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CE 341

Principles of Soil Mechanics

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N.B: এই note এ যে কোন জায়গায় যে কোন ভুল থাকতে পারে। যতটুকু সম্ভব নির্ভুলভাবে করার চেষ্টা করা হয়েছে। ভুল থাকলে নিজ দায়িত্বে সংশোধন করার জন্য অনুরোধ করা হচ্ছে। যে কোন ধরনের ভুলের জন্য ক্ষমাপ্রার্থী।

Xclusive Formulas

Weight volume Relationship

- Moisture content, $\omega = \frac{W_w}{W_s} \times 100\%$

- Void ratio, $e = \frac{V_v}{V_s} = \frac{V_v/V}{(V-V_v)/V} = \frac{n}{1-n}$

- Porosity, $n = \frac{V_v}{V} = \frac{V_v/V_s}{(V_s+V_v)/V_s} = \frac{e}{1+e}$

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

- Degrees of saturation, $S_r = \frac{V_w}{V_v} \times 100\% = \frac{G_s\omega}{e}$

- Submerged unit weight / effective unit weight, $\gamma' = \gamma_{sat} - \gamma_w$

- Air content / percentage of air void, $n_a = \frac{V_{air}}{V}$

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

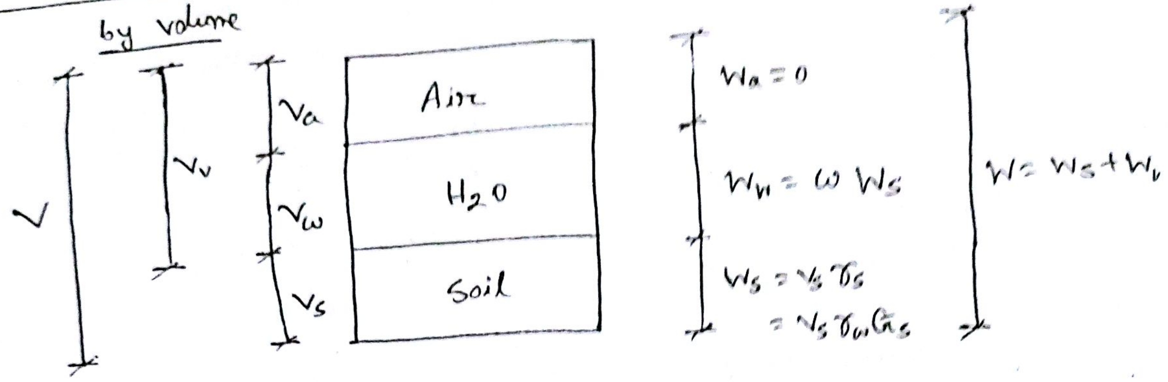
- $\omega = \frac{W_w}{W_s} = \frac{V_w\gamma_w}{V_s\gamma_s} = \frac{V_w\gamma_w}{V_s\gamma_w G_s} = \frac{V_w}{V_s G_s} = \frac{\frac{V_w}{V_v} \times \frac{V_v}{V_s}}{G_s} = \frac{S_r \times e}{G_s}$

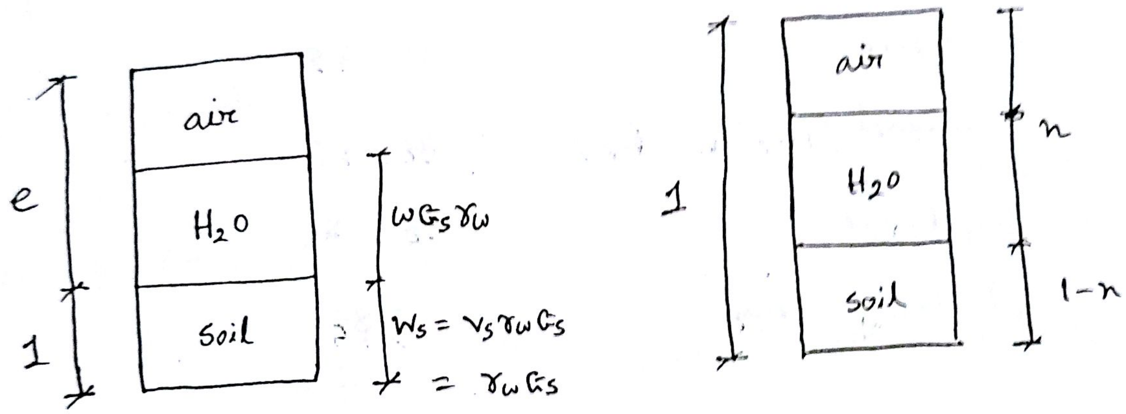
  $\therefore S_r = \frac{G_s\omega}{e}$.

phase diagram, [illegible]

by volume

by weight





$$\gamma_{bulk} = \frac{W}{V} = \frac{\omega G_s \gamma_w + \gamma_w G_s}{1+e} = \frac{\gamma_w G_s(1+\omega)}{1+e} = (1+\omega)\gamma_d$$

$$\gamma_{bulk} = \frac{\gamma_w G_s + \omega G_s \gamma_w}{1+e} = \frac{\gamma_w G_s + e\gamma_w}{1+e} = \frac{e+G_s}{1+e}\gamma_w$$

$$\gamma_d = \frac{W_s}{V} = \frac{\gamma_w G_s}{1+e}$$

$$\gamma_{sat} = \frac{W}{V} = \frac{\gamma_w G_s + e\gamma_w}{1+e} = \frac{e+G_s}{1+e}\gamma_w$$

$$\gamma' = \gamma_{sub} = \gamma_b = \gamma_{sat} - \gamma_w = \frac{(e+G_s)}{1+e}\gamma_w - \gamma_w = \frac{G_s - 1}{1+e}\gamma_w$$

⊞ Density Index / Relative Density, $I_D = \frac{e_{max} - e_f}{e_{max} - e_{min}}$

$$= \frac{V_{max} - V_f}{V_{max} - V_{min}} = \frac{\frac{1}{\gamma_{d\,min}} - \frac{1}{\gamma_{df}}}{\frac{1}{\gamma_{d\,min}} - \frac{1}{\gamma_{d\,max}}}$$

Sate of compaction:	I_d %
1) very loose	0 – 15
2) Loose	15 - 35
3) compact / Medium dense	35 – 65
4) Dense	65 – 85
5) Very dense	> 85

⊞ Degrees of compaction / Relative compaction (RC) $= \frac{\gamma_{df}}{\gamma_{d\,max}} \times 100$ %

Hydraulic properties of soil.

- $K = \frac{QL}{hAt}$ [constant head permeameter]

- $K = \frac{aL}{At} \log \frac{h_0}{h_1}$ [Falling head permeameter]

- $\sigma' = \gamma' z \pm i z \gamma_w$ [upward → -ve, downward → +ve]

- Hydraulic gradient $= \frac{h}{N_d \text{ (No. of drops)}}$

- F.S (Terzaghi's method) $= \frac{\text{Submerged weight of the soil in the heave zon}}{\text{Magnitude of seepage pressure}}$

$= \frac{W'}{U} = \frac{\gamma'}{i_{av} \gamma_w}$ [Recommended → 4 to 5]

F.S (Hazra's method) $= \frac{i_{cr}}{i_{exit}}$ [Recommended → 3 to 4]

$i_{cr} = \frac{\gamma'}{\gamma_w} = \frac{G_s - 1}{1+e}$ = Critical gradient

$i_{exit} = \frac{\text{magnitude of each head}}{L}$ = exit gradient

- Seepage velocity, $v' = \frac{v}{n}$ → discharge velocity

Seepage ~~velocity~~ loss; $q = K h \frac{N_f}{N_d}$

- $i_{exit} > i_{cr}$ → piping failure.

Consolidation characteristic of soil

- $a_v \rightarrow$ coefficient of compressi
- $m_v \rightarrow$ coefficient of volume compr
- $C_c \rightarrow$ Compression index
- $C_s \rightarrow$ Swelling index
- $C_v \rightarrow$ coefficient of consolidation

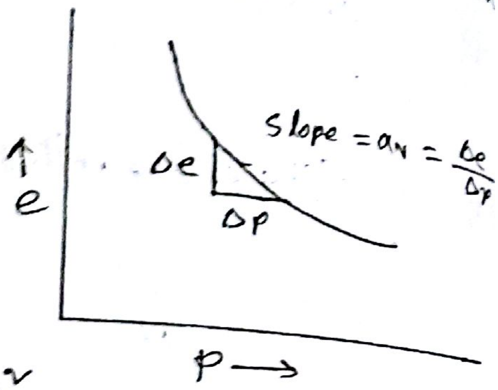
- $S = \frac{\Delta e}{1+e_0} H$

  $S = \frac{a_v}{1+e_0} \Delta P H$

  $S = m_v \Delta P H$

- a_v = slope of e logp e vs p curve

  $= \frac{\Delta e}{\Delta P} \Rightarrow \Delta e = a_v \Delta P$



- $\Delta e = C_c \log \frac{P_0 + \Delta P}{P_0}$

- $S = \frac{C_c H}{1+e_0} \log \frac{P_0 + \Delta P}{P_0}$

- $U < 60$, $T_v = \frac{\pi}{4}\left(\frac{U\%}{100}\right)^2$

  $U > 60$, $T_v = 1.781 - 0.933 \log(100 - U)$

- $C_v = \frac{T_v H_{dr}^2}{t}$

  [Both way drainage, $H_{dr} = \frac{\text{Height of soil sample}}{2}$

  one way drainage; H_{dr} = height of soil sample]

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

- $\frac{S_1}{S_2} = \frac{U_1}{U_2} = \sqrt{\frac{t_1}{t_2}}$

- $C_v = \frac{K}{\gamma_w m_v}$

(v) $U = \frac{S_t}{S_f} \times 100\,\%$

(vi) Secondary compression index, $C_\alpha = \dfrac{\Delta e}{\log(t_2/t_1)}$

Settlement, $S_s = C_\alpha' \, H \, \log\left(\frac{t_2}{t_1}\right)$

$$C_\alpha' = \frac{C_\alpha}{1+e_p}$$

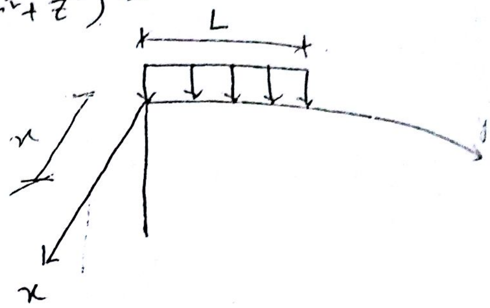
↓

coefficient of secondary consolidation

Stress Distribution of Soil

- Boussinesq $\longrightarrow$ $\sigma_z = \dfrac{3Q}{2\pi} \dfrac{z^3}{(r^2+z^2)^{5/2}}$

- Finite line load:



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

$$m = \frac{x}{z}$$

$$\sigma_z = \frac{q}{z} P_0$$

where $P_0 = \dfrac{1}{2\pi(m^2+1)^2}\left[\dfrac{3n}{\sqrt{n^2+1+m^2}} - \left(\dfrac{n}{\sqrt{m^2+n^2+1}}\right)^3\right]$

- Rectangular / square footing:

$$m = \frac{L}{z}, \quad n = \frac{B}{z}$$

for $m^2+n^2+1 > m^2n^2$

$$\sigma_z = \frac{q}{4\pi}\left[\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+m^2n^2+1} \cdot \frac{m^2+n^2+2}{m^2+n^2+1} + \sin^{-1}\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+m^2n^2+1}\right]$$

for $m^2+n^2+1 < m^2n^2$,

$$\sigma_z = \frac{q}{4\pi}\left[\frac{2mn\sqrt{m^2+n^2+1}}{(m^2+n^2+m^2n^2+1} \cdot \frac{m^2+n^2+2}{m^2+n^2+1} + \pi - \sin^{-1}\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+m^2n^2+1}\right]$$

$$\sigma_z = \frac{3Q}{2\pi} \cdot \frac{z^3}{(r^2+z^2)^{5/2}}$$ $\longrightarrow$ Boussinesq's equation.

$$\sigma_z = \frac{3Q}{2\pi} \cdot \frac{1}{z^2} \cdot \cos^5\psi$$

$\psi = 39.23°$ for maximum.

$\sigma_z/\sigma q$	a/z
0	0
0.1	0.27
0.2	0.40
0.3	0.52
0.4	0.64
0.5	0.77
0.6	0.92
0.7	1.11
0.8	1.39
0.9	1.97
1.0	∞

Capillary rise:

Fine gravel	0.02 to 0.1
coarse sand	0.1 to 0.15
medium sand	0.15 to 0.3
Fine sand	0.3 to 1
silt	1 to 10
clay	10 to 30 m.
colloid	> 30 m.

maximum height of cappillary rise, $h_c = \frac{C}{e D_{10}}$

Relative Density:

~~very~~ very loose	< 15
loose	15 - 35
medium dense	35 - 65
dense	65 - 85
very dense	> 85.

Weight Volume Relationship

Example - 1:

A natural soil deposit has a bulk unit weight of 18.44 KN/m³ and a moisture content of 5%. Calculate the volume of water required to raise the moisture content to w = 15%. Also calculate the volume of water required to make 1 m³ of soil sample fully saturated from w = 15%. Given $G_s = 2.67$. Also calculate S_r at w = 15%.

Ans:

Here, $\gamma_{bulk} = 18.44$ KN/m³ ; $\omega = 0.05$

$$\gamma_d = \frac{\gamma_{bulk}}{1+\omega} = \frac{18.44}{1+0.05} = 17.56 \text{ KN/m}^3$$

Assuming volume of soil = 1 m³

$$\gamma_d = \frac{W_s}{V} = \frac{17.56}{\cancel{1}} \quad \therefore W_s = 17.56 \text{ KN}.$$

Additional water required from $W_{0.05}$ to $W_{0.15}$

$$= (0.15 - 0.05) \times 17.56$$

$$= 1.756 \text{ KN}.$$

Additional volume of H_2O $= \frac{1756}{9.81} = 0.179 \text{ m}^3 = \boxed{179 \text{ Litres}}$ of water.

Now, $\gamma_d = \frac{\gamma_w G_s}{1+e} \Rightarrow 17.56 = \frac{9.81 \times 2.67}{1+e}$

$\therefore e = 0.49$

At fully saturated condition,

$S_r = 1 = \frac{w G_s}{e}$

$\Rightarrow w = \frac{e}{G_s} = \frac{0.49}{2.67} = 18.35\%$

Additional water required for $w_{15\%}$ to $w_{18.35\%}$ =

$= (0.1835 - 0.15) \times 17.56$

$= 0.59$ KN.

Additional volume of water required $= \frac{0.59}{9.81} = 0.06\ m^3$

$= \boxed{60 \text{ Litres.}}$

Example 2:

A soil having moisture content of 15%. Its dry unit weight is given as 17.5 kN/m³. Assume $G_s = 2.7$. Compute bulk density, saturated density, effective unit weight. Also calculate the quantity of water required to make 100 m³ of soil sample fully saturated.

Ans:

$\omega = 15\%$, $\gamma_d = 17.5\ \text{KN/m}^3$, $G_s = 2.7$

$$\gamma_{bulk} = \gamma_d(1+\omega) = 17.5 \times (1+0.15) = \boxed{20.1\ \text{KN/m}^3}$$

$$\gamma_d = \frac{\gamma_w G_s}{1+e} \Rightarrow 17.5 = \frac{9.81 \times 2.7}{1+e} \quad \therefore e = 0.51$$

$$\gamma_{sat} = \frac{G_s + e}{1+e}\gamma_w = \frac{2.7+0.51}{1.51} \times 9.81 = \boxed{20.85\ \text{KN/m}^3}$$

$$\gamma_{effective} = \gamma_{sat} - \gamma_w = 20.85 - 9.81 = \boxed{11.04\ \text{KN/m}^3}$$

$$\gamma_d = \frac{W_s}{V} \Rightarrow 17.5 = \frac{W_s}{100} \quad \therefore W_s = 1750\ \text{KN.}$$

$$S_r = \frac{\omega G_s}{e} \Rightarrow 1 = \frac{\omega \times 2.7}{0.51} \quad \therefore \omega = 18.9\%$$

So, addition water required $= (0.189 - 0.15) \times 1750$

$$= \boxed{68.25\ \text{KN.}}$$

Alter approach

From bulk to saturated, for 1 m³ sample, water required

$= (20.85 - 20.1) = 0.75$ KN.

$\therefore$ for 100 m³ sample $= \boxed{75\ \text{KN}}$.

xample-3:

Soil is to be excavated from a borrow pit which has density of 1.75 gm/cc at $w = 12\%$ with $G_s = 2.7$ Anot This soil is compacted so that $w = 18\%$ and dry density of 1.65 gm/cc. For $1000 m^3$ soil in the fill area estimate volume of soil to be transported from borrow pit area. Also calculate quantity of water to be added.

Ans:

Borrow

$\rho_b = 1.75 \text{ gm/cc} = 1750 \text{ Kg/m}^3$

$\gamma_b = \frac{1750 \times 9.81}{1000} \text{ KN/m}^3 = 17.17 \text{ KN/m}^3$

$\gamma_d = \frac{\gamma_b}{1+w} = \frac{17.17}{1+0.12} = 15.33 \text{ KN/m}^3$

$\gamma_d = \frac{W_s}{V_b} \Rightarrow 15.33 = \frac{16187}{V}$

$\therefore \boxed{V_b = 1056 \text{ m}^3}$

Fill

$\rho_d = 1.65 \text{ gm/cc}$

$\gamma_d = \frac{1.65 \times 9.81}{1000} = 16.187$

$\gamma_d = \frac{W_s}{V_f} \Rightarrow 16.187 = \frac{W_s}{1000}$

$\therefore W_s = 16187 \text{ Kg}$

Wt. of water required $= 16187 \times (0.18 - 0.12)$

$= \boxed{971 \text{ KN}}$

Another approach:

$\rho_f = \rho_{bulk(fill)} = 1.75$ gm/cc, $\rho_{dry(fill)} = 1.65$ gm/cc

Finished volume, $V_f = 1000\ m^3$

Now, $V_b \propto (1+e_b)$

$V_f \propto (1+e_f)$

$$\therefore \frac{V_b}{V_f} = \frac{1+e_b}{1+e_f}$$

$$\rho_{dry(b)} = \frac{\rho_{bulk(b)}}{1+w}$$

$$\Rightarrow \rho_{dry(b)} = \frac{1.75}{1+0.12} = 1.56 = \frac{G_s \rho_w}{1+e_b} = \frac{2.7\times 1}{1+e_b}$$

$$\Rightarrow e_b = 0.728$$

$$-\ \cancel{\frac{V_b}{1000}} = \rho_{dry(f)} = \frac{G_s \rho_w}{1+e_f} \quad \therefore e_f = 0.636$$

Now, $\dfrac{V_b}{1000} = \dfrac{1+0.728}{1+0.636}$ $\quad \therefore \boxed{V_b = 1056\ m^3}$

$$V_b = V_s + V_v$$

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

$$\therefore V_s = \frac{1056}{1+0.728} = 611\ m^3$$

Wt. of soil $= V_s \gamma_w G_s = 611 \times 9.81 \times 2.7 = 16184$ kN

Water added $= 16184 \times 0.06 = \boxed{971\ \text{kN}}$

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Example - 4:

Find out the mechanical energy needed in standard proctor test and Modified proctor test.

Ans

Standard proctor test: Layer = 3, No. of blow = 25

Wt. of hammer = 5.5 lb, Height of fall = 1'

~~So, energy = mgh = 4.5 ×~~

Volume = 1/30 ft^3.

$\therefore$ Energy required per unit volume $= \frac{mgh}{V} = \frac{5.5 \times 3 \times 25 \times 1}{1/30}$

$= 12375$ lb/cft

Modified Proctor test:

Layer = 5, No. of blows = 25, wt. of hammer = 10 lb

Height of fall = 1.5', Volume = 1/30 ft^3

$\therefore$ Energy required per unit volume $= \frac{5 \times 25 \times 10 \times 1.5}{1/30}$

$= 56250$ lb/cft

So, Modified test required 4.5 times more energy than standard test.

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Problem 10 : Peck - hanson :

Given, Degrees of compaction $= \frac{\gamma_{df}}{\gamma_{dmax}} \times 100 = 95$

$$\Rightarrow \frac{\gamma_{df}}{124} \times 100 = 95$$

$$\therefore \gamma_{df} = 117.8$$

Again, $\gamma_{df} = \frac{G_s \gamma_w}{1+e_f}$

$$\Rightarrow 117.8 = \frac{2.65 \times 62.4}{1+e_f}$$

$$\therefore e_f = 0.404$$

Given, $e_b = 0.6$, $V_f = 1 \ ft^3$

$$\therefore \frac{V_b}{V_f} = \frac{1+e_b}{1+e_f}$$

$$\Rightarrow V_b = \frac{1+0.6}{1+0.4} = \cancel{1.2} \boxed{1.14 \ ft^3}$$

Term Question Solve

[Weight Volume Relationship]

~~2009~~

$\boxed{2010-11.\ 5(c)}$

A	B
$e_A = 0.80$	$e_B = 0.70$
$\omega_A = 20\%$	$\omega_B = 15\%$

Fill area:

$$\gamma_{bulk} = 2\ ton/m^3 = \frac{2 \times 1000 \times 9.81}{1000}\ KN/m^3$$

$$= 19.62\ KN/m^3$$

$\omega = 0.22$

$$\gamma_d = \frac{\gamma_{bulk}}{1+\omega} = \frac{19.62}{1+0.22} = 16.08$$

Again, $\gamma_d = \dfrac{\gamma_w G_s}{1+e_f}$

$$\Rightarrow 16.08 = \frac{9.81 \times 2.65}{1+e_f}$$

$\therefore e_f = 0.62$

Given that, $V_f = 10,000\ m^3$

So, $\frac{V_A}{V_f} = \frac{1+e_A}{1+e_f}$

$\Rightarrow \frac{V_A}{10,000} = \frac{1+0.8}{1+0.62}$

$\Rightarrow \boxed{V_A = 11,111\ m^3}$

Again, $\frac{V_B}{V_f} = \frac{1+e_B}{1+e_f}$

$\Rightarrow V_B = \frac{1+0.7}{1+0.62} \times 10,000 = \boxed{10494\ m^3.}$

cost of $A = \frac{2000}{100} \times 11111 = 2,22,220$ TK

cost of $B = \frac{2200}{100} \times 10494 = 2,30,868$ TK.

So, Bororow area A is more economical.

Now, $\gamma_{d,A} = \frac{G_s \gamma_w}{1+e_A} = \frac{2.65 \times 9.81}{1+0.8} = 14.44$

$\gamma_{d,A} = \frac{W_s}{V} \Rightarrow 14.44 = \frac{W_s}{100}$

$\therefore W_s = 1444$ KN. , For fully saturated, $S_r = \frac{wG_s}{e_A} = 1$

$\Rightarrow w = \frac{0.8}{2.65} = 0.3$

Addition quantity of water ~~= 1444×~~

$= 1444 \times (0.3 - 0.2)$

$= \boxed{144.4\ KN}$

$\boxed{2009-10 . 5(d)}$

Given, $e_A = 0.79$, $e_B = 0.65$

$\omega_A = 18\%$, $\omega_B = 15\%$

$V_f = 4 \times 10^6 \, m^3$

$\cancel{\gamma_{bulk(f)}} = \rho_{bulk(f)} = 2 \text{ ton}/m^3 = 2000 \text{ kg}/m^3$

$\gamma_{bulk(f)} = \frac{2000 \times 9.81}{1000} \text{ KN}/m^3 = 19.62 \text{ KN}/m^3$

$\omega_f = 21\%$.

$\gamma_{d,A} = \frac{\gamma_w G_s}{1+e_A} = \frac{9.81 \times 2.65}{1.79} = 14.52 \text{ kN}/m^3$

$\frac{V_A}{V_f} = \frac{1+e_A}{1+e_f}$

Now, $\gamma_{d(f)} = \frac{\gamma_{bulk(f)}}{1+\omega_f} = \frac{19.62}{1.21} = 16.22$

Again $\gamma_{d(f)} = \frac{\gamma_w G_s}{1+e_f}$

$\therefore e_f = \frac{9.81 \times 2.65}{16.22} - 1 = 0.6$

Now, $\frac{V_A}{V_f} = \frac{1+e_A}{1+e_f} \Rightarrow V_A = \frac{1.79}{1.6} \times 4 \times 10^6$

$= 4.475 \times 10^6 \, m^3$

VA(

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$V_{A\,(truck)} = 4.475\times10^6 + 4.475\times10^6\times\frac{10}{100}$

$= 4.92\times10^6\ m^3.$

Again $\frac{V_B}{V_f} = \frac{1+e_B}{1+e_f} \Rightarrow V_B = \frac{1.65}{1.6}\times4\times10^6$

$= 4.125\times10^6\ m^3.$

$\therefore V_{B\,(truck)} = 4.125\times10^6 + 4.125\times10^6\times\frac{10}{100}$

$= 4.54\times10^6\ m^3.$

$\gamma_{d,af} = \frac{W_s}{V_f} \quad \therefore W_s = \cancel{14.52}\ 16.22\times4\times10^6 = 64.88\times10^6\ kN.$

Water required for $A = 64.88\times10^6\times(0.21-0.18)$

$= 1.946\times10^6\ kN = \frac{1.946\times10^6}{9.81} = 1.98\times10^5\ m^3$

Water required for $B = \frac{64.88\times10^6\,(0.21-0.15)}{9.81} = 3.97\times10^5\ m^3.$

Assume that, each truck carries $10\ m^3$ of water.

$\therefore$ cost of A site $= \frac{4.92\times10^6}{10}\times1000 + \frac{1.98\times10^5}{10}\times300$

$= 497.96\times10^6$ TK.

cost of B site $= \frac{4.54\times10^6}{10}\times1150 + \frac{3.97\times10^5}{10}\times300$

$= 534\times10^6$ TK.

$\therefore$ $\boxed{\text{A site is more economical.}}$

Confirm

$\boxed{2008\text{-}09.\ 5(c)}$

$w_b = 10\%$, $\gamma_{bulk(b)} = 1.8 \times 9.81 = 17.66\ kN/m^3$

$w_f = 18\%$, $\gamma_{d(f)} = 1.85 \times 9.81 = 18.15\ kN/m^3$

$V_b = 1m^3$

$$\gamma_{d(b)} = \frac{17.66}{1+0.1} = 16.06\ kN/m^3 = \frac{W_s}{V_b}$$

$\therefore W_s = 16.06\ kN.$

Quantity of water to be added $= 16.06 \times (0.18 - 0.1)$

$= 1.285\ kN$

$$= \frac{1.285 \times 1000}{9.81} = \boxed{131\ \text{litre}}$$

$$\gamma_{d(b)} = \frac{\gamma_w G_s}{1+e_b} \Rightarrow 16.06 = \frac{9.81 \times 2.65}{1+e_b}$$ [Assume, $G_s = 2.65$

$\therefore e_b = 0.62$

$$\gamma_{d(f)} = \frac{\gamma_w G_s}{1+e_f} \Rightarrow 18.15 = \frac{9.81 \times 2.65}{1+e_f}$$

$\therefore e_f = 0.43$

Now, $$\frac{V_b}{V_f} = \frac{1+e_b}{1+e_f} \Rightarrow \frac{V_b}{1} = \frac{1+0.62}{1+0.43}$$

$$\therefore \boxed{V_b = 1.133\ m^3}$$

confirm

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~~2008-09. 5(d)~~

2007-08 5(b)

$V_b = 3.3 \times 10^5 \ m^3$, $e_b = 1.20$

$V_f = ?$ $e_f = 0.7$.

$$\frac{V_f}{V_b} = \frac{1+e_f}{1+e_b}$$

$$\therefore V_f = \cancel{8}\,3.3 \times 10^5 \times \frac{1.7}{2.2} = \boxed{2.55 \times 10^5 \ m^3} \ \cancel{4.675 \times 10^5 \ m^3}$$

$$\gamma_{d,(f)} = \frac{\gamma_w G_s}{1+e_f} = \frac{9.81 \times 2.7}{1+0.7} = \overset{15.58}{\cancel{13.85}} \ kN/m^3$$

$$\gamma_{d.(f)} = \frac{W_s}{V_f} \Rightarrow \overset{15.58}{\cancel{13.85}} = \frac{W_s}{\underset{2.55}{\cancel{4.675}} \times 10^5}$$

$$\therefore W_s = \overset{\overset{3.97}{\cancel{3.53}}}{\cancel{6.67}} \times 10^6 \ kN.$$

$$= \frac{\overset{3.5397}{\cancel{6.67}} \times 10^6 \times 10^3}{9.81} = \boxed{\cancel{3.6 \times 10^8 \ kg}} = \boxed{4.05 \times 10^8 \ kg}$$

confirm

2007-08 . 5(c)

$\gamma_{d(min)} = 13.34 \ kN/m^3$, $\gamma_{d,(max)} = 21.40 \ kN/m^3$

$$n = 0.3, \quad e = \frac{n}{1-n} = \frac{0.3}{1-0.3} = 0.43$$

$$\gamma_{d,f} = \frac{\gamma_w G_s}{1+e} = \frac{9.81 \times 2.67}{1+0.43} = 18.32 \ kN/m^3$$

Relative density, $I_D = \dfrac{1/\gamma_{dmin} - 1/\gamma_{d,f}}{\dfrac{1}{\gamma_{dmin}} - \dfrac{1}{\gamma_{dmax}}} = \dfrac{\frac{1}{13.34} - \frac{1}{18.32}}{\frac{1}{13.34} - \frac{1}{21.4}} = \boxed{72.2\%}$

As the relative density falls in between 65-85%, so the soil sample is $\boxed{\text{dense.}}$

confirm

$\boxed{2006-07\ 5(d)}$

Given, $\gamma_{bulk} = 2.06 \times 9.81 = 20.2\ kN/m^3$

$\omega = 0.14$

$$\gamma_d = \frac{\gamma_{bulk}}{1+\omega} = \frac{20.2}{1+0.14} = \boxed{17.7\ KN/m^3}$$

At, zero air voids, $S_r = 1$

$$\therefore \gamma_d = \frac{\gamma_w G_s}{1+e} = \frac{\gamma_w G_s}{1+\frac{\omega G_s}{S_r}} = \frac{9.81 \times 2.67}{1+\frac{0.14 \times 2.67}{1}}$$

$$= \boxed{19.07\ KN/m^3}$$

Now, $\gamma' = \gamma_{sat} - \gamma_w = \frac{G_s - 1}{1+e}\gamma_w$

Now, $\gamma_d = \frac{G_s \gamma_w}{1+e} \Rightarrow 17.7 = \frac{2.67 \times 9.81}{1+e}$

$$\therefore e = 0.48$$

again, $\gamma_{sat} = \frac{G_s - 1}{1+e}\gamma_w + \gamma_w \ / \ \frac{G_s + e}{1+e}\gamma_w$

$$= \frac{2.67-1}{1+0.48} \times 9.81 + 9.81$$

$$= \boxed{20.88\ KN/m^3}$$

confirm

2004-05. 1(a)

$\gamma_{bulk,A} = 101\ lb/ft^3$, $w_A = 10\%$

$\gamma_{bulk,B} = 96\ lb/ft^3$, $w_B = 14\%$

$V_f = 350,000\ ft^3$, $w_f = 14\%$

$\gamma_{d,A} = \frac{101}{1+0.1} = 91.8$, $\gamma_{d,B} = \frac{96}{1.14} = 84.2\ lb/ft^3$.

$\gamma_{d,A} = \frac{\gamma_w G_s}{1+e_A} \Rightarrow 91.8 = \frac{\cancel{9.81}\ 62.4 \times 2.65}{1+e_A}$ $\therefore e_A = 0.8$

$\gamma_{d,B} = \frac{\gamma_w G_s}{1+e_B} \Rightarrow 84.2 = \frac{62.4 \times 2.65}{1+e_B}$ $\therefore e_B = 0.96$

$\gamma_{bulk,f} = 126\ lb/ft^3$, $\gamma_{d,f} = \frac{126}{1.14} = 110.53$

Again, $\gamma_{d,f} = \frac{\gamma_w G_s}{1+e_f}$ $\therefore e_f = 0.496$

Now, $\frac{V_A}{V_f} = \frac{1+e_A}{1+e_f}$ $\therefore V_A = \frac{1+0.8}{1+0.496} \times 350,000 = \boxed{4.21 \times 10^5}$

$V_B = \frac{1+0.96}{1+0.496} \times 350,000 = \boxed{4.6 \times 10^5\ ft^3}$

$W_s = \gamma_{d,f} \times V_f = 110.53 \times 350,000 = 38.7 \times 10^6\ lb$

water required for A site $= 38.7 \times 10^6 \times (0.14 - 0.1) = \frac{1.548 \times 10^6\ lb}{62.4\ lb/ft^3}$

$= 24.8 \times 10^3\ ft^3$

cost for A site $= 4.21 \times 10^5 \times 7.5 + 24.8 \times 10^3 \times 1 = \boxed{3.18 \times 10^6\ Tk.}$

cost for B site $= 4.6 \times 10^5 \times 7.5 = \boxed{3.45 \times 10^6\ Tk}$

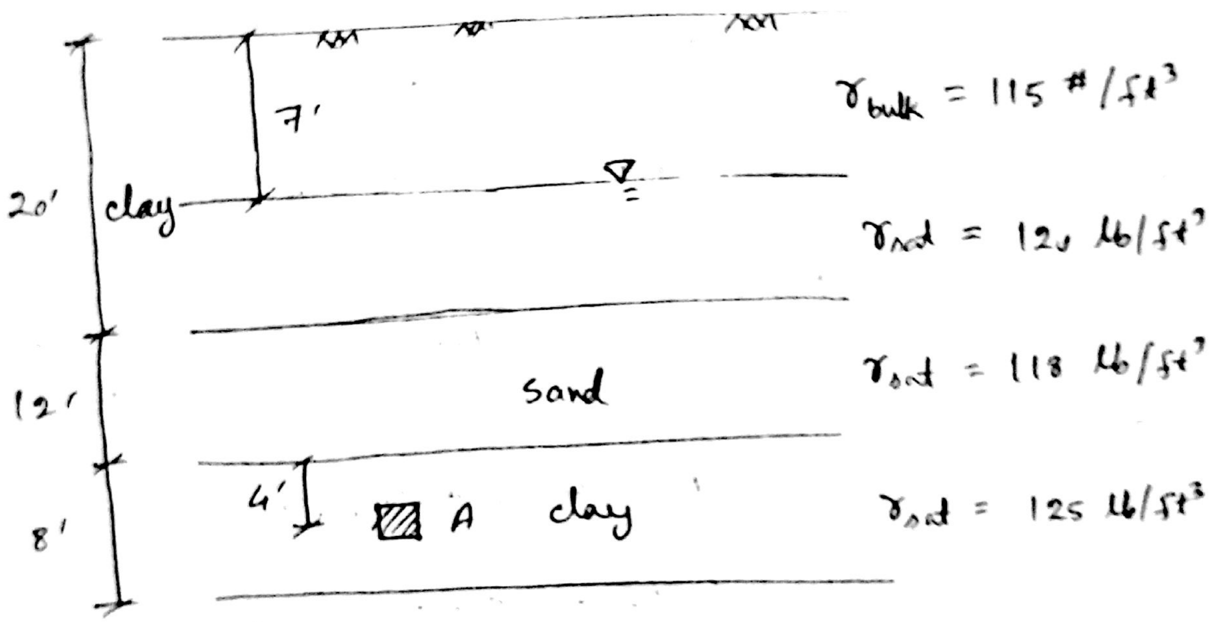
confirm

Hydraulic Properties of soil

Fluid flow through Porous media

Class Lectures:

Example 1: Find effective stress at point A (using total stress approach & effective stress approach)



Effective stress, $\bar{P}_A = 7 \times 115 + 13 \times (120 - 62.4) + 12 \times (118 - 62.4)$

$+ (125 - 62.4) \times 4 = \boxed{2471.4 \text{ lb/ft}^2}$

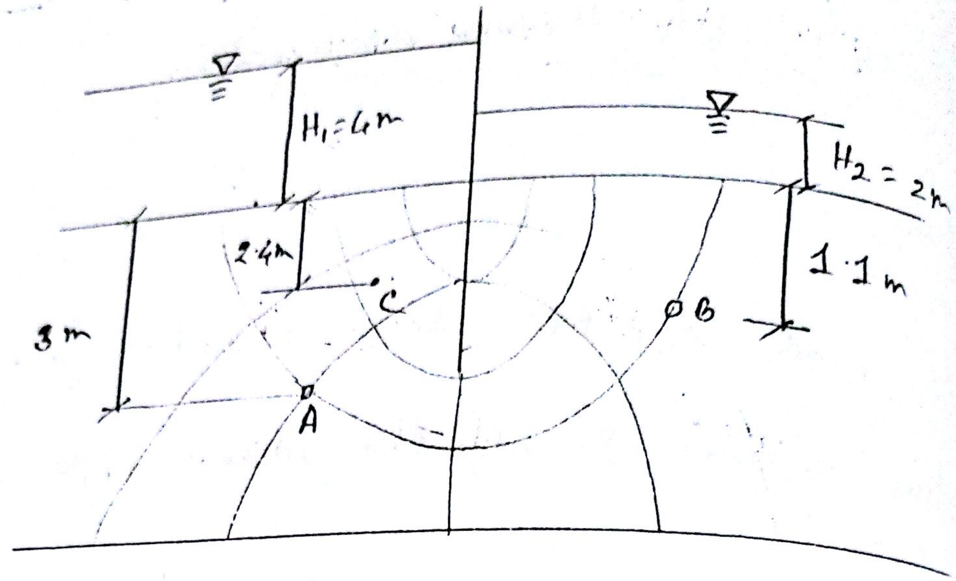
Total stress, $P_A = 7 \times 115 + 13 \times 120 + 118 \times 12 + 125 \times \cancel{8}^{\,4} = 4281$ ps

Pore water pressure, $P_{A.w} = 13 \times 62.4 + 12 \times 62.4 + 4 \times \cancel{125}^{\,62.4} = 1809.6$ ps

$\therefore$ Effective stress = Total stress − pore water pressure

$= 4281 - 1809.6 = \boxed{2471.4 \text{ psf}}$ Ans.

Example 2: Find σ_A', σ_B' & σ_C'



Given, $\gamma_{sat} = 20\ KN/m^3$

σ_A: No. of drops, $N_d = 6$, $H = 2m$.

hydraulic gradient, $i = \frac{H}{N_d} = \frac{2}{6} = 0.33$

σ_A: Total stress, $\sigma_A = 4\gamma_w + 3\gamma_{sat}$

$= 4 \times \cancel{62.4}\ 9.81 + 3 \times 20 = \cancel{309.6\ Kpa}\ 99.24$

Pore water pressure, $\sigma_{w,A} = 7\gamma_w - \cancel{03}\ iz\gamma_w$

$= 7 \times \cancel{62.4}\ 9.81 - 0.33 \times 2 \times \cancel{62.4}\ 9.81$

$= 62.2$ Kpa.

$\therefore$ Effective stress, $\sigma_A' = 99.24 - 62.2 = \boxed{37.04\ KPA}$

Another method, $\sigma'_A = \cancel{4\times9.81} + 3\times(20-9.81) + 0.33\times2\times9.81$

$= \boxed{37.04\text{ kpa}}$

$\cancel{\sigma_B}\ \sigma_C = 4\times9.81 + 2.4\times20 = 87.24\text{ kpa}$

$\sigma_{w,c} = 6.4\times9.81 - 0.33\times1.5\times9.81 = 57.93\text{ kpa}.$

$\sigma'_{c} = 87.24 - 57.93 = 29.31\text{ kpa}.$

Another method, $\sigma'_{c} = 2.4\times(20-9.81) + 0.33\times1.5\times9.81$

$= \boxed{29.31\text{ kpa}}$

$\sigma_B = 2\times9.81 + 1.1\times20 = 41.62\text{ kpa}.$

$\sigma_{w,B} = 3.31\times9.81 + 0.33\times1\times9.81 = \cancel{35.61}\ 33.65\text{ kpa}.$

$\sigma'_B = 41.62 - \cancel{35.61}\ 33.64 = \cancel{6.01\text{ kpa}}\ 7.97\text{ kpa}$

Another method, $\sigma'_B = 1.1\times(20-9.81) - 0.33\times1\times9.81$

$= \boxed{7.97\text{ kpa.}}$

σ'_B from upstream:

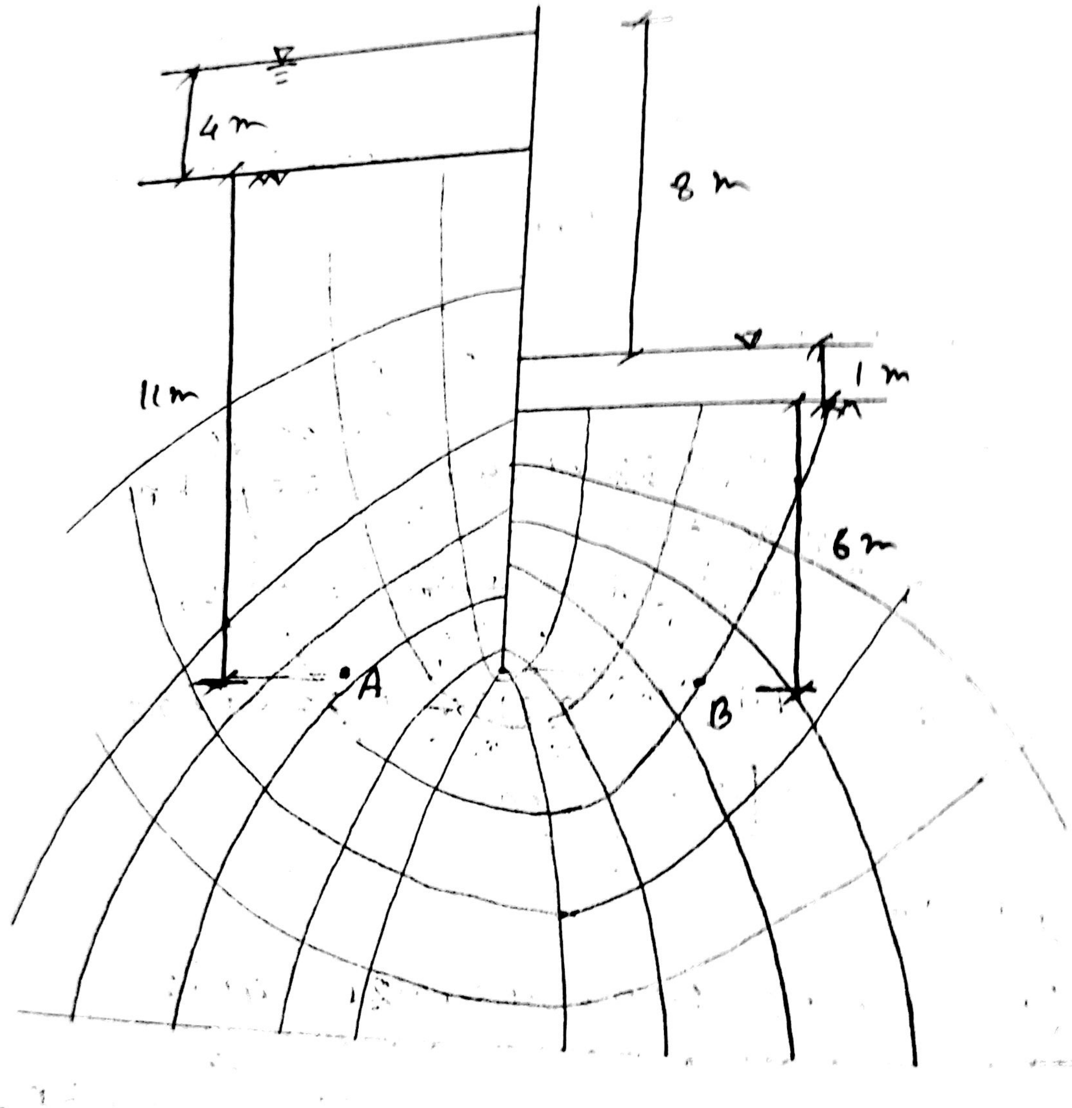
$\sigma_B = \cancel{(2+1.1)\times9.8}\ 2\times9.81 + 1.1\times20 = 41.62$

$\sigma_{w,B} = (4+1.1)\times9.81 - 0.33\times5\times9.81 = \cancel{37.77}\ 33.85\text{ kpa}.$

$\therefore \sigma'_B = 41.62 - 33.85 = \boxed{7.8\text{ kpa}}$

xample 3.3 [Craig - 101]

The flow net for seepage under a sheet pile wall is as shown in figure. The saturated unit weight of soil is 20 kN/m³. Determine the values of effective vertical stress at A and B.



