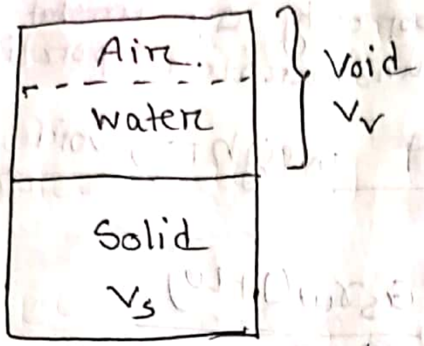
Soil Properties



Soil Volume

$V_{Total} = V_s + V_v$

$= V_s + V_a + V_w$

Weight, $W_T = W_s + W_a$

$= W_s + W_w$ [$W_{air} = 0$]

For dry soil

$V_{dry} = V_{solid} + V_{air}$

$W_d = W_s$ [$W_{air} = 0$]

For saturated soil

Saturated = void filled with water

$V_{sat} = V_s + V_w$

$W_{sat} = W_s + W_w$

Density & Unit weight

Density, $\rho = \frac{M}{V}$ (kg/m^3)

Unit weight, $\gamma = \frac{W}{V}$ (kN/m^3)

$\gamma = \rho g$

Sp. gravity of solids, $G_s = \frac{\gamma_{solid}}{\gamma_{water}}$

Unit weight of water, $\gamma_w = 9.81\ kN/m^3$

$= 62.43\ lb/ft^3$

Soil characteristics Parameters:

moisture content, $w(\%) = \frac{W_w}{W_s} = \frac{M_w}{M_s}$

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

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

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

degree of saturation, $S_r = \frac{V_w}{V_v} \times 100(\%)$

Air content, $A(\%) = \frac{V_a}{V} = n(1-S)$

[porosity এর unsaturated portion]

<u>Relationship between unit weight, void ratio & sp. gravity</u>

(moist) bulk unit weight, $\gamma_b = \frac{G_s \gamma_w (1+w)}{1+e}$

dry unit weight, $\gamma_d = \frac{G_s \gamma_w}{1+e}$

$\therefore \gamma_b = \gamma_d (1+w)$

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

*** water content & saturation percentage relationship

$Se = wG_s$ [$S=1$ for 100% saturation]

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

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

<u>In case of density</u>

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

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

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

* $\rho_w = 1000\, kg/m^3$
$= 1\, gm/cc$

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Max^m dry unit weight:

we know, $\gamma_{dry} = \frac{G_s \gamma_w}{1+e}$

we can see, γ_{dry} will be max^m if e is minimum.

again, $Se = wG_s$, then, e is minimum. when S is max i.e. $S=1$,

$\therefore e_{min} = wG_s$

$\therefore \gamma_{dry(max)} = \frac{G_s \gamma_w}{1+wG_s}$

It is also called "zero air void" unit weight

$\therefore \gamma_{ZAV} = \frac{G_s \gamma_w}{1+wG_s}$

Relative Density:

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

$$= \frac{\frac{1}{\gamma_{d(min)}} - \frac{1}{\gamma_d}}{\frac{1}{\gamma_{d(min)}} - \frac{1}{\gamma_{d(max)}}} \times 100\%$$

Volume of borrow pit & Volume of fill:

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

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Numericals

(i) Saturated soil, $w = 30\%$, $G_s = 2.65$, Determine dry unit weight. [PGCB'17]

Soln: $\gamma_{dry} = \dfrac{\gamma_w G_s}{1 + w G_s}$

$se = wG_s$
$\therefore 1 \cdot e = wG_s$
$\Rightarrow e = wG_s$

$= \dfrac{9.81 \times 2.65}{1 + 0.3 \times 2.65}$

$= 14.48\ KN/m^3$

(2) Find the unit weight of saturated soil, γ_{sat}. Given, $\gamma_{dry} = 110\ pcf$ & $G_s = 2.7$ [PGCL'17]

Soln: $\gamma_{dry} = \dfrac{G_s \gamma_w}{1+e}$

$\Rightarrow 110 = \dfrac{2.7 \times 62.43}{1+e}$

$\Rightarrow e = 0.53$

$\therefore \gamma_{sat} = \dfrac{(G_s + e)\gamma_w}{1+e} = \dfrac{(2.7 + 0.53) \times 62.43}{1 + 0.53} = 131.8\ pcf$ Ans

(3) Water content of a fully ~~standard~~ staurated soil sample is 25% , specific gravity, $G_s = 2.65$. Find moist unit weight, dry unit weight & void ratio [SGFL'17] [DNCC'16] [JGTDSL'21] ~~& porosity~~.

