m_v = 0.25 * 10^{-3}\ m^2/kN$

$\gamma_{sand} = 20\ kN/m^3$

$\gamma_{clay} = 18\ kN/m^3$

Initial pressure at centre of clay when WT is at G.L,

$\bar{\sigma}_0 = 5*20 + 1*18 - 6*9.81$

$= 59.14\ kN/m^2$

When WT is lowered by 2m from G.L permanently, final pressure,

$\bar{\sigma} = 5*20 + 1*18 - 4*9.81$

$= 78.76\ kN/m^2$

$\therefore$ Change in pressure

$\Delta\bar{\sigma}_0 = 78.76 - 59.14$

$= 19.62\ kN/m^2$

$\therefore H_0 = 2m$

Now,

Settlement at centre of clay is given by

$S = m_v H_0 \Delta\bar{\sigma}_0$

$= 0.25 * 10^{-3} * 2 * 19.62$

$= 9.81 * 10^{-3}\ m$

$= 9.81\ mm$

Rakesh Kumar Sah

14. A stratum of clay is 2m thick and has an initial overburden pressure of 50 kN/m² at its middle. Determine the final settlement due to an increase of 40 kN/m² at the middle of the clay layer. The clay is over consolidated, with a preconsolidation pressure of 75 kN/m². The values of the coefficients of recompression and compression index are 0.05 and 0.25 respectively. Take initial void ratio as 1.40.

→ Solution:-

Given that:

Thickness of clay layer (H_0) = 2m

Initial overburden pressure (σ_0') = 50 kN/m²

Increase in pressure ($\Delta\sigma'$) = 40 kN/m²

Pre-compression pressure (σ_c') = 75 kN/m²

Coefficient of recompression (C_r) = 0.05

Compression index (C_c) = 0.25

Initial void ratio (e_0) = 1.4

Since, clay layer is over-consolidated and $\sigma_0' < \sigma_c' < (\sigma_0' + \Delta\sigma')$,

The final settlement is given by:

$$H = \frac{C_r}{1+e_0} \cdot H_0 \log_{10}\left(\frac{\sigma_c'}{\sigma_0'}\right) + \left(\frac{C_c}{1+e_0}\right) H_0 \log\left(\frac{\sigma_0' + \Delta\sigma'}{\sigma_c'}\right)$$

$$= \frac{0.05}{1+1.4} * 2 * \log_{10}\left(\frac{75}{50}\right) + \frac{0.25}{1+1.4} * 2 \log\left(\frac{50+40}{75}\right)$$

$= 0.02384$

$\therefore H \simeq 23.84$ mm

15)

→ Solution:-

Given that: time (t_1) = 6 months = 180 days

$$U_1 = \frac{\text{settlement in 6 months}}{\text{Total settlement}} * 100\% = \frac{1.6}{6.5} * 100\% = 24.6\%$$

Now,

For $U_1 = 24.6\%$, $(T_v)_1 = \pi/4 * (0.246)^2 = 0.0475$

(i) $U_2 = 50\%$, $(T_v)_2 = \pi/4 * 0.5^2 = 0.1963$

$$\therefore \frac{t_2}{t_1} = \frac{(T_v)_2}{(T_v)_1} = \frac{0.1963}{0.0475} = 4.1326$$

i.e. $t_2 = t_{50} = 180 * 4.1326 = 743.87$ days

$\simeq 2.038$ yrs.

(ii) Also, when time = 6 months i.e. t = 6+6 = 12 months,

$$T_v = \left(\frac{t}{t_1}\right) * (T_v)_1 = \frac{12}{6} * 0.0475$$

$\therefore T_v = 0.095$

For $T_v = 0.095$, $U_1 = \sqrt{\frac{T_v \times 4}{\pi}} = \sqrt{\frac{0.095 * 4}{\pi}} = 34.78\%$

$\therefore$ Settlement in 12 months,

$S = U * S_f$

$= 0.3478 * 6.5$

$\simeq 2.26$ cm.

$\therefore$ Settlement in 12 months (next 6 months add)

$= 2.26$ cm

$\therefore$ Total settlement in next 6 months

$= 2.26 - 1.6$

$= 0.66$ cm

16. A saturated clay stratum 4m thick lies above an impervious stratum and below a pervious stratum. It has a void ratio of 1.2 at an initial pressure of 200 kN/m². Compute the change in void ratio due to an increase in stress of 50 kN/m² assuming that soil is normally consolidated with compression index 0.2. What would be the time required for 50% consolidation. Take coefficient of permeability = $3*10^{-4}$ cm/s.