$\gamma_{sat} = 20 \text{ kN/m}^3$

Hydraulic gradient, $i = \frac{h}{N_d} = \frac{8}{12} = 0.67$

Now, $\sigma_A' = 11\times(20-9.81) + 0.67\times 3.8\times 9.81$ [downward movement

$= \boxed{137\ kN/m^2}$ [from upstream side]

Again, from downstream side,

$\sigma_A' = 4\times 9.81 + 11\times 20 - (7\times 9.81 + 0.67\times 8.2\times 9.81)$

$= \boxed{137\ KN/m^2}$

Now, σ_B'. From downstream side,

$\sigma_B' = 6\times(20-9.81) \cancel{+} - 0.67\times 2.6\times 9.81$

$= \boxed{44\ KN/m^2}$

From upstream side,

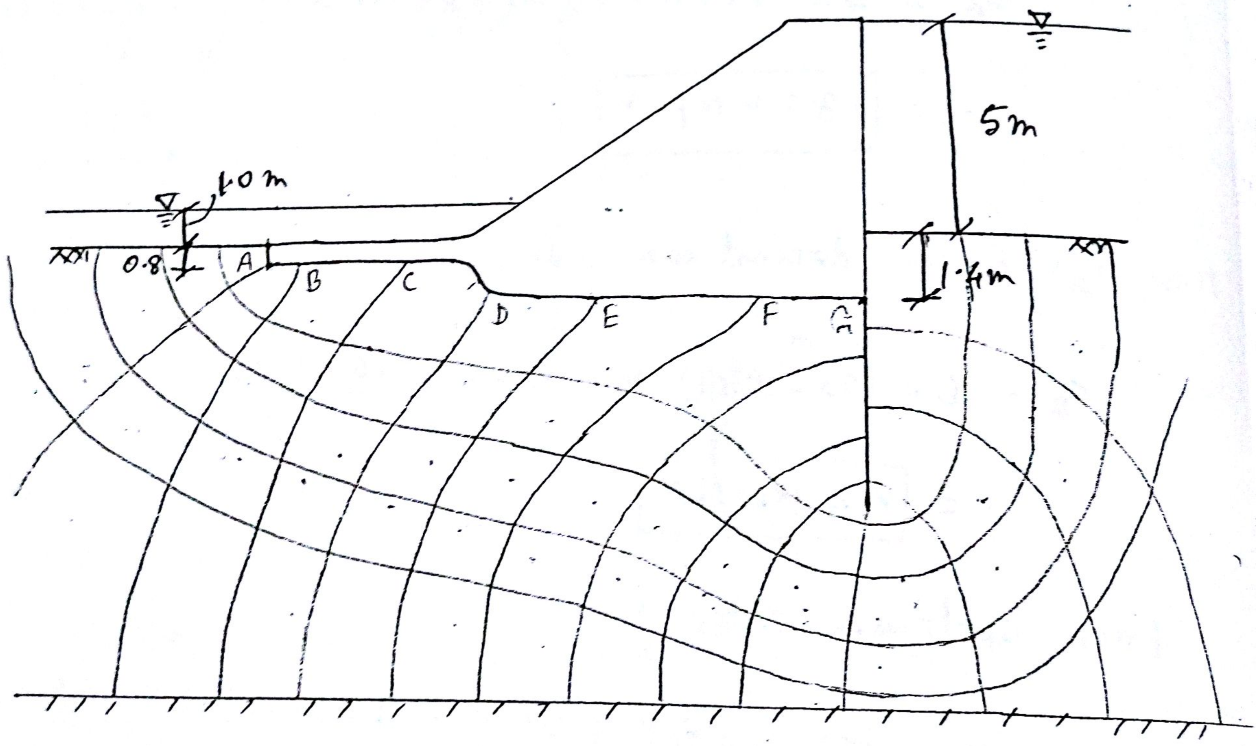
$\sigma_B' = \cancel{7\times 8}\ 1\times 9.81 + 6\times 20 - 15\times 9.81 + 0.67\times 9.4\times 9.81$

$= \boxed{44.4\ KN/m^2}$

Ans

Example – 2.2 [Craig - 58] Determine the quantity of seepage under the dam and plot the distribution of uplift pressure on the base of the dam. $K = 2.5 \times 10^{-5}$ m/s

Uplift pressure Distribution



quantity of seepage, $q = k\,h.\dfrac{N_f}{N_d}$

$$= 2.5 \times 10^{-5} \times 4 \times \frac{4.2}{\cancel{10.5}\ 14} = \boxed{3 \times 10^{-5}\ m^3/s\ \text{(per m)}}$$

~~$= 2.8 \times 10^{-5}$ m³/s (per m)~~

$$i = \frac{h}{N_d} = \frac{4}{14} = 0.286$$

pore water pressure at A, $\sigma_A = \cancel{1.8 \times \gamma_w} + h\gamma_w + iz\gamma_w$

$$= 1.8 \times 9.81 + \frac{4}{14} \times 1 \times 9.81$$

$$= 20.5 \ KN/m^2 .$$

$$\sigma_{w,B} = 1.8 \times 9.81 + \frac{4}{14} \times 2 \times 9.81$$

$$= 23.3 \ kpa .$$

$$\sigma_{w,c} = 1.8 \times 9.81 + \frac{4}{14} \times 3 \times 9.81$$

$$= 26.1 \ Kpa$$

$$\sigma_{w,D} = \overset{2.3}{\cancel{1.3}} \times 9.81 + \frac{4}{14} \times 4 \times 9.81 = \overset{33.8}{\cancel{23.96}} \ kpa .$$

$$\sigma_{w,E} = 2.4 \times 9.81 + \frac{4}{14} \times 5 \times 9.81 = 37.6 \ kpa .$$

$$\sigma_{w,F} = 2.4 \times 9.81 + \frac{4}{14} \times 6 \times 9.81 = 40.36 \ kpa .$$

$$\sigma_{w,E} = 2.4 \times 9.81 + \frac{4}{14} \times 6.6 \times 9.81 = 42.04 \ Kpa .$$

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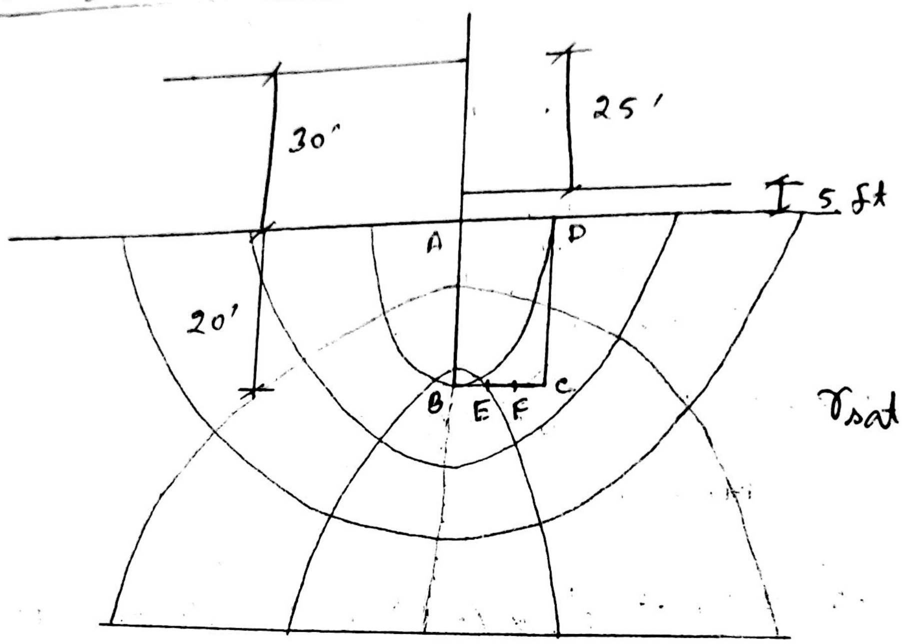
Stability Analysis of sheet pile:

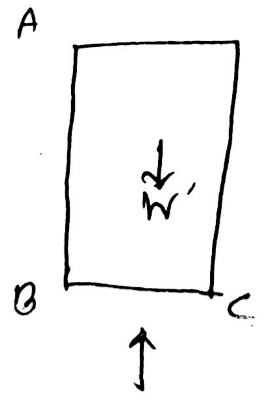
Example - 5.3 [B.M. Das - 82] → By Terzaghi method



$\gamma_{sat} = 112.32 \text{ lb/ft}^3$

Figure shows the flow net for seepage of water around a single row of sheet piles driven into a permeable layer. Calculate the F.S against downstream heave.

Ans:



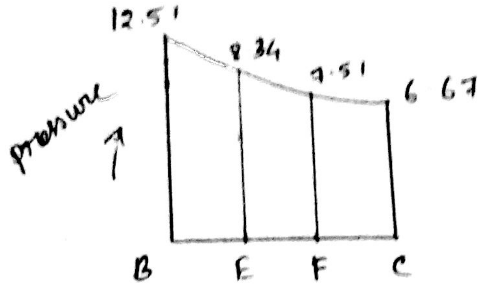
W' = submerged weight of soil in the heave zone.

$= \frac{L}{2} \times L \times 1 \times \gamma'$

U = magnitude of seepage pressure

$= i\gamma_w \times L \times L/2 \times 1$

So, $F.S = \frac{W'}{U} = \frac{\frac{L}{2} \times L \times \gamma'}{i\gamma_w \times L \times L/2} = \frac{\gamma'}{i_{av}\gamma_w}$



magnitude of each drop ~~head~~ $= \frac{25}{6} = 4.17$

At. B. driving head $= 3 \times 4.17 = 12.51$

At. C, driving head $= 1.6 \times 4.17 = 6.67$

At, E, driving head $= 2 \times 4.17 = 8.34$

At, F, driving head $= 1.8 \times 4.17 = 7.51$

$\cancel{\therefore j_{av} = \frac{12.51+6.67+8.34+7.51}{4} = 8.76}$

$i_{av} = \frac{h_{av}}{L} = \frac{(12.51+6.67+8.34+7.51)/4}{20} = 0.438$

$\therefore F.S = \frac{\gamma'}{i_{av}\gamma_w} = \frac{112.32 - 62.4}{0.437 \times 62.4} = \boxed{1.83}$

Recommended F.S in Terzoghis method is 4 to 5.

But our result is 1.83. So, we have to increase F.S.

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To increase F.S.:

1. Increasing W' by -

   a) adding more soil (but it should not obstruct flo

   b) using filter material for added weight.

2. Decreasing U by increasing length of the embankment (L

3. By giving thick clay layer at the upstream side to increase the length of the drainage path.

F.S by Hazra's method:

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

$$i_{cr} = \frac{\gamma'}{\gamma_w} = \frac{112-62.4}{62.4} = 0.8$$

$$i_{exit} = \frac{\text{magnitude of each head}}{L} = \frac{4.17}{20} = 0.21$$

$$\therefore F.S. = \frac{0.8}{0.21} = 3.8$$

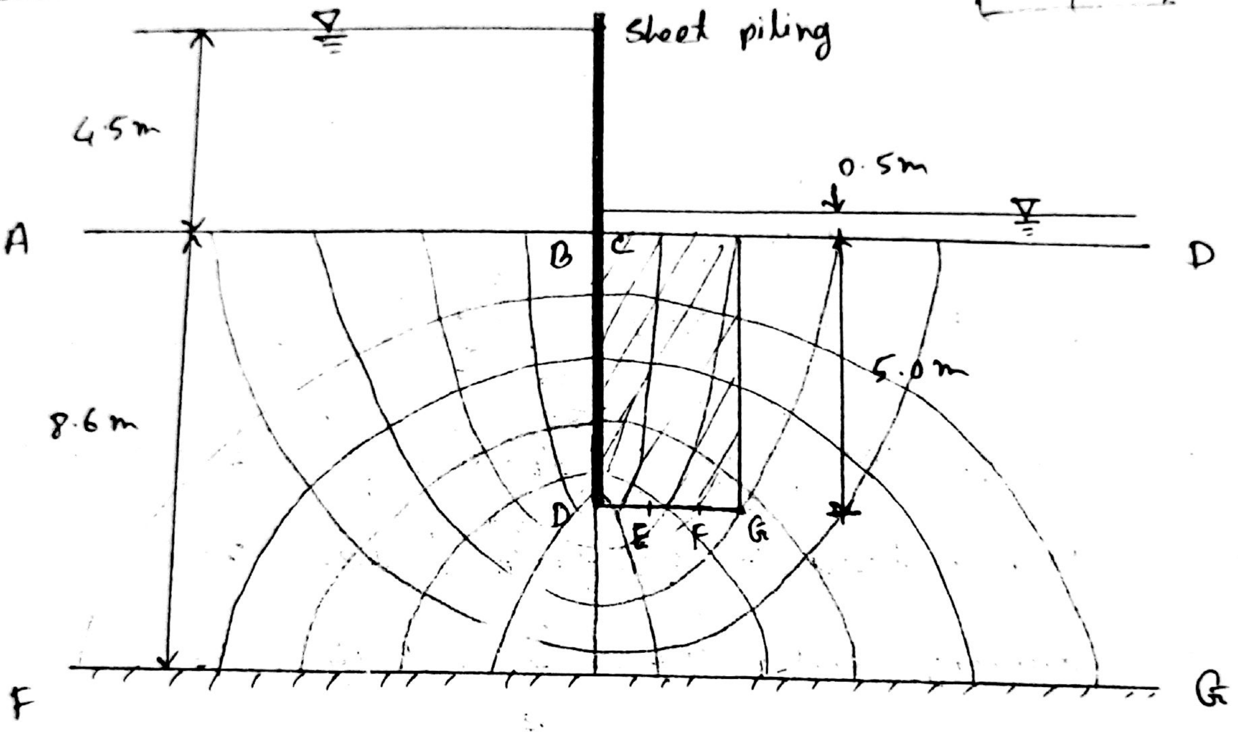
Recommended F.S in Hazra's method = 3 to 4.

So, F.S satisfied.

Term Question Solve

Hydraulic Properties of soil

2010-11 7(a)



Seepage loss in 6 months $= Kh \frac{N_f}{N_d} \times t$

$= 2.5\times10^{-5} \times 4 \times \frac{4}{12} \times 6\times30\times24\times60^2$

~~= 622 m³ (per m)~~

~~= 570.2 m³ (per m)~~

$= \boxed{518.4\ m^3\ (\text{per m})}$

confirm

F.S using Terzaghi's method:

$$F.S = \frac{\text{Submerged weight of soil in the heave zone } (W')}{\text{magnitude of seepage pressure } (U)}$$

$$= \frac{L \times L/2 \times 1 \times \gamma'}{i\gamma_w \times L \times L/2 \times 1}$$

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

Now, magnitude of each drop $= \frac{4}{12} = \cancel{0.4}\ \cancel{0.37}\ 0.33$

At D, driving head $= \cancel{0.4}\ 0.33 \times 6 = \cancel{1.85}\ \cancel{1.65}\ \cancel{1.98}\ 2$

At E, driving head, $= 4.2 \times \cancel{0.4}\ 0.33 = \cancel{1.68}\ 1.386$

At F, driving head $= 3.6 \times \cancel{0.4}\ 0.33 = \cancel{1.44}\ 1.188$

At G, driving head $= 3.2 \times \cancel{0.4}\ 0.33 = \cancel{1.28}\ 1.056$

$$\therefore i_{av} = \frac{(2 + \cancel{1.68}\ 1.386 + \cancel{1.44}\ 1.188 + \cancel{1.28}\ 1.056)/4}{5} = \cancel{0.32}\ \cancel{0.264}\ 0.28$$

$$\therefore F.S. = \frac{\gamma'}{i_{av}\gamma_w} = \frac{21.5 - 9.81}{\cancel{0.32}\ 0.28 \times 9.81} = \boxed{\cancel{3.72}}\ \cancel{4.5}\ \boxed{4.3}$$

Cud in Terzaghi's method, recommended F.S should be 4 to 5.

So, our result is ~~not~~ satisfied. ~~In this case, we have to increase F.S.~~

confirm

$\boxed{2009-10.\ 6(c)}$

Given that, $G_s = 2.65$, $n = 0.35$

$$e = \frac{n}{1-n} = \frac{0.35}{1-0.35} = 0.54$$

$F.S = 2,$

$$i_{cr} = \frac{G_s - 1}{1+e} = \frac{2.65-1}{1+0.54}$$

$$= 1.07$$

$$F.S = \frac{i_{cr}}{i_{exit}} \Rightarrow 2 = \frac{1.07}{i_{exit}}$$

$$\therefore i_{exit} = 0.536$$

$$\text{Again, } i_{exit} = \frac{\text{magnitude of each head}}{L} = \frac{\text{seepage head}}{L}$$

$$\Rightarrow 0.536 = \frac{1.9}{h_1 + 2}$$

$$\therefore \boxed{h_1 = 1.54 \text{ m}}$$

checked

Another approach:

$$F.S = \frac{\text{Submerged wt. of soil}}{\text{Seepage force}}$$

$$\Rightarrow 2 = \frac{(2+x) \times (20.33 - 9.81)}{1.9 \times 9.81}$$

$$\therefore \boxed{x = 1.544 \text{ m}}$$

2008-09	8(c)

Given, $t = 15$ min $= 15 \times 60$ s

$Q = 500$ mL

$A = \frac{\pi}{4} \times 5^2$ cm^2.

Head difference. $\Delta H = 40$ cm.

length of soil sample. $L = 15$ cm.

For constant head permeameter,

$$k = \frac{QL}{hAt} = \frac{500\,\text{mL} \times 15\,\text{cm}}{40\,\text{cm} \times \frac{\pi}{4} \times 5^2\,\text{cm}^2 \times 15 \times 60}$$

$$= \frac{500\,\text{cc} \times 15}{40 \times \pi/4 \times 5^2 \times 15 \times 60} = \boxed{10.6 \times 10^{-3}\ \text{cm/s}}$$

Discharge velocity, $v = \frac{Q}{A} = \frac{500/15 \times 60}{\frac{\pi}{4} \times 5^2} = \boxed{28.3 \times 10^{-3}\ \text{cm/s}}$

$W_d = 486$ gm.

$\cancel{\gamma_d =}$ $\rho_d = \frac{W_d}{V} = \frac{486}{\frac{\pi}{4} \times 5^2 \times 15} = 1.65$

Now, $\rho_d = \frac{\rho_w G_s}{1+e} \Rightarrow 1.65 = \frac{1 \times 2.65}{1+e}$

$\therefore e = 0.61$

$n = \frac{e}{1+e} = 0.38$

Now, Seepage velocity, $v' = \frac{v}{n} = \frac{28.3 \times 10^{-3}}{0.38} = \boxed{74.5 \times 10^{-3}\ \text{cm/s}}$

Confirm

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[2007-08| 8(c)]

(i) Given No of flow channel $N_f = 4$

No. of drops = 12

Head difference, $h = 10$ m, $K = 10^{-4}$ cm/s $= 10^{-6}$ m/s

~~Sap~~ Seepage loss, $q = Kh \cdot \frac{N_f}{N_d}$

Seepage loss in a year, $q = 10^{-6} \times 10 \times \frac{4}{12} \times 365 \times 24 \times 60^2$

$= \boxed{105.12 \text{ m}^3 \text{ (per m)}}$

(ii) Pore water pressure at A, $\sigma_{w,A} = 2 \times 9.81 + \frac{10}{12} \times 5.5 \times 9.81$

$= \boxed{64.58 \text{ KN/m}^2}$

(iii) Effective stress at B, $\sigma_B' = 5 \times (21 - 9.81) - \frac{10}{12} \times 1.5 \times 9.81$

$= \boxed{43.687 \text{ KN/m}^2}$

Seepage pressure at B $= i z \gamma_w = \frac{10}{12} \times 1.5 \times 9.81 = \boxed{12.26 \text{ KN/m}^2}$

(iv) $F.S = \frac{i_{cr}}{i_{exit}}$ [by Hazra's method]

Now, $i_{cr} = \frac{\gamma'}{\gamma_w} = \frac{21 - 9.81}{9.81} = 1.14$

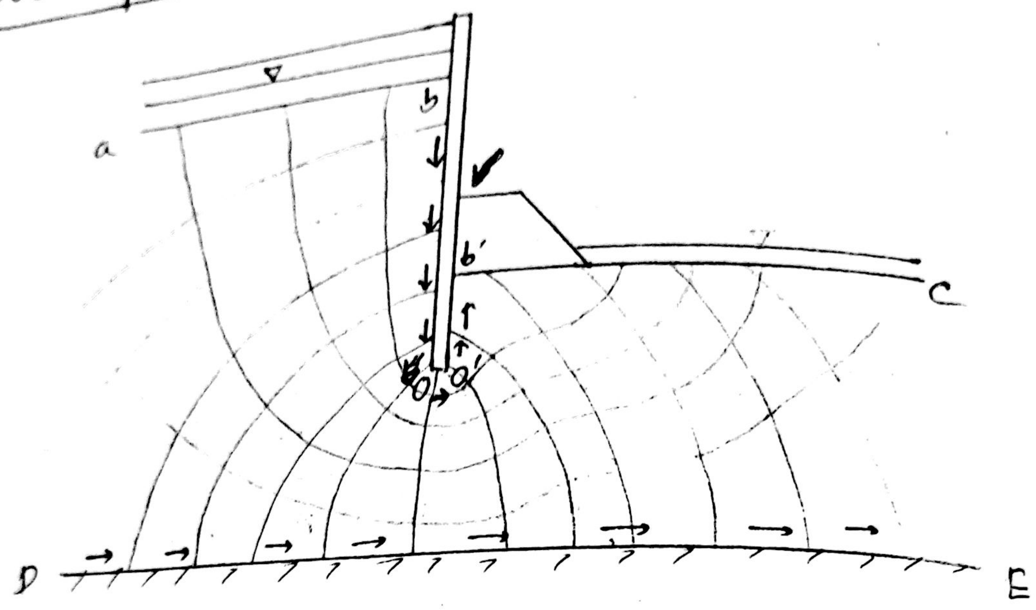
$i_{exit} = \frac{\text{driving head}}{L} = \frac{10/12}{6}$ [From figure, length $L = 6$ m (if it is drawn on true scale)]

Same m

$= 0.14$

$\therefore F.S = \frac{1.14}{0.14} = \boxed{8.14}$

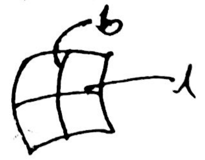
2006-07 | 8(c)



The flow net diagram was incorrect because:

1) Each flow element is not in curvilinear shape.

i.e. $l \neq b$



2). Each equipotential line is not perpendicular to flow line. For example; on the flow lines bo, b'o', the equipotential lines are not perpendicular.

3) Simillarly, flow lines are not orthogonal to equipotential line. For example, on the equipotential lines ab or bc flow lines are not orthogonal.

4) Flow channel doesn't intersect each other. But in the figure, the bottom flow line intersects with the flow line DE.

That's why, the flow net diagram is incorrect.

2004-05	2(e)

Given that: $k = 3 \times 10^{-7}$ m/s

$$n = 0.4 \quad \therefore e = \frac{n}{1-n} = \frac{0.4}{0.6} = 0.67$$

$$\gamma_{bulk} = 21 \text{ KN/m}^3 \rightarrow w = 31\%.$$

$$\therefore \gamma_d = \frac{\gamma_{bulk}}{1+w} = \frac{21}{1+0.31} = 16.03 \text{ KN/m}^3$$

Again. $\gamma_d = \frac{\gamma_w G_s}{1+e} \Rightarrow 16.03 = \frac{9.81 \times G_s}{1+0.67}$

$$\therefore G_s = 2.73$$

Now, $\gamma_{sat} = \frac{G_s + e}{1+e} \gamma_w = \frac{2.73+0.67}{1+0.67} \times 9.81$

$$= 19.97 \text{ KN/m}^3$$

$\therefore$ effective stress, $\gamma' = \gamma_{sat} - \gamma_w = 19.97 - 9.81$

$$= 10.16$$

Now, for quick sand condition,

$$\bar{P}_{a-b} = \gamma' z - i_c z \gamma_w = 0$$

$$\Rightarrow \gamma' z = i_c z \gamma_w$$

$$\therefore i_c = \frac{\gamma'}{\gamma_w} = \frac{10.16}{9.81} = 1.036$$

At quick sand condition, $i_{cr} = i_{crit} = 1.036$

or, $i_{cr} = \frac{G_s - 1}{1+e} = \frac{2.73-1}{1+0.67} = 1.036.$

Again, $i_{cr} = \frac{h}{L}$ ie, $i = \frac{h}{L}$

$$\Rightarrow 1.036 = \frac{h}{3}$$

head, $h = \boxed{3.12 \text{ m}}$

we know that, $K = \frac{QL}{hAt}$

$$\Rightarrow \frac{Q}{t} = \frac{KhA}{L}$$

$$\Rightarrow \frac{Q}{t} = \frac{3\times10^{-7} \times 3.12 \times 1}{3}$$

$$= \boxed{3.12 \times 10^{-7} \text{ m}^3/\text{s}}$$

Another approach:

$$Q = Av$$

$$\Rightarrow Q = A.\, k i_{cr}$$

$$\Rightarrow Q = 1 \times 3\times10^{-7} \times 1.036$$

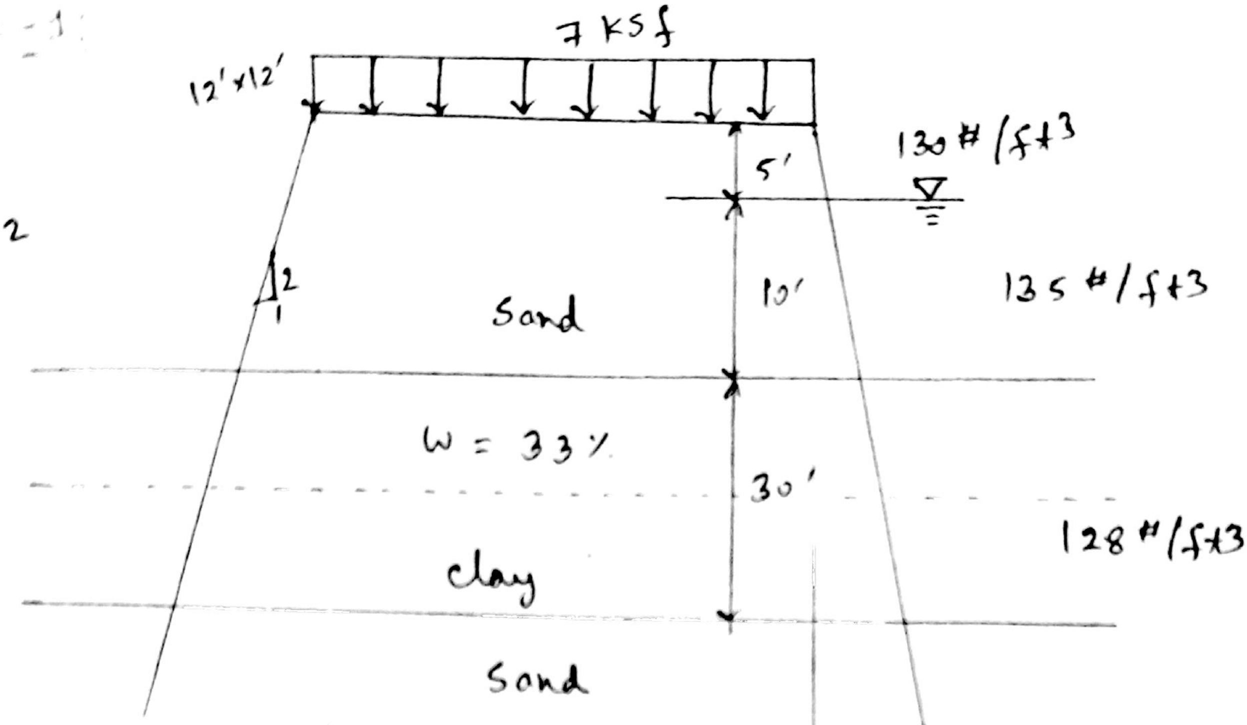
$$\therefore Q = \boxed{3.11 \times 10^{-7} \text{ m}^3/\text{s}}$$

Consolidation Characteristics of soil

Class Lectures:

Example - 1:

$C_c = 0.12$



Find out the settlement of clay layer due to 12'×12' footing foundation.

$e_0 = wG_s = 0.33 \times 2.7 = 0.891$

$P_0 = 5 \times 130 + 10 \times (135 - 62.4) + 15 \times (128 - 62.4)$

$= 2360$ psf

Load, $P = 7 \times 12 \times 12 = 1008$ kips.

∴ Area at mid ~~layer~~ point of clay layer $= (12 + 15 \times 2)^2$

$= 1764$.

$$\Delta e = \frac{1.78}{1.264} = 0.57[illegible]$$

max settlement, $S = \frac{C_c H}{1+e_0} \log \frac{\sigma_0 + \Delta\sigma}{\sigma_0}$

$$= \frac{0.12 \times 15}{1+0.89} \log \frac{2.36 + 4.57}{2.36}$$

$$= 0.089'$$

$$= \boxed{1.073''}$$

Problem-2 An 18 mm thick specimen drained at the top and bottom reaches 25% in 10 minutes. How long it will take the sample to reach 50% consolidation.

Ans

$$H_{dr} = \frac{18}{2} = 9 \text{ mm}$$

$U = 25\%$

$$\therefore T_v = \frac{\pi}{4}\left(\frac{U}{100}\right)^2 = \frac{\pi}{4} \times \left(\frac{25}{100}\right)^2 = 0.049$$

$$C_v = \frac{T_v H_{dr}^2}{t_{25}} = \frac{0.049 \times 9^2}{10} = 0.397 \text{ mm}^2/\text{min}$$

Again, for $U = 50\%$, $T_v = \frac{\pi}{4} \times \left(\frac{50}{100}\right)^2 = 0.196$

$$\therefore t_{50} = \frac{T_v H_{dr}^2}{C_v} = \frac{0.196 \times 9^2}{0.397} = \boxed{40 \text{ min}}$$

Another approach:

$$C_v = \frac{T_v H_{dr}^2}{t}$$

$$\therefore \frac{t_1}{t_2} = \frac{T_{v_1}}{T_{v_2}}$$

Again, $T_v = \frac{\pi}{4} \times \left(\frac{U}{100}\right)^2 \quad \therefore \frac{T_{v1}}{T_{v2}} = \frac{U_1^2}{U_2^2}$

$$\therefore \frac{t_{25}}{t_{50}} = \frac{T_{v25}}{T_{v50}} = \frac{U_{25}^2}{U_{50}^2}$$

$$\Rightarrow \frac{10}{t_{50}} = \frac{25^2}{50^2} \quad \therefore t_{50} = \boxed{40 \text{ min}}$$

Problem-3: A building constructed on a compressible clay layer with double drainage settled by 80 mm in 4 years. What will be the settlement in 9 years & if the final settlement is ~~30 mm~~ 300 mm. What time will be required to settle by 210 mm and what will be the settlement in 25 years?

Ans: ~~$H_{dr} =$~~ ~~80~~ $\frac{S_1}{S_2} = \frac{U_1}{U_2} = \sqrt{\frac{t_1}{t_2}}$

$$\Rightarrow \frac{S_1}{80} = \sqrt{\frac{9}{4}} \quad \therefore S_1 = 120 \text{ mm}.$$

$$U = \frac{S_c}{S_f} = \frac{120}{300} \times 100\% = 40\%$$

So, the initial assumption is correct.

So, settlement in 9 years will be $\boxed{120 \text{ mm}}$

For 210 mm settlement, $U = \frac{210}{300} \times 100\% = 70\%$

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

$= 1.781 - 0.933 \log(100 - 70)$

$= 0.403$

For $U = 40\%$, $T_v = \frac{\pi}{4} \times \left(\frac{40}{100}\right)^2$

$= 0.126$

~~Now, $C_v = \frac{T_v H_{dr}^2}{t}$~~

~~$\Rightarrow C_v = 0.126 \times$~~

Now, $\frac{T_{v_1}}{T_{v_2}} = \frac{t_1}{t_2}$

$\Rightarrow \frac{0.126}{0.403} = \frac{9}{t_2}$ $\therefore t_2 = \boxed{28.8 \text{ years}}$

Again, $\frac{t_1}{t_2} = \frac{T_{v_1}}{T_{v_2}}$

$\Rightarrow \frac{25}{9} = \frac{T_{v_1}}{0.126}$ $\therefore T_{v_1} = 0.35$

Again, $T_{v_1} = 1.781 - 0.933 \log(100 - U) = 0.35$

$\therefore U = 65.82 = \frac{S_c}{S_f} \times 100$

$\therefore S_c = \frac{65.82 \times 300}{100} = \boxed{197.46 \text{ mm}}$

Scanned by CamScanner

[4]: A saturated soil with $G_s = 2.65$, $w = 18\%$ such thickness 1.9 cm was tested on consolidation apparatus. The test shows the compression of 0.05 cm when the load is increased from 40 to 80 kN/m^2. Find C_c & m_v.

Ans: Settlement, $s = 0.05$ cm, $H = 1.9$ cm

$$S = m_v \, \Delta P \, H$$

$\therefore$ coefficient of volume compressibility, $m_v = \dfrac{S}{\Delta P H}$

$$\Rightarrow m_v = \frac{0.05}{40 \times 1.9} = \boxed{6.58 \times 10^{-4} \ m^2/kN}$$

$$e_0 = w G_s = 0.18 \times 2.65 = 0.477$$

Now, $S = \dfrac{C_c H}{1+e_0} \log \dfrac{P_0 + \Delta P}{P_0}$

$$\Rightarrow 0.05 = \frac{C_c \times 1.9}{1+0.477} \log \frac{40+40}{40}$$

$$\therefore \boxed{C_c = 0.129}$$

Another approach:

$$S = \frac{\Delta e}{1+e_0} H \Rightarrow 0.05 = \frac{\Delta e}{1+0.477} \times 1.9$$

$$\therefore \Delta e = 0.039$$

Now, $m_v = \dfrac{\Delta e}{(1+e_0)\Delta P} = \dfrac{0.039}{(1+0.477) \times 40} = \boxed{6.6 \times 10^{-4} \ m^2/kN}$

Scanned by CamScanner

Problem-5: A 3m thick layer beneath a build... is overlaying by a permeable stratum (layer) and underlying by a impervious layer. The coefficient of consolidation was found to be 0.025 cm^2/min. The final expected settlement of the layer is 8cm.

1) How much time it will take 80% of the final settlement to take this.

2) Determine the time required to settle 2.5 cm to occur.

3) Find the settlement to occur in one year.

Ans:

$H_{dr} = 3$ m. $c_v = 0.025$ cm^2/min,

Settlement, $S_f = 8$ cm

For 80% consolidation.

$$T_v = 1.781 - 0.933 \log(100-80)$$

$$= 0.567$$

1) $$C_v = \frac{T_v H_{dr}^2}{t}$$

$$\therefore t = \frac{0.567 \times 300^2}{0.025} = 2.04 \times 10^6 \text{ min} = \boxed{3.88 \text{ years}}$$

a) $S_c = 2.5$ cm

$\therefore U = \frac{2.5}{8} \times 100 = 31.25\% < 60\%$

$\therefore T_v = \frac{\pi}{4} \times \left(\frac{U}{100}\right)^2 = \cancel{0.0??}\ 0.077$

$\therefore t = \frac{T_v H_{dr}^2}{C_v} = \frac{0.077 \times 300^2}{0.025} = \cancel{\ldots}\ 2.76 \times 10^5$

$= \boxed{192 \text{ days}}$

b) $t = $ one year $= 365 \times 24 \times 60$

$\therefore C_v = \frac{T_v \times H_{dr}^2}{t}$

$\Rightarrow T_v = \frac{0.025 \times 365 \times 24 \times 60}{300^2} = 0.146$

Now, $T_v = \frac{\pi}{4} \times \left(\frac{U}{100}\right)^2 \Rightarrow U = 43.12\%$

Again, $U = \frac{S_c}{S_f} \times 100 \Rightarrow 43.12 = \frac{S_c}{8} \times 100 \Rightarrow$

$\therefore \boxed{S_c = 3.45 \text{ cm}}$

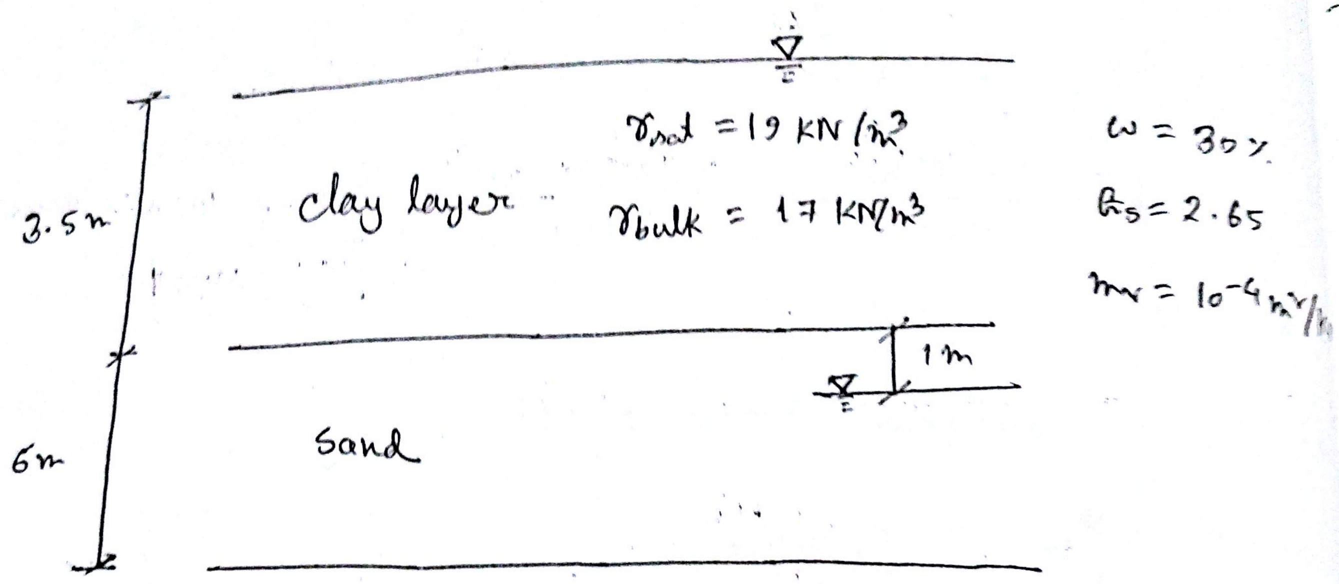
→ <u>Another approach:</u>

$\frac{S_1}{S_2} = \frac{U_1}{U_2} = \sqrt{\frac{T_{v1}}{T_{v2}}} = \sqrt{\frac{t_1}{t_2}}$

$\Rightarrow \frac{S_1}{2.5} = \sqrt{\frac{1 \times 365}{192}} \Rightarrow$

$\therefore \boxed{S_1 = \cancel{2}\ 3.45 \text{ cm}}$

Problem - 6: Due to Fluctuation in water table, find settlement magnitude in clay layer



$$P_1 = \frac{3.5}{2} \times (19 - 9.81) = 16.083\ KN/m^2$$

$$P_2 = \frac{3.5}{2} \times 17 = 29.75\ KN/m^2$$

$$\therefore \Delta P = P_2 - P_1 = 13.67\ KN/m^2$$

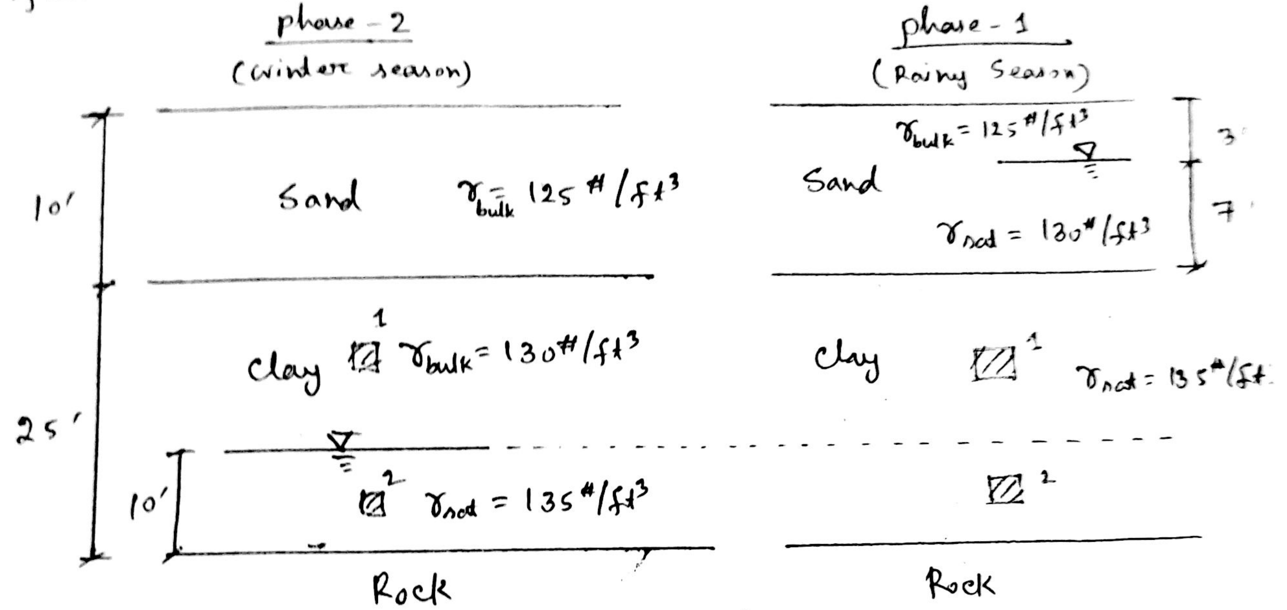
Now, $S = m_v\ \Delta p\ H$

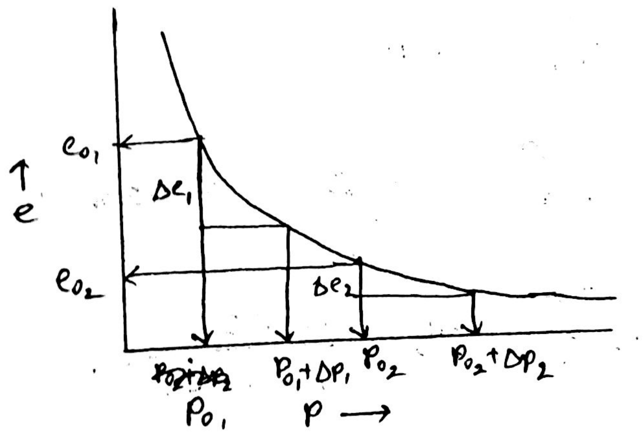
$$= 10^{-4} \times 13.67 \times 3.5$$

$$= 4.78 \times 10^{-3}\ m$$

$$= \boxed{0.478\ cm}$$

Problem 7: Find the settlement of clay layer due to fluctuation of water level in rainy and winter season.