Soln: $Se = wG_s$

$\therefore e = 0.25 \times 2.65 = 0.6625$

$\therefore \gamma_{moist} = \dfrac{G_s \gamma_w (1+w)}{1+e} = \dfrac{2.65 \times 9.81 \times (1 + 0.25)}{1 + 0.6625} = 19.55\ KN/m^3$

$\gamma_{dry} = \dfrac{\gamma_{moist}}{1+w} = \dfrac{19.55}{1+0.25} = 15.637\ KN/m^3$ Ans

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(4) The difference between max^m & min^m void ratio of sand sample is 0.30. If the relative density of this sample is 66.67%. & field void ratio is 0.4. Find out the saturated density at its loosest condition. [CPGCBL'18] [RPGCL'17] [BPDB'16]
Take, $G_s = 2.7$

Solⁿ: We know,

$$D_r = \frac{e_{max} - e_0}{e_{max} - e_{min}}$$

$$\Rightarrow 0.6667 = \frac{e_{max} - 0.4}{0.3}$$

$$\Rightarrow e_{max} = 0.6$$

saturated density at loosest condition,

$$\gamma_{sat(min)} = \frac{(G_s + e_{max})\gamma_w}{1 + e_{max}}$$

$$= \frac{(2.7 + 0.6) \times 9.81}{1 + 0.6} = 20.23 \text{ KN/m}^3$$

(5) $100 cm^3$ sample of moist soil has a mass of 212.25gm. Water content 16%. Specific gravity of soil is 2.68. Calculate bulk unit weight of soil, dry unit weight of soil, void ratio & porosity. [RPGCL'22] [DNCC'16]

Solⁿ:

$$\gamma_{moist} = \frac{212.25}{100} = 2.12 g/cm^3$$

$$\gamma_{dry} = \frac{2.121}{1 + 0.16} = 1.83 g/cm^3$$

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

$$\Rightarrow e = \frac{G_s \gamma_w}{\gamma_{dry}} - 1 = \frac{2.68 \times 1}{1.83} - 1 = 0.465 = 46.5\%$$

$$\text{porosity}, n = \frac{e}{1+e} = \frac{0.465}{1 + 0.465} = 0.317 \text{ Ans.}$$

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(6) In a natural state, a moist soil has a volume of $0.33 ft^3$ & weight 39.9 lb. The oven dry weight of soil is 34.54 lb. If $G_s = 2.67$, calculate dry unit weight, γ_d and void ratio. [BCIC'17] [PGCB'15] [RPGCL'22]

Solⁿ: (i) Dry unit weight, $\gamma_d = \frac{W_s}{V}$

$= \frac{34.54}{0.33}$

$= 104.7 \ lb/ft^3$

(ii) $V_s = \frac{W_s}{G_s \gamma_w}$

$= \frac{34.54}{2.67 \times 62.43}$

$= 0.207 ft^3$

$$\boxed{\begin{aligned} \gamma_s &= \frac{W_s}{V_s} \\ \Rightarrow G_s \gamma_w &= \frac{W_s}{V_s} \end{aligned}}$$

$$\boxed{\begin{aligned} \text{or, } \gamma_d &= \frac{G_s \gamma_w}{1+e} \\ \Rightarrow e &= 0.59 \end{aligned}}$$

$V_v = V - V_s$

$= 0.33 - 0.207 = 0.123 ft^3$

$e = \frac{V_v}{V_s} = \frac{0.123}{0.207} = 0.59$ A

(7) The dry density of a soil is 1.58 gm/cc. If the saturation water content is 50% then what would be its saturated density & submerged density [BPDB'16]