→ Solution:-

Given that:-

$H_0 = 4m$ (one way drain) $\bar{\sigma}_0 = 250\ kN/m^2$

$C_c = 0.2$ $\sigma_1 = 50\ kN/m^2$

$k = 3*10^{-4}$ cm/s. $e_0 = 1.2$

$= 3*10^{-6}$ m/s.

a) $C_c = \frac{\Delta e}{\Delta \log \sigma'} = \frac{e_0 - e_1}{\log \sigma_1 - \log \sigma_0}$

$\therefore e_0 - e_1 = C_c \log\left(\frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_0}\right) = 0.2 \log\left(\frac{250}{200}\right) = 0.0194$

b) $\Delta H = \left(\frac{C_c}{1+e_0}\right) H_0 \log_{10}\left(\frac{\sigma_1}{\sigma_0}\right)$

$a_v = \frac{\Delta e}{\Delta \bar{\sigma}} = \frac{0.0194}{50} = 3.88*10^{-4}\ m^2/kN.$

c) for $U = 50\%$, $T_v = \pi/4 * U^2 = \pi/4 * 0.5^2 = 0.196$

$m_v = \left(\frac{\Delta e}{1+e_0}\right) \cdot \frac{1}{\Delta\sigma'} = \frac{a_v}{1+e_0} = \frac{3.88*10^{-4}}{1+1.2} = 1.7636*10^{-4}\ m^2/kN$

$C_v = \frac{K}{m_v \gamma_w} = \frac{3*10^{-6}}{9.81*1.7636*10^{-3}} = 1.734*10^{-3}\ m^2/s$

Also,

$t_{50} = \frac{T_v * d^2}{C_v} = \frac{0.196*4^2}{1.734*10^{-3}} = 1808.54$ sec $\simeq 30.14$ minutes

17.

→ Solution:-

Given that:-

$\gamma_{clay,sat} = 18.5\ kN/m^3$

$C_c = 0.22$

$e_0 = 1.4$

$\Delta\bar{\sigma} = 40\ kN/m^2$.

Case I:-

$S = \frac{C_c}{1+e_0} H_0 \log_{10}\left(\frac{\bar{\sigma}_0 + \Delta\sigma}{\bar{\sigma}_0}\right)$ $H_0 = 3.5m$

where, $\bar{\sigma}_0 = 1.75*18.5 = 32.375\ kN/m^2$.

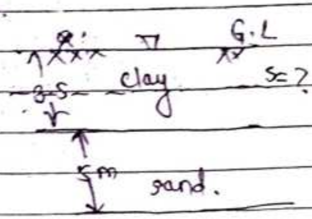
$S = \frac{0.22}{1+1.4} * 3.5 \log\left(\frac{32.375+40}{32.375}\right)$

$= 112.09$ mm.

Case II:-

when WT rises to G.L,

Settlement = ?



Here,

$\bar{\sigma}_0 = 1.75*18.5 - 1.75*9.81$

$= 15.208\ kN/m^2$.

Now,

$S = \frac{C_c}{1+e_0} H_0 \log_{10}\left(\frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_0}\right)$

$= \frac{0.22}{141.9} * 3.5 * \log_{10}\left(\frac{15.208 + 40}{15.208}\right)$

$= 0.17964\ m.$

$\simeq 179.64\ mm$

Additional practise (2016 Spring) same as above solution.

Tutorial - 8

Rakesh Shah

Ch-11 Stability of Slopes:-

1. Calculate the safe height of an embankment rising 70° to the horizontal and to be made with the clayey soil having unit weight of 2.1 gm/cc, $\phi = 15°$ and a cohesion of 0.2 kg/cm². The factor of safety as 2.5 and value of stability number corresponding to slope angle 70° and $\phi = 15°$ is 0.14.

→ Solution:-

Given that:-

Since the factor of safety given in the problem is with respect to shear strength, it is applicable to both C and ϕ.

$\gamma = 2.1\ gm/cc = 21\ kN/m^3$

$\phi = 15°$

$c = 0.2\ kg/cm^2 = 20\ kN/m^2$

$F_c = 2.5$

$S_n = 0.14$ for $i = 70°$

We have, ~~$F = \frac{\tan\phi}{\tan\phi_m}$~~

~~$\therefore \phi_m = \tan^{-1}$~~

$S_n = \frac{C_m}{\gamma H_c} = \frac{C}{F \gamma H_c}$

or, $H_c = \frac{C}{F \gamma S_n} = \frac{20}{2.5 * 21 * 0.14} = 2.721\ m.$

Hence, the safe height of an embankment is 2.721 m.