Ans:

$$P_{o1} = 125\times3 + 7\times(130-62.4) + 7.5\times(135-62.4) = 1390 \text{ psf}$$

$$P_{o2} = 125\times3 + 7\times(130-62.4) + 20\times(135-62.4) = 2300 \text{ psf}$$

$$P_{o1} + \Delta P_1 = 125\times10 + 7.5\times130 = 2225 \text{ psf}$$

$$P_{o2} + \Delta P_2 = 125\times10 + 15\times130 + 5\times(135-62.4) = 3560 \text{ psf}$$

$H_1 = 15\text{ m}$, $H_2 = 10\text{ m}$

Now, $\Delta P_1 = 2225 - 1390 = 835$ psf

$\Delta P_2 = 1260$ psf.

From the e vs p graph, calculate, $e_{01}, e_{02}, \Delta e$, 2010

Sol^n: Let, $e_{01} = 0.8$, $e_{02} = 0.45$,

$\Delta e_1 = \cancel{0.25}\ 0.05$, $\Delta e_2 = \cancel{0.220}\ 0.02$

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

$\therefore S_{o1} = \frac{\Delta e_1}{1+e_{o1}} \times H_1$

$= \frac{0.05}{1+0.8} \times 15 = 0.42$ m.

$S_{o2} = \frac{\Delta e_2}{1+e_{o2}} \times H_2 = \frac{0.02}{1+0.45} \times 10 = 0.138$ m.

$\therefore$ Total settlement, $S = S_{o1} + S_{o2} = 0.42 + 0.138 = \boxed{0.56\text{m}}$

If m_{v1} & m_{v2} is given, then,

$S_{o1} = m_{v1}\, \Delta P_1\, H_1$

$S_{o2} = m_{v2}\, \Delta P_2\, H_2$.

Term Question Solve

Consolidation Charecteristics of soil

$\boxed{2010-11.\ 6(c)}$

$$H_{dr} = \frac{4}{2} = 2\,m$$

$U = 5\%$

$$T_v = \frac{\pi}{4} \times \left(\frac{5}{100}\right)^2 = 1.96 \times 10^{-3}$$

$K = 0.02\ m/yr$

$$C_v = \frac{T_v H_{dr}^2}{t} = \frac{1.96 \times 10^{-3} \times 2^2}{1} = 7.85 \times 10^{-3}$$

Again, $C_v = \dfrac{k}{m_v \gamma_w}$

$$\Rightarrow 7.85 \times 10^{-3} = \frac{0.02}{m_v \times 9.81 \times 1000}$$

$\therefore m_v = 2.6 \times 10^{-4}\ m^2/N$

Now, $S = m_v \Delta p H = 2.6 \times 10^{-4} \times 55 \times \cancel{2}\,4 \times 1000$

$= \boxed{28.55\ m} \times 2 = 57.4\ m$

Again, for $S =$ [illegible] mm, [illegible]

$m_v =$ [illegible] $\cdot 9 \times 10^{-3}$ [illegible]

[illegible] $= 5.23 \times 10^{-4}$

$t =$ [illegible] 15 years

2010-11. 8(c)

$e_0 = 0.8$, $C_c = 0.28$, $P_0 = 2650\ lb/ft^2$

$\Delta P = 970\ lb/ft^2$, $C_a = 0.02$, $H = 8.5\ ft$

$$\Delta e = \frac{C_c}{\cancel{1+e_0}} \log \frac{P_0 + \Delta P}{P_0}$$

$$= 0.28 \log \frac{2650 + 970}{2650} = 0.038$$

$\therefore e_p = e_0 - \Delta e = 0.762$

Now, $C_a' = \frac{C_a}{1+e_p} = \frac{0.02}{1+0.762} = 0.0114$

Secondary settlement, $S_s = C_a' H \log (t_2/t_1)$

$$= 0.0114 \times 8.5 \log \left(\frac{5}{1.5}\right)$$

$$= \cancel{0.025'} = 0.05' = 0.6''$$

primary consolidation, $S_p = \frac{\Delta e}{1+e_0} H = \frac{0.038}{1+0.8} \times 8.5 = 2.15''$

$\therefore$ Total settlement $= S_p + S_s = 0.6 + 2.15$

$$= \boxed{2.75''}$$

Confirm

009-10 | 7(

C_c

(i)

(ii)

(ii

2009-10 7(c)

$C_c = 0.27$, $e_0 = 2.04$, $P_0 = 125\ KN/m^2$

$K = 3.5 \times 10^{-10}\ m/s$, $P_0 + \Delta P = 187.5\ KN/m^2$

ans (i) $\Delta e = c_c \log \frac{P_0 + \Delta P}{P_0} = 0.27 \log \frac{187.5}{125} = 0.0475$

(ii) $S = \frac{\Delta e}{1+e_0} H = \frac{0.0475}{1+2.04} \times 5 = 0.078\ m = \boxed{7.82\ cm}$

(iii) T_v (for 50%) $= \frac{\pi}{4} \times \left(\frac{50}{100}\right)^2 = 0.196$

$m_v = \frac{\Delta e}{\Delta P(1+e_0)} = \frac{0.0475}{(187.5-125) \times (1+2.04)}$

$= 2.5 \times 10^{-4}\ m^2/KN$

$C_v = \frac{K}{\gamma_w m_v} = \frac{3.5 \times 10^{-10}}{2.5 \times 10^{-4} \times 9.81} = 1.43 \times 10^{-7}\ m^2/s$

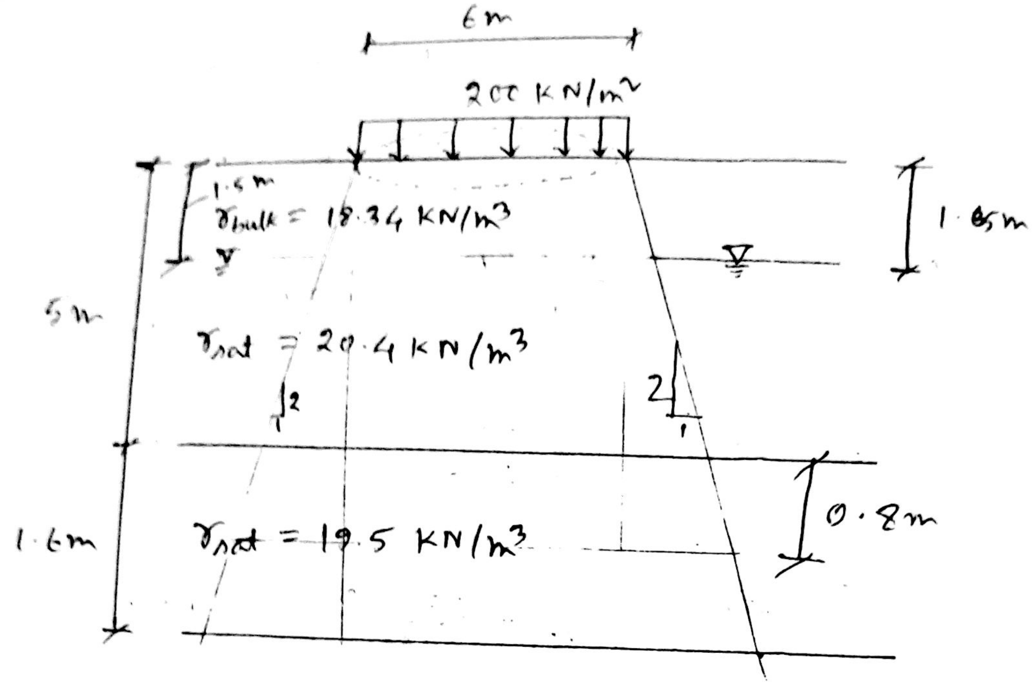
$t = \frac{T_v H_{dr}^2}{C_v} = \frac{0.196 \times 5^2}{1.43 \times 10^{-7}} = 34.34 \times 10^6$ sec

$= \boxed{1.1 \text{ years}}$

(i) & (ii) will be same for both way drainage

(iii) both way $\rightarrow \frac{1}{4}$ th,

2008-09. 6(b)



Load $= 200 \times \frac{\pi}{4} \times 6^2 = 5654.87$ KN

~~Area~~ Diameter $= \frac{5.8}{2} \times 2 + 6 = 11.8$ m

$\therefore$ Area $= \frac{\pi}{4} \times (11.8)^2 = 109.36$ m²

$\therefore$ $\Delta P = \frac{5654.87}{109.36} = 51.7$ KN/m²

$P_0 = 18.34 \times 1.65 + (20.4 - 9.81) \times 3.5 + 0.8 \times (19.5 - 9.81)$

$= 72.33$ KN/m².

From graph, for $P_0 = 72.33$ KN/m², $e_0 = 0.71$

for $P_0 + \Delta P = 72.33 + 51.7 = 124.03$ KN/m²,

$e_0 - \Delta e = 0.666$

$\therefore$ $\Delta e = 0.044$

Nos. So

For Ar

(ii) Fo

For

(iii)

Now, settlement, $s = \frac{\Delta e}{1+e_0} \cancel{H}\,H = \frac{0.044}{1+0.71} \times \cancel{}\,1.6 = \cancel{0.02\text{ m}}\ \boxed{0.041\text{ m}}$

~~For settlement of the~~ foundation, $H = \cancel{1.6}\ 0.8\text{ m}$

(ii) For 90% consolidation, $T_v = 1.781 - 0.933 \log(100-90) = 0.848$

$$c_v = \frac{T_v H_{dr}^2}{t} = \cancel{\frac{0.848 \times 1.6^2}{16t}} =$$

$$\Rightarrow t \propto H_{dr}^2$$

For consolidation test sample, $H_{dr(s)} = \frac{20}{2}\text{ mm} = 10\text{ mm}$

time for 90% consolidation, $t_s = 106$ min

$H_{dr(field)} = 1.6$ m

$$\text{Now, } \frac{t_s}{t_f} = \frac{H_{dr(s)}^2}{H_{dr(f)}^2}$$

$$\Rightarrow t_f = 106 \times \frac{1.6^2}{(10 \times 10^{-3})^2} = 2.71 \times 10^6 \text{ min} = \boxed{5.16 \text{ years}}$$

(iii) $$m_v = \frac{a_v}{1+e_0} = \frac{\Delta e}{\Delta p(1+e_0)} = \frac{0.044}{51.7 \times (1+0.71)} = 5 \times 10^{-4}\ \text{m}^2/\text{kN}$$

$$\text{Now, } c_v = \frac{T_v H_{dr}^2}{t} = \frac{0.848 \times 1.6^2}{\cancel{106}\ 5.16} = 0.42\ \text{m}^2/\text{yr}$$

$$\text{Again } c_v = \frac{k}{m_v \gamma_w} \qquad \therefore k = c_v m_v \gamma_w = 5 \times 10^{-4} \times 0.42 \times 9.81$$

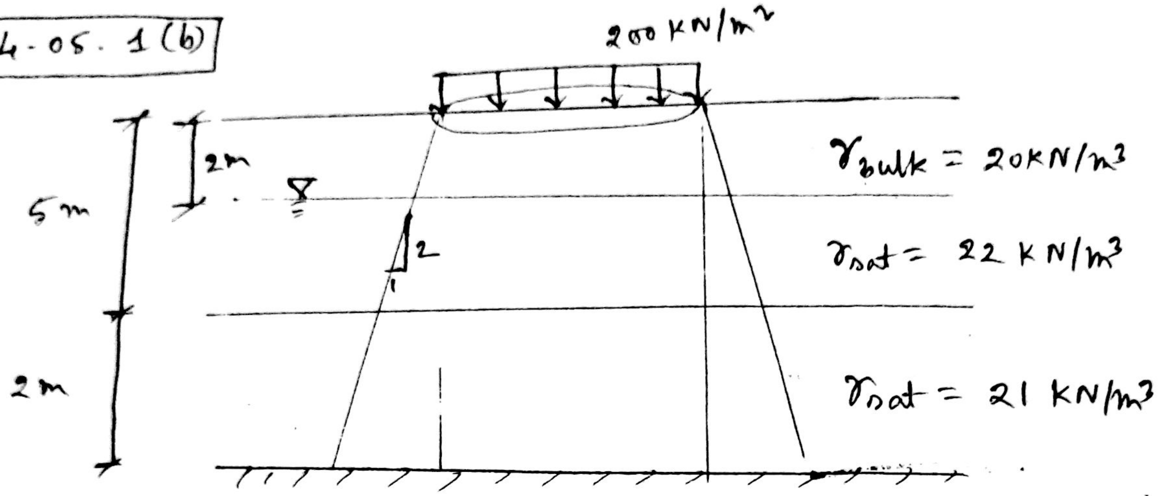
$$= \boxed{2.06 \times 10^{-3}\ \text{m/yr}}$$

Settlement of [illegible]

$\boxed{2007\text{-}08 \cdot 6(c)}$ → Same as 2009-10

$\boxed{2006\text{-}07 \cdot 6(c)}$ → Same as 2008-09 · 6(b)

$\boxed{2004\text{-}05 \cdot 1(b)}$



$P_o = 20 \times 2 + (22 - 9.81) \times 3 + (21 - 9.81) \times 1$

$= 87.76 \ kN/m^2$

$Load = 200 \times \frac{\pi}{4} \times 6^2 = 5654.87 \ kN$

$Area = \left(\frac{6}{2} \times 2 + 6\right)^2 \times \frac{\pi}{4} = 113.1 \ m^2$

$\therefore \ \Delta P = \frac{5654.87}{113.1} = 50 \ kN/m^2$

$C_c = 0.12, \quad e_o = 0.3 \times 2.7 = 0.81$

Now, settlement, $S = \frac{C_c H}{1 + e_o} \log \frac{P_o + \Delta P}{P_o}$

$= \frac{0.12 \times 1}{1 + 0.81} \times \log \frac{87.76 + 50}{87.76}$

$= 0.013 \ m = \boxed{1.3 \ cm}$

Scanned by CamScanner

$\boxed{2(11)\,4 \cdot 05 \cdot 3(c)}$

$C_c = 0.28$, $K = 3.5 \times 10^{-6}$ m/sec

$P_0 = 150$ KN/m^2, $e_0 = 1.95$ KN/m^2.

(i) $\Delta e = C_c \log \frac{P_0 + \Delta P}{P_0}$

$= 0.28 \log \frac{210}{150} = \boxed{0.041}$

(ii) T_v (for 50%) $= \frac{\pi}{4} \times \left(\frac{50}{100}\right)^2 = 0.196$

T_v (for 90%) $= 0.848$

$m_v = \frac{\Delta e}{\Delta P(1+e_0)} = \frac{0.041}{(210-150) \times (1+1.95)} = 2.32 \times 10^{-4}$ m^2/KN

$C_v = \frac{K}{m_v \gamma_w} = \frac{3.5 \times 10^{-6}}{2.32 \times 10^{-4} \times 9.81} = 1.54 \times 10^{-3}$

$t_{50} = \frac{T_v H_{dr}^2}{C_v} = \frac{0.196 \times 6^2}{1.54 \times 10^{-3}} = 4581.1$ sec $= \boxed{1.27 \text{ hr}}$

$t_{90} = \frac{0.848 \times 6^2}{1.54 \times 10^{-3}} = 19820$ sec $= \boxed{5.51 \text{ hrs}}$

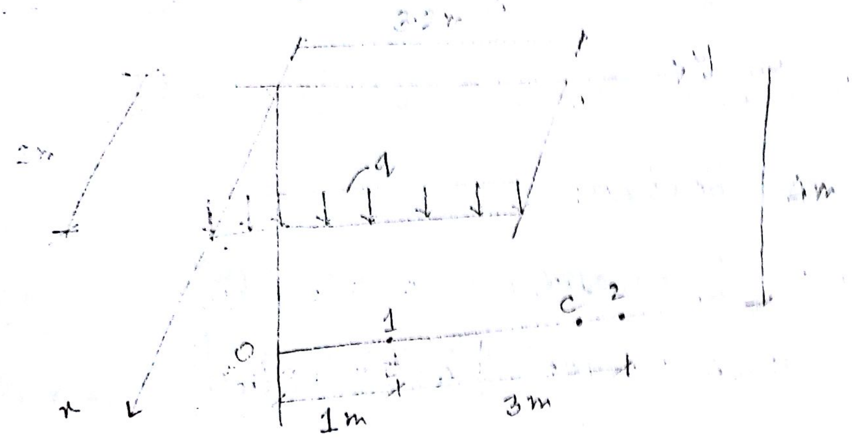
Stress Distribution of Soil

class Lectures:

Example - 8.4: (From sheet)

Given, $q = 100\ kN/m^2$,

Find σ_z at points 0, 1 and 2 shown in the figure.



Ans: a) at point 0:

$$m = \frac{x}{z} = \frac{2}{4} = 0.5, \quad n = \frac{y}{z} = \frac{3.2}{4} = 0.8$$

Now, $$P_0 = \frac{1}{2\pi(m^2+1)^2}\left[\frac{3mn}{\sqrt{m^2+n^2+1}} - \left(\frac{n}{\sqrt{m^2+n^2+1}}\right)^3\right]$$

$$= \frac{1}{2\pi(0.5^2+1)^2}\left[\frac{3\times0.8}{\sqrt{0.5^2+0.8^2+1}} - \left(\frac{0.8}{\sqrt{0.5^2+0.8^2+1}}\right)^3\right]$$

$$= 0.15775.$$

So, $$\sigma_{z,0} = \frac{q}{z} P_0 = \frac{100}{4} \times 0.15775 = \boxed{3.944\ kN/m^2}$$

b) at point 1

$\sigma_{z,1} = \sigma_{z,L}$ (due to load on left) + $\sigma_{z,R}$ (due to load

Now, for $\sigma_{z,L}$, $m = \frac{x}{z} = 0.5$, $n = \frac{1}{4} = 0.25$

$\therefore P_0 = 0.0656$

$\therefore \sigma_{z,L} = \frac{q}{z} P_0 = \frac{100}{4} \times 0.0656 = 1.64 \ KN/m^2$

Again, for $\sigma_{z,R}$, $m = 0.5$, $y = \frac{2.2}{4} = 0.55$

$\therefore P_0 = 0.1261$

$\therefore \sigma_{z,R} = \frac{100}{4} \times 0.1261 = 3.153 \ KN/m^2$.

So, $\sigma_{z,2} = 1.64 + 3.153 = \boxed{4.79 \ KN/m^2}$

c) at point 2:

$\sigma_{z,2} = \sigma_{z,0-2} - \sigma_{z,c-2}$

Now, for $\sigma_{z,0-2}$, $m = 0.5$, $n = \frac{4}{4} = 1$, $P_0 = 0.17354$

$\therefore \sigma_{z,0-2} = \frac{100}{4} \times 0.17354 = 4.338 \ KN/m^2$

Again, for $\sigma_{z,c-2}$, $m = 0.5$, $n = \frac{0.8}{4} = 0.2$, $P_0 = 0.0534$

$\therefore \sigma_{z,c-2} = \frac{100}{4} \times 0.0534 = 1.331 \ KN/m^2$.

$\therefore \sigma_{z,2} = 4.338 - 1.331$

$= \boxed{3.0067 \ KN/m^2}$

Problem

line

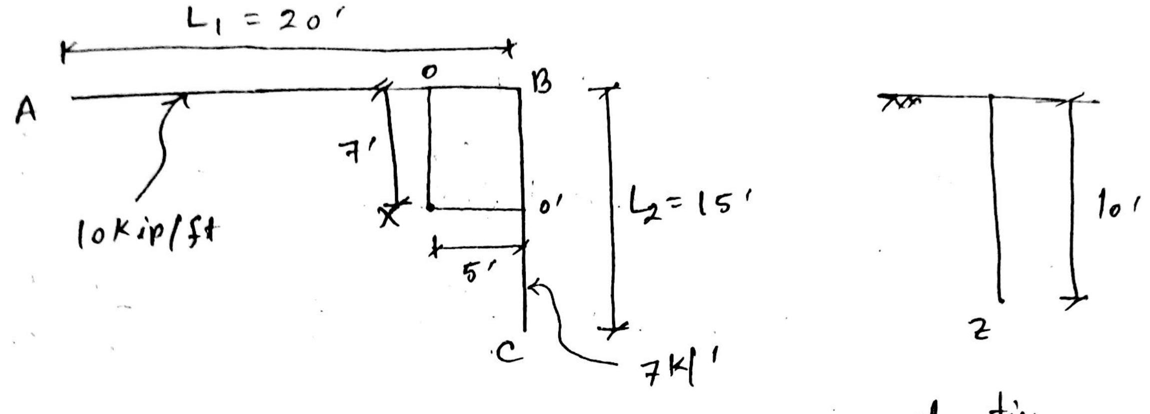
por

Ans:

n right,

Problem-2: Find σ_z at point X in the figure. The line load for AB portion is 10 kip/ft and BC portion is 7 kip/ft. $z = 10'$

Ans:



plan view

elevation

For AB:

$$\sigma_{z(A-B)} = \sigma_{z,(A-O)} + \sigma_{z(B-O)}$$

A to O, $L = 15'$, $x = 7'$, $z = 10'$

$$\therefore m = \frac{x}{z} = 0.7, \quad n = \frac{15}{10} = 1.5$$

$$\therefore P_0 = 0.1333$$

$$\therefore \sigma_{z(A-O)} = \frac{10}{10} \times 0.1333 = 0.1333 \text{ kip/ft}^2$$

O to B: $L = 5'$, $x = 7'$, $z = 10'$

$$\therefore m = 0.7, \quad n = 0.5$$

$$\therefore P_0 = 0.0776$$

$$\therefore \sigma_{z(O-B)} = \frac{q_0}{z} \times P_0 = \frac{10}{10} \times 0.0776 = 0.0776 \text{ kip/ft}^2$$

$$\therefore \sigma_{z(A-B)} = 0.1333 + 0.0776 = 0.2109 \text{ kip/ft}^2$$

For BC

$$\sigma_{z(B-C)} = \sigma_{z(B-O')} + \sigma_{z(O'-C)}$$

B to O': $x = 5'$, $L = 7'$, $z = 10'$

$$\therefore m = 0.5, \quad n = 0.7$$

$$\text{Now, } p_o = \frac{1}{2\pi(m^2+1)^2}\left[\frac{3n}{\sqrt{m^2+n^2+1}} - \left(\frac{n}{\sqrt{m^2+n^2+1}}\right)^3\right]$$

$$= \frac{1}{2\pi(0.5^2+1)^2}\left[\frac{3\times0.7}{\sqrt{0.5^2+0.7^2+1}} - \left(\frac{0.7}{\sqrt{0.5^2+0.7^2+1}}\right)^3\right]$$

$$= 0.14694$$

$$\therefore \sigma_{z(B-O)} = \frac{q_o}{z}\, p_o = \frac{7}{10} \times 0.1494 = 0.10286 \text{ k/ft}^2$$

O' to C: $L = 8'$, $x = 5'$, $z = 10'$

$$m = 0.5, \quad n = 0.8$$

$$\text{Now, } p_o = \frac{1}{2\pi(0.5^2+1)^2}\left[\frac{3\times0.8}{\sqrt{0.5^2+0.8^2+1}} - \left(\frac{0.8}{\sqrt{0.5^2+0.8^2+1}}\right)^3\right]$$

$$= 0.15775$$

$$\therefore \sigma_{z,(O'-C)} = \frac{7}{10} \times 0.15775 = 0.11042$$

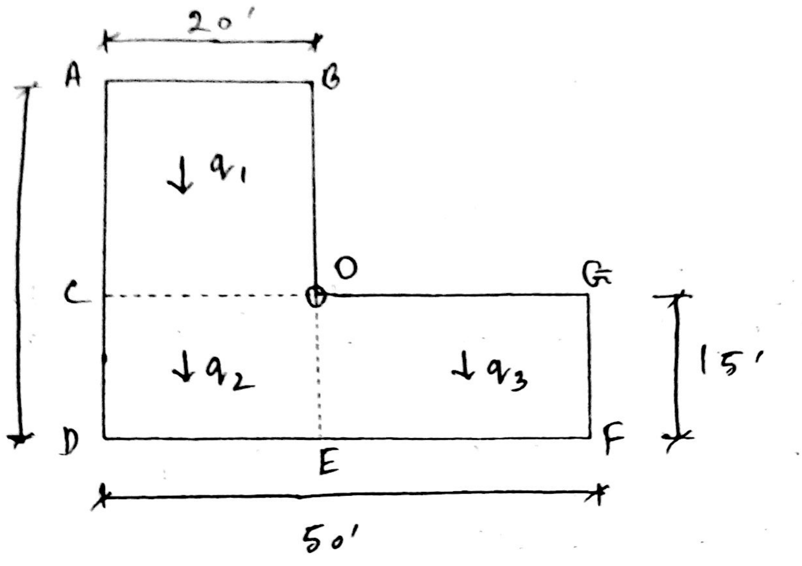
So, $\sigma_{z(B-C)} = 0.10286 + 0.11042 = 0.21328$ k/ft^2

$$\text{So, } \sigma_z = \sigma_{z(A1-B)} + \sigma_{z(B-C)}$$

$$= 0.2109 + 0.21328$$

$$= \boxed{0.424 \text{ Kip/ft}^2}$$

III Problem 3: Find σ_z at point O in the following diagram where $q_1 = 20\ k/ft^2$, $q_2 = 15\ kip/ft^2$, $q_3 = 10\ k/ft^2$, $Z = 10'$



For ABCO:

$$m = \frac{L}{Z} = \frac{20}{10} = 2, \quad n = \frac{B}{Z} = \frac{25}{10} = 2.5$$

Now, $m^2 + n^2 + 1 = 2^2 + 2.5^2 + 1 = 11.25$

$m^2 n^2 = 2^2 \times 2.5^2 = 25.$

So, $m^2 + n^2 + 1 < m^2 n^2$

$$\therefore \sigma_{z_1} = \frac{q_1}{4\pi}\left[\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+m^2n^2+1} \cdot \frac{m^2+n^2+2}{m^2+n^2+1} + \pi - \sin^{-1}\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+m^2n^2+1}\right]$$

$$= \frac{20}{4\pi} \times \left[\frac{2\times 2\times 2.5\sqrt{2^2+2.5^2+1}}{2^2+2.5^2+2^2\times 2.5^2+1} \times \frac{2^2+2.5^2+2}{2^2+2.5^2+1} + \pi - \sin^{-1}\frac{2\times 2\times 2.5\times\sqrt{2^2+2.5^2+1}}{2^2+2.5^2+2^2\times 2.5^2+1}\right]$$

$$= \frac{20}{4\pi} \times \left[1.0075 + \pi - 67.71 \times \frac{\pi}{180}\right]$$

$$= 4.7227\ kip/ft^2.$$

r CDEO:

$$m = \frac{L}{z} = \frac{20}{10} = 2, \quad n = \frac{15}{10} = 1.5, \quad z = 10'$$

I Prob

$$m^2+n^2+1 = 2^2+1.5^2+1 = 7.25$$

$$m^2n^2 = 2^2 \times 2.5^2 = 10$$

So, $m^2 + n^2+1 < m^2n^2$

$$\therefore \sigma_{z_2} = \frac{q_2}{4\pi}\left[\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+m^2n^2+1}\,\frac{m^2+n^2+2}{m^2+n^2+1} + \pi - \sin^{-1}\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+m^2n^2+1}\right]$$

$$= \frac{15}{4\pi}\left[1.1313 + \pi - 83.818 \times \frac{\pi}{180}\right]$$

$$= 3.3542 \text{ kip/ft}^2.$$

For EFEO:

$$m = \frac{30}{10} = 3, \quad n = \frac{15}{10} = 1.5$$

$$m^2+n^2+1 = 3^2+1.5^2+1 = 12.25$$

$$m^2n^2 = 20.25.$$

So, $m^2+n^2+1 < m^2n^2$.

$$\therefore \sigma_{z_3} = \frac{q_3}{4\pi}\left[\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+m^2n^2+1}\,\frac{m^2+n^2+2}{m^2+n^2+1} + \pi - \sin^{-1}\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+m^2n^2+1}\right]$$

$$= \frac{10}{4\pi} \times \left[1.04835 + \pi - 75.75 \times \frac{\pi}{180}\right]$$

$$= 2.2822 \text{ kip/ft}^2.$$

$$\therefore \sigma_z = \sigma_{z_1} + \sigma_{z_2} + \sigma_{z_3} = 4.7227 + 3.3542 + 2.2822$$

$$= \boxed{10.359 \text{ kip/ft}^2}$$

Problem - 4: solve the previous problem using Newmark's chart

$\frac{\sigma_z}{q}$	a/z	$a = z \times$
0	0	0
0.1	0.27	2.7
0.2	0.40	4.0
0.3	0.52	5.2
0.4	0.64	6.4
0.5	0.77	7.7
0.6	0.92	9.2
0.7	1.11	11.1
0.8	1.39	13.9
0.9	1.91	19.1
100	∞	∞

Scale : 1 cm = 1.91 ft

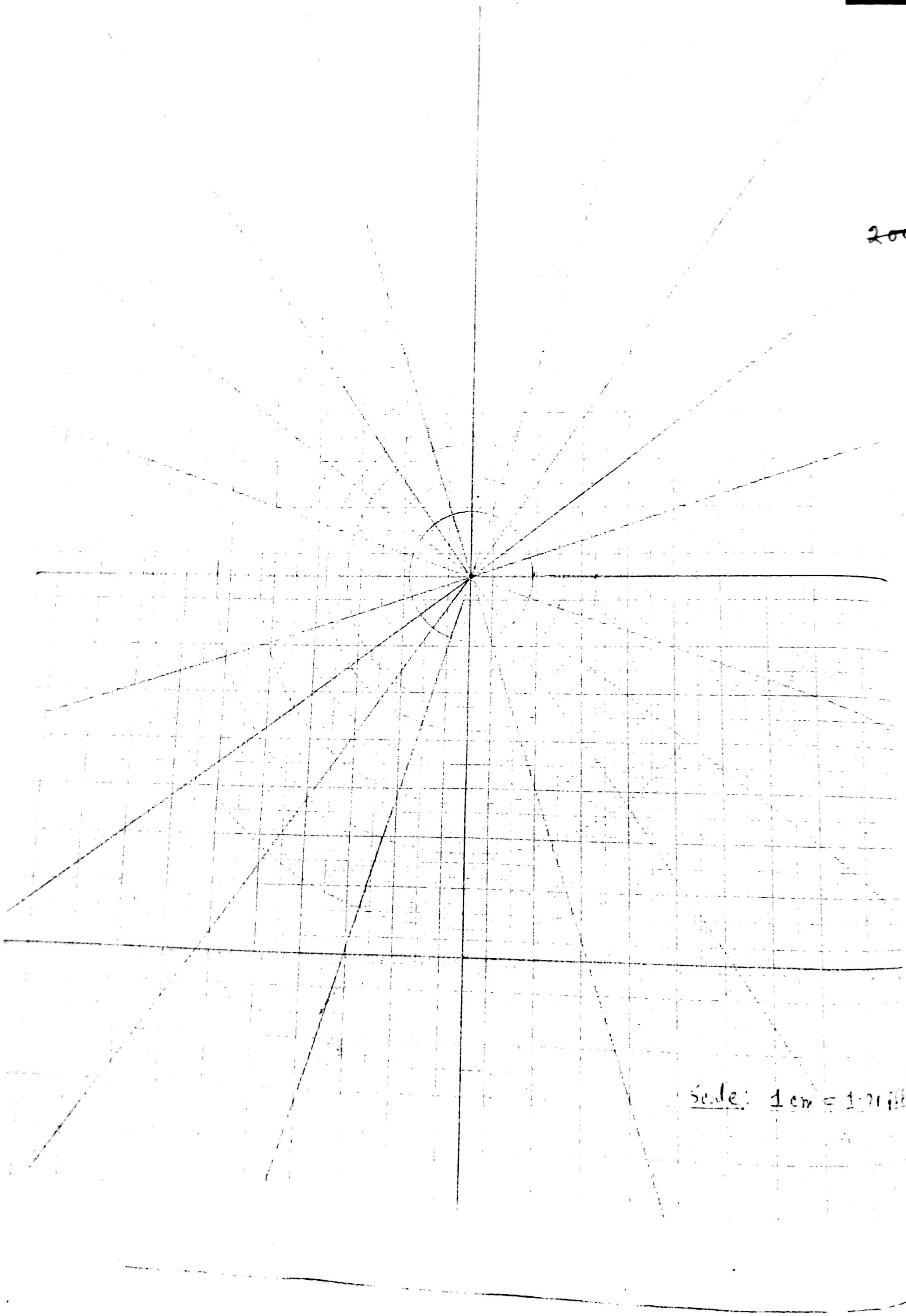
Now, $\sigma_z = \frac{a_1}{200} \times$ No. of division $+ \frac{a_2}{200} \times$ No. of division

$+ \frac{a_3}{200} \times$ No. of division.

$$= \frac{20}{200} \times 50 + \frac{15}{200} \times 45 + \frac{10}{200} \times 45$$

$$= \boxed{10.625 \text{ Kip/ft}^2}$$

2001

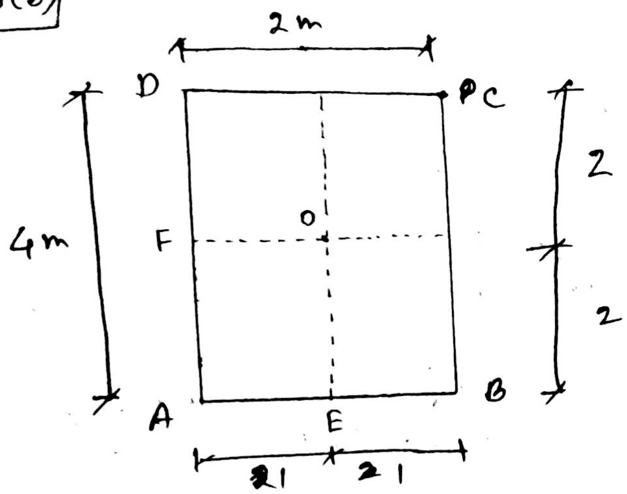


Scale: 1 cm = 1·71

Term Question solve

Stress Distribution of Soil

~~2001~~ 2010-11 | 8(b)



Given that, $q = 8 \text{ ton/m}^2$, $z = 5\text{ m}$,

At the centre:

for area AEOF, $m = \frac{L}{z} = \frac{1}{5} = 0.2$, $n = \frac{B}{z} = \frac{2}{5} = 0.4$

$m^2 + n^2 + 1 = 0.2^2 + 0.4^2 + 1 = 1.2$

$m^2 n^2 = 0.2^2 \times 0.4^2 = 6.4 \times 10^{-3}$

So, $m^2 + n^2 + 1 > m^2 n^2$.

$$\therefore \sigma_{z_1} = \frac{q}{4\pi}\left[\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+m^2n^2+1}\,\frac{m^2+n^2+2}{m^2+n^2+1} + \sin^{-1}\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+m^2n^2+1}\right]$$

$$= \frac{8}{4\pi}\left[\frac{2\times0.2\times0.4\sqrt{0.2^2+0.4^2+1}}{0.2^2+0.4^2+0.2^2\times0.4^2+1} \times \frac{0.2^2+0.4^2+2}{0.2^2+0.4^2+1} + \sin^{-1}\frac{2\times0.2\times0.4\times\sqrt{0.2^2+0.4^2+1}}{0.2^2+0.4^2+0.2^2\times0.4^2+1}\right]$$

$$= \frac{8}{4\pi}\left[0.26635 + \overset{8.35375}{\cancel{75.75}} \times \frac{\pi}{180}\right]$$

$$= 0.2624 \text{ ton/m}^2$$

So, for whole area ABCD,

$$\cancel{q_z =}\ \sigma_z = 0.2624 \times 4 = \boxed{1.0495\ \text{Ton}/\text{m}^2.}$$

509-10

<u>At the corner:</u>

for area ABCD, $m = \frac{2}{5} = 0.4$, $n = \frac{4}{5} = 0.8$, $z = 5$

So, $m^2 + n^2 + 1 > m^2 n^2$

$$\text{So, } \sigma_z = \frac{q_0}{4\pi}\left[\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+m^2n^2+1} \cdot \frac{m^2+n^2+2}{m^2+n^2+1} + \sin^{-1}\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+m^2n^2+1}\right]$$

Giv

$$= \frac{8}{4\pi}\left[\frac{2\times0.4\times0.8\times\sqrt{0.4^2+0.8^2+1}}{0.4^2+0.8^2+0.4^2\times0.8^2+1} \times \frac{0.4^2+0.8^2+2}{0.4^2+0.8^2+1} + \sin^{-1}\frac{2\times0.4\times0.8\times\sqrt{0.4^2+}}{0.4^2+0.8^2+0.4^2\times0.8}\right]$$

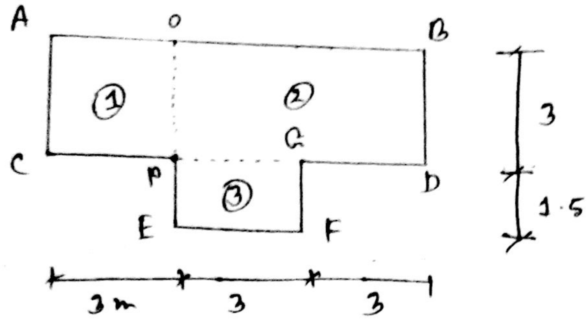
Ans

$$= \frac{8}{4\pi}\left[0.7021 + 26.8304 \times \frac{\pi}{180}\right]$$

$$= \boxed{0.745\ \text{ton}/\text{m}^2}$$

checked with Nafis.

2009-10 | 8(c)



Given, $q = 120\ kN/m^2$, $z = 5\ m$.