Solⁿ:

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$\therefore \cancel{G_s = 2.52}$

$\therefore \cancel{\gamma_{sat} = \frac{(G_s+e)\gamma_w}{}}$

Solⁿ:

$\rho_{sat} = \rho_d(1+w)$ [যেহেতু fully saturated নয় তাই most unit weight (density) হিসেবে ধরতে হবে।]

$= 1.5(1+0.5)$

$= 2.25\ g/cc$

$\rho' = \rho_{sat} - \gamma_w$

$= 2.25 - 1$

$= 1.25\ g/cc$ A

(8) A soil sample has a porosity of 50% and specific gravity of 2.69. Calculate the dry density, void ratio and bulk density of soil, if the soil sample is 50% saturated.

Solⁿ: $n = 50\% = 0.5$

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

$\rho_d = \frac{G_s \rho_w}{1+e} = \frac{2.69 \times 1}{1+1} = 1.345\ gm/cc$

$Se = wG_s$

$\Rightarrow 0.5 \times 1 = w \times 2.69$

$\therefore w = 0.186$

$\rho_b = \rho_d * (1+w)$

$= 1.345(1+0.186)$

$= 1.595\ gm/cc$

(9)

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(9) percent of void is 50%. If specific gravity of soil is 2.7 then calculate submerged unit weight. [BPDB'15]

Soln: $\gamma' = \gamma_w \left(\frac{G_s - 1}{1+e}\right)$

$= 1 \times \frac{2.7-1}{1+0.5}$

$= 1.133$ gm/cc

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

(10) A moist soil sample of $100\,cm^3$ has a mass of ~~2.43~~ 178 gm. Water content 16% and specific gravity 2.68. Find (i) void ratio (ii) degree of saturation (iii) dry unit weight [PGCB'18]

Soln: $\gamma_{moist} = \frac{178}{100} = 1.78$ g/cc

$\gamma_{dry} = \frac{\gamma_{moist}}{1+w} = \frac{1.78}{1+0.16} = \frac{1.78}{1.16} = 1.534$ gm/cc

again, $\gamma_d = \frac{G_s \gamma_w}{1+e}$

$\Rightarrow 1.534 = \frac{2.68 \times 1}{1+e}$

$\Rightarrow e = 0.747$

$Se = wG_s$

$\Rightarrow S \times 0.747 = 0.16 \times 2.68$

$\Rightarrow S = 0.574 \quad \therefore S = 57.4\%$

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(11) A soil sample has a dry weight 40lb & moist weight 50lb. volume of soil is $0.2 ft^3$. Find the NMC (normal moisture content), bulk unit weight & dry unit weight. [GTCL'16]

Solⁿ: Normal moisture content, $NMC = \frac{50-40}{40} \times 100\%$
$= 25\%$

Bulk unit weight, $\gamma = \frac{50}{0.2} = 250\ lb/ft^3$

Dry unit weight, $\gamma_d = \frac{40}{0.2} = 200\ lb/ft^3$

(12) Water content, sp. gravity and void ratio of a soil sample are 20%, 2.68, 0.78 respectively. Now calculate (i) bulk unit weight (ii) Degree of saturation (iii) Porosity [BIWTA'19]

Solⁿ: (i) $\gamma_b = \frac{\gamma_w G_s (1+w)}{1+e}$

$= \frac{1 \times 2.68 \times (1+0.2)}{1+0.78} = 1.807\ gm/cc$

(ii) $Se = wG_s$

$\Rightarrow S = \frac{wG_s}{e} = \frac{0.2 \times 2.68}{0.78} = 0.6872$

$\therefore S = 68.72\%$

(iii) $n = \frac{e}{1+e} = \frac{0.78}{1+0.78} = 0.438$

$= 43.8\%$ A

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(13) In a field, a hole is cut off volume is $1.1\ ft^3$ and the wet mass of the hole is 130 lb and dry mass is 119 lb. Determine the degree of saturation if specific gravity is 2.7? [BWDB'16]