2. Calculate the factor of safety wrt cohesion of a clay slope laid at 1 in 2 to a height of 10 m, if the angle of internal friction $\phi = 10°$, $C = 25\ kN/m^2$ and $\gamma = 19\ kN/m^3$. What will be the critical height of the slope in this soil?

→ Solution:-

Given that:-

$\phi = 10^\circ$

$C = 25\ kN/m^2$ $\quad \gamma = 19\ kN/m^3$ $\quad H = 10m$ and $\tan i = 1/2$

$\Rightarrow i = 26.5^\circ$

For $\phi = 10^\circ$, & $i = 26.5^\circ$ from table, $S_n = 0.064$.

Also, we have,

$$S_n = \frac{C}{F_c \gamma H}$$

$$\therefore F_c = \frac{C}{S_n \gamma H} = \frac{25}{0.064 * 19 * 10} = 2.06$$

Again,

$$H_c = \frac{C}{\gamma S_n} = \frac{25}{19 * 0.064} = 20.6\ m$$

3. A slope is to be constructed at an inclination of 30° with the horizontal. Determine the safe height of the slope at factor of safety of 1.5. The soil has following properties. $\phi = 22.5^\circ$, $C = 15\ kN/m^2$ & $\gamma = 19\ kN/m^3$.

→ Solution:-

We have, for $\phi = 22.5^\circ$ & $i = 30^\circ$, $S_n =$ ~~0.18~~

$$S_n = \frac{C}{F_c \gamma H} = \text{~~}\frac{15}{1.5*19*}\text{~~}$$

or, $0.18 = \dfrac{15}{1.5 * 19 * H}$

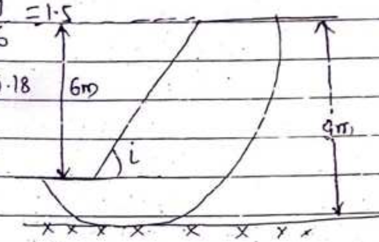
or, $H =$

4. The soil at a site has $C = 15\ kN/m^2$ and $\gamma = 18\ kN/m^3$. A hard stratum exists at a depth of 9m below the ground surface. If a cutting 6m deep is to be made in the soil, find factor of safety if a 30° slope is safe. What will be the slope angle if factor of safety of 1.5 is required?

→ Solution:-

Given that:-

Depth factor $(D_f) = \dfrac{9}{6} = 1.5$



From stability chart,

for $i = 30^\circ$ & $D_f = 1.5$, $S_n = 0.18$

Now,

$$S_n = \frac{C}{F_c \gamma H}$$

$$\therefore F_c = \frac{C}{S_n \gamma H} = \frac{15}{0.18 * 18 * 6} = 0.77.$$

Therefore, the proposed slope is unsafe.

For $F_c = 1.5$, the required stability number is

$$S_n = \frac{C}{F_c \gamma H} = \frac{15}{1.5 * 18 * 6} = 0.092.$$

Now,

for $S_n = 0.092$ and $D_f = 1.5$

Slope angle $(i) = 9^\circ$

5>.

6). A granular soil has $\gamma_{sat} = 19\ kN/m^3$, $\phi = 30°$. A slope has to be made of this soil. If a factor of safety of 1.2 needed against slope failure, determine the safe angle of the slope,

i) when the slope is dry or submerged without seepage.

ii) If seepage occurs at and parallel to the slope.

iii) If seepage occurs parallel to the slope with water table at a depth of 1.5m, what is the factor of safety available on a slope parallel to the ground surface at a depth of 4m? Assume a slope angle of 28°.

→ Solution:-

Given that,

$c = 0\ kN/m^2$,

$\gamma_{sat} = 19\ kN/m^3$

$\phi = 30°$

$F = 1.2$

i) When the slope is dry.

$$F = \frac{\tan\phi}{\tan i}$$

$$\text{or, } 1.2 = \frac{\tan 30°}{\tan i}$$

$$\text{or, } i = 25.69°$$

ii). When water level is at surface and seepage occurs parallel to surface,

$$F = \frac{\gamma'}{\gamma_{sat}} * \frac{\tan\phi}{\tan i}$$

$$\text{or, } \tan i = \frac{19 - 9.81}{19} * \frac{\tan 30°}{1.2}$$

$$\therefore i = 13.1°$$

And,

11. A new canal is excavated to a depth of 5m below ground level, through a soil having the following characteristics: $C = 14\ KN/m^2$, $\phi = 15°$, $e = 0.8$ and $G = 2.70$. The slope of the banks is 1 in 1. Calculate the factor of safety wrt cohesion when the canal runs full. If it is suddenly and completely emptied, what will be factor of safety?