Ans Analytical solution:

For ACPO:

$$m = \frac{3}{5} = 0.6,\quad n = \frac{3}{5} = 0.6,\quad z = 5m$$

$m^2+n^2+1 > m^2n^2$

$$\therefore \sigma_{z_1} = \frac{q}{4\pi}\left[\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+m^2n^2+1} \times \frac{m^2+n^2+2}{m^2+n^2+1} + \sin^{-1}\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+m^2n^2+1}\right]$$

$$= \frac{120}{4\pi}\left[0.80735 + 30.7 \times \frac{\pi}{180}\right]$$

$$= 12.826\ kPa.$$

For OPDB:

$$m = \frac{6}{5} = 1.2,\quad n = \frac{3}{5} = 0.6$$

$m^2+n^2+1 = 2.8,\quad m^2n^2 = 0.5$

$\therefore m^2+n^2+1 < m^2n^2$

$$\therefore \sigma_{z_2} = \frac{120}{4\pi}\left[0.98546 + 46.563 \times \frac{\pi}{180}\right]$$

$$= 17.171\ kPa.$$

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For EFGP:

$$m = \frac{3}{5} = 0.6, \quad n = \frac{1.5}{5} = 0.3, \quad z = 5\,m$$

$m^2 + n^2 + 1 < m^2 n^2$.

$$\therefore \sigma_{z_3} = \frac{120}{4\pi} \times \left[\overset{0.494}{\cancel{0.726}} + \overset{17.0035}{\cancel{26.6047}} \times \frac{\pi}{180} \right]$$

$$= \cancel{11.367\ \text{kPa}}\ 7.55\ \text{kPa}.$$

$$\therefore \sigma_z = \sigma_{z_1} + \sigma_{z_2} + \sigma_{z_3} = 12.826 + 17.171 + 7.55 = \boxed{37.55\ \text{kPa}}$$

checked with

Newmark's chart:

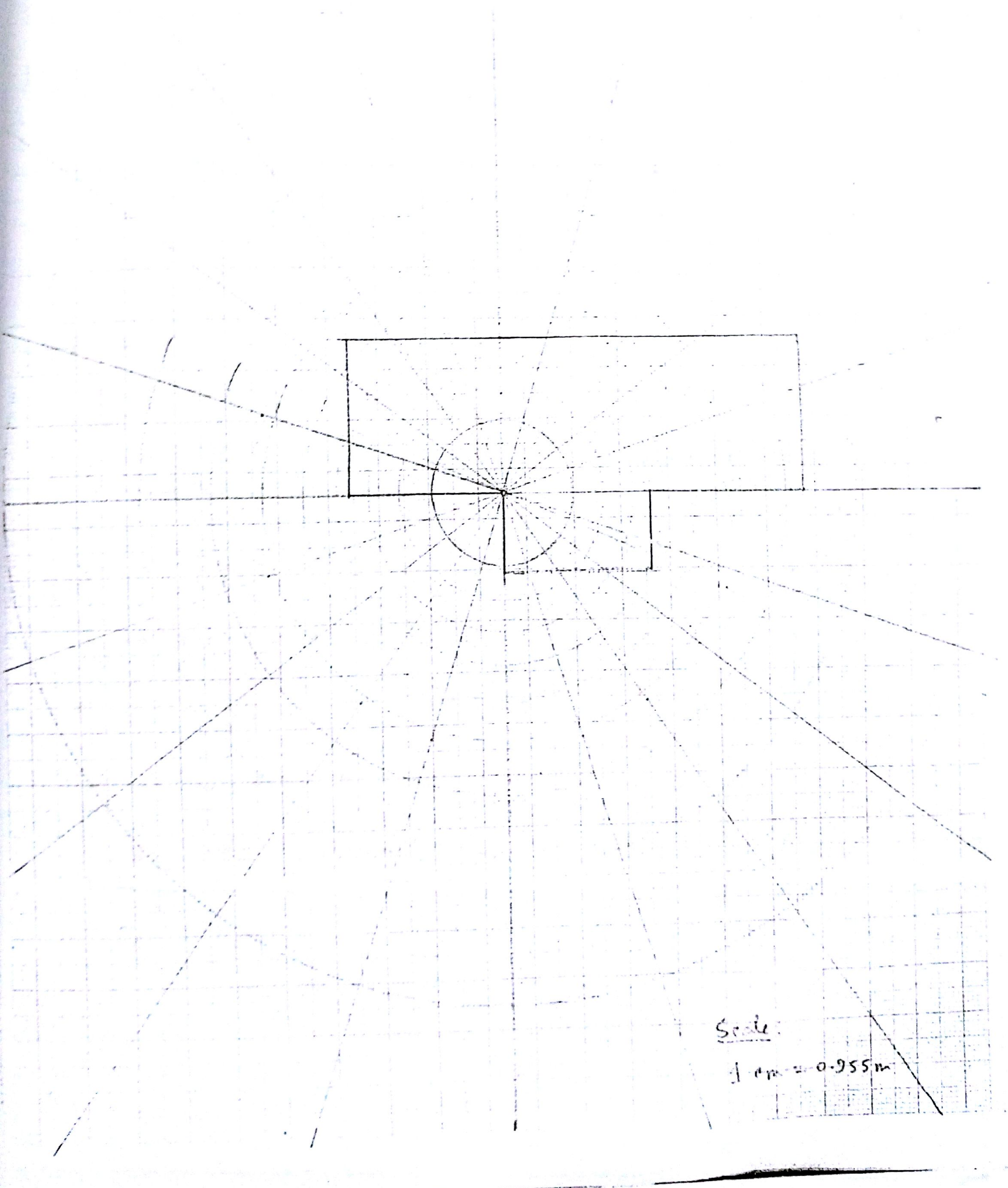
$$\frac{a}{z} = \left[\left(1 - \frac{\sigma_z}{q}\right)^{-2/3} - 1 \right]^{1/2}, \quad z = 5\,m.$$

$\frac{\sigma_z}{q}$	a/z	a (m)
0	0 ~~27~~	0
0.1	0.27	1.35
0.2	0.40	2
0.3	0.52	2.6
0.4	0.64	3.2
0.5	0.77	3.85
0.6	0.92	4.6
0.7	1.11	5.55
0.8	1.39	6.95
0.9	1.91	9.55
1	∞	∞

Scale: 1 cm = 0.955 m

No. of influence units = 62, Influence factor = $\frac{1}{200}$

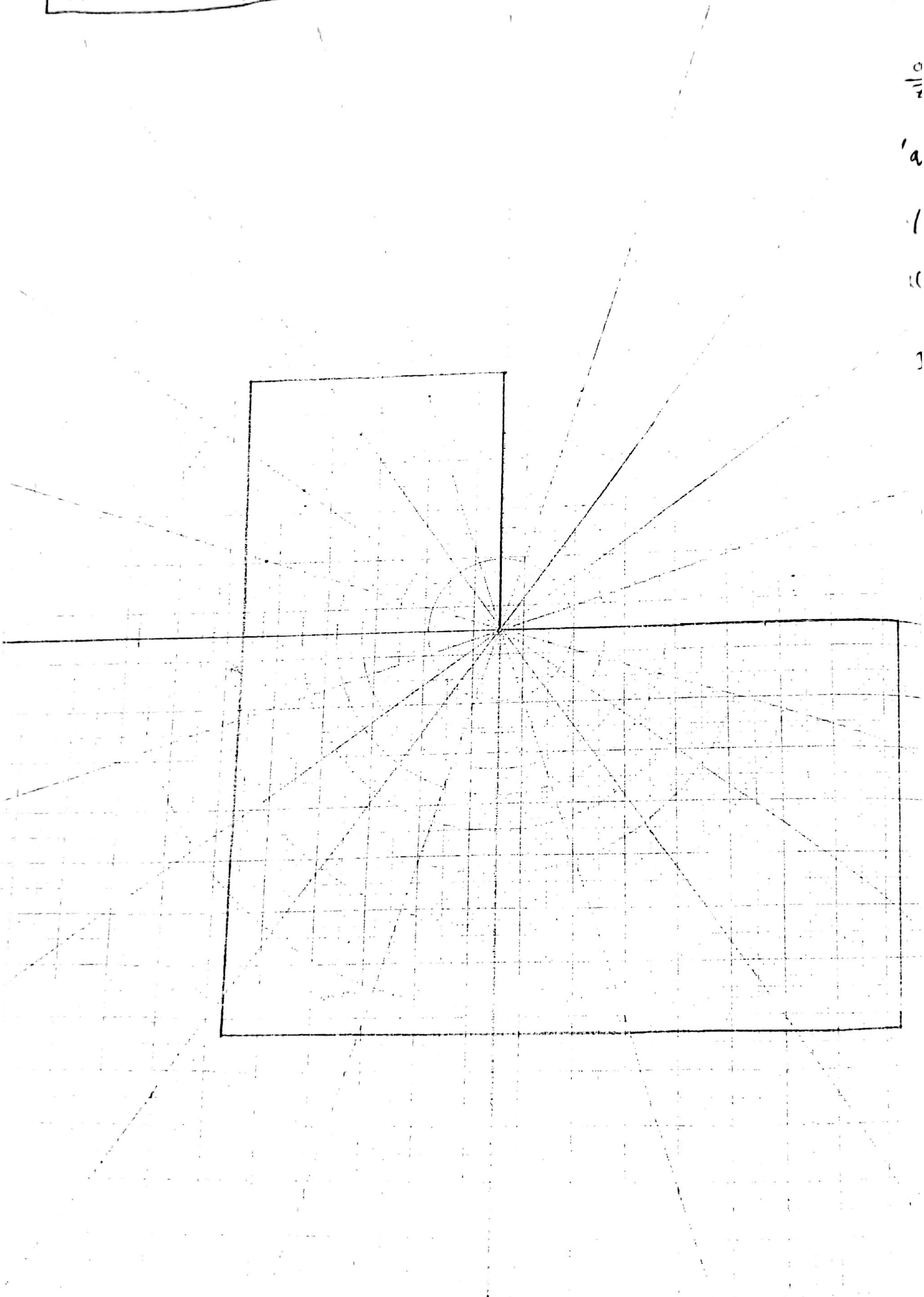
$$\therefore \sigma_z = \frac{62}{200} \times 120 = 37.2\ \text{kPa}.$$



Scale:

1 cm = 0.955 m

2008-09 · 7(b)



$$\frac{a}{z} = \left[\left(1-\frac{\sigma_z}{q}\right)^{-2/3}-1\right]^{1/2}, \quad q = 75\ kN/m^2, \quad z = 10\ m.$$

σ_z/q	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	∞
a/z	0	0.27	0.40	0.52	0.64	0.77	0.92	1.11	1.39	1.91	∞
$a(m)$	0	2.7	4	5.2	6.4	7.7	9.2	11.1	13.9	19.1	∞

Influence factor $= \frac{1}{200}$, No. of influence units $= 115$

$$\therefore \sigma_z = \frac{115}{200} \times 75 = \boxed{43.125}$$

checked with Nafis

Check:

$A_1 \rightarrow m = \frac{10}{10} = 1, \; n = \frac{10}{10} = 1, \quad m^2+n^2+1 > m^2n^2$

$$\sigma_{z_1} = \frac{q}{4\pi}\left[\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+m^2n^2+1} \times \frac{m^2+n^2+2}{m^2+n^2+1} + \sin^{-1}\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+m^2n^2+1}\right]$$

$$= \frac{75}{4\pi}\left[1.1547 + 60 \times \frac{\pi}{180}\right] = 13.142$$

$A_2 \rightarrow m = 1, \; n = 1.5, \quad m^2+n^2+1 > m^2n^2$

$$\sigma_{z_2} = \frac{75}{4\pi}\left[1.175365 + \frac{\pi}{180} \times 72.08\right] = 14.523$$

$A_3 \rightarrow m = 1.5, \; n = 1.5,$

$$\sigma_{z_3} = \frac{75}{4\pi}\left[1.1808 + \frac{\pi}{180} \times 87.626\right] = 16.175$$

$$\sigma_z = \boxed{43.84\ Kpa}$$

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2006-07	7(c)

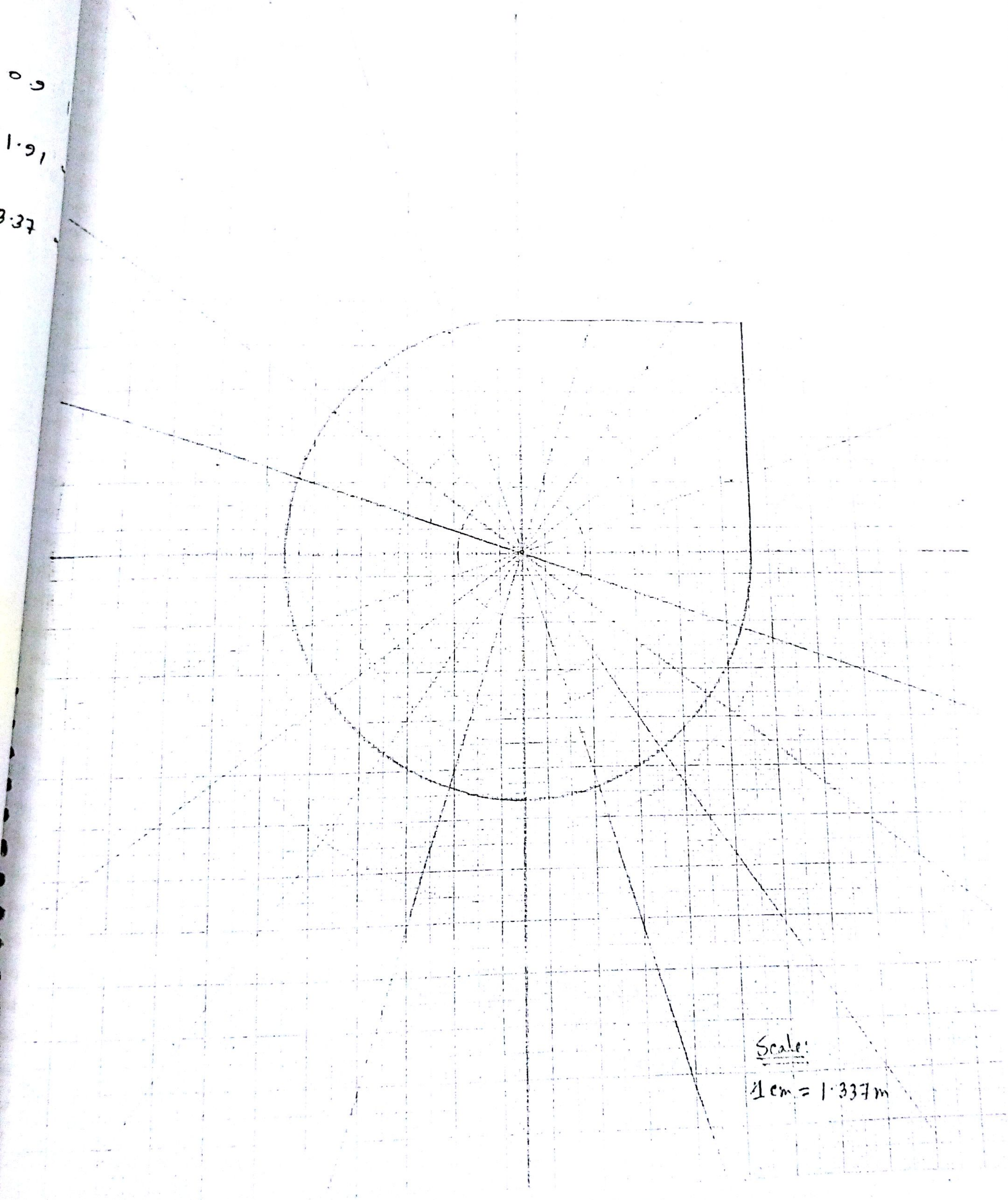
$$\frac{a}{z} = \left[\left(1 - \frac{\sigma_z}{q}\right)^{-2/3} - 1\right]^{1/2} \quad , z = 7m$$

$\frac{\sigma_z}{q}$	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
a/z	0	0.27	0.4	0.52	0.64	0.77	0.92	1.11	1.39	1.9
a (m)	0	1.89	2.8	3.64	4.48	5.39	6.44	7.77	9.73	[illegible]

σ_z Influence factor $= \frac{1}{200}$

No. of subdivision $= 129$

$$\therefore \sigma_z = \frac{129}{200} \times 100 = \boxed{64.5 \text{ KN/m}^2}$$



Scale:

1 cm = 1.337 m

2004-05 4(c)

$q = 50\,kpa$

$z = 12\,m$

$\therefore \sigma_{AFG}$

For CE

D C E

2m

For

$\sigma_z(\text{at } G) = \sigma_{FGED} + - \sigma_{AFGI} - \sigma_{CEGH} + \sigma_{BIGH}$

For FGED:

$m = \frac{L}{z} = \frac{10}{12} = 0.83$, $n = \frac{B}{z} = \frac{10}{12} = 0.83$

$m^2 + n^2 + 1 > m^2 n^2$.

$$\therefore \sigma_{FGED} = \frac{q}{4\pi}\left[\frac{2mn\sqrt{m^2+n^2+1}}{(m^2+n^2+m^2n^2+1)} \times \frac{m^2+n^2+2}{m^2+n^2+1} + \sin^{-1}\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+m^2n^2+1}\right]$$

$$= \frac{50}{4\pi}\left[1.06065 + \frac{\pi}{180} \times 48.389\right.$$

$$= 7.58054\ Kpa.$$

For AFGI:

$m = \frac{L}{z} = \frac{10}{12} = 0.83,\quad n = \frac{2}{12} = \frac{1}{6}$

$m^2 + n^2 + 1 > m^2 n^2$.

$\therefore \sigma_{AFGI} = \frac{50}{4\pi}\left[0.33086 + \frac{\pi}{180} \times 12.0826\right]$

$= 2.1555$ Kpa.

For CEGH:

$m = \frac{2}{12} = \frac{1}{6}$, $n = \frac{10}{12} = 0.83$

$\therefore \sigma_{AFGI} = \frac{50}{4\pi}\left[0.33086 + \frac{\pi}{180} \times 12.0826\right]$

$= 2.1555$ Kpa.

For BIGH:

$m = \frac{2}{12} = \frac{1}{6}$, $n = \frac{1}{6}$

$\therefore \sigma_{BIGH} = \frac{50}{4\pi} \times \left[0.105225 + \frac{\pi}{180} \times 3.09745\right]$

$= 0.6338$ Kpa

$\therefore \sigma_z = 7.58054 - 2.1555 \times 2 + 0.6338$

$= \boxed{3.903 \text{ Kpa}}$

checked with Nafis

بسم الله الرحمن الرحيم

Dr. MD. Zoynul Abedin

Chapter-3 : Soil Type & Classification

- Coefficient of uniformity, $\boxed{C_u = \frac{D_{60}}{D_{10}}}$

  Coefficient of curvature/gradation, $\boxed{C_z = \frac{D_{30}^2}{D_{60} \times D_{10}}}$

- Liquid limit : one point method.

  Kapre & Kulkarni → $\boxed{w_L = w\left(\frac{N}{25}\right)^{0.1}}$

  Nagraz & Jayadeva → $\boxed{w_L = \frac{w}{1.3215 - 0.23 \log n}}$

  Cone pentrometer method → $\boxed{w_L = \frac{w}{0.65 + 0.0175 D}}$

- **Shrinkage limit:**

  Shrinkage limit, $w_s = \frac{\text{weight of water at shrinkage limit}}{\text{weight of soil solid}}$

  $$\boxed{w_s = \frac{(W_1 - W_d) - (V_1 - V_d)\gamma_w}{W_d}}$$

  or, $\boxed{W_s = \frac{\gamma_w V}{W_s} - \frac{G_T}{G_s}}$

  if G_s is known, $\boxed{w_s = \frac{V_d \gamma_w - \frac{W_d}{G_s}}{W_d}}$

(i) Linear shrinkage $= \left(1 - \dfrac{\text{final length}}{\text{initial length}}\right) \times 100$, $\boxed{I_p = 2.13\, LS}$

(j) Indices

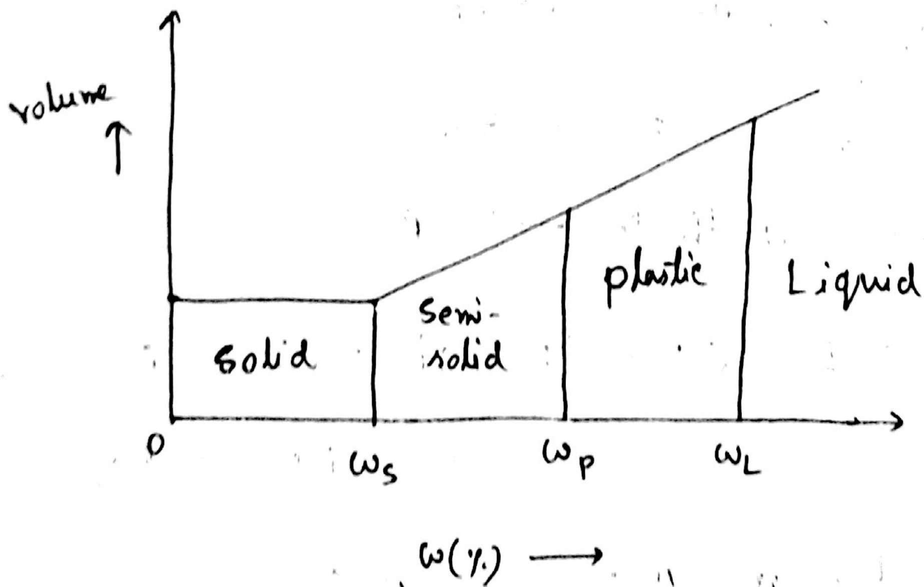
1. Flow index, $\boxed{I_f = \dfrac{w_1 - w_2}{\log\left(\frac{n_2}{n_1}\right)}}$

2. plasticity Index, $\boxed{I_p = w_L - w_p}$

3. Liquidity Index, $\boxed{I_L = \dfrac{w_n - w_p}{w_L - w_p}}$ → water plasticity ratio

4. Toughness index, $\boxed{I_T = \dfrac{I_p}{I_f}}$

5. Consistency index, $\boxed{I_c = \dfrac{w_L - w_n}{I_p}}$



Consistency Index, I_c <0 → liquid

0-100 → plastic

>100 → Semisolid

$I_c + I_L = 100\%$

% retained on #200 seive > 50% → coarse grained
" " " " < 50% → Fine grained

% retained #4 seive > 50% → Gravel (G)
% " " " < 50% → Sand (S)

% Finer #200 ≤ 5% → clean gravel / sand
% " " 5 - 12% → Dual classification
% " " > 12 - 50% → Gravel with Fine / Sand with Fine

A-line → $PI = 0.73(LL - 20)$

Above A-line → clay, below A line, → silt.

$C_u \geq 4$, $C_z = 1$ to 3 → GW
meets neither of the two → GP

$C_u \geq 6$, $C_z = 1$ to 3 → SW
meets neither of the two → SP

Group Index, $GI = (F-35)[0.2 + 0.005(W_L - 40)] + 0.01(F-15)(I_p - 10)$

For subgroup A-2-6 & A-2-7,
partial group index, $PGI = 0.01(F-15)(I_p - 10)$

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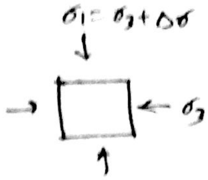
Chapter - 7 : Shear Strength of Soil

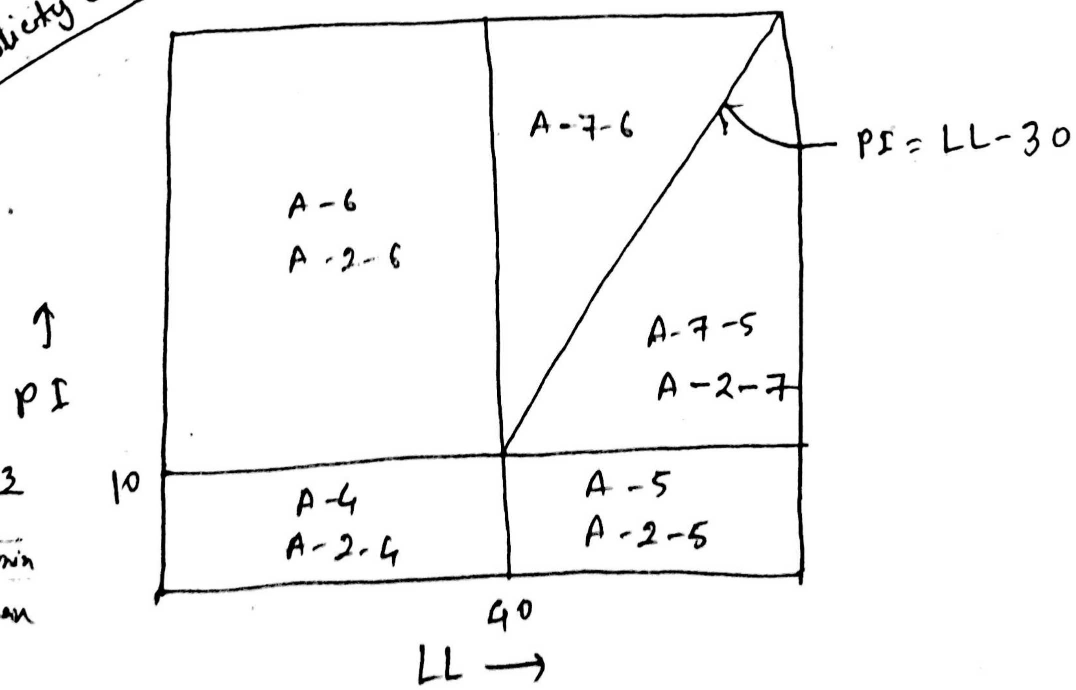


- $\sigma_1' = \sigma_3' \tan^2(45 + \frac{\phi'}{2}) + 2c' \tan(45 + \frac{\phi'}{2})$
- $\theta = 45 + \frac{\phi'}{2}$ [θ = failure plane]
- $\sigma' = \frac{\sigma_1' + \sigma_3'}{2} + \frac{\sigma_1' - \sigma_3'}{2} \cos 2\theta$ (Stress at failure plane)
- $\tau_f = \frac{\sigma_1' - \sigma_3'}{2} \sin 2\theta$
- $\sigma_{max/min} = \frac{\sigma_1 + \sigma_3}{2} \pm \sqrt{\left(\frac{\sigma_1 - \sigma_3}{2}\right)^2 + \tau_{xy}^2}$
- Vane Shear test, $T = \pi C_u \left(\frac{d^2 h}{2} + \frac{a d^3}{4}\right)$, $C_u = \frac{6T}{7\pi d^3}$
- $B = \frac{\Delta u_c}{\Delta \sigma_c}$, $\Delta u = B[\underbrace{\Delta\sigma_3}_{\text{confining pressure}} + A(\underbrace{\Delta\sigma_1 - \Delta\sigma_3}_{\text{deviator stress}})]$

$$= \Delta u_c + \Delta u_d$$

$$= B\Delta\sigma_3 + \bar{A}(\Delta\sigma_1 - \Delta\sigma_3)$$

AASHTO plasticity chart



	A-1-a	A-1-b	A-3
No-4	50 max		
No-10	30 max	50 max	51 min
No-40	15 max	25 max	10 max

Chapter - 9: Lateral Earth pressure

① ~~Ear~~ Earth Pressure at rest:

$$\text{Jacky}\begin{cases} K_0 = (1-\sin\phi) & [\text{for NC soil}] \\ K_0 = (1-\sin\phi)\,OCR^{\sin\phi} & [\text{for OC soil}] \end{cases}$$

$$\text{Alpan}\begin{cases} K_0 = 0.19 + 0.233 \log I_p & [\text{NC}] \\ K_{0(oc)} = K_{0(NC)}\sqrt{OCR} & [\text{OC soil}] \end{cases}$$

For sloping surface $(1+0.5\tan\beta)^2$ must be multiplied.

② Cohesionless soil:

$$K_a = \frac{1-\sin\phi}{1+\sin\phi} = \tan^2(45-\frac{\phi}{2}) \quad [\text{for active earth pressure}]$$

$$K_p = \frac{1+\sin\phi}{1-\sin\phi} = \tan^2(45+\frac{\phi}{2}) \quad [\text{for passive earth pressure}]$$

③ Partially Cohesive Soil: ($c-\phi'$ soil)

Active earth pressure, $P_{ac} = K_a\gamma h - 2c\sqrt{K_a}$

Critical height, $h_c = \frac{2c}{\gamma\sqrt{K_a}} = \frac{2c}{\gamma}\tan(45+\frac{\phi}{2})$.

Theoretical unsupported height, $H_u = 2h_c = \frac{4c}{\gamma\sqrt{K_a}}$

Active earth thrust (force), $P_{ac} = \frac{1}{2}K_a\gamma h^2 - 2cH\sqrt{K_a} + \frac{2c^2}{\gamma}$

passive earth pressure, $P_{pc} = K_p\gamma h + 2c\sqrt{K_p}$

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Cohesionless backfill with sloping surface

$$K_a = \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}$$

Active thrust, $P_a = \frac{1}{2} K_a \gamma H^2 \cos\beta$

Horizontal active thrust, $P_{ah} = \frac{1}{2} K_a \gamma H^2 \cos^2\beta$

$$K_p = \frac{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}$$

passive thrust, $P_p = \frac{1}{2} K_p \gamma H^2 \cos\beta$

Active → downward & outward

passive → inward & upward

Point load:

For $m > 0.4$,

$$P_h = \frac{1.77Q}{H^2} \frac{m^2 n^2}{(m^2+n^2)^3}$$

For $m \le 0.4$,

$$P_h = \frac{0.28Q}{H^2} \frac{0.4^2 n^2}{(0.4^2+n^2)^3}$$

$$P_h' = P_h \cos^2(1.1\alpha)$$

Line load:

For $m > 0.4$,

$$P_h = \frac{4q}{\pi H} \frac{m^2 n}{(m^2+n^2)^2}$$

For $m \le 0.4$,

$$P_h = \frac{4q}{\pi H} \frac{0.4^2 n}{(0.4^2+n^2)^2}$$

$\left.\begin{array}{l} LL \rightarrow 12 \text{ to } 25 \\ PI \rightarrow 4 \text{ to } 7 \end{array}\right\}$ CL-ML (Low plastic clayey silt)

$$K_a = \frac{\sin^2(\alpha + \phi)}{\sin^2(\alpha)\,\sin^2(\alpha - \delta)\left[1 + \sqrt{\dfrac{\sin(\phi + \delta)\,\sin(\phi - \beta)}{\sin(\alpha - \delta)\,\sin(\alpha + \beta)}}\right]^2}$$

where, $\alpha \rightarrow$ angle of wall with horizontal

$\delta \rightarrow$ angle of wall friction

$\beta \rightarrow$ slope of soil surface.

For sand, $c = 0$

NC clay ; $c = 0$

saturated clay , $\phi = 0$

purely cohesive soil, $\phi = 0$

sand / Granular materials, $c = 0$

Culman's line, $\psi = 90 - \theta - \delta$

Sieve No.		Sieve opening
#4	—	~~0.4274~~ 4.75
#10	—	2
#40	—	0.425
#100	—	0.15
#200	—	0.075

Sensitivity

< 2	—	Insensitive
2 - 4	—	Medium sensitive
4 - 8	—	Sensitive
8 - 16	—	Very (extra) sensitive
16 - 32	—	slightly quick
32 - 64	—	Medium quick
> 64	—	quick.

$q_u = \frac{c_u}{2}$ $\quad c_u = \frac{q_u}{2}$

$c_u \longrightarrow$ undrained shear strength

$q_u \longrightarrow$ unconfined compression strength

$$K_a = \frac{\cos^2(\alpha + \phi) \text{ [illegible]}}{\text{[illegible]} \left[1 + \text{[illegible]} \right.}$$

Atterberg Limits

Given that, $w = 45\%$, No. of blows $= 28$

$w_p = 18.2\%$, $w_n = 40\%$

Now liquid limit, $w_L = w\left(\frac{N}{25}\right)^{0.1}$

Round figure

$$= 0.45 \times \left(\frac{28}{25}\right)^{0.1} = 45.5\% \approx 46\%$$

consistency index, $I_c = \dfrac{w_L - w_n}{w_L - w_p}$

$$= \frac{\cancel{45.5}\,46 - 40}{\cancel{45.5}\,46 - 18.2} = \boxed{\cancel{20.2\%}} \quad \boxed{22\%}$$

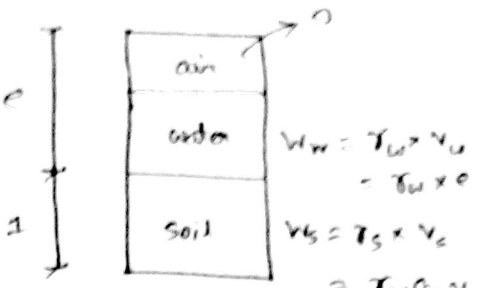
so, the soil is in plastic condition.

plasticity index, $I_p = (45.5 - 18.2) = 27.3\%$

Now, $I_p = 2.13 \times LS$ (Linear Shrinkage)

$$\Rightarrow LS = \frac{27.3\%}{2.13} = \boxed{0.1282}$$

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



$\Rightarrow V_v = e, \quad V_s = 1$

Now, shrinkage limit $= \frac{\text{weight of water at shrinkage limit}}{\text{weight of soil solid}}$

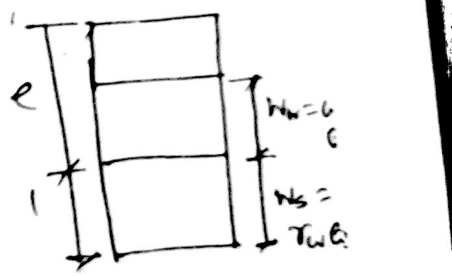
$\Rightarrow w_s = \frac{W_w}{W_s}$

$\Rightarrow w_s = \frac{\gamma_w e}{\gamma_w G_s} = \frac{0.4}{2.7} = \boxed{14.8\%}$

$\approx 15\%$

<u>Another approach:</u>

As the G_s is known, so, $w_s = \frac{V_d \gamma_w - \frac{W_d}{G_s}}{W_d}$



~~Now e~~ given, $e = 0.4$.

Let's assume, total volume $= 1+e = 1.4$

$\gamma_d = \frac{\gamma_w G_s}{1+e} = 18.92 = \frac{W_d}{V_d}$

$V_d = 1+e = 1.4, \quad W_d = 26.487$ KN.

Now, $w_s = \frac{1.4 \times 9.81 - \frac{26.487}{2.7}}{26.487} \times 100 = \boxed{14.8\%} \approx 15\%$

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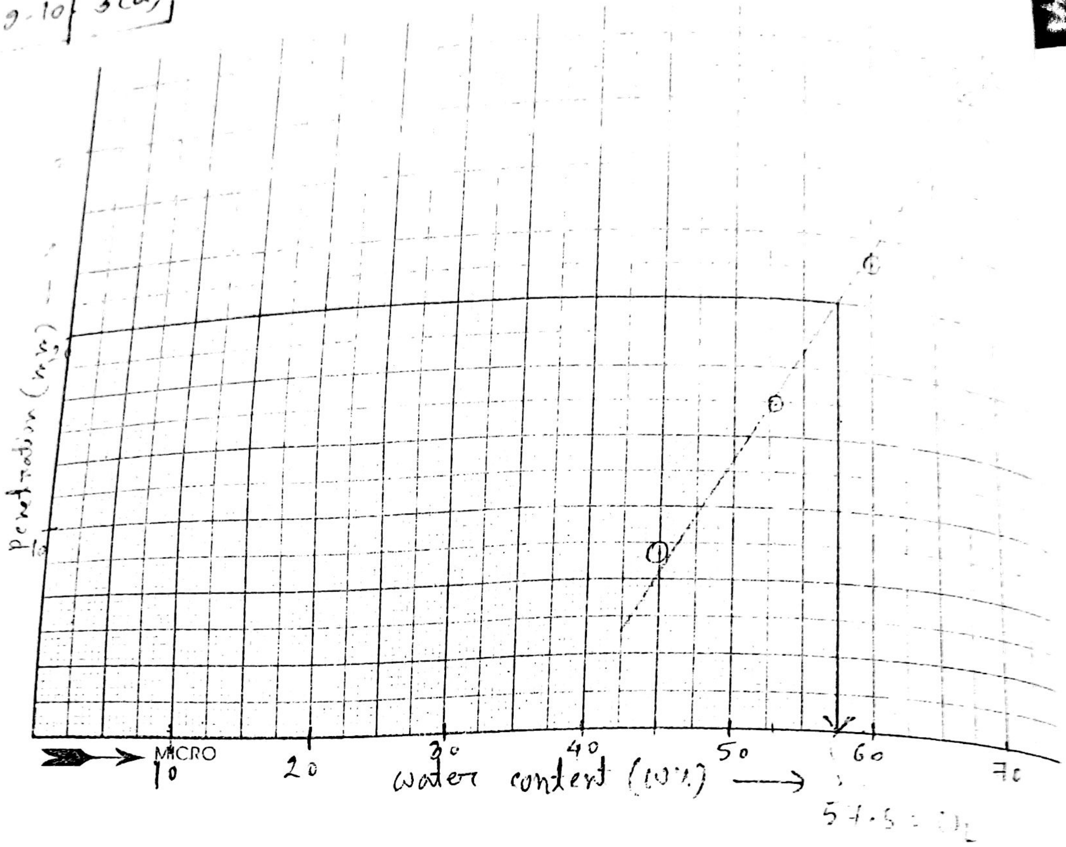
2009-10| 3(a)]

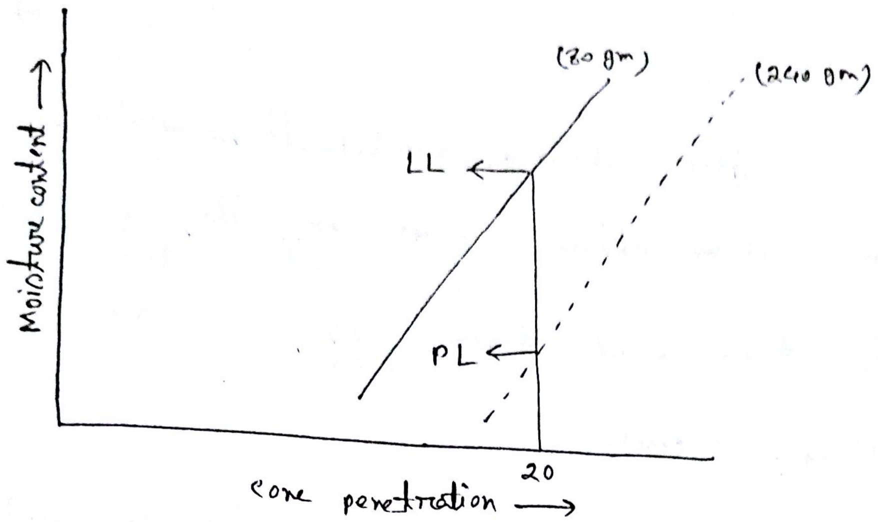


For cone penetration method, LL is at 20 mm penetration.

from graph, LL = 57.5% ≃ 58%

<u>Determination of plastic limit by cone penetration method:</u>

To determine plastic limit, a cone of similar geometry (35 mm long with an apex angle of 30°) but with a mass of 240 gm (in stead of 80 gm). is used. Three or four tests at varying moisture content



are conducted and the corresponding cone penetrations are determined. The moisture content corresponding to a cone penetration of 20 mm is the plastic limit.

Give, $PL = 35\%$, $w_n = 40\%$.

$$\therefore \text{Liquidity index, } I_L = \frac{w_n - w_p}{w_L - w_p} = \frac{40 - \overset{35}{\cancel{57.5}}}{57.5 - 35}$$

$$= 22.2\%$$

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2008-09 [1(c)]

After shrinkage limit there would be no ~~vote~~ volume change occur in the soil. As the S is dried from 9%. So, no volume change occurs.

[2004-05] 5(b)

No. of blows = 16, water content = 45%.

So, liquid limit = $w \times \left(\frac{N}{25}\right)^{0.1}$

$= 45 \times \left(\frac{16}{25}\right)^{0.1}$

$= 43\%$.

Liquid limit is between 39 to 50%. So, it has medium swelling potential.

Compressibility of a soil generally increases with the increase ~~of~~ in liquid limit.

Soil Classification

Illustrative examples: (From sheet)

Example - 1:

$D_{10} = 0.06\,mm$, $D_{30} = 0.20\,mm$, $D_{60} = 0.40\,mm$.

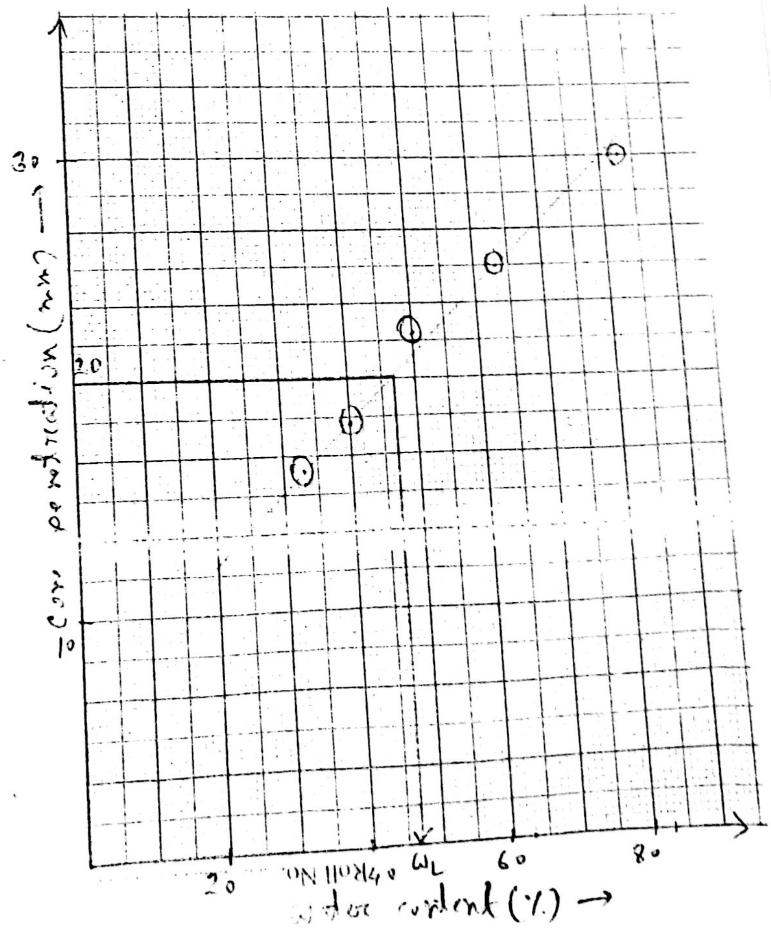
uniformity coefficient. $C_u = \frac{D_{60}}{D_{10}} = \frac{0.4}{0.06} = 6.67$

$$C_z = \frac{D_{30}^2}{D_{10} \times D_{60}} = \frac{0.2^2}{0.06 \times 0.4} = 1.67.$$

For sand, $C_u \geq 6$, $C_z \rightarrow 1$ to $3 \rightarrow$ well graded.

So, the soil is well graded.

Example - 2:



From graph, $w_L = 47\%$

From one point method.

$$w_L = \frac{w}{0.65 + 0.0175 D}$$

$$= \frac{40}{0.65 + 0.0175 \times 18}$$

$$= 42$$

Therefore, one point method gives a lower value of liquid limit (42) as compared to the value of 46.

Example 3

To determine the shrinkage limit, G_s is unknown,

$$\therefore \text{shrinkage limit} = \frac{\text{weight of water at shrinkage limit}}{\text{weight of soil solid}}$$

$$= \frac{(117.3 - 25.6) - (63.5 - 47.2) \times 1}{25.6}$$

$$= \boxed{19.9\%}$$

Example - 4:

USCS classification:

% retained on #200 seive = 90% (> 50%)

So, the soil is coarse grained.

Now, % of coarse grained soil retained on #4 seive $= \frac{4}{0.9} = 4.44\%$ (< 50)

So, the soil is sand.

Again, % finer on #200 is 10% (5 - 12%)

Hence dual classification is applicable.

Now, $C_u = \frac{D_{60}}{D_{10}} = \frac{2}{0.075} = 26.7$

$C_c = \frac{D_{30}^2}{D_{60} \times D_{10}} = \frac{0.425^2}{0.075 \times 2} = 1.2$

As $C_u \geq 6$ and C_c is between 1 to 3.

So, the soil sample is well graded.

LL = 40%, PL = 20%. PI = 20

At A line. $PI = 0.73(LL - 20) = 0.73 \times (40 - 20)$
$= 14.6$

So, PI is above the A-line.

So, the soil is $\boxed{\text{SW-SC}}$

AASHTO classification:

% retained on #200 = 90% (> 35%)

So, the soil is granular material.

% passing on #10 = 60 [A-1-a is rejected]

% passing on #40 = 30 [A-3 is rejected]

% passing on #200 = 10

LL = 40 [A-2-5 & A-2-7 is rejected]

PI = 20 [A-1, A-2-4, A-2-5 is rejected].

A-2-6. meets all criterias. So, the soil is silty or clayey gravel or sand.

For A-2-6. Partial group index is needed.

So, $PGI = 0.01(F-15)(I_p-10)$

$= 0.01(10-15)(20-10)$

$= -0.5 = 0$

So, the soil is $\boxed{A-2-6(0)}$

<u>Exercise - 3.13</u>

% retained on #200 = 40 (< 50)

So, the soil is ~~fine~~ coarse grained (silt or clay).

LL = 60, PL = 20, PI = 40

For A-line. PI = 0.73(60 − 20) = 29.2

So, PI is above A-line.

Exercise - 3.13

% retained on #4 sieve = 18

% " " #200 " = 40

$\therefore$ % finer on #4 = 82%

% finer on #200 = 100 − (40 + 18) = 42%

Major Division:

% Finer than 0.075 mm is 42 (<50). So, the soil is coarse grained.