Sol^n: $w = \frac{(130-119)}{119} = 0.0924$

$\gamma_d = \frac{119}{1.1} = 108.2\ lb/ft^3$

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

$\Rightarrow e = \frac{G_s \gamma_w}{\gamma_d} - 1 = \frac{2.7 \times 62.43}{108.2} - 1 = 0.558$

$\therefore S = \frac{wG_s}{e} = \frac{0.0924 \times 2.7}{0.558} \times 100 = 44.71\%$

(14) Prove that $\gamma_{sat} = \left(\frac{e}{w}\right)\left(\frac{1+w}{1+e}\right)\gamma_w$ [BWDB'19]

$\because$ saturated
$\therefore V_v = V_w$

Sol^n: $\gamma_{sat} = \frac{W}{V} = \frac{W_s + W_w}{V_s + V_w}$

$= \cancel{\frac{G_s\gamma_w + wG_s\gamma_w}{1+e}}$

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

$= \frac{\frac{W_s}{V_s} + \frac{wW_s}{V_s}}{1+e}$

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

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

$$\boxed{\begin{aligned} &\gamma_s = \frac{W_s}{V_s} \\ &\Rightarrow G_s\gamma_w = \frac{W_s}{V_s} \\ &\therefore G_s = \frac{\gamma_s}{\gamma_w} \end{aligned}}$$

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

[$\because$ fully saturated $\therefore S=1$
$\therefore$ $Se = wG_s$
$\therefore$ $e = wG_s$
$\Rightarrow G_s = \frac{e}{w}$]

(15) A sample of a sand measured volume $12cm^3$ & weight 32.3gm. After oven dried the weight of ~~sample~~ same sample is to be measured 31.2gm. If the solid sand volume is $8.48 cm^3$, calculate the void ratio of the sample [DPDC'20]

Solⁿ: $w = \frac{32.3-31.2}{31.2} = 0.035$

$\gamma_d = \frac{\text{Solid weight}}{\text{Total volume}} = \frac{31.2}{12} = 2.6 gm/cc$

~~porosity~~ void ratio, $e = \frac{V_v}{V_s} = \frac{12-8.48}{8.48}$

$= 0.415$

(16) Saturated unit weight & moisture content are $19.3 KN/m^3$ & 28% respectively. Calculate void ratio e. [SGFCL'21] [JGTDSL'21]

Solⁿ: $Se = wG_s$

$\Rightarrow e = 0.28 G_s$

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

$\Rightarrow 19.3 = \frac{(G_s+0.28G_s)\times 9.81}{1+0.28G_s}$

$\Rightarrow 19.3 + 5.404G_s = 12.5568 G_s$

$\Rightarrow G_s = 2.7$

$\therefore e = 0.28\times 2.7 = 0.756$ Ans.

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(17) The volume & weight of moist soil are $0.006 m^3$ & 10kg. If the moisture content in soil & specific gravity are 13% and 2.71 then. Calculate the porosity [Titas'21] [JGTDSL'21]

Sol^n: $\gamma_{moist} = \frac{10}{0.006} = 1666.67 \, kg/m^3$

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

$\cancel{\Rightarrow 1666.67 = \frac{(2.71+e)\times 1000}{1+e}}$

$$\gamma_{moist} = \frac{G_s \gamma_w (1+w)}{1+e} \qquad \Big| \; \gamma_w = 1000 kg/m^3$$

$$\Rightarrow 1666.67 = \frac{2.71 \times 1000 (1+0.13)}{1+e}$$

$$\Rightarrow e = 0.837$$

(18) Show the saturated moisture content is $W_{sat} = \gamma_w \left[\frac{1}{\gamma_d} - \frac{1}{\gamma_s} \right]$

Sol^n:
we know,

$$se = wG_s \quad \text{---(i)} \quad [\because \text{for saturation, } s=1]$$

$$\therefore w_{sat} = \frac{e}{G_s}$$

again, $\gamma_d = \frac{G_s \gamma_w}{1+e}$ $\quad e = \frac{G_s \gamma_w}{\gamma_d} - 1$ ---(ii)