→ Solution:-

$$\gamma_{sat} = \left(\frac{G+e}{1+e}\right)\gamma_w = \left(\frac{2.70+0.8}{1+0.8}\right) 9.81 = 19.075\ kN/m^3$$

$$\gamma' = (\gamma_{sat} - \gamma_w) = 19.075 - 9.81 = 9.265\ kN/m^3.$$

$i = 45°$

$\phi = 15°$

i) Submerged condition,

for $i = 45°$ & $\phi = 15°$, $S_n =$

Now,

$$S_n = \frac{C}{\gamma' H}$$

$$\therefore C = S_n \gamma' H$$

$$=$$

$\therefore$ Factor of safety wrt cohesion,

$$F_c = \frac{C}{C_m} = \frac{14}{}$$

ii) Rapid emptied.

$$\phi_w = \frac{\gamma'}{\gamma_{sat}} * \phi = \frac{9.265}{19.075} * 15° = 7.285°$$

for $i = 45°$ & $\phi = 7.28°$, $S_n =$

$$\therefore S_n = \frac{C}{\gamma_{sat} H}$$

$$\text{or, } C =$$

∴ Factor of safety wrt. Cohesion,

$F_c = \frac{C}{C_m} = 14$

12.

R.K. SHAH

Tutorial-7

Ch-10 - Shear strength of soil

1. Two triaxial tests were done on a soil sample. In the first test, all round pressure was 2.4 kg/cm² and the failure occured at axial stress of 7.5 kg/cm². In another test, all round pressure was 4 kg/cm² and the failure occured at total axial ~~strength~~ stress of 16 kg/cm². Determine the value of cohesion and angle of internal friction at failure.

→ Solution:-

Given that:-

First test	2nd test
$\sigma_f = 7.5\ kg/cm^2$	$\sigma_3 = 4\ kg/cm^2$
$\sigma_c = 2.4\ kg/cm^2$	$\sigma_f = 16\ kg/cm^2$
$\sigma_1 = 7.5 + 2.4 = 9.9$	$\sigma_1 = 20$

We know,

$\sigma_1 = \sigma_3 N_\phi + 2C\sqrt{N_\phi}$

or, $9.9 = 2.4 N_\phi + 2 \times C\sqrt{N_\phi}$ —(I) $\quad 20 = 4N_\phi + 2CN_\phi$ —(II)

Subtracting (I) from (II), we get,

$10.1 = 1.6 N_\phi$

∴ $N_\phi = 6.3125$

or, $\tan^2\left(45° + \frac{\phi}{2}\right) = 6.3125$

or, $45° + \phi/2 = 68.31$

∴ $\phi = 46.62$

Date
Page

2. An embankment is planned to be constructed from a soil with $c' = 4.0\ kg/cm^2$ and $\phi' = 26°$. Evaluate the shear strength of the material on a horizontal plane at a point 10m below the surface of the embankment. The bulk density of the soil is 2.3gm/cc and the pore pressure at that point is $1.8\ kg/cm^2$.

→ Solution:-

Given that,

Normal pressure caused by the weight of the fill at a depth of 10m,

$$\therefore \sigma_z = \gamma * h = \frac{2.3}{1000} * 1000 = 2.3\ kg/cm^2$$

Pore water pressure $(u) = 1.8\ kg/cm^2$.

$\therefore$ Effective normal pressure $(\bar{\sigma}) = \sigma_z - u = 2.3 - 1.8 = 0.5\ kg/cm^2$.

Now, shear strength,

$$\tau_f = c + \bar{\sigma} \tan\phi$$
$$= 4 + 0.5 \tan 26°$$
$$= 4.244\ kg/cm^2$$

3. Keeping the minor principal stress constant as $2\ kg/cm^2$, the major principal stress on a cylindrical soil sample was increased till failure occured. If the cohesion of the soil was $2.5\ kg/cm^2$ and the angle of internal friction as 20°. Calculate (a) max. axial load at failure (b) Shear and normal stress along the failure plane (c) Angle made by the failure plane with horizontal.

→ Solution:-

Given that:-