58% of the soil is coarse grained.

% of coarse grained soil retained on #4 = $\frac{18}{0.58}$ = 31 (<50)

So, the soil is sand.

Now, % finer on #200 = 42 (12 − 50)

So, the probable classification is SC or SM

LL = 60, PL = 20, PI = 40

For A-line, PI = 0.73 × (60 − 20) = 29.2 < 40.

So, the soil falls above A-line.

$\therefore$ The soil is $\boxed{\text{SC (clayey sand)}}$

Exercise 3.14

% passing on # 10 = 82%.

% " " # 40 = 54%.

% " " # 200 = 14%.

200 passing = 14%. (<35%)

so, the soil is granular materials.

The soil may be A-1, A-2 or A-3.

10 passing = 82%. (>50%) [So, the soil is not A...]

40 passing = 54%. [So, the soil is not A-1-b].

200 passing = 14% [So, the soil is not A-3].

LL = 60 [it is not A-2-4 & A-2-6]

PI = 40

So, the soil is surely A-2-7.

Now, $PGI = 0.01(F-15)(I_p-10)$

$= 0.01 \times (14-15)(40-10)$

$= -0.3 = 0$

∴ Soil is $\boxed{A\text{-}2\text{-}7\ (0)}$

Exer...

a)

Exercise - 3.15

a) The compressibility of soil ~~gene~~ generally increases with an increase in liquid limit.

So, soil 2 is more compressible.

c) The greater value of flow index i.e. steeper slope ~~possesses~~ possesses lower shear strength.

So, soil 2 is a better foundation soil.

b) As soil 2 is more compressible, so, soil 1 is better for foundation.

Exercise - 3.16:

$$\text{Shrinkage limit} = \left(\frac{V_d}{W_d} - \frac{1}{G_s \gamma_w}\right)\gamma_w$$

$$= \left(\frac{13.5}{27} - \frac{1}{2.7 \times 1}\right) \times 1$$

$$= 12.96\% = \boxed{13\%}$$

As the shrinkage limit is in 10-16%, it has medium swelling potential.

~~10-10-26~~

010-11 | 2(b)

$\because$ % finer #200 = 96.9% (>50)

So, the soil is fine grained.

LL = 20, PI = 6.

For A line, PI = 0.73 (~~40~~ 20 − 20) = 0.

So, the soil is above A-line.

So, it is clay. | As. LL is ~~12~~ in between 12 to ... PI is in between 5 to ... So, it is CL-...

~~As LL < 50, so it is low plastic~~

~~$\therefore$ Soil is CL (low plastic clay)~~

ASTM

% finer on #200 = 96.9% (>35%)

So, it is silt-clay materials.

Now, LL = 20 So, A-4 & ~~A A6~~ A-6

PI = 6, So, soil is A-4

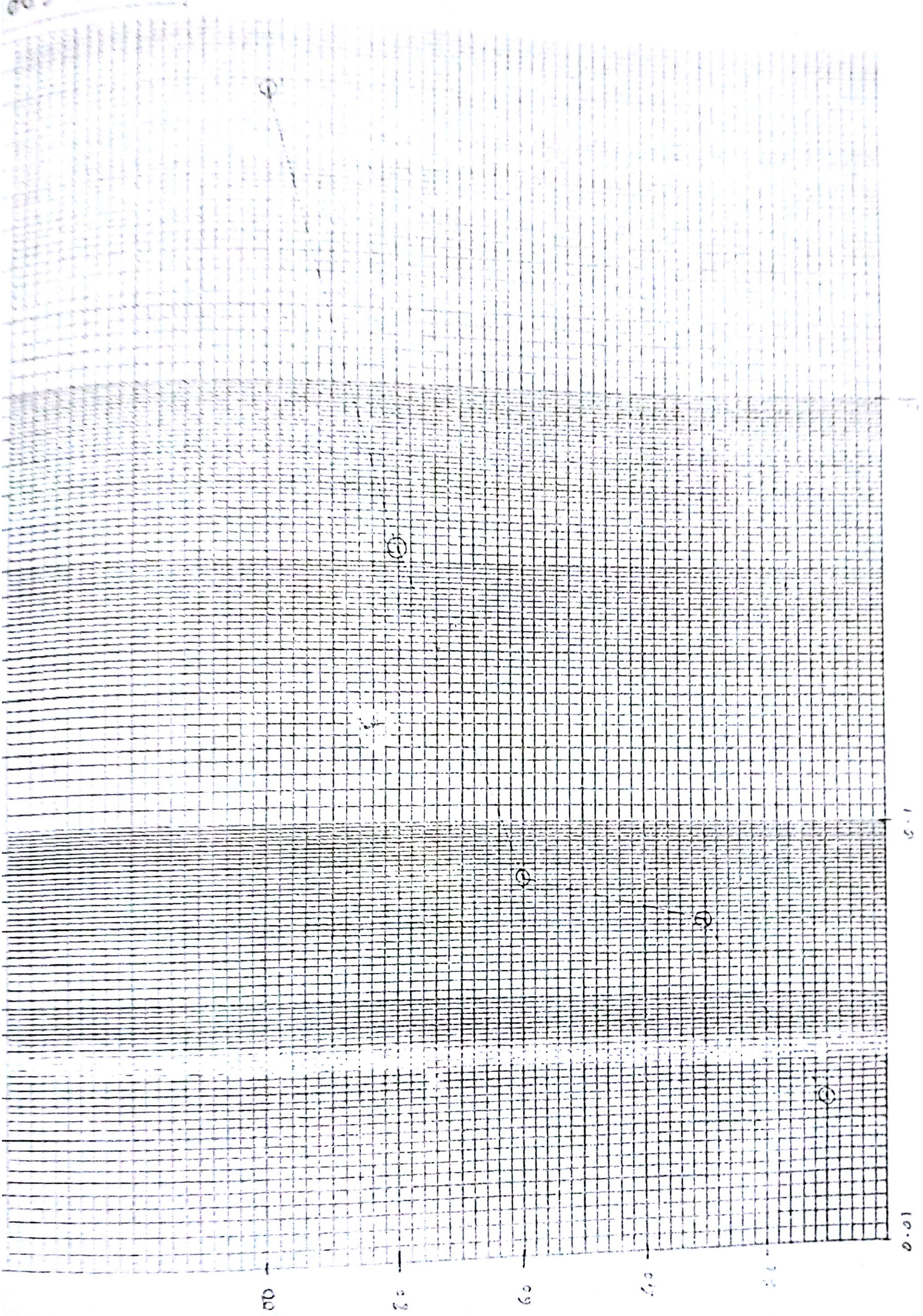
$$G.I = (F-35)[0.2+0.005(W_L-40)] + 0.01(F-15)(I_p-10)$$

$$= (96.9-35)[0.2+0.005(20-40)] + 0.01(96.9-15)(6-10)$$

$$= 2.9 \approx 3$$

So, the soil is $\boxed{A-4(3)}$ confirm

009-10.2(a)



$D_{60} = 0.075$ mm, $\quad C_u = \dfrac{D_{60}}{D_{10}}$

$$\Rightarrow 3 = \frac{0.075}{D_{10}}$$

$$\therefore D_{10} = 0.025 \text{ mm.}$$

$$C_z = \frac{D_{30}^2}{D_{60} \times D_{10}} \Rightarrow 1.92 = \frac{D_{30}^2}{0.025 \times 0.075}$$

$$\therefore D_{30} = 0.06$$

% passing in # 200 = 60% (>35%)

so, it is silt - clay material.

Now, LL = 43 (>40) so, A-5 or A-7 may be.

PI = 43 - 15 = 28 so, it is A-7

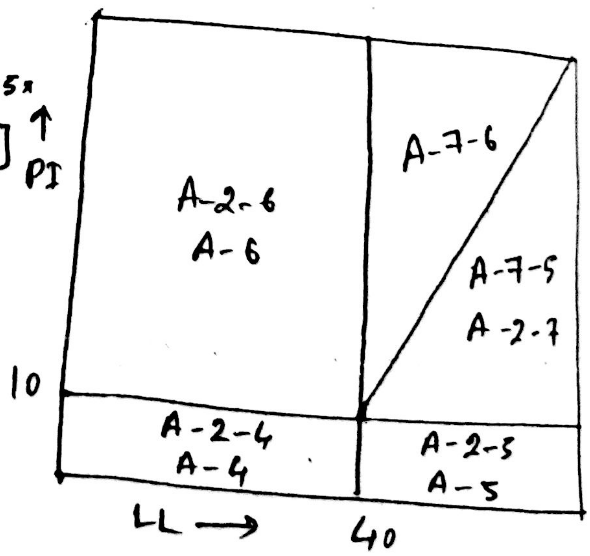
Now, PI = LL - 30 = 43 - 30 = 13

Since PI is above the line, so, the soil is A-7-6

$$GI = 0.01(F-15)(PI-10) + \frac{F-35}{100}[20 + 0.5 \times (LL-40)]$$

$$= \cancel{0.01 \times (60-15) \times (28-10)}$$

$$= \cancel{2.1 \approx 8} \quad \cancel{7.43 \approx 7} \quad 13.5 \approx 14$$



$\therefore$ soil $\boxed{A-\cancel{7-6\,(7)}}$

$$\boxed{A-7-6\,(14)}$$

2008-09. 1(b)

200 passing = 60% (> 50%)

So, it is fine soil (silt / clay).

$LL = 19$, $PL = 14$, $PI = 4$

For A-line, $PI = 0.73(19 - 20) = \cancel{-0.73}$ Not possible

~~So, PI is above A line. ∴ the soil is clay.~~

~~Now, LL = 19 is (< 40) (< 50).~~

~~So, it is low plastic clay (CL)~~

But here, LL is between 12 to 25

PI is between 4 to 7.

∴ Soil is is $\boxed{\text{Low plastic clayey silt (CL–ML)}}$

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008.09.2(b)

% passing #200 = 8 (<35%)

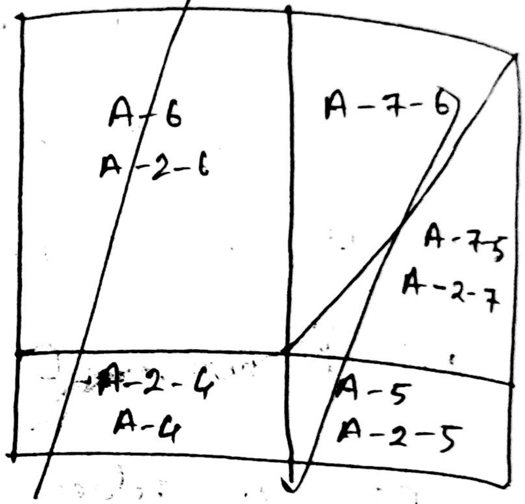
So, it is granular material

10 passing = 60 (So, it is not A-1-a)

40 passing = 30 (So, it is not A-3)

200 passing = 8

LL =



USCS classification:

#200 passing = 8 (< 50)

So, it is coarse ~~sand~~ (sand / gravel)

~~# 4 π~~

~~Now, % of coarse grained soil retained on 4 =~~

Now percent passing # 10 = 60

Since # 4 seive is bigger than # 10, so the materials that passes # 10 seive will also pass # 4 sieve.

∴ % passing to # 4 seive ≥ 60% which is greater than 50 %. So, the coarse material will be sand.

Now, #200 passing = 8%. (5-12%)

So, Dual classification is needed.

LL = 27, PI = 12

PI for A-line, PI = 0.73 (27-20) = 5.11

So, the soil is above a A-line. so, clayey sand.

Again, $C_u = \frac{D_{60}}{D_{10}} = \frac{2}{0.15} = 13.33$

$C_z = \frac{D_{30}^2}{D_{60} \times D_{10}} = \frac{0.425^2}{0.15 \times 2} = 0.6$

$C_u > 6$ but C_z is not between 1 to 3.

So, it is poorly graded clayey sand.

$\boxed{SP-SC}$.

[2007-08] 1(b)

#200 passing = 30% (< 35%).

The possible classifications are: (as per AASHTO)

A-1-a

A-1-b

A-3

A-2-4

A-2-5

A-2-6

A-2-7.

2007-08 | 3(a)

$\# 200$ passing $= 10\%$ ($< 50\%$)

So, it is coarse grained (sand / gravel).

% of coarse grained soil retained on $\# 4 = \frac{40}{0.9} = 44.44 < 50$

So, the soil is sand.

$\# 200$ passing $= 10\%$ ($5 - 12\%$) so, dual classification.

Now, $LL = 45$, $PI = 24$

For A-line, $PI = 0.73(LL - 20) = 18.25$.

$\therefore$ PI is above A-line.

Hence the finer soil is clay.

Again, $C_u = \frac{D_{60}}{D_{10}} = \frac{4.76}{0.075} = 63.47 > 6$

$\therefore C_z = \frac{D_{30}^2}{D_{60} \times D_{10}} = \frac{0.425^2}{4.76 \times 0.075} = 0.5 < 1$

As C_z is not within 1 to 3.

$\therefore$ ~~It~~ the soil is $\boxed{SP - SC}$

confirm

[2006-07 | 4(e)]

200 passing = 40 (> 35)

∴ the soil is silt clay material.

~~# 10 passing = 80~~ (so, the soil is not A-1-a)

~~# 40 passi~~

LL = 30 (∴ the soil is A-4 or A-6)

PI = 8 (<10)

∴ the soil is $\boxed{A-4 (2)}$.

The soil is not much suitable for for highway construction

According to USCS, possible classifications are

$\boxed{SC}$

$$GI = (F-35)[0.2 + 0.005(W_L - 40)] + 0.01(F-15)(I_p - ...$$

$$= (40-35)[0.2 + 0.005 + (30-40)] + 0.01(40-15)(8- ...$$

$$= 1.75 \approx 2$$

2

[2008-09 | 1(d)]

Initial

$e_1 = 0.5$, $\gamma_{d_1} = 17.8\ kN/m^3$

$\phi_1 = 30°$

$$K_{p_1} = \frac{1+\sin 30}{1-\sin 30} = 3$$

$P_1 = \frac{1}{2} K_{p_1} \gamma_{d_1} H_1 \times H_1$

$= 3 \times 17.8 \times 8^2 \times \frac{1}{2}$

$= 1708.8\ KN$

Final (after compaction)

$e_2 = 0.4$, $\gamma_{d_2} = 18.8\ KN/m^3$

$\phi_2 = 35°$

$$K_p = \frac{1+\sin 35}{1-\sin 35} = 3.7$$

lets the height of the soil in $H_2 (< 8m)$ after compaction

so, $P_2 = \frac{1}{2} K_{p_2} \gamma_{d_2} \times H_2^2$

Now, $\frac{V_2}{V_1} = \frac{1+e_2}{1+e_1} \Rightarrow \frac{H_2 \times A}{H_1 \times A} = \frac{1+0.4}{1+0.5}$

$\Rightarrow H_2 = 7.47\ m$

$\therefore P_2 = \frac{1}{2} \times 3.7 \times 18.8 \times 7.47^2 = 1940.76\ KN.$

$\therefore$ ratio of initial to final passive thrust.

$$\frac{P_1}{P_2} = \frac{1708.8}{1940.76} = \boxed{0.88}$$

confirm

[2008-09] 1(a)

$LL = 40\%$, $PL = 25\%$.

$\therefore$ PI, or $I_p = 40 - 25 = 15$

$OCR = 2.5$, $H = 6m$

~~Later~~ ~~R~~

As the wall is rigid, so there is no movement of the wall. Hence the earth pressure is at rest.

$\therefore K_o = (0.19 + 0.233 \log 15) \times \cancel{\;} \sqrt{2.5}$

$= 0.734$

$\therefore$ lateral thrust $= \frac{1}{2} \times K_o \gamma h \times h$

$= \frac{1}{2} \times 0.734 \times \gamma \times 6^2$

$= \boxed{13.212\gamma \ \text{kN/m}}$

confirm

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2008-09 1(g)

Given, $\gamma = 22\ KN/m^3$, $\phi = 0$.

cased in → mean soil fi failed.

Now unsupported heigh, $H_u = \frac{4c}{\gamma} \tan(45 + \frac{\phi}{2})$

$\Rightarrow 4 = \frac{4 \times c}{22} \times 1$

$\therefore \boxed{c = 22\ KN/m^2}$

confirm

2008-09 | 2(a) ফিগার নং Am. 53.01

$H = 4.5m$, $\gamma = 18.6\ KN/m^3$. $\phi = 32°$, $\delta = 20°$, $\theta = 0$

$\therefore \psi = 90 - \cancel{\theta} - \delta = 90 - 20 = 70°$

$\Delta ABC_1 = \cancel{\frac{1}{2} \times 4.5 \times 1 = 2.25}$ area of $ABC_1 \times \gamma \times 1$

$= \frac{1}{2} \times 4.5 \times 1 \times 18.6 = 41.85\ KN$

$\Delta ABC_2 = \frac{1}{2} \times 4.5 \times 2 \times 18.6 = 83.7\ KN$

$\Delta ABC_3 = \frac{1}{2} \times 4.5 \times 3 \times 18.6 = 125.55\ KN$

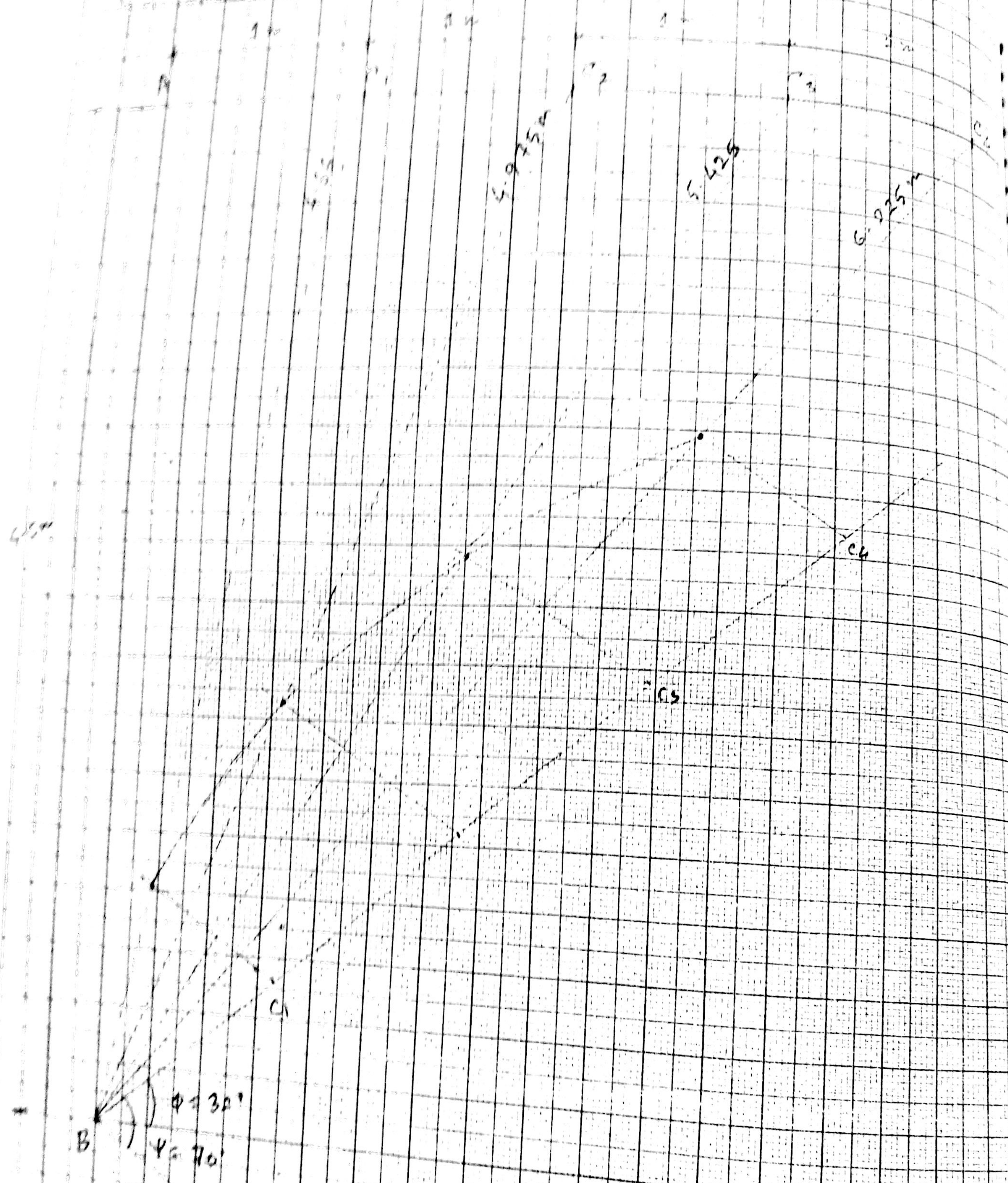
$\Delta ABC_4 = \frac{1}{2} \times 4.5 \times 4 \times 18.6 = 167.4\ KN$.

$\therefore$ total active earth pressure $= C_a C_a' \times$ load scale

$= 4.2\ cm \times \frac{200\ KN}{20\ cm}$

$= \boxed{42\ KN}$

2008-09 | 2(a)



Scale:

Length : 1m = 2 cm

Weight : 2 cm = 100 kN

2004-05 | 5(c)

possible soil groups as USCS are —

(i) SP – SC

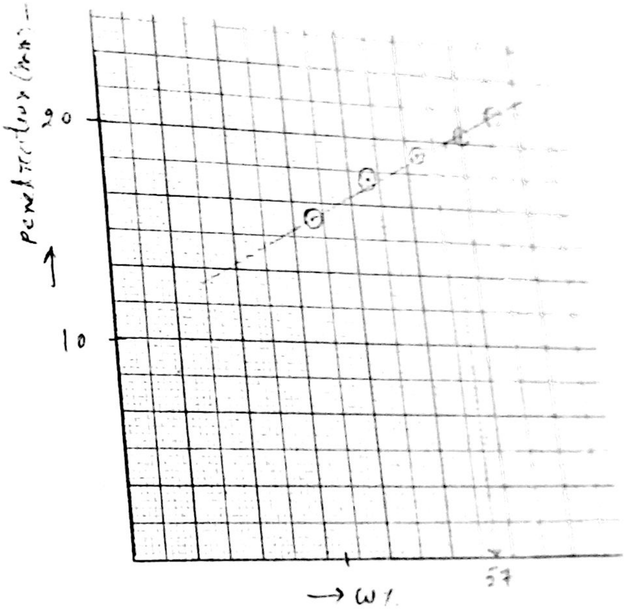
(ii) SP – SM

(iii) SW – SC

(iv) SW – SM

(v) SC – SM – silty clayey sand.

2004-05 | 6(a)



$LL = 57\%$, $PL = 25\%$, $PI = 32\%$.

AASHTO classification:

#200 passing = 54 (>35)

So, it is silt clay material.

LL = 57 (A-5 or A-7)

PI = 32 ∴ A-7

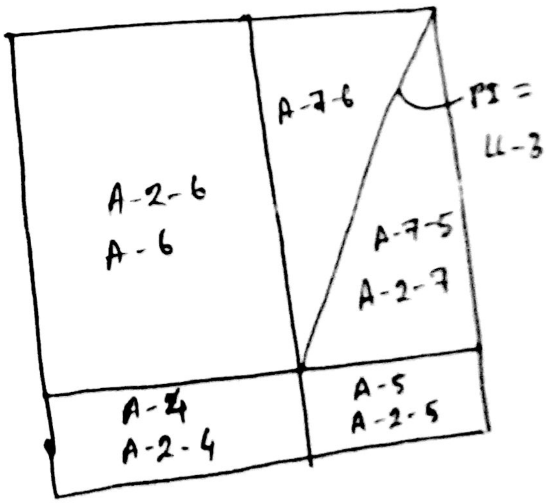
Now, LL − 30 = 27 < 32.

∴ the soil is A-7-6

$\therefore G = (F-15)[0.2+0.005(w_L-40)]$

$+0.01(F-15)(I_p-10)$

$=19.7 \approx 20$



∴ soil is $\boxed{A-7-6\ (20)}$

USCS classification:

#200 passing = 54 (>50)

So, it is silt or clay.

LL = 57, PI = 32

For A-line, $PI = 0.73(57-20) = 27 < 32$

∴ the soil is clay.

LL = 57 (>50).

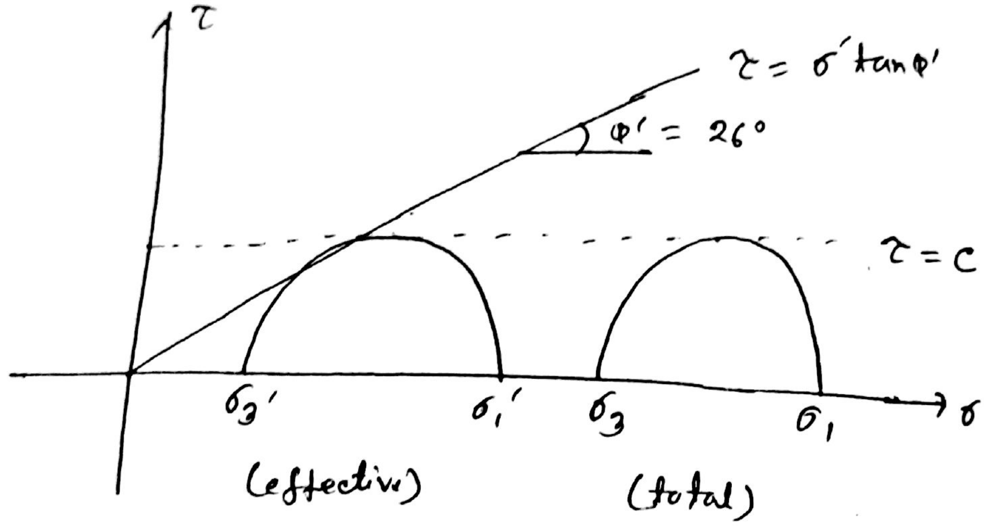
So, the soil is $\boxed{\text{CH (high plastic clay)}}$

Stress, strain, strength characteristics of soil

Example:

In an UU test, undrained strength = 17.5 kPa. $c' = 0$, $\phi = 26°$. pore pressure at failure, $u_d = 43$ kPa. Find cell pressure used.

Ans:



Now, undrained shear strength = 17.5 = $\cancel{\frac{\sigma_3 - \sigma_1}{2}} = \cancel{\sigma_3'}$

$$17.5 = \frac{\sigma_1' - \sigma_3'}{2} = \frac{\sigma_1 - \sigma_3}{2} \quad \text{—— (i)}$$

Again, $\tau = c' + \sigma' \tan\phi'$

$$\Rightarrow \tau = \sigma' \tan 26°.$$

$$\sigma_1' = \sigma_3' \tan^2\left(45 + \frac{\phi'}{2}\right) + 2c' \tan\left(45 + \frac{\phi'}{2}\right)$$

$$\Rightarrow \sigma_1' = \sigma_3' \tan^2\left(45 + \frac{26}{2}\right)$$

$$\therefore \sigma_1' = 2.561\, \sigma_3' \quad \text{—— (ii)}$$

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Again, from (i),

$\sigma_1' = 35 + \sigma_3'$

$\Rightarrow 2.561\, \sigma_3' = 35 + \sigma_3'$

$\therefore \sigma_3' = 22.4$ Kpa

$\sigma_1' = 57.42$ Kpa.

$\therefore$ cell pressure, $\sigma_3 = 22.4 + 43 = \boxed{65.4 \text{ Kpa}}$ Ans

Example 11.2: (B.M. Das)

chamber confining pressure, $\sigma_3 = 16$ lb/in²,

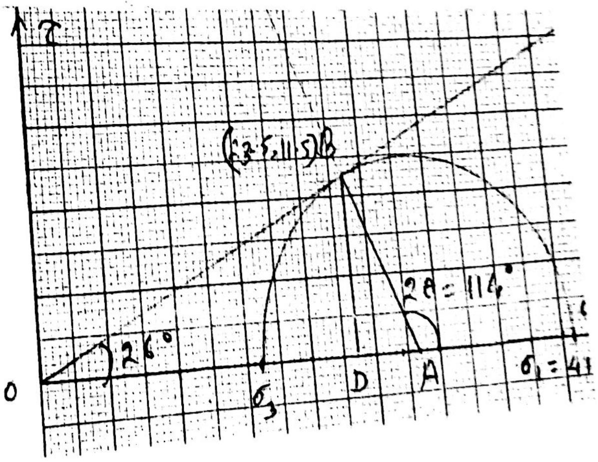
Deviator stress, ~~σ_3~~ $\Delta\sigma = 25$ lb/in²

$\therefore \sigma_1 = \sigma_3 + \Delta\sigma = 16 + 25 = 41$ lb/in².

For NC clay, $c = 0$.

Now, $\sin\phi' = \dfrac{\frac{\sigma_1 - \sigma_3}{2}}{\frac{\sigma_1 + \sigma_3}{2}} = \dfrac{(41-16)}{(41+16)}$

$\therefore \boxed{\phi = 26°}$



From graph, $2\theta = 114°$.

$\therefore \theta = 57°$.

or, From ~~tran~~ triangle OAB,

$26 + 90 + (180 - 2\theta) = 180$

$\therefore \boxed{\theta = 58°}$

or, from direct formula, $\theta = 45 + \frac{\phi'}{2} = 45 + \frac{26}{2} = \boxed{58°}$

Example - 11.3

a) For failure plane,

$$\sigma' = \frac{\sigma_1' + \sigma_3'}{2} + \frac{\sigma_1' - \sigma_3'}{2} \cos 2\theta$$

$$= \frac{41+16}{2} + \frac{41-16}{2} \cos(58 \times 2)$$

$$= \boxed{23 \ lb/in^2}$$

Again, from graph, $\sigma' = 23.5 \ lb/in^2$.

~~or, AD = OB sin 2θ =~~

shear stress $\tau_f = \frac{\sigma_1' - \sigma_3'}{2} \sin 2\theta$

$$= \frac{41-16}{2} \sin(2 \times 58) = \boxed{11.24 \ lb/in^2}$$

From graph, $\tau_f = 11.5 \ lb/in^2$

b) At maximum shear stress, effective normal stress,

$$\sigma' = \frac{\sigma_1' + \sigma_3'}{2} = \frac{41+16}{2} = \boxed{28.5 \ lb/in^2}$$

Now, we know,

$$\sigma_1' = \sigma_3' \tan^2\left(45+\frac{\phi}{2}\right) + 2c' \tan\left(45+\frac{\phi}{2}\right)$$

$$\Rightarrow 200 = 70\tan^2\left(45+\frac{\phi}{2}\right) + 2c' \tan\left(45+\frac{\phi}{2}\right) \quad \text{--- (1)}$$

And, $383.5 = 160 \tan^2(45+\phi/2) + 2c' \tan(45+\phi/2)$ --- (2)

solving (1) & (2),

$$383.5 - 200 = (160-70)\tan^2\left(45+\frac{\phi}{2}\right)$$

$\therefore \boxed{\phi = 20^\circ}$

and, $\boxed{c = 20 \text{ Kpa}}$

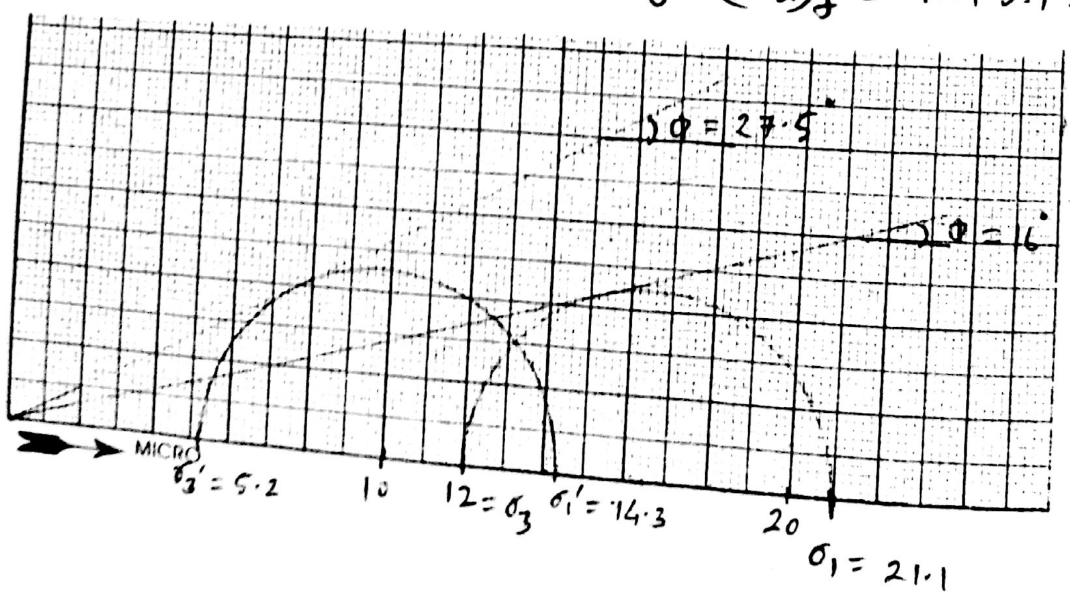
From graph, $\phi = 20^\circ$ & $c = 21$ Kpa.

Ans

Example - 11.6

CU test:

for NC clay, $c=0$. $\sigma_1 = \sigma_3 + (\Delta\sigma_d)_f = 12+9.1 = 21.1$ Kpa



$\sigma_1 = \sigma_3 \tan^2(45 + \phi/2) + 2c \tan(45 + \phi/2)$

$\Rightarrow 81.1 = 18 \tan^2(45 + \phi/2)$

$\therefore \boxed{\phi = 16°}$

$\sigma_3' = \sigma_3 - (\Delta u_d)_f = 12 - 6.8 = 5.2$ kPa

Again, $\sigma_1' = \sigma_3' \tan^2(45 + \frac{\phi'}{2}) + 2c' \tan(45 + \frac{\phi'}{2})$

$\Rightarrow 14.3 = 5.2 \tan^2(45 + \frac{\phi'}{2})$

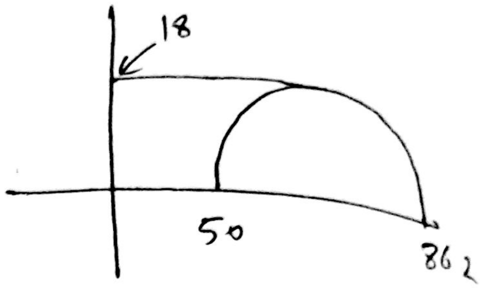
$\therefore \boxed{\phi' = 27.8°}$

[Question solve - ch 7]

[10-11. 2(a)]

For saturate clay, $\phi = 0$.

For the first specimen, $c_u = 18$ Kpa.



As the similar specimen, so, c_u will be same.

Now, for unconfined compression test,

$\cancel{q_u = \frac{c_u}{2}}$ $\quad c_u = \frac{q_u}{2}$

$\therefore q_u = 2 \times 18 = \boxed{36 \text{ Kpa}}$

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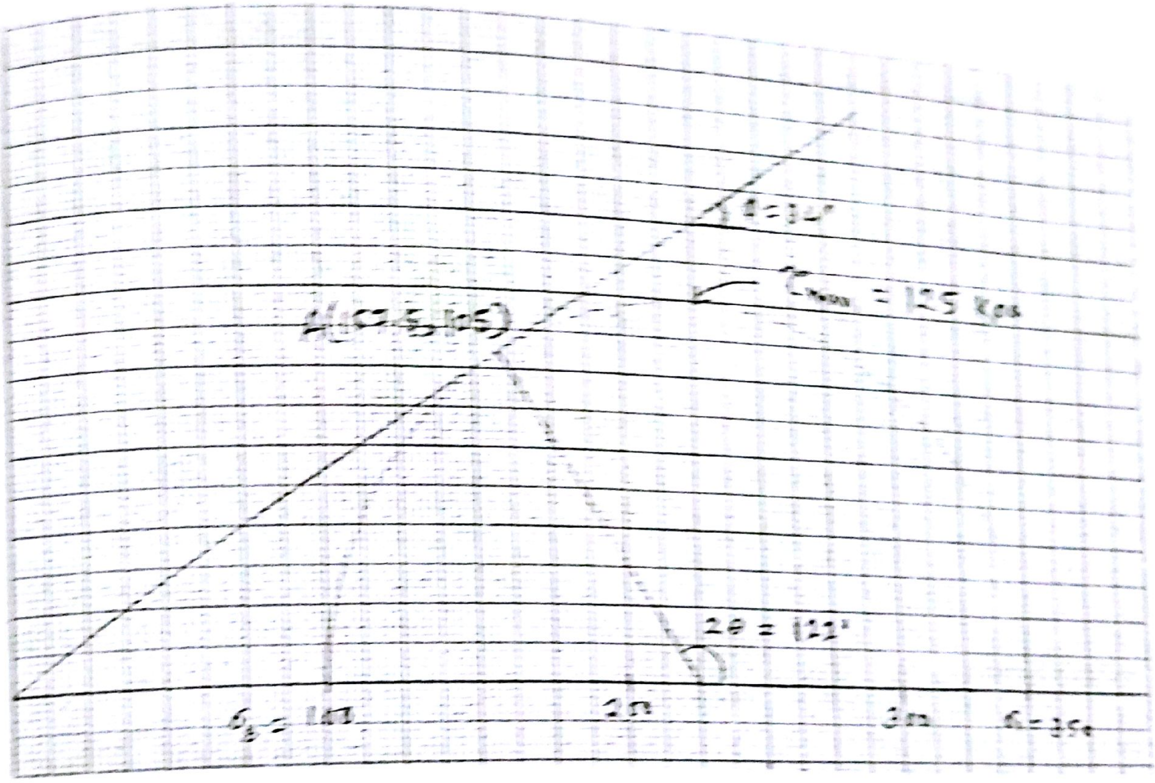
For saturated sand, $c = 0$.

Deviator stress, $\Delta\sigma_d' = 250$ kPa.

$\sigma_3' = 100$ kPa, $\sigma_1' = 250 + 100 = 350$ kPa.

For drained conditions, there is no pore water pressure.
So all the stresses are effective stress.



Now, $\sin\phi = \dfrac{(\sigma_1 - \sigma_3)/2}{(\sigma_1 + \sigma_3)/2} = \dfrac{350 - 100}{350 + 100}$

$\therefore \phi = 33.75°$

So, shear strength parameters, $\boxed{c = 0,\ \phi = 33.75°}$

From graph, $\phi = 34°$.

Theoretical inclination of failure plane, $\theta = 45 + \phi/2$

$$= 45 + \frac{33.75}{2}$$

$$\boxed{\theta = 61.88^\circ}$$

From graph, $2\theta = 122^\circ$ $\therefore$ $\theta = 61^\circ$

Shear stress on the failure plane, $\tau_f = \frac{\sigma_1' - \sigma_3'}{2} \sin 2\theta$

$$= \frac{350 - 100}{2} \times \sin(61.88 \times 2)$$

$$\therefore \boxed{\tau_f = 103.92 \text{ Kpa}}$$

maximum shear stress, $\tau_{max} = \cancel{\frac{\sigma_1' + \sigma_3'}{2}} \; \frac{\sigma_3' - \sigma_1'}{2}$

$$= \frac{350 - 100}{2}$$

$$\therefore \boxed{\tau_{max} = 125 \text{ Kpa}}$$

orientation $= \frac{90}{2} = \boxed{45^\circ}$

[2015 S] [4(b)]

$\sigma = \gamma h = 19.8 \times 15 = 297$ kPa

$\sigma' = (\gamma - \gamma_w) \times h = (19.8 - 9.81) \times 15 = 149.85$ kPa

a) The shearing stress builds up rapidly (CU test) — Q test

$\tau = C_{cu} + \sigma \tan \phi_{cu}$

$= 48.2 + 297 \times \tan 13 = \boxed{116.87 \text{ kPa}}$

b) The shearing stress builds up very slowly: (C-D test / S test)

$\tau' = C_d + \sigma' \tan \phi'_d$

$= 41.4 + 149.85 \times \tan 23$

$= \boxed{105 \text{ kPa}}$

[2012-13] [2(c)]

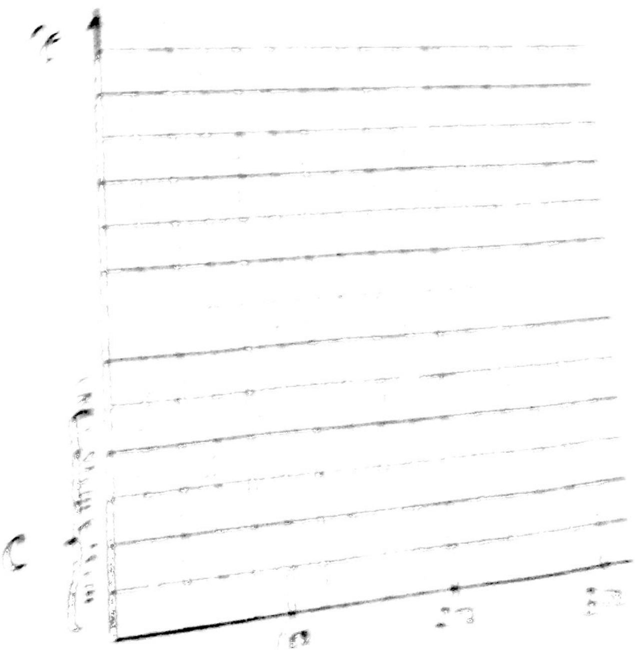
For saturated clay, $\phi = 0$

So, $C = \frac{\sigma_1 - \sigma_3}{2} = \frac{300 - 100}{2}$

$= 100$ kPa

So, strength parameters of the sample are:

$\boxed{\begin{array}{l} \phi = 0, \\ C = 100 \text{ kPa} \end{array}}$



009-10 [2008-09 | 3(a)]

Sample - 1

$\sigma_3 = 170$ Kpa

$\sigma_1 = 170 + 125 = 295$ Kpa

$\sigma_3' = 170 - 110 = 60$ Kpa

$\sigma_1' = 295 - 110 = 185$ Kpa

Sample - 2

$\sigma_3 = 430$ Kpa

$\sigma_1 = 310 + 430 = 7\cancel{8}40$ Kpa

$\sigma_3' = 430 - 270 = 160$ Kpa

$\sigma_1' = 7\cancel{8}40 - 270 = 4\cancel{2}70$ Kpa

Now, $\sigma_{\cancel{3}1} = \sigma_3 \tan^2(45 + \phi/2) + 2c \tan(45 + \frac{\phi}{2})$

$\Rightarrow \cancel{B}\, 295 = 170 \tan^2(45 + \phi/2) + 2c \tan(45 + \frac{\phi}{2})$ —— (1)

and, $7\cancel{8}40 = 430 \tan^2(45 + \frac{\phi}{2}) + 2c \tan(45 + \frac{\phi}{2})$ —— (2)

Solving (1) & (2), $\boxed{\phi = 15.21^\circ,\; c = 1.56 \text{ Kpa}}$

Again, $\sigma_{\cancel{3}1}' = \sigma_3' \tan^2(45 + \frac{\phi'}{2}) + 2c' \tan(45 + \frac{\phi'}{2})$

$185 = 60 \tan^2(45 + \frac{\phi'}{2}) + 2c' \tan(45 + \frac{\phi'}{2})$ —— (3)

$470 = 160 \tan^2(45 + \frac{\phi'}{2}) + 2c' \tan(45 + \frac{\phi'}{2})$ —— (4)

Solving (3) & (4), $\boxed{\phi' = 28.72^\circ,\; c' = 4.15 \text{ Kpa}}$

Now, $\Delta u = \Delta u_c + \Delta u_d$

$= B \Delta\sigma_3 + \bar{A}(\Delta\sigma_{\cancel{3}1} - \Delta\sigma_3)$

$\Rightarrow \Delta u = B \Delta\sigma_3 + AB(\Delta\sigma_1 - \Delta\sigma_3)$

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$\Rightarrow (270 - 119) = 1 \times (430 - 170) + A \times 1 \times [(740 - 295) - (430 - 170)]$

$\therefore \boxed{A = -0.54}$

$\boxed{\text{Ans} \rightarrow 4(b)}$

for sand, $c = 0$.