From eq^n (1)

$$w_{sat} = \frac{\frac{G_s \gamma_w}{\gamma_d} - 1}{G_s} = \frac{\gamma_w}{\gamma_d} - \frac{1}{G_s} = \frac{\gamma_w}{\gamma_d} - \frac{\gamma_w}{\gamma_s}$$

$$\boxed{\therefore w_{sat} = \gamma_w \left[\frac{1}{\gamma_d} - \frac{1}{\gamma_s} \right]}$$

Sub:

(19) If a soil sample has a dry unit weight of 19.5 KN/m^3, moisture content of 8% & a specific gravity of solids particles is 2.67. Calculate

(i) the void ratio

*** (ii) moisture & saturated unit weight

(iii) The mass of water to be added to cubic meter of soil to reach 80% saturation.

(iv) Is it possible to reach 20% water content without changing the void ratio?

Sol^n:

(i) $\gamma_{dry} = \frac{G_s \gamma_w}{1+e}$ $\therefore e = \frac{G_s \gamma_w}{\gamma_d} - 1 = \frac{2.67 \times 9.81}{19.5} - 1 = 0.343$

(ii) $\gamma_{moist} = \gamma_{dry} * (1+w) = 19.5 * (1+0.08) = 21.06\ KN/m^3$

$w_{sat} = \frac{Se}{G_s} = \frac{1 \times 0.343}{2.67} = 0.1285$

$\therefore \gamma_{sat} = \gamma_{dry}(1+w_{sat}) = 19.5 * (1+0.1285) = 22\ KN/m^3$

(iii) $\gamma_{moist} = 21.06\ KN/m^3$

$w_{80\%} = \frac{Se}{G_s} = \frac{0.8 \times 0.343}{2.67} = 0.10277$

$\therefore \gamma_{80\% sat} = 19.5 * (1+0.10277) = 21.5\ KN/m^3$

weight of water to be added $= 21.5 - 21.06 = 0.44\ KN/m^3$

$= \frac{0.44 \times 1000\ N}{9.81}$

$= 44.85\ Kg/m^3$ Ans

(iv) $S = \frac{w G_s}{e} \times 100\% = \frac{0.20 \times 2.67}{0.343} \times 100\%$

$= 155\% > 100\%$ which is not possible

So reaching 20% moisture content with same void ratio is not possible.

P.T.O

(v) Is it possible to compact the soil sample to a dry unit weight of 25 kN/m³

Soln: max^m dry density,

$$\gamma_{ZAV} = \frac{G_s \gamma_w}{1+e_{min}}$$

$$= \frac{2.67 * 9.81}{1+0.2136}$$

$$= 21.58\ kN/m^3 > 25\ kN/m^3$$

(Not possible)

$$e_{min} = \frac{G_s \cdot w}{S_{max}} = \frac{2.67 \times 0.08}{1} = 0.2136$$

(20) A borrow material has a volume of 191000 m³ and void ratio of 1.2. After compaction its new void ratio is 0.7, find the corresponding volume.

$\left[\text{i.e. } e = \frac{V_v}{V_s} \right]$

Soln: $e_1 = \frac{V_{T_1} - V_s}{V_s}$

$$\Rightarrow 1.2 = \frac{191000 - V_s}{V_s}$$

$$\Rightarrow V_s = 86818.18\ m^3$$

compact করার air, water বের হয়ে total volume কমে যায়। কমেনা কিন্তু volume of solid এটার সব ক্ষেত্রে স্থায়ী থাকে।

$$e_2 = \frac{V_{T_2} - V_s}{V_s}$$

$$\Rightarrow 0.7 = \frac{V_{T_2} - 86818.16}{86818.18}$$

$$\therefore V_{T_2} = 147590.906\ m^3 \quad \text{Ans}$$

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

$$\therefore V_2 = \frac{1+0.7}{1+1.2} * 191000 = 147590.91\ m^3 \quad \text{Ans}$$

(21) Spacific gravity of a soil is 2.65. Would it be possible to compact the above soil at a water content of 13.5% to a dry unit weight of 2t/m³?

Check if $\gamma_{ZAV} \geq 2.2 t/m^3$

$\gamma_{ZAV} = \frac{2.65 \times 1}{1+0.135 \times 2.65} = 1.95 t/m^3 < 2 t/m^3$ not possible

$\gamma_w = 1 \text{ ton}/m^3$

Soln:

lowest possible void ratio for this soil

$$e_{min} = \frac{G_s \cdot w}{S_{max}} = \frac{2.65 \times 0.135}{1} = 0.3577$$

$$\gamma_{dry} = \frac{G_s \gamma_w}{1+e} \Rightarrow 2 \times 9.81 \; kN/m^3 = \frac{2.65 \times 9.81}{1+e}$$

$$\therefore e = 0.325 < e_{min}$$

যেহেতু প্রাপ্ত e এর value lowest possible (not possible) value এর চেয়েও কম অতএব এক্ষেত্রে 2t/m³ dry unit weight attain করা possible না।

(22) 270000 ft³ আয়তনের একটি গর্ত মাটি দ্বারা ভরাট করতে হবে। এই স্থান থেকে মাটি কেটে আনা হয় ঐ স্থানের মাটির ইউনিট ওয়েট – $\gamma_{moist} = 105 \, lb/ft^3$, $w = 16\%$, $G_s = 2.75$ আর ভরাটকৃত মাটির ইউনিট ওয়েট – $\gamma_{dry} = 103.5 \, lb/ft^3$, $w = 18\%$, $G_s = 2.75$, তাহলে মাটি কোন স্থান থেকে কত আয়তনের মাটি কাটতে হবে?

Soln: ভরাটকৃত মাটির জন্য,

$$\gamma_d = \frac{G_s \gamma_w}{1+e_{fill}} \Rightarrow e_f = \frac{2.75 \times 62.43}{103.5} - 1 = 0.66$$

borrow pit এর জন্য,

$$\gamma_{moist} = \frac{G_s \gamma_w (1+w)}{1+e_{borrow}} \Rightarrow e_b = \frac{2.75 \times 62.43 (1+0.16)}{105} - 1 = 0.896$$

Now, $\frac{V_b}{V_f} = \frac{1+e_b}{1+e_f}$ $\therefore V_b = \frac{1+0.896}{1+0.66} \times 270000 \, ft^3$

$= 308385.54 \, ft^3$ A

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* An uncompacted heap of soil has a volume of $10,000 m^3$ and void ratio of 1. If the soil is compacted to a volume of $7500 m^3$. Then the corresponding void ratio of the compacted soil is ________ ?

Solⁿ:

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

$$\Rightarrow \frac{7500}{10000} = \frac{1+e_2}{1+1}$$

$$\Rightarrow 1.5 = 1+e_2$$

$$\Rightarrow e_2 = 0.5 \quad \text{Ans.}$$

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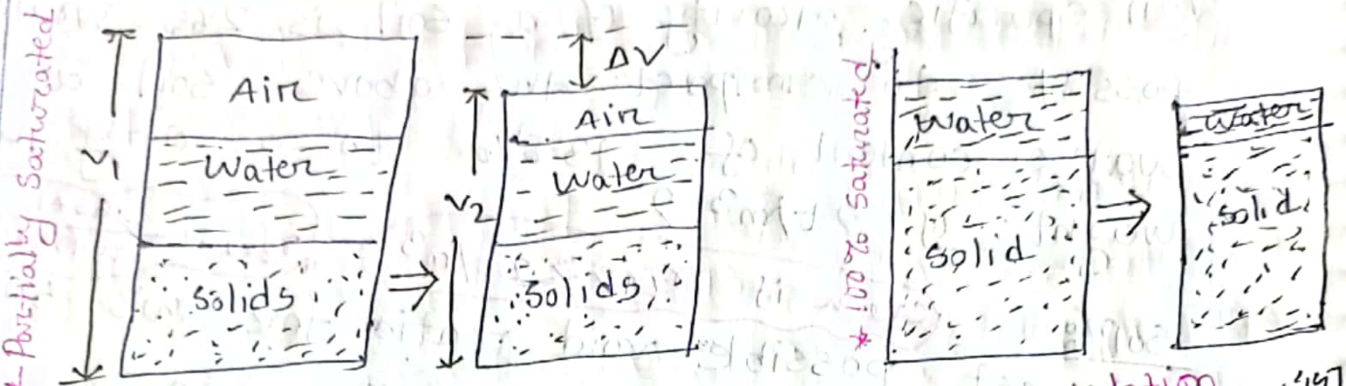