$\sigma = \gamma h = 19.8 \times 4.5 = 89.1$ Kpa.

shear strength, $\tau = c + \sigma \tan\phi$

$= 0 + 89.1 \tan 32 = 55.68$ Kpa.

So, $\boxed{\tau_{capacity} = 55.68 \text{ Kpa}}$

After load application,

applied $\sigma = 89.1 + 65 = 154.1$ Kpa

applied $\tau = 55.68 + 50 = 105.68$ kpa.

$\tau_{(capacity)} = 154.1 \tan 32 = 96.3$ Kpa < 105.68 Kpa.

As the applied shear is greater than the shear capacity, so the structure will fail.

confirm

For saturated clay $\phi = 0$

$\sigma_3 = 100$ kPa $\sigma_1 = 290$ kPa.

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2007-08 . 1 (g)

For sand sample, $c = 0$

$\sigma = \dfrac{0.36}{36\times10^{-4}} = 100\,\text{Kpa}$, $\tau = \dfrac{0.18}{36\times10^{-4}} = 50\,\text{Kpa}$.

Now, $\tau = c + \sigma \tan\phi$

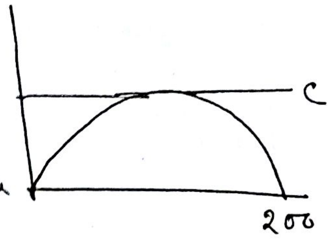
$\Rightarrow 50 = 0 + 100 \tan\phi$

$\therefore \boxed{\phi = 26.57^\circ}$

confirm

2007-08 | 2(b)

Unconfined compressive:

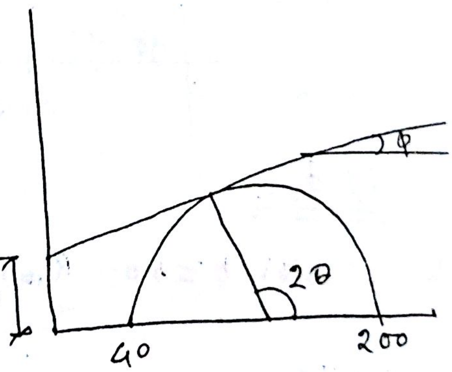


unconfined compressive strength, $q_u = 200$ Kpa

$\therefore c = \dfrac{200}{2} = 100\,\text{Kpa}$.

Triaxial compression test:

here, $\sigma_1 = 200$, $\sigma_3 = 40$ Kpa.



Now, $\sigma_1 = \sigma_3 \tan^2(45+\frac{\phi}{2}) + 2c\tan(45+\phi/2)$

$\Rightarrow 200 = 40\tan^2(45+\phi/2) + 200\tan(45+\phi/2)$

$\therefore \tan(45+\phi/2) = 0.854, -5.85$

$\Rightarrow 45+\phi/2 = 40.5$

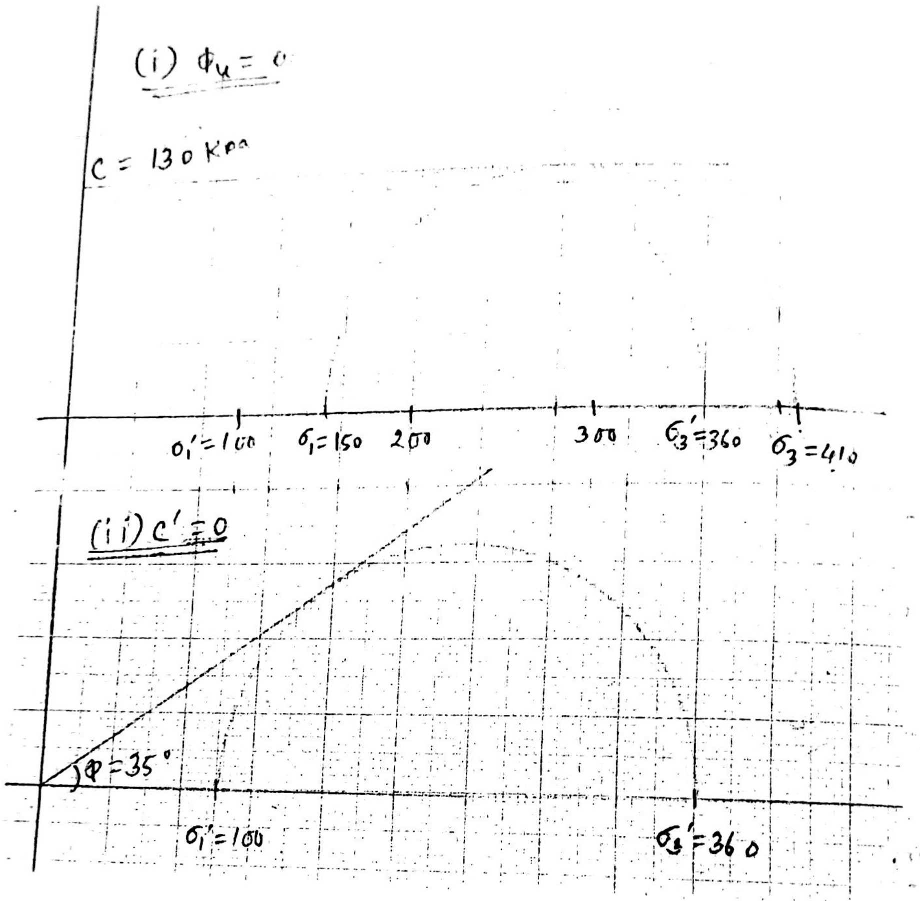
$\therefore \boxed{\phi = -9^\circ}$

There might be some correction in the question paper

07-08. 4(b)

Given, $\sigma_3 = 150$ Kpa, $\Delta\sigma_d = 260$ Kpa, $\sigma_1 = 410$ Kpa

Now, $\sigma_3' = 100$ Kpa, $\sigma_1' = 360$ Kpa.



(i) $\phi_u = 0$:

when $\phi = 0$, $C_u = \frac{\sigma_1 - \sigma_3}{2} = \frac{410 - 150}{2} = \boxed{130 \text{ Kpa}}$

(ii) $C' = 0$:

when $c' = 0$, take all effective stress.

Now, $\sin\phi' = \frac{(\sigma_1' - \sigma_3')/2}{(\sigma_1' + \sigma_3')/2} = \frac{360 - 100}{360 + 100}$

$\therefore \boxed{\phi' = 34.4°}$

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2006-07 | 1(c)

Specimen - 1

$\sigma_3 = 250$ kpa,

$\Delta\sigma_d = 750$ kpa.

$\sigma_1 = 750 + 250 = 1000$ kpa

Sepecimen - 2

$\sigma_3 = 400$ Kpa

~~$\Delta\sigma_d$ = 1600 kpa~~

$\sigma_1 = 1600$ Kpa ~~2000 kpa.~~

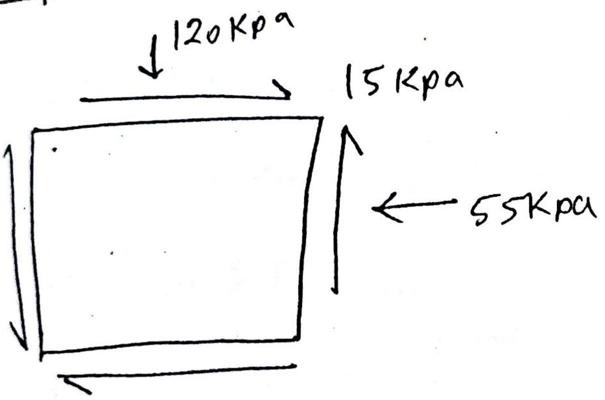
Now $\sigma_1 = \sigma_3 \tan^2(45 + \frac{\varphi}{2}) + 2c \tan(45 + \frac{\varphi}{2})$

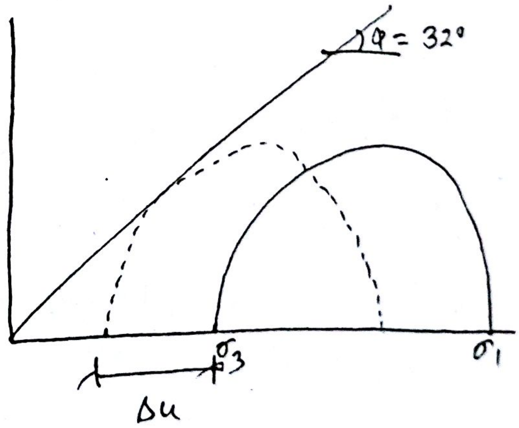
$\Rightarrow 1000 = 250 \tan^2(45 + \frac{\varphi}{2}) + 2c \tan(45 + \frac{\varphi}{2})$ —(1)

and, $\cancel{2000}\ 1600 = 400 \tan^2(45 + \varphi/2) + 2c \tan(45 + \frac{\varphi}{2})$ —(2)

solving 1 & 2, $\boxed{\varphi = 36.87°,\ c = 0}$

2006-07 | 2(b)





$$\sigma_{(\text{principle planes})} = \frac{\sigma_1 + \sigma_3}{2} \pm \sqrt{\left(\frac{\sigma_1 - \sigma_3}{2}\right)^2 + \tau_{xy}^2}$$

$$= \frac{120 + 55}{2} \pm \sqrt{\left(\frac{120 - 55}{2}\right)^2 + 15^2}$$

$$= 123.3,\ 51.7 \text{ Kpa.}$$

as $\sigma_3 < \sigma_1$ so, $\sigma_3 = 51.7$ Kpa. $\sigma_1 = 123.3$ kpa.

Now, $\sigma_1' = \sigma_3' \tan^2(45 + \frac{\phi}{2}) + 2c \tan(45 + \frac{\phi}{2})$

$\Rightarrow \cancel{123.3 = 51.7 \tan^2(45 + \frac{\phi}{2}) + 0}$

$\Rightarrow 123.3 - \Delta u = (51.7 - \Delta u) \tan^2(45 + \frac{32}{2})$

$\therefore \Delta u = 19.94$ Kpa.

So, change in pore water pressure $= \boxed{19.94 \text{ Kpa}}$

2006-07 | 3(b)

2004-05 | 5(e)

$$T = \pi Cu \left(\frac{d^2 h}{2} + \frac{a d^3}{4} \right)$$ [Cu = undrained shear strength]

Now $Cu \propto T$

$\therefore$ sensitivity $= \frac{Cu(\text{undisturbed})}{Cu(\text{remoulded})} = \frac{T_{\text{undisturbed}}}{T_{\text{remoulded}}}$

$= \frac{45}{20} = 2.25$.

so, sensitivity is in between 2 & 4. $\therefore$ the soil is medium sensitive. $\therefore$ Remoulded strength is a good approximation ~~of~~ for actual strength.

2004-05 | 6(b)

$c' = 25 \text{ kPa} > 0$, so over consolidated clay.

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

$\Rightarrow 90 = 25 + \sigma' \tan 27$

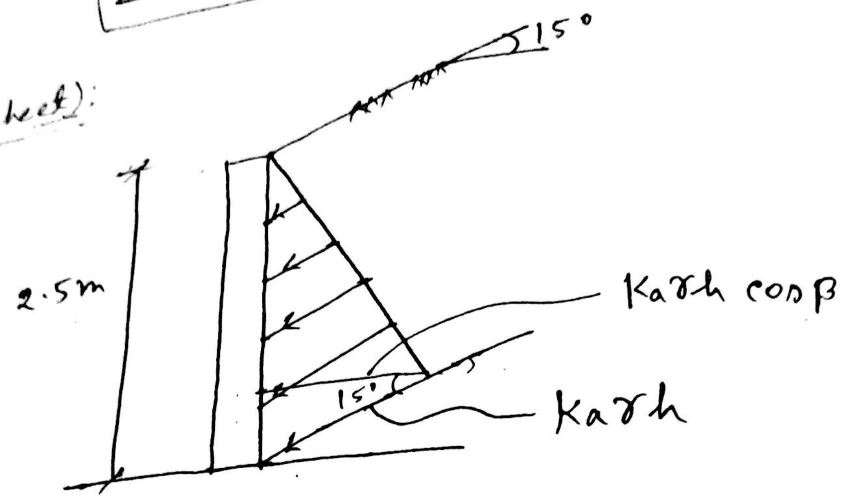
$\therefore \sigma' = 128 \text{ kPa}.$

$\therefore$ ultimate pore pressure, $u = \sigma - \sigma' = 180 - 128 = \boxed{52 \text{ kPa}}$ Ans

2004-05 | 8(b)

Lateral Earth Pressure

Example (sheet):



Given that, $H = 2.5\,m$, $\phi = 35°$, $\gamma = 18\,KN/m$, $OCR = 2$.

$\beta = 15°$.

Let, the ~~b~~ earth pressure is at rest

so, $K_0 = (1 - \sin\phi)(OCR)^{\sin\phi}(1 + 0.5\tan\beta)^2$

$= (1 - \sin 35) \times 2^{\sin 35} \times (1 + 0.5 \tan 15)^2$

$= 0.816$

Total force, $P_0 = \frac{1}{2} \times K_a \gamma h \times \cos\beta \times h$

$= \frac{1}{2} \times 0.816 \times 18 \times 2.5 \times \cos 15 \times 2.5$

$= 44.34\ KN/m$

Now, horizontal thrust, $P_H = P_0 \cos\beta$

$= 44.34 \cos 15$

$= \boxed{42.83\ KN/m}$

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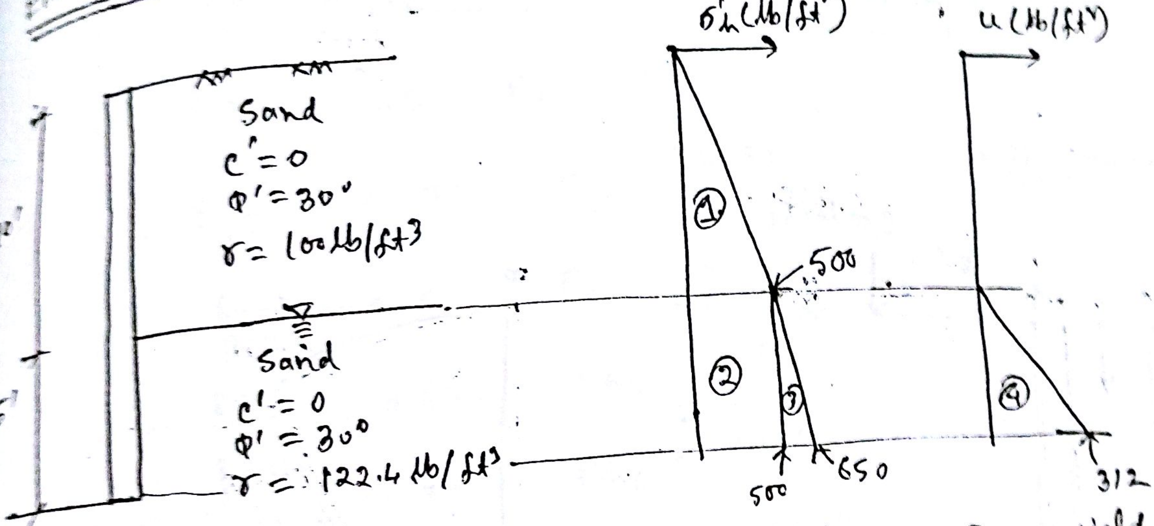
From Alpan's formula,

$$K_0 = [0.19 + 0.233 \log I_p] \times \sqrt{OCR} \times (1+ \cancel{\tan}\ 0.5 \tan\beta)^2$$
$$= (1-\sin\phi) \times 2^{0 \sin\phi} \times (1+0.5\tan\beta)^2$$

$$\Rightarrow [0.19 + 0.233 \log I_p] \times \sqrt{OCR} = (1-\sin 35) \times 2^{\sin 35}$$

$\therefore$ $\boxed{I_p = 12.9}$ (plasticity index)

Example 12.1 (B.M. Das



~~K_0 = 1-~~ As, the wall is restrained from yield,

so, ~~it is it~~ earth pressure is at rest.

Now, $K_0 = \dfrac{1-\sin 30}{\cancel{1+\sin 30}} = \cancel{\dfrac{1}{3}}\ \dfrac{1}{2}$

at $z=10$, $\sigma_1' = K_0 \gamma h = \frac{1}{2} \times 100 \times 10 = 500$

$u_1 = 0$

at $z = 15$, $\sigma_1' = 500 + \frac{1}{2} \times (122.4 - 62.4) \times 5$

$= 650$ lb/ft³

$u_2 = K_0 \gamma h = 1 \times 62.4 \times 5 = 312$ lb/ft³.

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total lateral force $= \frac{1}{2} \times 500 \times 10 + 5 \times 500 + \frac{1}{2} \times 150 \times 5 + \frac{1}{2} \times 5 \times 312$

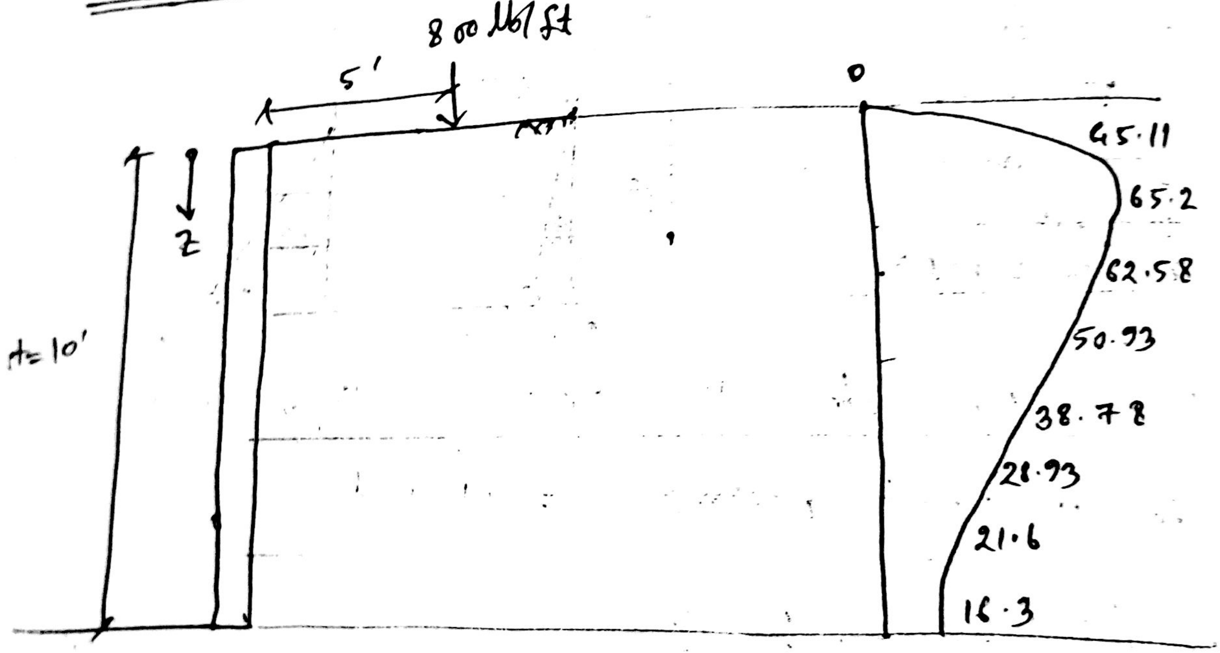
$$\boxed{P_o = 6155 \text{ lb/ft}}$$

Loch Location, $\bar{x}$ from the ground (bottom wall),

$$\bar{x} = \frac{\frac{1}{2} \times 500 \times 10 \times \left(\frac{10}{3} + 5\right) + 5 \times 500 \times 2.5 + \frac{1}{2} \times 150 \times 5 \times \frac{5}{3} + \frac{1}{2} \times 5 \times 312 \times \frac{5}{3}}{6155}$$

$= \boxed{4.71 \text{ ft (from the bottom)}}$ Ans

Example - 12.2



For line load, $m = \frac{x}{H} = \frac{5}{10} = 0.5$

So, $m = 0.5 > 0.4$

so, $P_h = \cancel{\frac{1.77Q}{H^2}\left(\frac{m^2 n^2}{(m^2+n^2)^3}\right)}$ & $\frac{4q}{\pi H}\frac{m^2 n}{(m^2+n^2)^2}$

$= \cancel{\frac{1.77Q}{10^2}\left(\frac{0.05^2 n^2}{(0.5^2+n^2)^3}\right)} = \frac{4 \times 800}{\pi \times 10}\frac{0.5^2 n}{(0.5^2+n^2)^2}$

let's divide the height into 8 equal parts.

so ~~n = 8, 1.25~~, $z = 0, 1.25, 2.5, 3.75, 5, 6.25, 7.5, 8.75, 10$

$n = \frac{z}{H}$

for, $n = \frac{0}{10} = 0$, $P_{h_0} = 0$

$n = \frac{1.25}{10} = 0.125$, $P_{h_1} = 45.11$

$n = \frac{2.5}{10} = 0.25$, $P_{h_2} = 65.2$

$n = \frac{3.75}{10} = 0.375$, $P_{h_3} = 62.58$

$n = \frac{5}{10} = 0.5$, $P_{h_4} = 50.93$

$n = \frac{6.25}{10} = 0.625$, $P_{h_5} = 38.78$

$n = \frac{7.5}{10} = 0.75$, $P_{h_6} = 28.93$

$n = \frac{8.75}{10} = 0.875$, $P_{h_7} = 21.6$

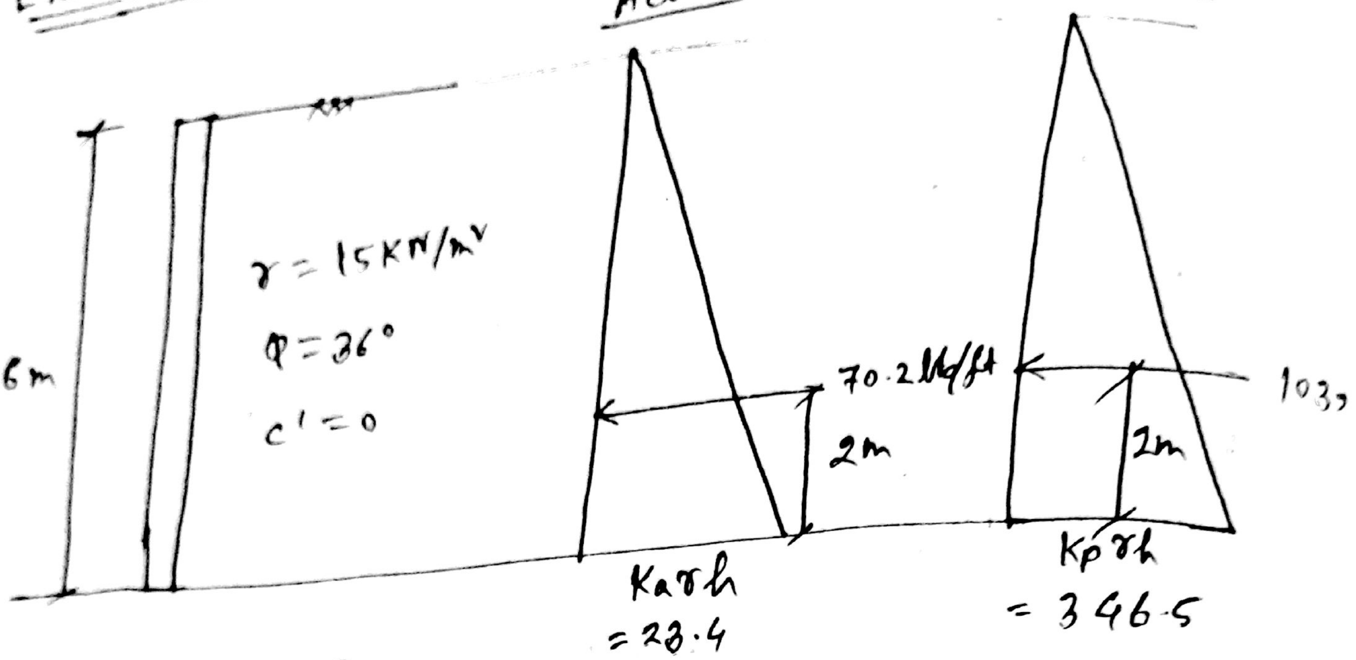
$n = \frac{10}{10} = 1$, $P_{h_8} = 16.3$

∴ Total increase in lateral force

$= \frac{1}{2} \times 1.25 \times [0 + 16.3 + 2 \times (45.11 + 65.2 + 62.58 + 50.93 + 38.78 + 28.93 + 21.6)]$

$= \boxed{401.6 \text{ lb/ft}}$

Example - 12.3



a) Active force

for cohesionless soil, $K_a = \frac{1-\sin\phi}{1+\sin\phi} = \frac{1-\sin 36}{1+\sin 36}$

$\therefore K_a = 0.26$

Now, $P_{ae} = \frac{1}{2} \times K_a \gamma h \times h$

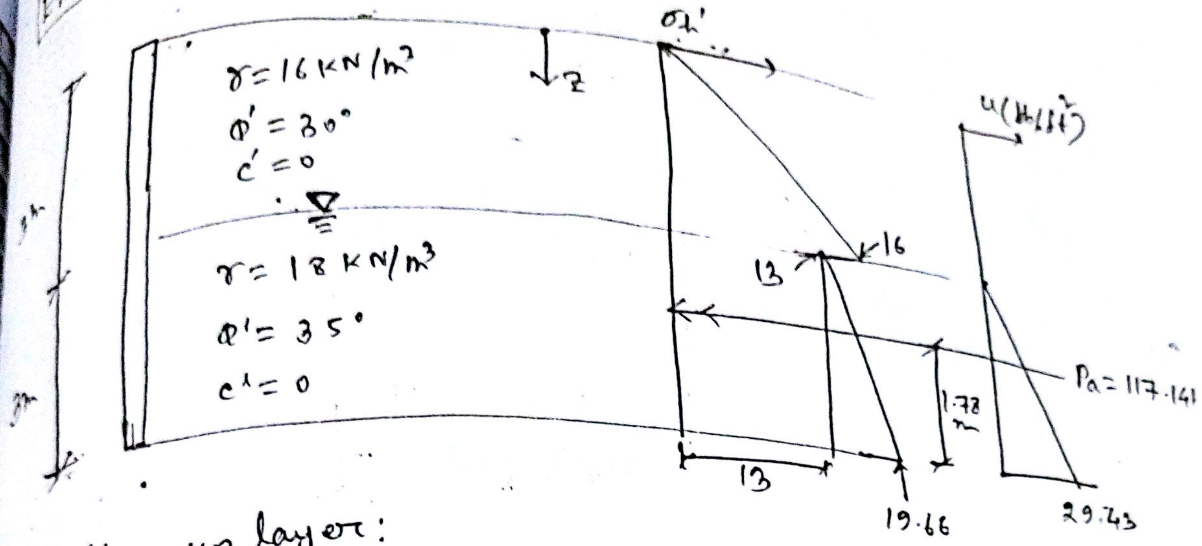
$= \frac{1}{2} \times 0.26 \times 15 \times 6^2 = \boxed{70.2 \text{ lb/ft}}$

b) passive force:

$K_p = \frac{1+\sin\phi}{1-\sin\phi} = 3.85$

$\therefore P_{pe} = \frac{1}{2} \times 3.85 \times 15 \times 6 \times 6 = \boxed{1039.5 \text{ lb/ft}}$

Example - 12.4



<u>For the up layer:</u>

$$K_a = \frac{1 - \sin 30}{1 + \sin 30} = 1/3$$

$$\sigma = K_a \gamma h = \frac{1}{3} \times 16 \times 3 = 16$$

<u>For the bottom layer:</u>

$$K_a = \frac{1 - \sin 35}{1 + \sin 35} = 0.271$$

at, $z = 3m$, $\sigma = K_a \gamma h = 0.271 \times 16 \times 3 = 13$

at, $z = 6m$, $\sigma = 13 + K_a \gamma' h = 13 + 0.271 \times (18 - 9.81) \times 3$

$= 19.66\ kN/m^2$

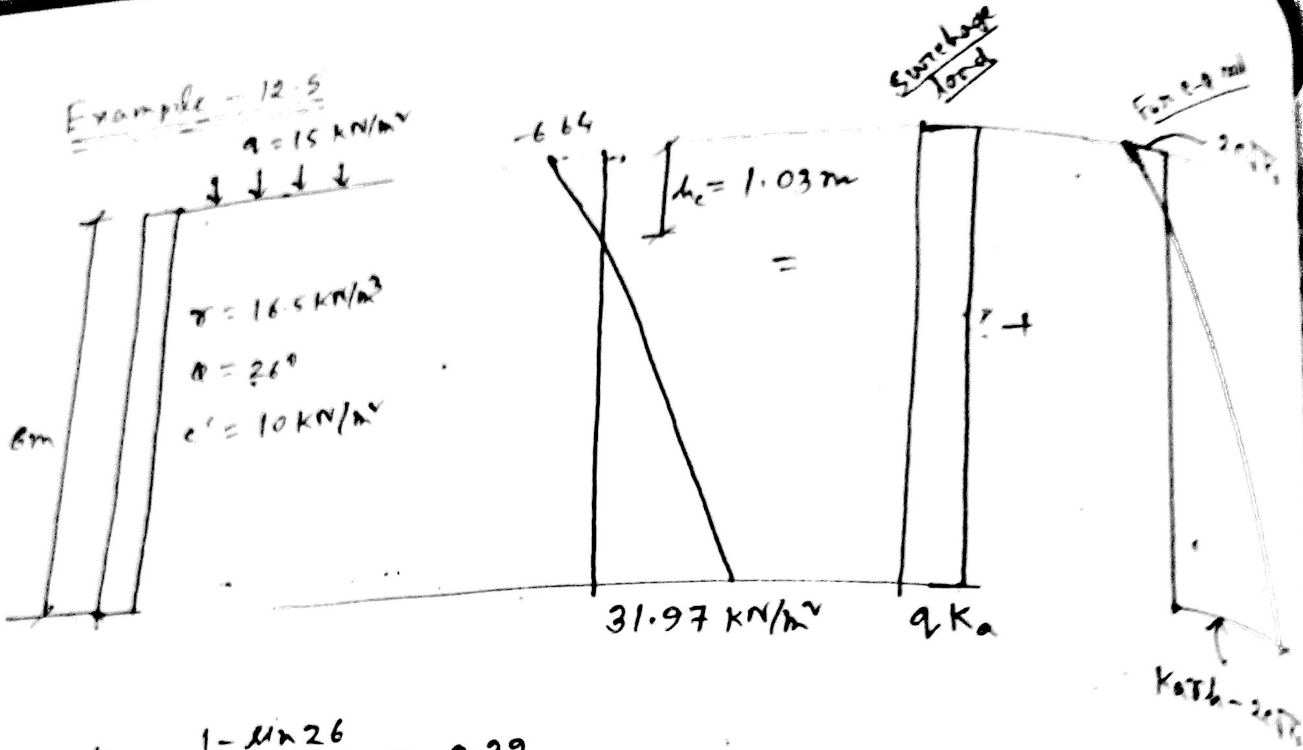
$u = 1 \times 9.81 \times 3 = 29.43\ kN/m^2$

$\therefore$ Force per unit width $= \frac{1}{2} \times 16 \times 3 + 13 \times 3 + \frac{1}{2} \times (19.66 - 13) \times 3 + \frac{1}{2} \times 29.43 \times 3$

$= \boxed{117.14\ kN/m}$

$$\bar{z} = \frac{\frac{1}{2} \times 16 \times 3 \times (3+1) + 13 \times 3 \times 1.5 + \frac{1}{2} \times (19.66 - 13) \times 3 \times 1 + \frac{1}{2} \times 29.43 \times 3 \times 1}{117.14}$$

$= \boxed{1.78\ m}$



$K_a = \frac{1-\sin 26}{1+\sin 26} = 0.39$

at $z = 0$, $\sigma_a' = qK_a - 2c\sqrt{K_a}$

$= 15 \times 0.39 - 2 \times 10 \times \sqrt{0.39} = -6.64$ kN/m²

at $z = 6$m, $\sigma_a' = qK_a + K_o\gamma h - 2c\sqrt{K_a}$

$= 15 \times 0.39 + 0.39 \times 16.5 \times 6 - 2 \times 10 \times \sqrt{0.39}$

$= 31.97$ kN/m²

Now, $\frac{31.97}{6-h_c} = \frac{6.64}{h_c}$ $\therefore h_c = 1.03$ m

$\therefore$ Active force $= \frac{1}{2} \times 31.97 \times (6-1.03)$ $\cancel{- \frac{1}{2} \times 1.03 \times 6.64}$

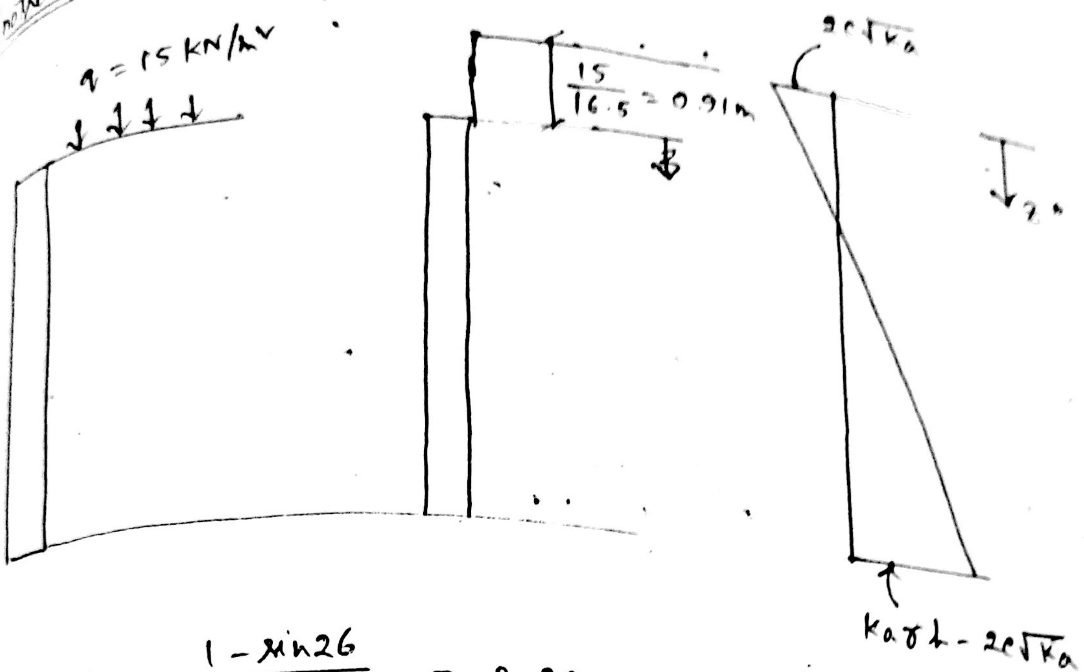
$= 79.45$ kN/m

After the tensile crack occurs,

$\boxed{P_a = 79.45 \text{ kN/m}}$

easier method

Another approach:



$$K_a = \frac{1-\sin 26}{1+\sin 26} = 0.39$$

at $z=0$, $\sigma_a' = 2 \times 10 \times \sqrt{0.39} = -12.49\ KN/m^2$

at $z = 6+0.91 = 6.91\ m$, $\sigma_a' = K_a \gamma h - 2c\sqrt{K_a}$

$$= 0.39 \times 16.5 \times 6.91 - 12.49$$

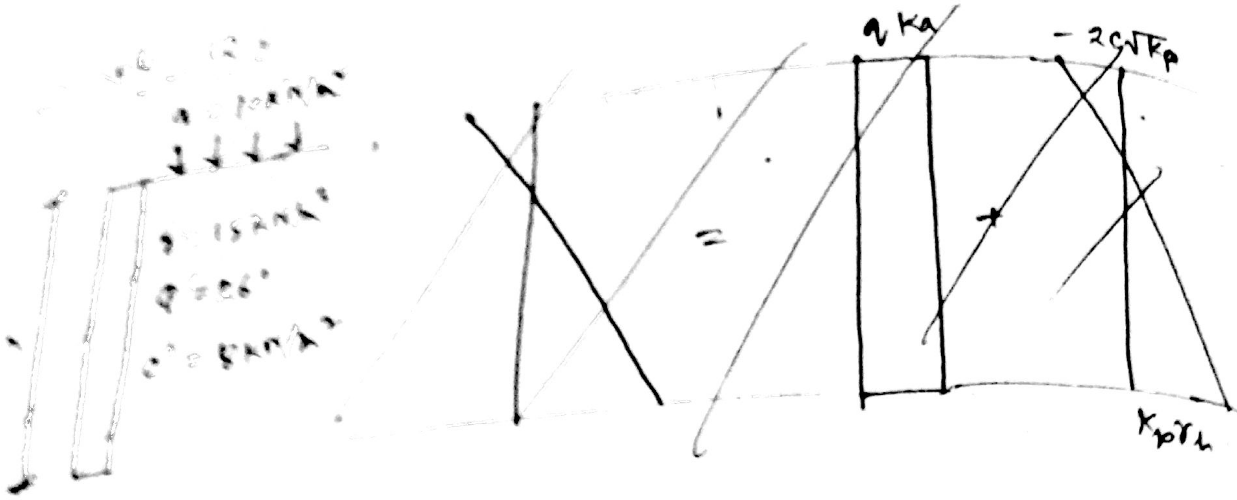
$$= 31.97\ KN/m^2$$

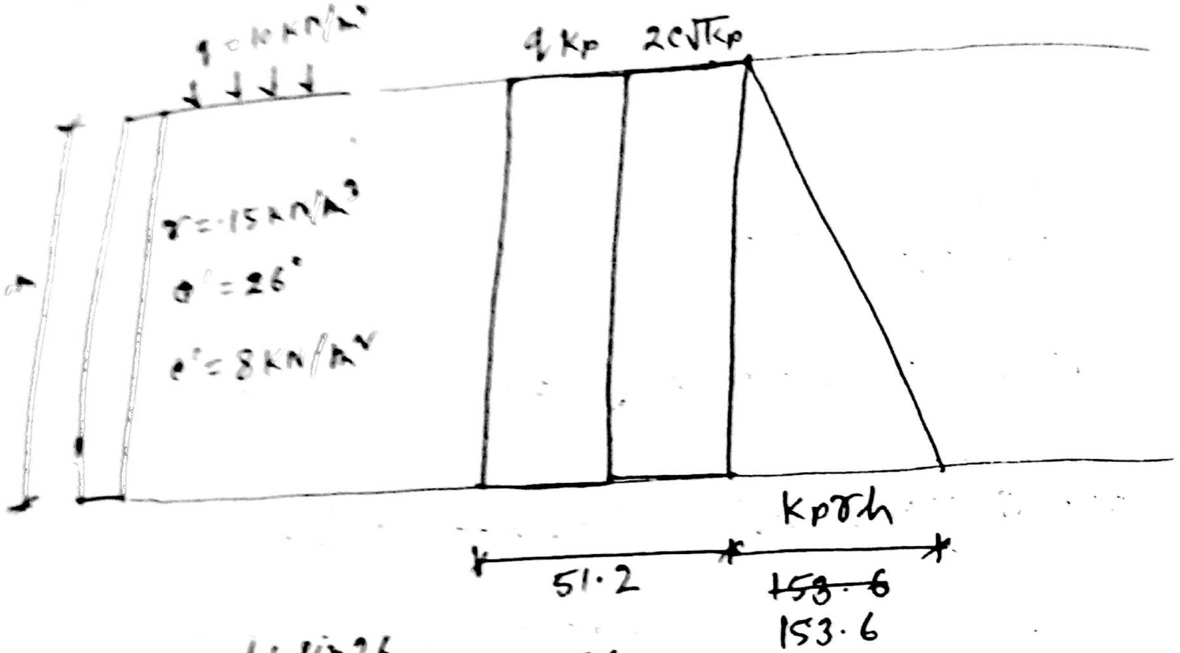
Now $\frac{31.97}{6.91-h_c} = \frac{12.49}{h_c}$ $\therefore h_c = 1.94$

$\therefore$ Active force $= \frac{1}{2} \times 31.97 \times (6.91 - 1.94)$

$$= \boxed{79.45\ KN/m}$$

Ans





$$K_p = \frac{1+\sin 26}{1-\sin 26} = 2.56$$

at $z=0$, $\sigma_p' = qK_p + 2c\sqrt{K_p} = 10 \times 2.56 + 2 \times 8 \times \sqrt{2.56}$

$= 51.2$ Kpa

at $z = 6$ m, $\sigma_p' = 51.2 + \cancel{153.6} = K_p\gamma h = 51.2 + 2.56 \times 15 \times 4$

$= 204.8$ Kpa.

∴ Passive resistance per unit width,

$$P_p = 51.2 \times 4 + \frac{1}{2} \times 153.6 \times 4 = \boxed{512 \text{ KN/m}}$$

$$\bar{z} = \frac{51.2 \times 4 \times 2 + \frac{1}{2} \times 153.6 \times 4 \times \frac{4}{3}}{512} = \boxed{1.6 \text{ m}}$$

Ans

Example 12.7. A 15 ft height retaining wall with a granular backfill. Given that, $\gamma = 100\ lb/ft^2$, $\phi = 35°$, $\theta = 5°$, $\delta = 10°$. Determine active thrust per foot of length of the wall.

Ans: For this problem, $\psi = 90 - \theta - \delta$

$= 90 - 5 - 10 = 75°$.