Compaction

Consolidation

Difference betwn compaction & consolidation [53 BMA] [Rajuk'15] [PGCB'17] [WZPDCL'16]

Compaction: Compaction is a mechanical process of densifying soil by reducing air voids.

* Compaction always occurs in partially saturated soil

* In the construction of different engineering structures, loose soil must be compacted to increase their stability & bearing capacity & to reduce settlements, permeability etc.

Consolidation: consolidation is an unwanted process of reduction in volume of sub-soil below the structure due to sustained static loading (self weight of structure)

* Consolidation always occurs in fully saturated soil.

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* Compaction is an instantaneous process whereas ~~compaction~~ consolidation is a time dependent process (লোডের জন্য নিচের মাটি থেকে pore water expulsion হয়ে আস্তে আস্তে বসে যায়)

* consolidation causes unwanted settlement & failure of structure whereas compaction is done to increase density, stability & shear strength of soil.

* Consolidation mainly occurs in cohesive soils ie clay whereas compaction can be done in any type of soil.

Decrease of Height due to compaction:

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

e_0 = initial void ratio

e_f = final void ratio

Compaction Tests:

(i) standard Proctor Test (SPT) method

(ii) Modified Proctor Test (MPT) method

Test method	Mould volume (ft^3)	weight of hammer lb	Height of Hammer (in)	No of layers	No of blows per layer
SPT	1/30	5.5	12	3	25
MPT	1/30	10 [DNCC'16]	18	5	25

Compaction Effort Comparison between SPT & MPT:

compaction effort বলতে বুঝায় energy per unit volume অর্থাৎ একক mould volume এর জন্য pound এর hammer দিয়ে যত উঁচু থেকে যত layer এ compact করা হয় তাকেই compaction effort.

$$\frac{E_m}{E_s} = \frac{5 \text{ layers}}{3 \text{ layers}} * \frac{10 \text{ lb}}{5.5 \text{ lb}} * \frac{18''}{12''} * \frac{25 \text{ blows}}{25 \text{ blows}}$$

$$= 4.54$$

[BPDB'16]

$\therefore E_m = 4.54 E_s$

অর্থাৎ Modified compaction test এর compaction effort standard compaction test এর compaction effort থেকে 4.54 গুণ বেশি।

* Energy of standard proctor test = 596 kJ/m³ [BIWTA'19] [CPGCBL'22]