~~$\Delta ABC_1 = \frac{1}{2} \times 4.4 \times 17.9 = 39.38$~~

$\Delta ABC_1 = (\text{Area of } ABC_1) \times \gamma \times 1$

$= \frac{1}{2} \times 4.4 \times 17.9 \times 100 = 3938\ lb$

$\Delta ABC_2 = \frac{1}{2} \times 2.4 \times 18.6 \times 100 + 3938 = 6170\ lb$

$\Delta ABC_3 = 6170 + \frac{1}{2} \times 2.2 \times 19.5 \times 100 = 8315\ lb$

$\Delta ABC_4 = 8315 + \frac{1}{2} \times 2.1 \times \cancel{22.2}\ 20.8 \times 100 = 10499\ lb$

$\Delta ABC_5 = 10499 + \frac{1}{2} \times 2 \times 22.2 \times 100 = 12719\ lb$

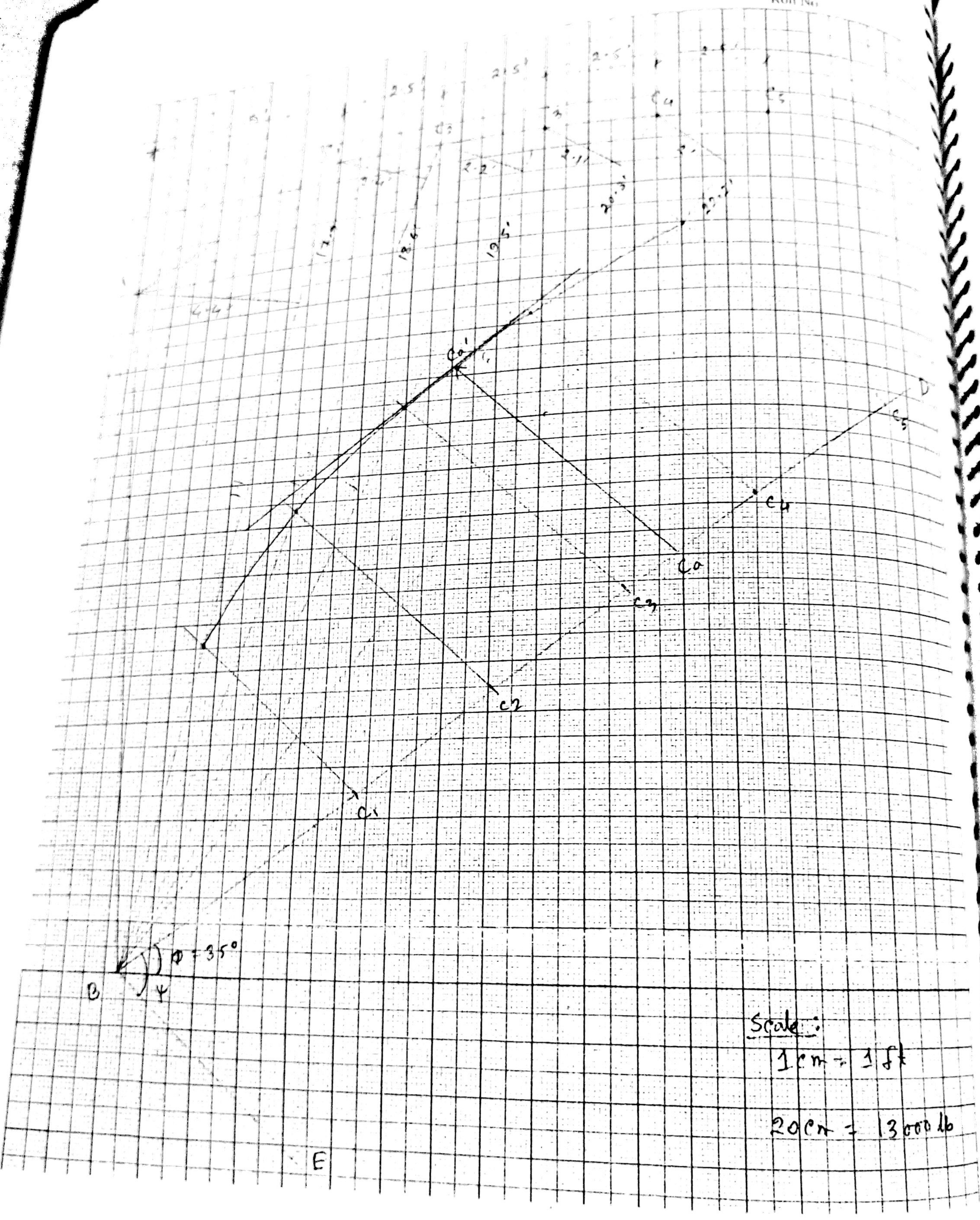
$\therefore BC_1 = 3938\ lb$, $BC_2 = 6170\ lb$, $BC_3 = 8315\ lb$,

$BC_4 = 10499\ lb$, $BC_5 = 12719\ lb$.

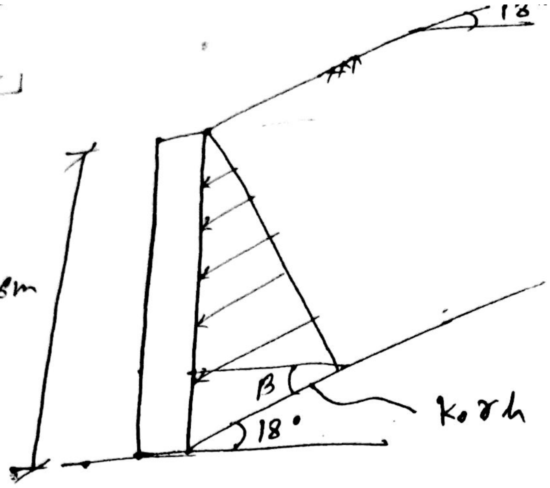
The active thrust per unit length of the wall

is $6.3\ cm = \frac{6.3}{20} \times 1300 = \boxed{4095\ lb}$

Roll No



C_0'
C_0
C_1
C_2
C_3
C_4
C_5
D
B
E
$\phi = 35°$
ψ
2.5
2.5'
2.5'
19.5'
20.3'
22.5'
Scale :
1 cm = 1 ft
20 cm = 1300 lb



$c = 0$
$\phi = 33°$
$\gamma = 18\ kN/m^3$
$OCR = 3$

(i) wall restricted from any movement:

$$K_o = (1-\sin\phi)(OCR)^{\sin\phi}(1+0.5\tan\beta)^2$$

$$= (1-\sin 33)\, 3^{\sin\phi}\,(1+0.5\tan 18)^2$$

$$= 1.12$$

Total thrust, $P_o = \frac{1}{2} \times K_o \gamma h \times h \times \cos\beta$

$$= \frac{1}{2} \times 1.12 \times 18 \times 6^2 \times \cos 18 = 345.12\ kpa$$

$\therefore$ Horizontal thrust, $P_{h_o} = 345.12 \times \cos 18 = \boxed{328.2\ kpa}$

(ii) backfill soil outward and downward direction:

$$\cancel{K_a = \frac{1-\sin\phi}{1+\sin\phi} = 0.2}$$

$$K_a = \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}} = \frac{\cos 18 - \sqrt{\cos^2 18 - \cos^2 33}}{\cos 18 + \sqrt{\cos^2 18 - \cos^2 33}} = 0.36$$

~~Total thrust, $P_a = \frac{1}{2} K_a \gamma h^2 - 2cH\sqrt{K_a}$~~

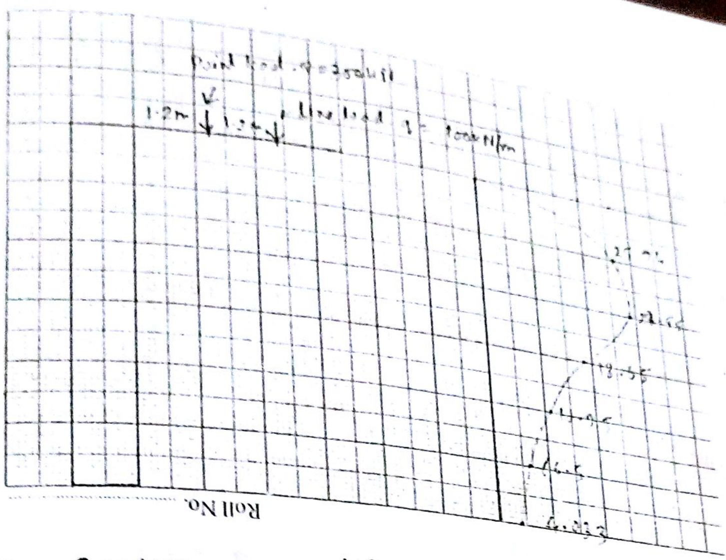
~~$= \frac{1}{2} \times 0.36 \times 18 \times 6^2 - 0 =$~~

Total thrust, $P_a = \frac{1}{2} \times h \times K_a \gamma h \cos\beta = 110.93$

$\therefore$ horizontal active thrust, $P_{a_h} = 110.93 \times \cos 18$

$$= \boxed{105.5\ Kpa}$$

2010-11 | 4(a)



For point load, $Q = 300\,KN$, $m = \frac{1.2}{6} = 0.2 < 0.4$

$$\therefore P_h = \frac{0.28Q}{H^2} \frac{n^2}{(0.16+n^2)^3}$$

For line load, $q = 100\,KN/m$, $m = \frac{2.4}{6} = 0.4 \leq 0.4$

$$\therefore P_h = \frac{4q}{\pi H} \frac{0.4^2 n}{(\cancel{2}\,0.4^2+n^2)^2} = \frac{\cancel{0.5}\,4 \times q}{\cancel{4}\,\pi H} \frac{0.16n}{(0.16+n^2)^2}$$

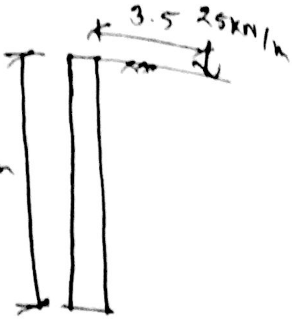
Let's divid the total depth into 6 areas.

for, $z = 1, 2, 3, 4, 5, 6\,m$, $n = \frac{z}{H} = \frac{1}{6}, \frac{1}{3}, 0.5, \frac{2}{3}, \frac{5}{6}, 1\,m$.

value of n	P_h (for point load)	P_h (line load)	Total P_h
$\frac{1}{6}$	~~1.51~~ 9.89	16.05	25.94
1/3	13.15	15.4	28.55
0.5	8.55	10.1	18.65
2/3	4.75	6.2	10.95
5/6	2.63	3.87	6.5
1	1.51	2.52	4.033

N.B lateral pressure

[Lateral earth pressure should be added as it mentioned that lateral pressure distribution. So, $K_a \gamma h$ must be added to the individual point and three loads should be added to get total P_h.]



$m = \frac{x}{H} = \frac{3.5}{6} = \frac{7}{12} > 0.4$

so, $P_h = \frac{4q}{\pi H} \frac{m^2 n}{(m^2 + n^2)^2}$

Lets divide the depth into 6 strips.

Depth (z)	$n = \frac{z}{H}$	$P_h = \frac{4 \times 25}{\pi \times 6} \times \frac{m^2 n}{(m^2 + n^2)^2}$
0	0	0
1	1/6	2.22
2	1/3	2.953
3	1/2	2.59
4	2/3	1.95
5	5/6	1.41
6	1	1

total lateral thrust = area under graph × unit thickness.

using trapezoidal rule.

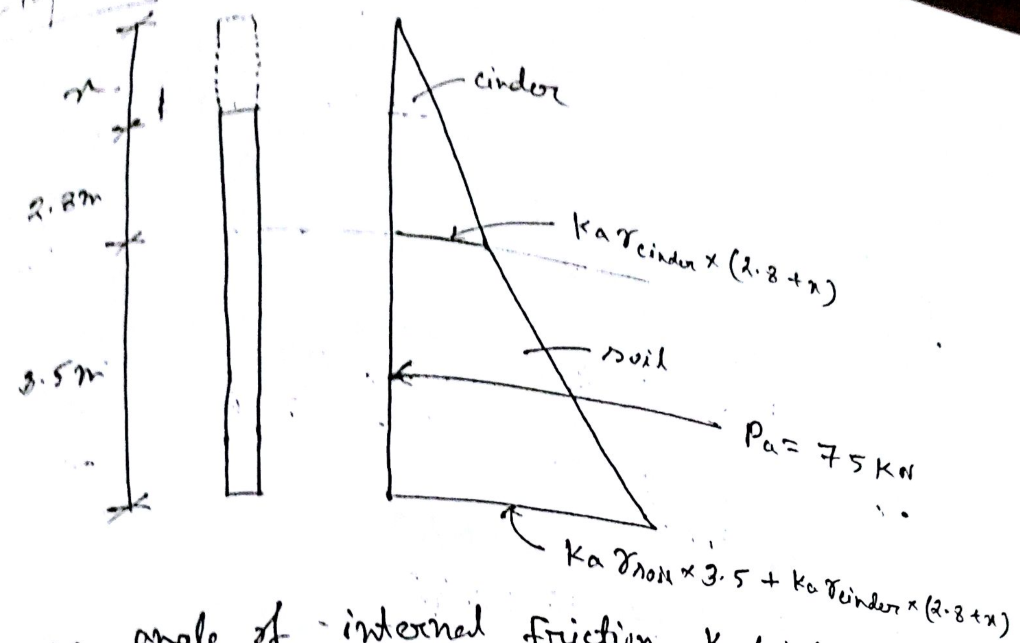
total lateral thrust

$$= \frac{1}{2} \times 1 \times \left[0 + 1 + 2 \times (2.22 + 2.953 + 2.59 + 1.95 + 1.41)\right]$$

$$= \boxed{11.623 \text{ KN/m}}$$

confirm

2009-10 | 3 (b)



for same angle of internal friction, Ka (cinder) = Ka (soil)

Now, <u>before adding cinder:</u>

$$\frac{1}{2} \times K_a \gamma_{soil} h \times h = 75$$

$$\Rightarrow K_a = \frac{75 \times 2}{6.3^2 \times 16.2} = 0.233$$

let. addition height = x m.

<u>after adding cinder:</u>

$$\cancel{\frac{1}{2} \times K_a \times \gamma \times (2.8+x) \times \frac{2.8+x}{3}}$$

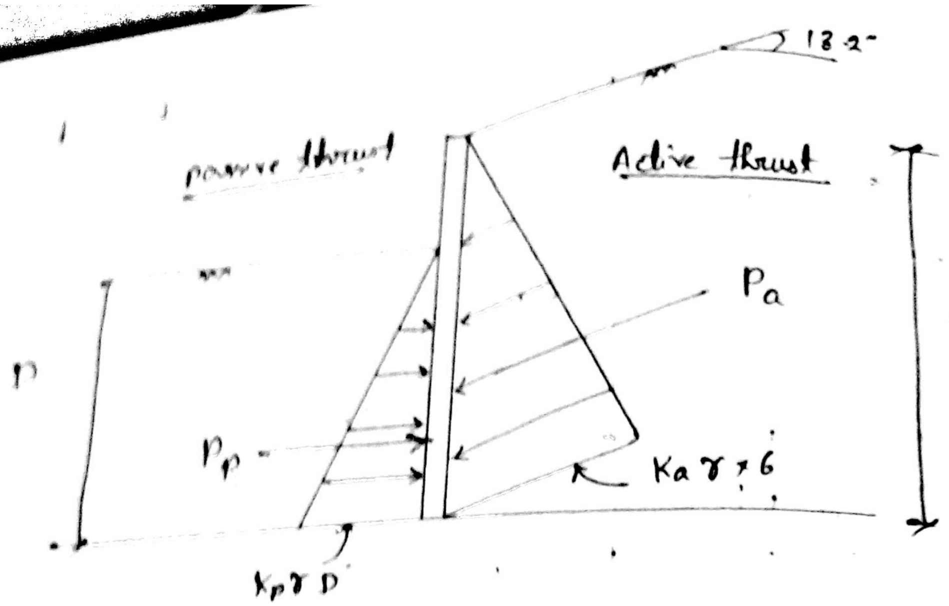
$$\frac{1}{2} \times K_a \gamma_c (2.8+x)(2.8+x) + K_a \gamma_c (2.8+x) \times 3.5$$
$$+ \frac{1}{2} \times K_a \gamma_s \times 3.5 = 75$$

$$\Rightarrow \frac{1}{2} \times 0.233 \times 8.2 \times (2.8+x)^2 + 0.233 \times 8.2 \times 3.5 \times (2.8+x)$$
$$+ \frac{1}{2} \times 0.233 \times 16.2 \times 3.5^2 - 75 = 0$$

$$\therefore 2.8 + x = \cancel{4.65,} \; 4.658, -11.66$$

$$\therefore x = 1.86 \text{ or } -14.46 \text{. but } x > 0. \therefore \boxed{x = 1.86\text{ m}}$$

confirm



here, $c = 0$, $\phi = 30°$, $\gamma = 17\ kN/m^3$

Now, $K_a = \dfrac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}} = \dfrac{\cos 18.2 - \sqrt{\cos^2 18.2 - \cos^2 30}}{\cos 18.2 + \sqrt{\cos^2 18.2 - \cos^2 30}}$

$= 0.417$

$K_p = \dfrac{1+\sin\phi}{1-\sin\phi} = \dfrac{1+\sin 30}{1-\sin 30} = 3$

For back fill :

active horizontal thrust, $P_{h,a} = \frac{1}{2} \times K_a \gamma h \times h \times \cos^2 18.2$

$= \frac{1}{2} \times 0.417 \times 6^2 \times \cos^2 18.2 = 6.774\ kN \times 17$
$\times 17$

For front side ground,

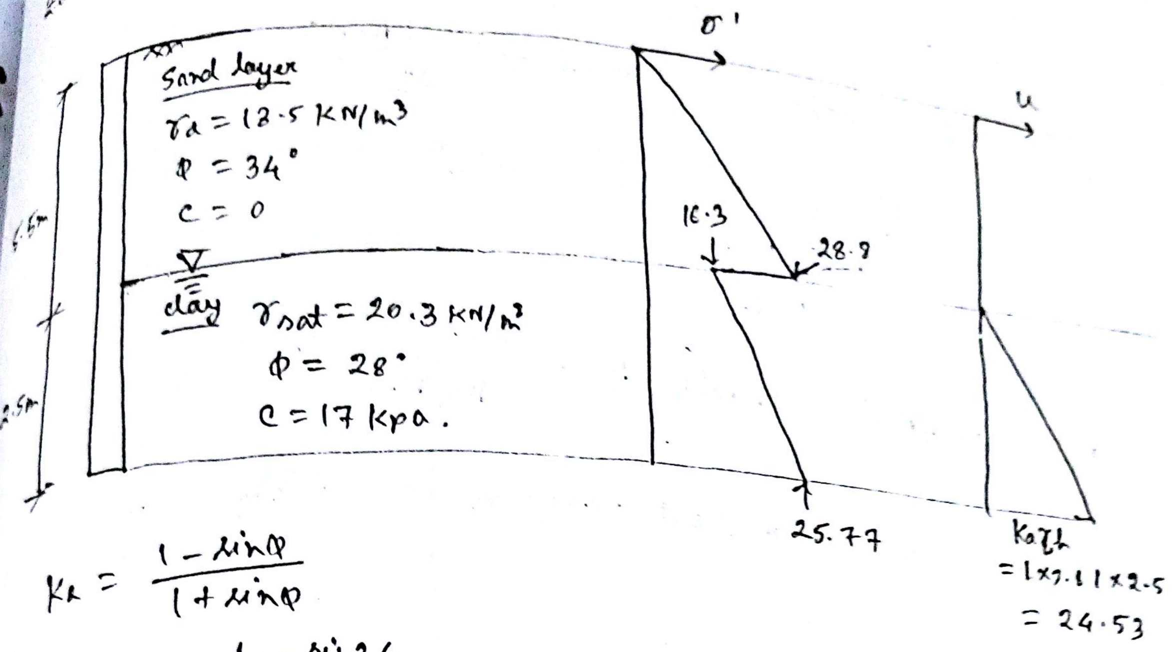
passive horizontal thrust, $P_p = \frac{1}{2} K_p \gamma D \times D = \frac{1}{2} \times 3 \times 17 \times D^2$

$\therefore \frac{1}{2} \times 3 \times 17 \times D^2 = 6.774 \times 17$

$\therefore \boxed{D = 2.13\ m}$

Confirm

[2008-09 | 4(a)]



$$K_a = \frac{1 - \sin\phi}{1 + \sin\phi}$$

$$K_a(\text{sand}) = \frac{1 - \sin 34}{1 + \sin 34} = 0.283, \quad K_a(\text{clay}) = \frac{1 - \sin 28}{1 + \sin 28} = 0.361$$

For sand layer:

$K_a = 0.283, \quad \sigma = K_a \gamma_d h = 0.283 \times 18.5 \times 5.5 = 28.8$ Kpa.

For clay layer:

$K_a = 0.361,$

~~at 5.5, $\sigma = K_a \gamma' h = 0.361 \times (20.3 - 9.81) \times 5.5 = 20$~~

at $z = 5.5$ m, $\sigma = K_a \gamma' h - 2c\sqrt{K_a}$

$= 0.361 \times (\overset{18.5}{\cancel{20.3 - 9.81}}) \times 5.5 - 2 \times 17 \times \sqrt{0.361}$

$= 16.3$ Kpa

at $z = 8$ m, $\sigma = 16.3 + K_p \gamma' h = 16.3 + 0.361 \times (20.3 - 9.81) \times 2.5$

$= 25.77$ kpa.

pore water pressure, $u = \cancel{\gamma_w} \times 9.81 \times 2.5 = 24.53$ Kpa.

$\therefore$ total active thrust = area of the pressure diagram

$= \frac{1}{2} \times 28.8 \times 5.5 + 16.3 \times 2.5 + \frac{1}{2} \times (25.77 - 16.3) \times 2.5 + \frac{1}{2} \times 24.53 \times 2.5$

$= \boxed{162.45\ KN/m}$ confirm

Scanned by CamScanner

2007-08 | 1(c)

unconfined compressive strength = $\frac{80 \text{ KN/m}^2}{2}$

$\therefore c = \frac{q_u}{2} = \frac{80}{2} = 40 \text{ KN/m}^2$

$\gamma = 16 \text{ KN/m}^3$

Now, unsupported height, $H_u = \frac{2c}{\gamma\sqrt{K_a}} \times 2$

Now, we assume that clay is a purely cohesive soil

$\therefore \phi = 0.$

$\therefore K_a = \frac{1 - \sin 0}{1 + \sin 0} = 1$

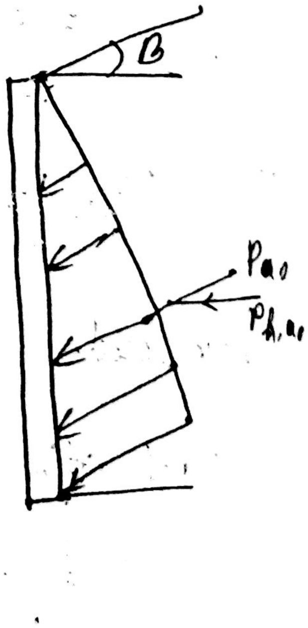
$\therefore H_u = \frac{2 \times 40}{16 \times \sqrt{1}} \times 2 = 10 \text{ m} > 6 \text{ m}.$

As the unsupported height is greater than the existing height, so retaining wall is not required.

2007-08	1(b)

For massive retaining wall, earth pressure at rest is at rest.

Total horizontal thrust,



$P_h = \frac{1}{2} \times K_0 \gamma h \cos\beta \times h \times \cos\beta$

$= \frac{1}{2} \times \cancel{\frac{1 - \sin 32}{1 + \sin 32}}$ Now, $K_0 = (1 - \sin\phi)(1 + 0.5 \tan\beta)^2$

$= 0.6045$

$\therefore P_h = \frac{1}{2} \times 0.6045 \times 16 \times 8^2 \times \cos^2 15$

$= \boxed{288.76 \text{ KN}}$

Scanned by CamScanner

2007-08 | 1(j)

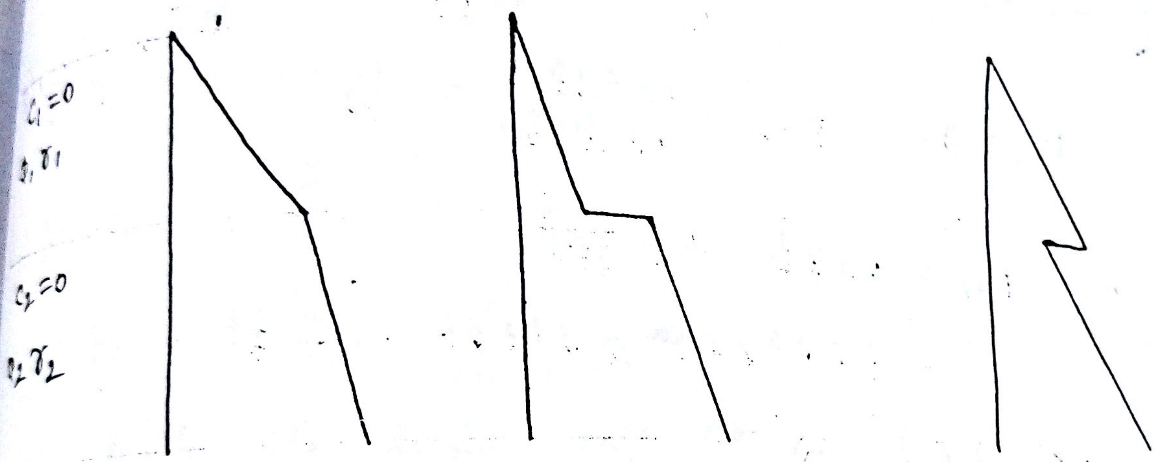
$$K_a = \frac{1-\sin\phi}{1+\sin\phi}$$

for active earth pressure, K_a decreases with increasing ϕ.

Again, $p = K_a \gamma h - 2c\sqrt{K_a}$

$= K_a \gamma h$ [as $c = 0$]

K's effect is negligible on slope.



(i) $\phi_1 = \phi_2$, $\gamma_1 > \gamma_2$, $K_1 = K_2$

(ii) $\phi_1 \gtrless \phi_2$, $\gamma_1 = \gamma_2$, $K_1 < K_2$

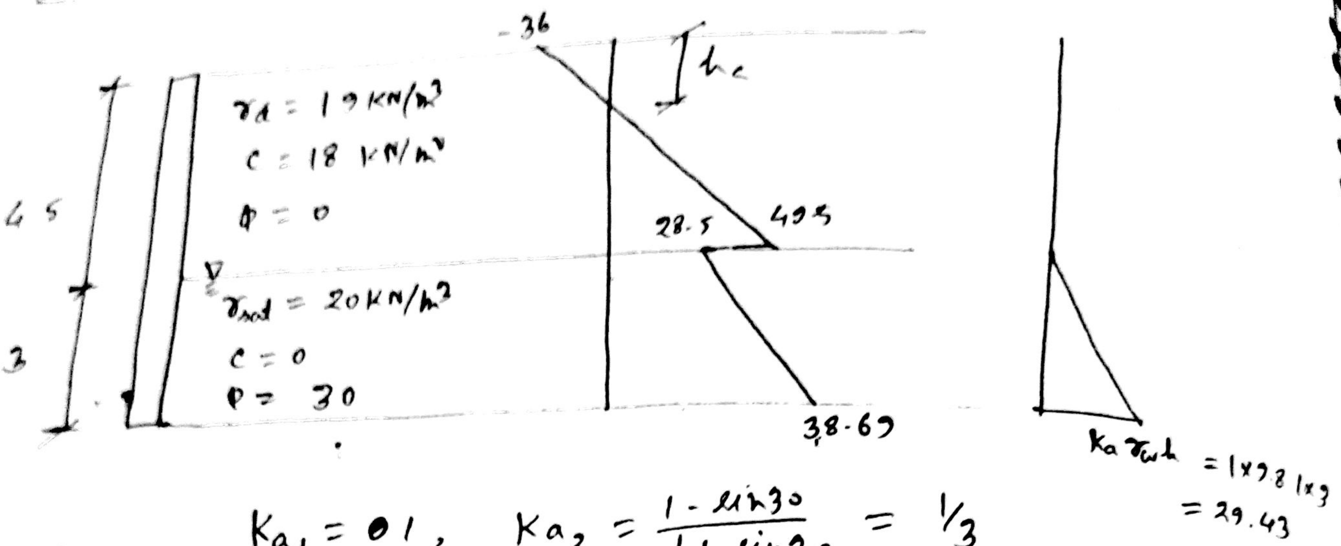
~~(iii)~~ $\phi_1 < \phi_2$, $\gamma_1 = \gamma_2$, $K_1 > K_2$

(iv) $\phi_1 > \phi_2$, $\gamma_1 > \gamma_2$, $K_1 < K_2$

(v) $\phi_1 < \phi_2$, $\gamma_1 > \gamma_2$, $K_1 > K_2$

(iii) $\phi_1 < \phi_2$, $\gamma_1 < \gamma_2$, $K_1 > K_2$

[2007-08 | 3(b) | (+, − দুটোই consider করতে হবে)



$$K_{a_1} = 1, \quad K_{a_2} = \frac{1 - \sin 30}{1 + \sin 30} = \frac{1}{3}$$

at $z = 0$, $\quad P_{ac} = K_a \gamma h - 2c\sqrt{K_a}$

$$= 1 \times 19 \times 0 - 2 \times 18 \times 1 = -36$$

at $z = 4.5$ (for 1st layer),

$$P_{ac} = 1 \times 19 \times 4.5 - 2 \times 18 = 49.5$$

at $z = 4.5$ (for 2nd layer),

$$P_{ac} = \frac{1}{3} \times 19 \times 4.5 - 0 = 28.5$$

at $z = 7.5$ m,

$$P_{ac} = \frac{1}{3} \times (20 - 9.81) \times 3 - 0 + 28.5 = 38.69$$

Now, $\frac{h_c}{36} = \frac{4.5 - h_c}{49.5} \quad \therefore h_c = 1.89$ m.

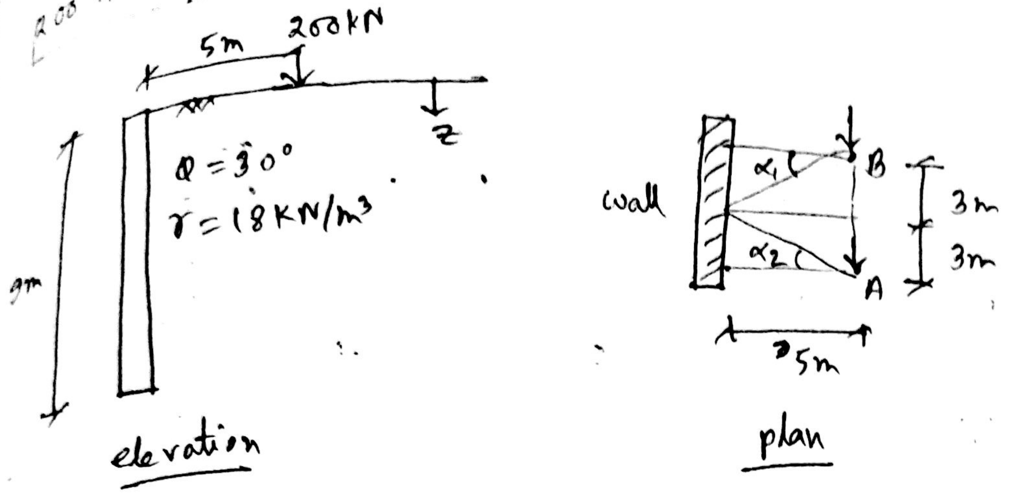
$\therefore$ Total thrust = area of the pressure diagram,

$$= \frac{1}{2} \times 49.5 \times (4.5 - 1.89) + 28.5 \times 3 + \frac{1}{2} \times (38.69 - 28.5) \times 3 + \frac{1}{2} \times 29.43 \times 3$$

$$= \boxed{209.53 \text{ KN/m}}$$

Ans

2007-08 | 4(a)



$\tan\alpha = \frac{3}{5} \quad \therefore \alpha_1 = 30.96 = \alpha_2$

$m = \frac{x}{H} = \frac{5}{9} = 0.556 > 0.4.$

$\therefore P_h = \frac{1.77Q}{H^2} \frac{m^2 n^2}{(m^2+n^2)^3} \cancel{\times 2}, \quad P_h' = P_h \cos^2(1.1\alpha) \times 2$ for both load

let's divide the height into equal 6 parts.

z	$n = \frac{z}{H}$	$P_h' = \frac{2\times1.77\times200}{9^2} \times \frac{0.556^2 \times n^2}{(0.556^2+n^2)^3} \times \cos^2(1.1\times30.9$
0	0	~~1.88~~ 0 Kpa
1.5	1/6	~~1.88 Kpa~~ 1.347 Kpa
3	1/3	~~3~~ 2.777
4.5	1/2	2.653
6	2/3	1.926
7.5	5/6	1.274
9	1	0.827

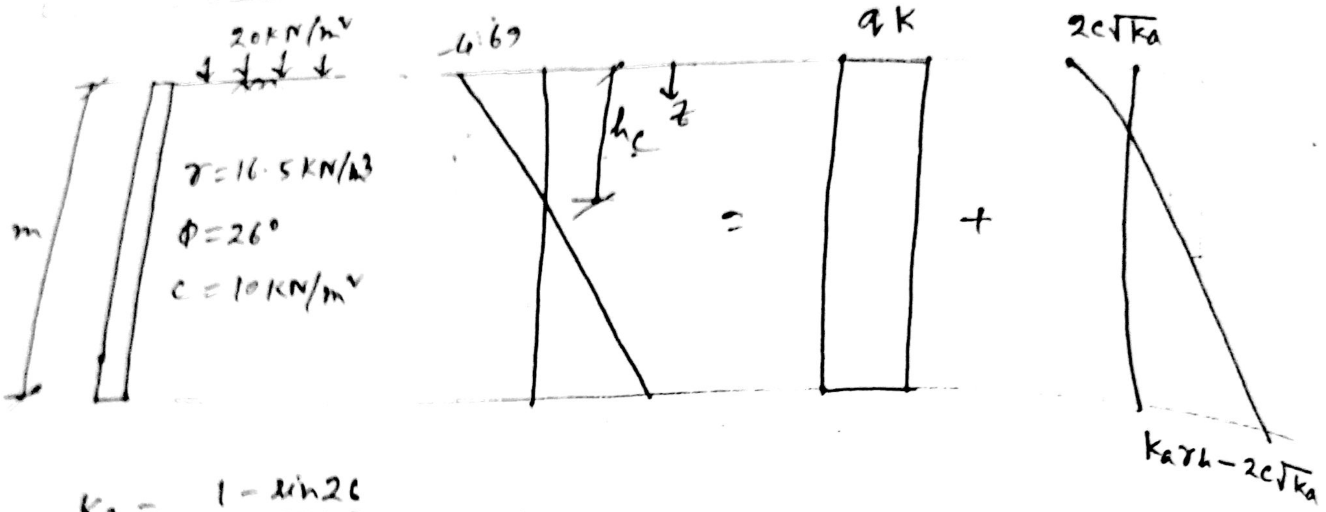
$\therefore$ Total active thrust $= \frac{1}{2} \times 1.5 \times [0 + 0.827 + 2\times(1.347 + 2.777 + 2.653 + 1.926 + 1.274)]$

$= \boxed{15.586 \text{ kN/m}}$ Ans

confirm

2006-07 | 3(c) | → Similar to 2007-08. 4(a)

2006-07. 4(b)



$$K_a = \frac{1 - \sin 26}{1 + \sin 26} = 0.39$$

at $z = 0$, $P_{ac} = qK - 2c\sqrt{K_a}$

$$= 20 \times 0.39 - 2 \times 10 \times \sqrt{0.39} = -4.69$$

at $z = \cdot 20 \times 0.39 + 0.39 \times 16.5 \times 8 - 2 \times 10\sqrt{0.39}$

$$= 46.79$$

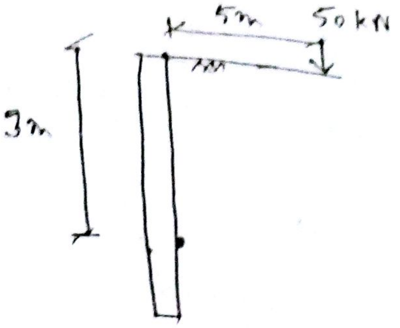
Now, $\frac{4.69}{h_c} = \frac{46.79}{8 - h_c}$ $\therefore h_c = 0.73\,m$.

$\therefore$ Total thrust $= \frac{1}{2} \times (8 - 0.73) \times 46.79$

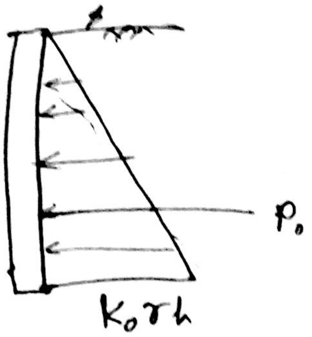
$$= \boxed{170.08 \text{ Kpa}}$$

2004-05 | 1(b)

$m = \frac{5}{2}$, $n = \frac{3}{2}$



2004-ᴵᶜ | 7(b)



$K_o = (0.19 + 0.233 \log I_P)\sqrt{OCR}$

Now, $I_P = LL - PL$

$= 35 - 15 = 20$

$\therefore K_o = (0.19 + 0.233 \log 20)\sqrt{2.5}$

$= 0.78$

~~For c-ϕ soil,~~

~~active earth pressure,~~ $P_a = K_o \gamma h$ ~~$- 2c\sqrt{K_o}$~~

$\therefore$ total thrust $= \frac{1}{2} K_o \gamma h \times h$

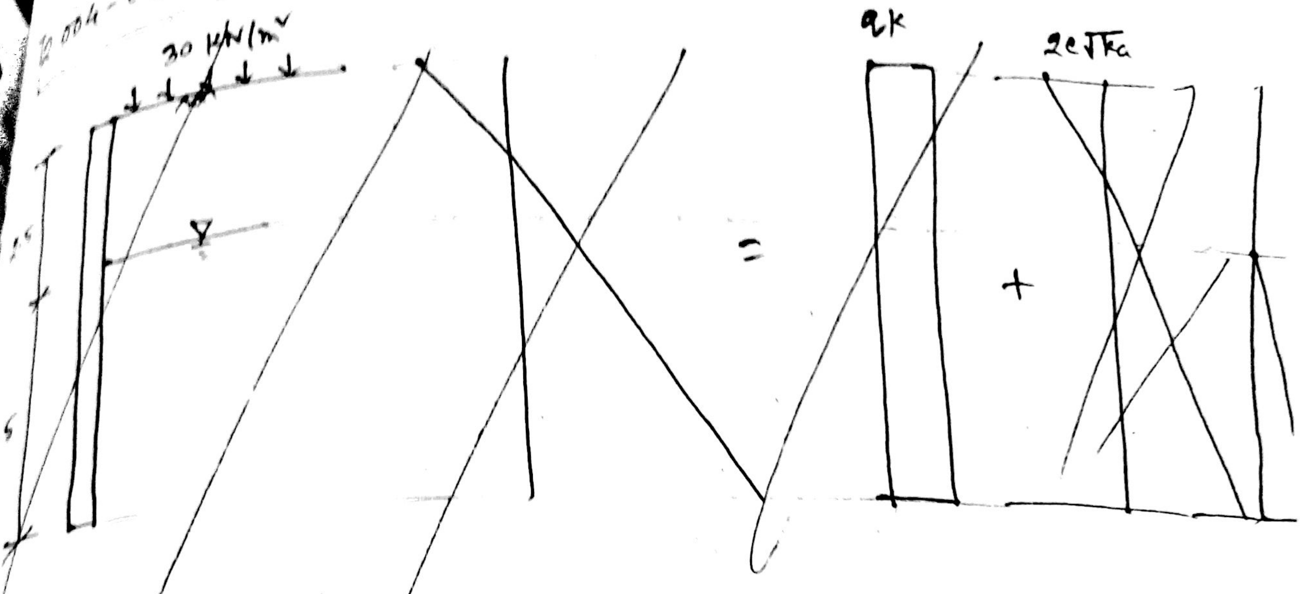
$= \frac{1}{2} \times \overset{0.74}{\cancel{0.78}} \times \gamma \times 6^2$

$= \boxed{\overset{13.34}{\cancel{14.04}}\ \gamma \text{ kN/m}}$

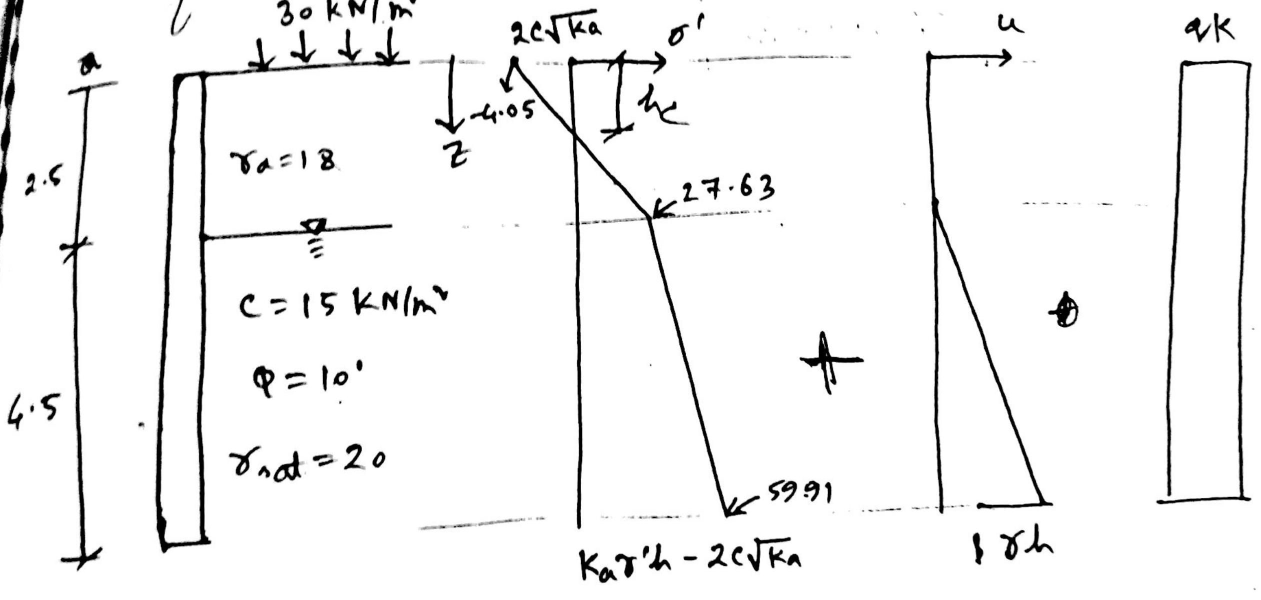
$K_o = (0.19 + 0.233 \log I_P)\sqrt{OCR}$ → This formula is only applicable for NC clay.

So, here, $K_o = \frac{1 - \sin 15}{\cancel{1+\sin 15}} = 0.74$

2004-05. 8(a)



$$K_a = \frac{1-\sin 10}{1+\sin 10} = 0.704$$



$$K_a = \frac{1-\sin 10}{1+\sin 10} = 0.704$$

at, z = 0 m, $P_{ae} = qK + K_a \gamma h - 2c\sqrt{K_a} = -2\times 15\times\sqrt{0.704} + 30\times 0.70$

$= -4.05$ kpa.

at, z = 2.5 m, $P_{ae} = 30\times 0.704 + 0.704\times 18\times 2.5 - 2\times 15\times\sqrt{0.704}$

$= 27.63$ kpa

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at $z = \cancel{3}\ \cancel{4.25m},\ 7m,$

$P_{ac} = \cancel{30 \times 0.704 + 0.704 \times (20-9.81) \times 4.5 - 2 \times 15 \times \sqrt{0.704}}$

$= 27.63 + 0.704 \times (20-9.81) \times 4.5$

$= 59.91\ KPa$

~~∴ Total thrust =~~ $\frac{6.05}{h_c} = \frac{27.63}{(2.5-h_c)}$

$\therefore h_c = 0.32\ m$

$\therefore$ Total thrust $= \frac{1}{2} \times (2.5 - 0.32) \times 27.63 + \frac{1}{2} \times 4.5 \times (27.63 + 59.91) + \frac{1}{2} \times 9.81 \times 4.5 \times 4.5$

$= \boxed{326.41\ KN/m}$

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List of Theoretical Questions 0904001

for compaction

① what is a compaction curve? Give its salient features? what is a zero air void line?

2010, 5(a) ② Distinguish between compaction and consolidation what are the factors affecting compaction? Discuss in brief

③ Write down the standard procedures for estimating the field density of compaction. Explain one of them

2009-10 ④ Explain the effect of compaction on soil type with neat sketches.

⑤ Define the term relative density of cohesionless soil. How this can be used to classify the compactness of soil [2008-09] 5(a)

→2007-08 6(a)

⑥ why compaction test is done in the laboratory?

⑦ Bring out the usefulness of compaction test in the lab in soil engineering practice. Define optimum moisture content and state factors on which it depends. → 2006-07; 5(c)

*** ⑧ why compaction is done?

By — Mahmudul Shawon
0904040

Muqtadir স্যারের Theoretical Questions
(compaction)

2010 - 2011

(5) (a) What is a compaction curve? Give its salie[nt] features. What is zero air void line?

Ans: Compaction curve is the plot of dry unit weight γ_d - Vs - moisture content ω (%). It shows the water/moisture content at which maximum density for a soil can be gained.

Salient Features:

Hydraulic properties of soil

Permeability

1. "The results of permeability tests on cohesionless materials are often misleading" — Explain

2. How do you determine co-efficient of permeability of clay?

3. Write down the different values of permeability of clay, silty clay and Fine sand.

wt volume Relationship

① Show the phase diagram of a soil and define the following –

(i) void ratio; (ii) Porosity; (iii) water content and (iv) Degree of saturation

② Prove that, $S_r = \dfrac{\omega}{\dfrac{\rho_w(1+\omega)}{\rho} - \dfrac{1}{G_s}}$

③ Define the terms S_r, γ_d, and n. Establish relation between e, ω and S_r and G_s.

④ What is phase diagram? What is its use? How void ratio of a soil sample can be determined.

⑤ Develop a relationship between γ_{sub}, G_s, e and γ_w.

Flow Net / Effective stress

① What is the effect of seepage pressure on effective stress? Give examples.

② Derive the equation for sepage calculation,
$q = k \dfrac{H N_f}{N_d}$ from a flow where the symbols carry their usual meanings.

③ Explain total stress, effective stress and pore water pressure.

④ What is quick sand condition? Under what circumstances does it occur?

⑤ What is a flow Net and what are its use in civil Engineering practice?

⑥ Why the quantity of sepage between two successive flow line is equal?

⑦ Write a short note on capilary rise of soils.

⑧ Define Seepage pressure. এর সাথে ① নং question

⑨ Why filter is used on the down stream side of ~~the~~ an earth structure? How would you design such a filter?

(10) what is reverse or graded filter.

(11) State the physical interpretation of basic laplace equation. In connection with the soil seepage problem in two dimensional flow.

(12) Prove that $\sigma_{effective} = \gamma_{sub} \times z$

Permeability

① The results of permeability tests are often misleadin. - Explain. [2010-11 7(a),

Ans: It is ~~not~~ difficult to simulate the field condition in the laboratory. Thats why the test result is an indicative value; not a precise one. soil gets disturbed while collecting sample.

- If entrapped air gets into the sample, it obstructs the flow of water through the soil.
- Finer particles of cohesionless soils get themselves arranged at the top and bottom of the soil sample. It obstructs the flow; reducing the value of permeability.

(9) Write down the different values of permeability of clay, silty clay and Fine Sand.

Ans:

Soil type	K cm/sec
Clay	< 0.000001
Silty clay	0.001 – 0.00001
Fine Sand	0.01 – 0.001

[Table 6.1; Page 148; BM Das ২য় (মোটা বই)]

Compaction

① What is a compaction curve? Give its salient features. What is zero air void line.

Ans: Compaction curve is the plot of dry density of the soil γ_d - Vs - w water content. It is drawn to find at which water content the maximum density of a sample can be gained.

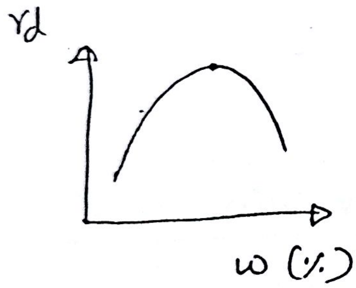


Fig — A typical compaction curve.

Salient features:

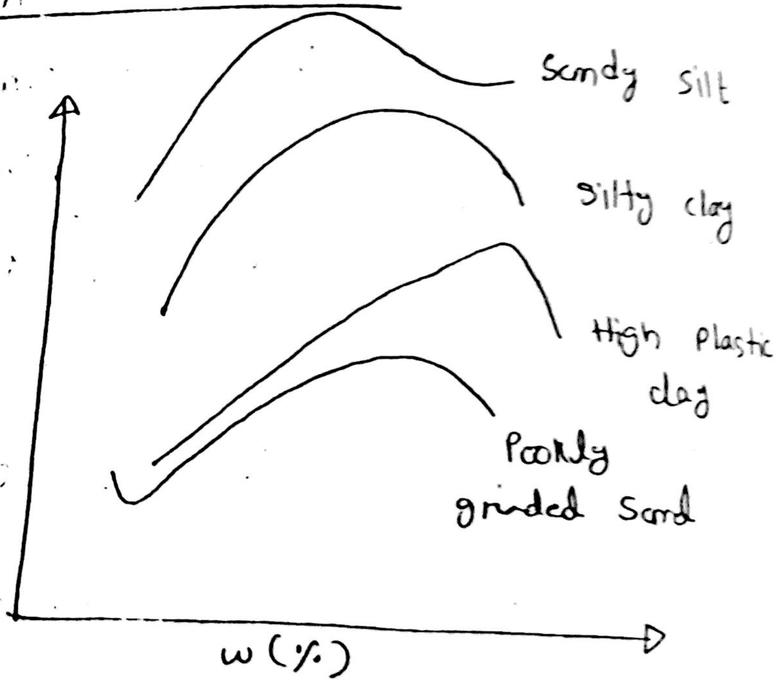
ZERO air void line: It is a line on the same plot as that of compaction curve. It indicates that water contents and Dry Densities at which there is no air void in the soil sample. It means all the void spaces are occupied by ~~walls~~ water. thus, the soil is fully saturated. this line never touches the compaction curve.

② Distinguish between compaction and consolidation. what are the factors affecting compaction? Discuss in brief. [2010-11;

Ans8 compaction is the process of reducing of the air voids from the soil sample. whereas it does not expell water. But consolidation is the process of expelling water from soil samples.

Factors affecting compaction:

I) Soil type:



soils of different type show different compaction curves.

Compaction Is the densification of soil imparting the mechanical energy to it. This energy expells the air voids out of the soil.

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② Moisture content: when water is added to dry soils, it creates a film over soil particles. This film allows soil particles to slide past one another. This is called lubrication. Due to this process, addition of water upto a certain water helps ~~the~~ compaction. Upto this point water replaces air voids. This increases the soil density. But, beyon this point, water occupies spaces that could have been occupied by soil particles. It decreases the soil density.

③ Mechanical Energy: The higher the mechanical energy imparted to the soil, the more compaction takes place.

⑤ Define the term Relative Density of cohesionless soil. How this can be used to classify compactness of soil. [2008-09; 5(a)]

Ans: Relative Density indicates the 'in situ' denseness or looseness of ~~granular s~~ cohesionless soil.

It is given by, $I_D = \frac{e_{max} - e_f}{e_{max} - e_{min}}$ where.

e_{max} = void ratio at the loosest state

e_{min} = " " " " densest "

e_f = " " " " field condition

usual values for Id is analogus to different state of compaction like the following -

State of compaction	Id (%)
① Very loose	0-15%
② Loose	15-35
③ Compact / Medium Dense	35-65
④ Dense	65-85
⑤ Very Dense	>85

Soil compactness can be classified if ~~comp~~ the test result is compared with the above table data.

... compaction is done in the laboratory?

Ans. The maximum field density that needs to be maintained in the field work must be 95% of the maximum dry density that was determined in the laboratory test. This is why compaction test is done in the laboratory.

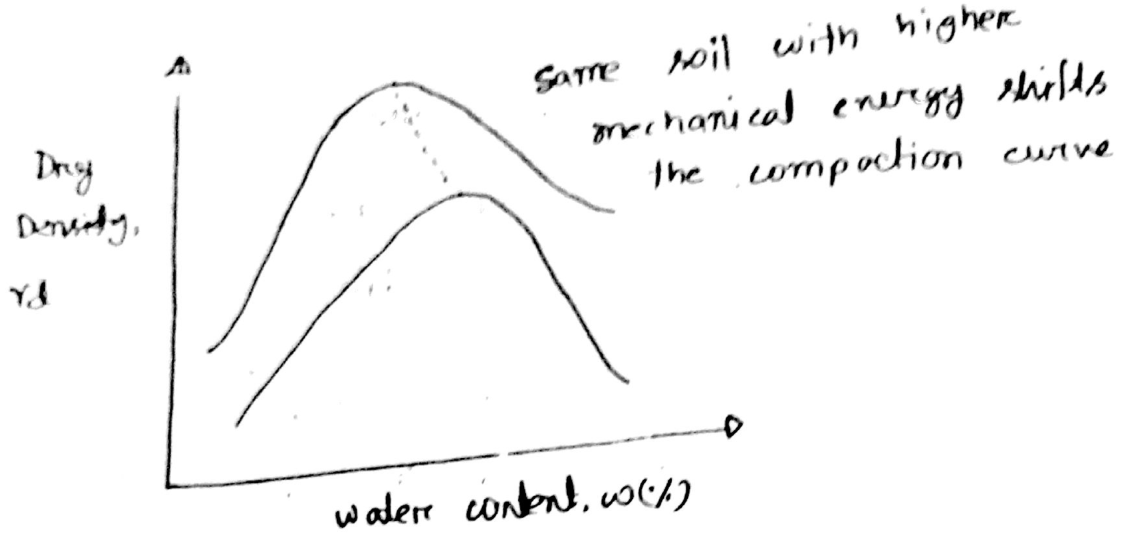
⑦ Bring out the usefulness of compaction test in soil engg. practice. Define optimum moisture content and state the factors on which it depends.

Ans: ~~By conducting the~~ compaction tests helps us to know what density in the field must be maintained in a project.

Optimum moisture content is the moisture content at which maximum dry density of a soil ~~gat~~ can be gained.

Factors:

① Amount of Mechanical Energy: ~~with~~ If More Mechanical Energy is put in on the same soil sample, less amount of water is required to gain the maximum dry density. Graphically, looking like this



means that the curve ~~shif~~ pick shifts towards the left.

② <u>Soil particle size:</u> Finer soils need more water than the relatively coarser soils to reach their optimum levels.

⑧ Why we do compaction on soil? ***

<u>Ans:</u> We compact soils for 3 basic reasons -

① To reduce the settlement of the structure built over it.

② To increase the shear strength.

③ To decrease permeability [for embankment like structures in particular]

Flow Net / Effective Stress [সাবেকের Lecture এর chronology অনুযায়ী দেওয়া হয়েছে]

Q: Explain Total stress, effective stress and pore water pressure.

A: Total stress is the stress acting on a point through a section of a saturated soil mass. It is contributed by the overlying material of the point of interest.

Effective stress is one of the parts that the total stress is composed of. It acts exclusively between the soil particles. It is solely responsible for inducing volume change behavior in soil. It also provides frictional resistance of the soil.

Pore water is the other component of the total stress. It acts ~~solid~~ in soil particles and water with equal magnitude in all direction. It is not at all responsible for volume change or frictional resistance in soil.

So, if effective stress is reduced, strength of the soil also falls.

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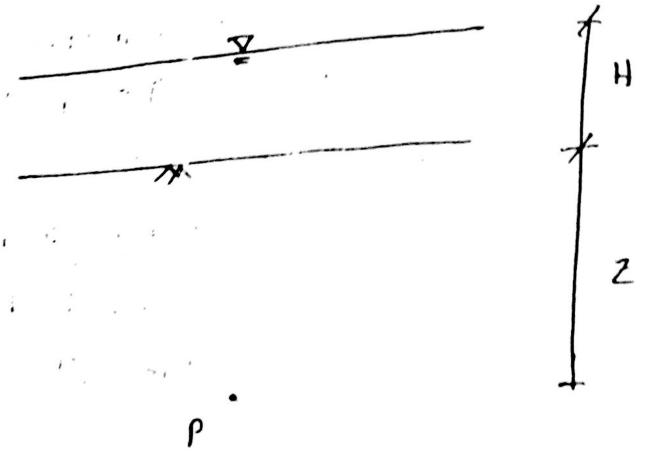
(10) Prove that $\sigma_{effective} = \gamma_{submerged} \times z$ where

z = height of the saturated soil sample above the concerned point.

Ans:



Let that we are discussing about the total stress on the point p. Height of overburden soil of p is z and above that we have water with height of H. As the soil is below water. we consider that it is saturated and water has reached all the way to 'p' through the continious voids of the soil mass. Assuming this, we can have the porewater pressure u_w at point p is $u_w = (H+z)\gamma_w$ [Here, γ_w is the unit weight of water]

and the total stress $\sigma_p = \gamma_w H + \gamma_{sat} z$

Hence, effective stress $\sigma_{eff} = \sigma_p - u_w$

$\Rightarrow \sigma_{effective} = H\gamma_w + \gamma_{sat} z - (H+z)\gamma_w$

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

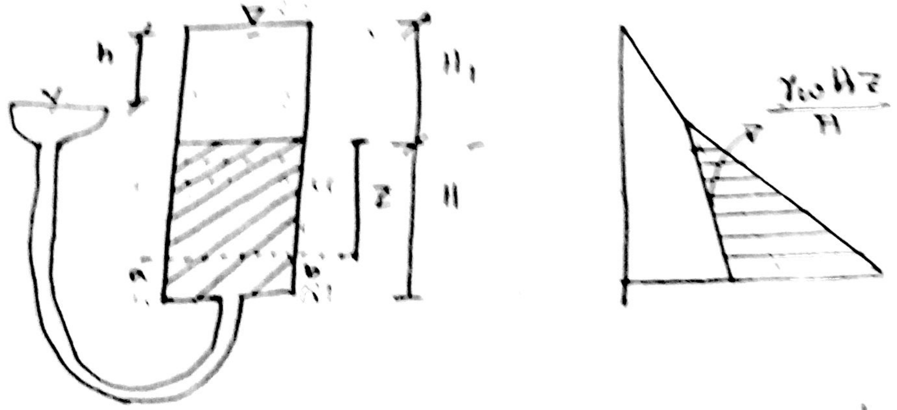
$= z(\gamma_{sat} - \gamma_w)$

$\sigma_{effective} = \gamma_{submerged} \, z$

[Proved]

Q) Define seepage pressure. Show ~~it~~ how it varies with effective stress.

A:



Seepage pressure is defined by the amount of increase in effective stress due to flow of water through the voids of sample.

In the figure, we have a container partly filled with granular material and completely filled with water. A water reservoir is connected to it through a flexible tube. In the pressure diagram, the unshaded area ~~x~~ means the pore water pressure and the shaded area means the effective stress. when the reservoir is lowered & by amount h to the container. flow will occur due to the head difference. the direction is from the container to the reservoir. Now, if we compare to with no flow condition; in which the container and the reservoir were at the same level, the amount of pore water pressure ~~was~~ fall is $\gamma_w h$. At plane a-b, total stress is governed by the weight of soil and water above it. So, the effective stress must increase by the same amount. This can be written as, $\frac{h \gamma_w z}{H}$. This is how seepage pressure varies with effective stress.

① What is quicksand condition? Under what situation it might occur?

A: When water flows in upward direction through the voids of a soil mass, quicksand occurs at a particular point.

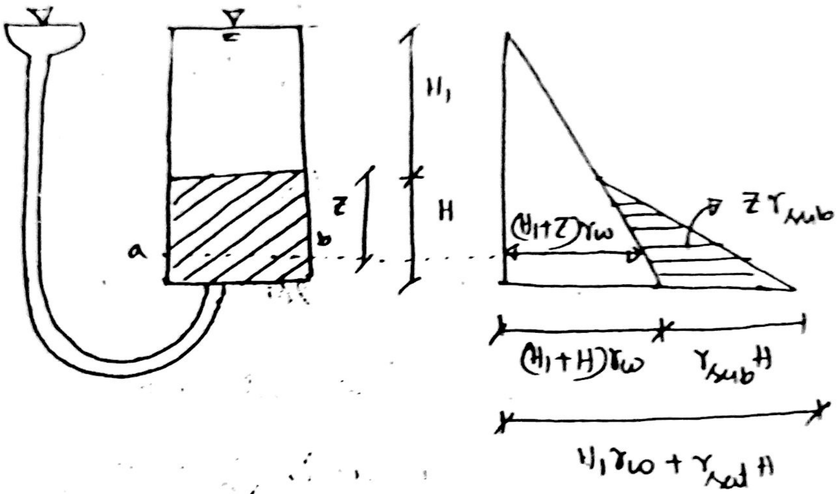


Fig 1. No flow condition

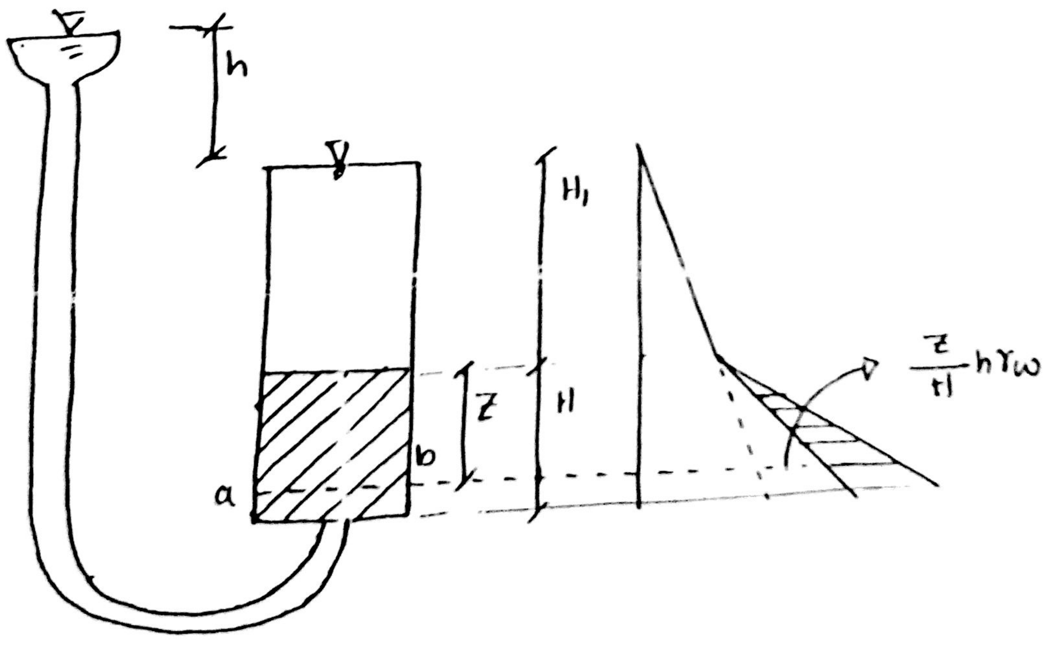


Fig 2. Upward flow ~~the~~ through the soil sample

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At No flow condition (fig 1), the container and the reservoir are at same level. If the reservoir raised above by 'h', the head difference will make the water flow from reservoir to container. This causes an increase in pore water pressure by the amount $\gamma_w h$. The more the reservoir is raised, as the more pore water pressure rises and of course the more effective stress drops. At a certain point, the effective stress will vanish. The hydraulic gradient $\left(\frac{h}{H}\right)$ at which it occurs is critical hydraulic gradient. ~~Under this~~

Under such conditions, a cohesionless soil can not take any weight on its surface. This is quick sand condition.

(9) xxx Derive the laplace equation for two dimensional flow.

A: What are the assumptions for a fluid flow porous media like soil?

Ans: The assumptions are –

① Soil is Isotropic and homogenious

② Soil is saturated completely

③ Darcy's Low is applicable for the flow

④ Water is incompressible

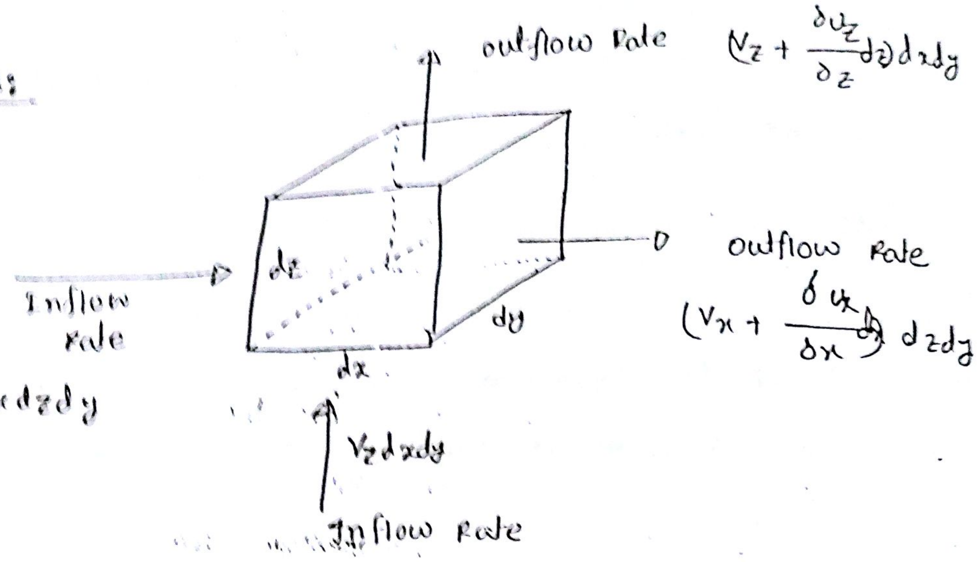
⑤ Soil skeleton is incompressible too.

⑥ Flow is 2-Dimensional

⑦ Net flow is zero. That means inflow is equal to outflow.

Q. Derive the Laplace Equation about two dimensional flow through a porous media?

Ans:



Lets consider an element of soil skeleton through which water is travelling at speed v. v_x is the component of the velocity at x direction ; v_y be that of ~~at the~~ in the y direction. The element having dimensions dx, dy, dz in the respective directions. Now in the x direction water is coming in at speed v_x and flows through the cross section of area dzdy. So Inflow according to continuity equation is.

$v_x dy dz$. Similarly ~~f~~ Inflow in the z direction is $v_z dx dy$. Now change in velocity in x direction becomes $\frac{\partial v_x}{\partial x} dx$ and that in z direction

is $\frac{\delta v_z}{\delta z} dz$. So, outflows in those directions are

$(v_x + \frac{\delta v_x}{\delta x} dx) dz dy$ and $(v_z + \frac{\delta v_z}{\delta z} dz) dx dy$ respectively.

Now to meet the assumptions made that the inflow and outflow rates will be equal. We have,

$$(v_x + \frac{\delta v_x}{\delta x} dx) dy dz - v_x dz dy - \{(v_z + \frac{\delta v_z}{\delta z} dz) dx dy - v_z dx dy\} = 0$$

$$\Rightarrow \frac{\delta v_x}{\delta x} + \frac{\delta v_z}{\delta z} = 0 \quad \text{(1)}$$

From Darcy's principle we find $v = ki$

and from the assumption that the soil is isotropic, permeability at all directions is equal (k, let)

$\therefore$ we have, $v_x = k i_x$ and $v_z = k i_z$

i_x and i_z are hydraulic gradients in the respective direction. Now, $i_x = \frac{\delta h}{\delta x}$; $i_z = \frac{\delta h}{\delta z}$ [where h is the potential function]

So, we have from (1),

$$\frac{\delta}{\delta x}(k \frac{\delta h}{\delta x}) + \frac{\delta}{\delta z}(k \frac{\delta h}{\delta z}) = 0$$

$$\Rightarrow \frac{\delta^2 h}{\delta x^2} + \frac{\delta^2 h}{\delta z^2} = 0$$

$$\Rightarrow \nabla^2 h = 0$$

(?) state physical interpretation of basic Laplace equation, in connection with soil seepage problem in two dimensional problem.

Ans: Mathematically the equation represents the algebraic sum of ~~ca~~ change of hydraulic gradient in two flow directions is zero.

Graphically, this equation represents two sets of orthogonal curves. One of them is flow line. It is a line through which water moves from upstream to downstream. The region bounded between two successive flow line is called flow channel.

The other set is equipotential line. ~~tut~~ These are lines through which all piezometre readings will be same; i.e, same pressure head.

The region bounded by two flow lines and flow ~~el~~ equipotential lines is flow element.

Flow Net and Seepage

lecture wise [illegible] [illegible]

[illegible] is a flow net and what are its use [illegible] civil Engineering practice? why is the [illegible]

[illegible] The combination of flow line, equipotential line and flow element with appropriate boundary condition is called flow Net diagram.

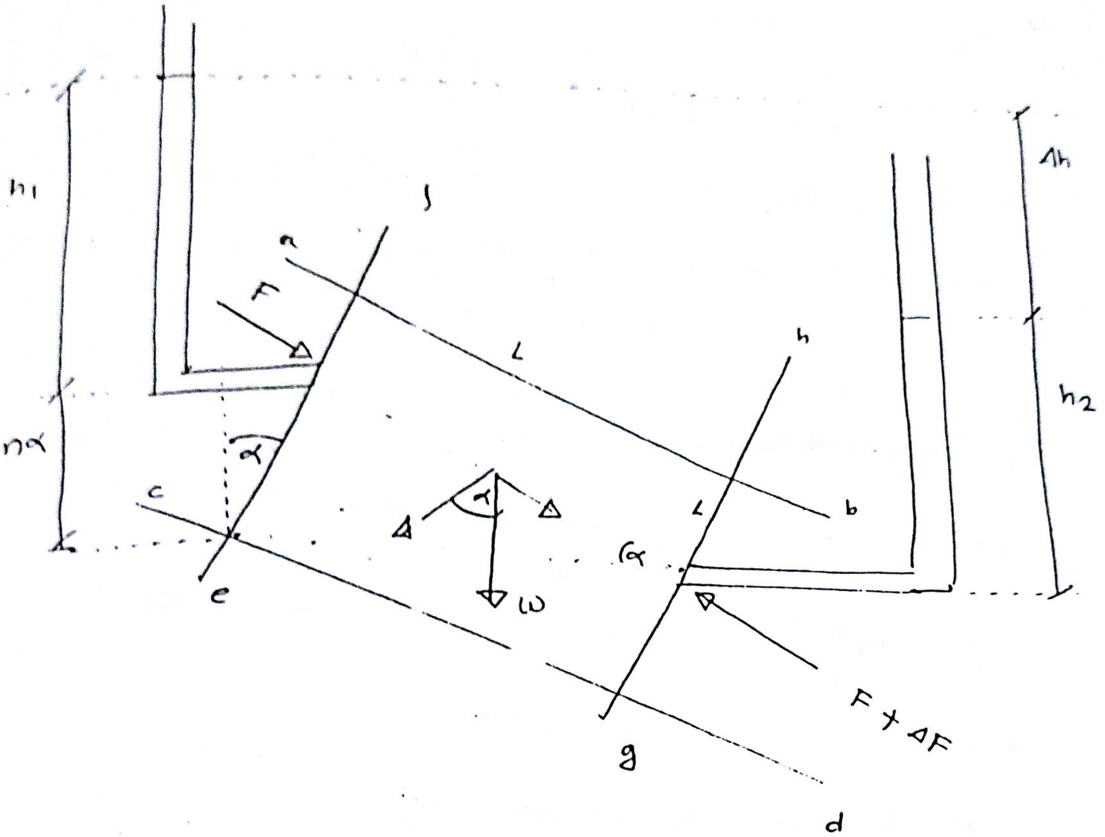
Seepage through the soil under an earthen structure like dam, embankment can be calculated quantitatively with its help.

(i) [illegible] seepage between two successive flow lines [illegible] equal?

... the equation $q \propto \frac{B N_f}{N_d}$ for doping calculation

where symbols carry there usual meanings

~~ans~~ Derive the expression for seepage force per unit volume of the soil.



To avoid seepage force per unit volume of soil, Let's consider a soil mass bounded by two flow lines ab and cd and two equipotential lines ef and gh as in the figure. The soil mass has unit ~~unit~~ thickness per normal to the section. Thus self weight w of the soil mass $W = L \times L \times 1 \times \gamma_1$

$$W = L^2 \gamma_{sat}$$

The hydrostatic force on the ~~ef and~~ gh ~~sides of the soil~~ ~~masses are~~ evaluated

side ef is $F = \text{pressure} \times \text{Area}$

$$= h_1 \gamma_w \times L \times 1$$

$\therefore F = h_1 \gamma_w L$

hydrostatic force on the side gh, F_2 = pressure head × Area

$F_2 = (h_2 \gamma_w) \times (L \times 1)$

$\therefore F_2 = h_2 \gamma_w L$

change in Force $\Delta F = F_2 - F_1$

$\Rightarrow F_2 = \Delta F + F_1$

For equilibrium,

$\Delta F = h_1 \gamma_w L + L^2 \gamma_{sat} \sin\alpha - h_2 \gamma_w L$

$\Delta F = h_1 \gamma_w L - h_2 \gamma_w L + L^2 \gamma_{sat} \sin\alpha \quad \text{--} (1)$

Now, $h_1 + L\sin\alpha = h_2 + \Delta h$

$\Rightarrow h_1 - h_2 = \Delta h - L\sin\alpha \quad \text{-} (2)$

putting (2) into (1),

$\Delta F = (\Delta h - L\sin\alpha)\gamma_w L + L^2 \gamma_{sat} \sin\alpha$

$= \Delta h \gamma_w L + L^2 \gamma_{sat} \sin\alpha - L^2 \gamma_w \sin\alpha$

$= \Delta h \gamma_w L + L^2 (\gamma_{sat} - \gamma_w) \sin\alpha$

$= \cancel{\Delta h +} \underbrace{L^2 \gamma_{submerged} \sin\alpha}_{\text{component of the submerged weight of the flow element in the flow direction}} + \underbrace{\Delta h \gamma_w L}_{\text{seepage force along the flow direction}}$

Now volume of the soil mass $V = L \times L \times 1 = L^2$

$\therefore$ seepage force per unit volume of the soil sample, $= \frac{\Delta h \gamma_w L}{L^2}$

$= \frac{\Delta h \gamma_w}{L}$

$= \frac{\Delta h}{L} \gamma_w$

$= i\gamma_w$ [i = hydraulic gradient]

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(8) Why filter is used on the downstream side of an earthen structure? [2006-07 ; 8(b)]

Ans: Filter material is used in the downstream to increase the soil weight. This extra weight increases the factor of safety against piping.

(9) How can you design a filter?

To Design a filter, the designer has to meet the following criteria -

① It should be coarse enough to get saturated readily. This way seepage force is avoided. For this condition to be fulfilled,

$$\frac{D_{F_{15}}}{D_{B\,15}} > 4 \text{ to } 5$$

[D_{F15} is the size of the filter material of which 15 is finer.]

② It also needs to be fine enough so that it can the base material from passing through it. for this,

$$\frac{D_{F_{15}}}{D_{B_{85}}} \leq 4 \text{ to } 5$$

[$D_{B\,15}$ and $D_{B\,85}$ are the sizes of the Base material of which 15%. are 85% are f...]

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(8) what is Reverse or graded filter?

if the filter material needs to be very fine to prevent the particles from the protect soil it is better to lay only a thin layer & then protect this filter material with a coarser material. Sometimes a filter may be constructed in several layer each of width 12". Each layer is designed to protect the one beneath it This is known as Graded filter

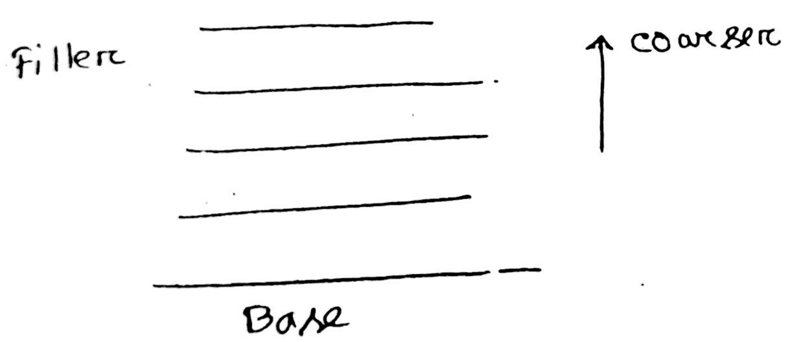

Fig - graded / Reverse filter.

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① Derive the equation $q = k\frac{HN_f}{N_d}$ for seepage calculation where symbols carry their usual meaning.

A:

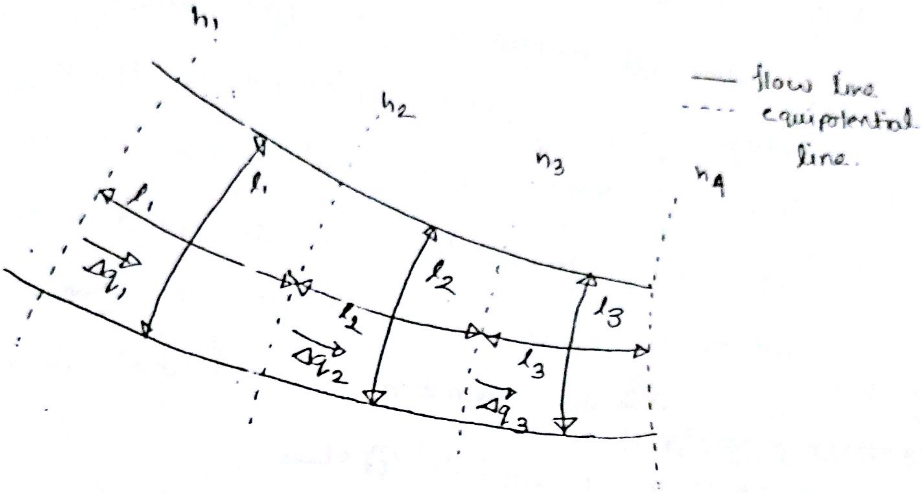


Fig - Seepage through flow channel

The figure above shows a flow channel that with equipotential lines. They form curvilinear square element & Let, h_1, h_2, h_3, h_4 are the piezometric readings of the equipotential lines. Since, there is no flow across the flow lines, $\Delta q_1 = \Delta q_2 = \Delta q_3 = \cdots\cdots = \Delta q$ (Let) -

From Darcy's law, we have flow rate, $q = kiA$

here, k = permeability

i = hydraulic gradient

A = x-sectional area Normal to the flow direction

So, for each flow element equation ① can be rewritten as-

$$\Delta q = k\left(\frac{h_1 - h_2}{l_1}\right) \times l_1 = k\left(\frac{h_2 - h_3}{l_2}\right) l_2 = k\left(\frac{h_3 - h_4}{l_3}\right) l_3 \cdots (

here, K = permeability

$\frac{h_1 - h_2}{l_1}$ = hydraulic gradient for the first element

$l_1 \times 1 = l_1$ = x-sectional area [unit width assumed]

equation (2) means if the flow lines are drawn approximately square, piezometric drops ~~of~~ between two successive equipotential lines is the same. So, if the head difference of head between the upstream and downstream of the flow is H and the Number of potential drops is Nd, we can rewrite (2) —

$$h_1 - h_2 = h_2 - h_3 = h_3 - h_4 = \cdots = \frac{H}{Nd} \quad \text{-- (3)}$$

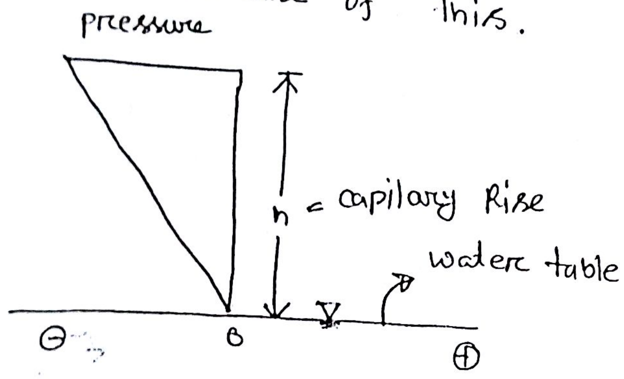
So we find $\Delta q = K \frac{H}{Nd}$

If the Number of flow channel in a flow Nd is N_f then we, can say that the the total flow $q = N_f \times \Delta q$

$$\therefore \boxed{q = K \frac{H}{Nd} N_f}$$

(6) Write a short Note on capilary Rise in soils.
[2007-08; 7(d)]

Ans: Because of surface tension of water, it gets sucked up into the soil pores (continious voids) above the water table. This is called capilary Rise. The soil above the water table remain moist because of this.



Pore water pressure in this zone is negative. It increases upto zero at the water table. Allen Hazen ~~propose~~ gave a formula to calculate the approximate height of capilary Rise —

$$h = \frac{c}{e D_{10}} ; \text{ where,}$$

e = void ratio

D_{10} = effective size

c = A constant varying from 10 - 50 mm^2

This equation is analogus to the relation $h \propto \frac{1}{d}$; where d is the diametre of the capilary tube. The more the pore dia, the less the capilary Rise. It is smallest for fine greavels (0.02~0.1m) and largest for clay (10~30m)

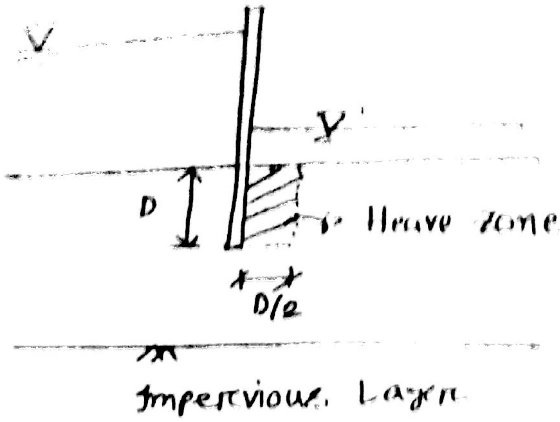
Q) Explain the mechanics of piping in hydraulic structures. What Methods can be used to increase the factor of safety against piping. [2007-08; 7(b)]

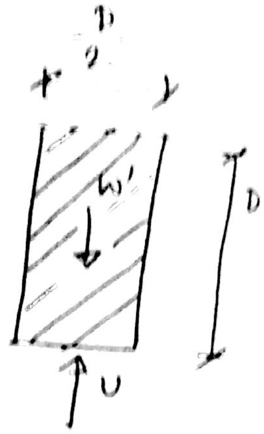
Ans: In the downstream side of a hydraulic structure seepage is usually upward. This causes a fall in effective stress. At a certain point, total stress becomes zero. Due to this, the soil in the downstream direction fails and bursts upward. This is called piping/ Heaving/ subsurface erosion

Two methods can be used to increase the safety factors against piping. 1st one is suggested by Karl Tarzaghi. He says, the factor of safety against piping is $F.S = \frac{W'}{U}$. where,

W' = Submerged weight of the soil in heave zone per unit width of the hydraulic structure.

U = ~~soil~~ Upword ~~soi~~ seepage force on same volume of soil.





Now, volume of the sieve zone $= \frac{D}{2} \times D \times 1$

$$= \frac{D^2}{2}$$

where D is the depth of embedment of the structure / pile.

$\therefore U = \frac{D^2}{2} i_{av} \gamma_w$. $\quad i_{av}$ = avg. hydraulic gradient in the soil block.

$i_{av} = \frac{h_{av}}{L}$

According to Terzaghi Factor of safety is about 4 to 5.

~~If the Factor of safety is not in that range. It is needed to be increased.~~ That can be done by 2 ways -

① Increasing the weight, but it should not exert extra seepage pressure or upward thrust.

① Increase the Embedment depth D of the pile. This will reduce the value of U; increase the Factor of safety.

③ or, increase the length of the drainage path; but it is more costly.

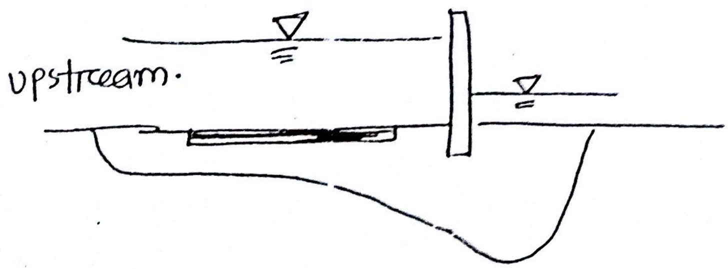
to increase W, ~~the~~ we use
~~If the base material is too fine use of~~ filter material to protect the base material. The filter material should not obstruct the flow.

~~If Fs doe snot do~~

If FS falls out of range, then we can –

① ~~②~~ use graded filter.

② Provide a 3'-4' thick layer of clay in the upstream. this will increase the drainage path's length



Filter material can also be used if the Base material needing protection is too fine

The 2nd Method is Harza's one. According to him

$$F.S = \frac{\text{critical hydraulic gradient, } i_c}{\text{Exit gradient, } i_{exit} \text{ of the nearest flow element}}$$

According to this method, F.S is about 3 to 4.