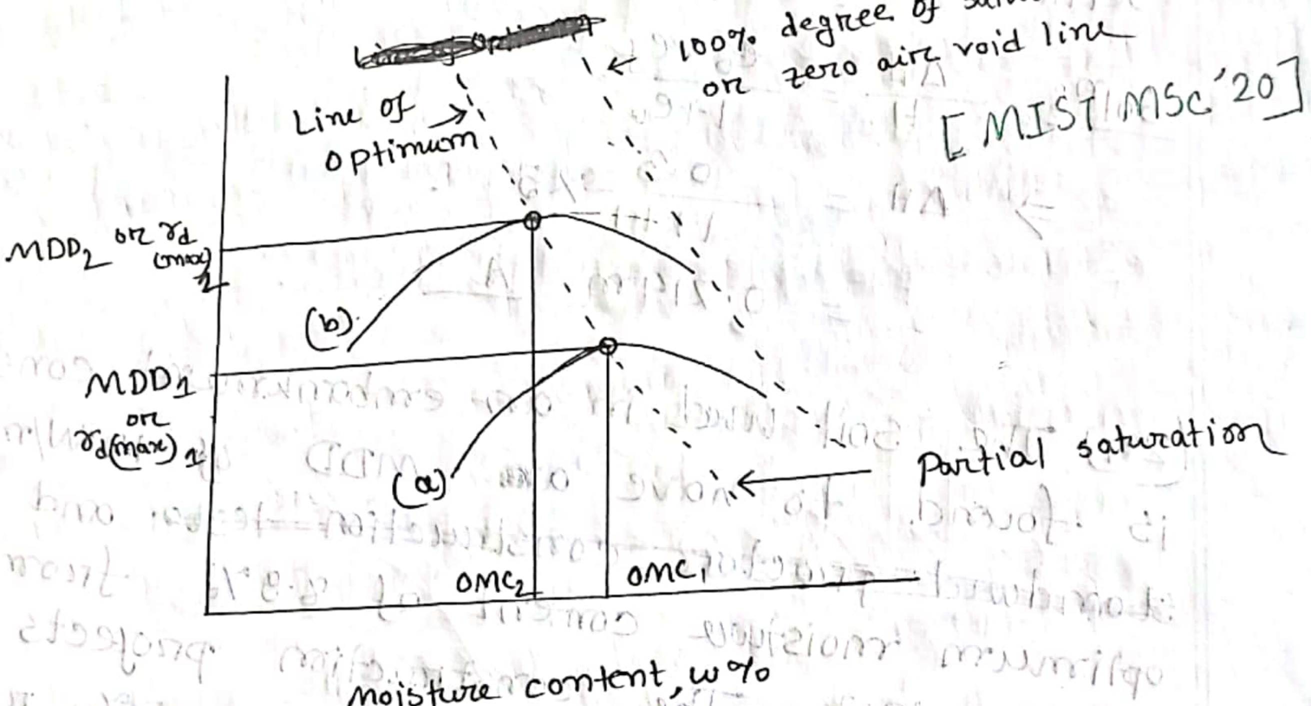
Some Important Aspects:

* moisture content বাড়ার সাথে সাথে dry density বাড়তে থাকে, একটি নির্দিষ্ট moisture content এর জন্য dry density maxm হয়। সেই moisture content কে optimum moisture content (OMC) বলে।
* moisture content এর পরও বাড়ানো হলে dry unit weight or dry density কমতে থাকে, আবার dry

Sub:

* compaction effort বেশি হলে ~~moisture content~~ Dry unit weight ও বাড়ে।

* $\boxed{\text{Max}^m \text{ dry density} = MDD = \gamma_{d(max)}}$



(a) Standard Proctor Test
(b) Modified Proctor Test

* Relative Compaction:

$$R = \frac{\gamma_{d(field)}}{\gamma_{d(max)}} \times 100\%$$

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(23) The initial void ratio of a 5m thick clay layer is 1.1. If the difference of void ratio is 0.3 than find out the settlement? [MSC'18]

Solⁿ: $\frac{\Delta H}{H} = \frac{e_0 - e_f}{1+e_0}$

$\Rightarrow \Delta H = \frac{0.3}{1+1.1} \times 5$

$= 0.714\,m$ A

(24) The soil used in an embankment construction ~~from~~ is found to have an MDD of $19\,kN/m^3$ ~~standard proctor construction tests~~. and an optimum moisture content of 8.9% from standard proctor test. The construction projects requires the soil to be compacted to 95% relative density.

After finishing compaction, a block of soil is extracted from the compacted layer using a pipe section of 100mm length & 75mm internal dia. Wet & dry weight of extracted soil is 871gm & 810gm respectively. Does the layer meet the required specifications? [DMICL'19]

Solⁿ: $\gamma_d = \frac{810 \times 10^{-3}}{\frac{\pi}{4} \times \left(\frac{75}{1000}\right)^2 \times \frac{100}{1000}} = 1833.465\,kg/m^3$

$= \frac{1833.465 \times 9.81}{1000}\,kN/m^3$

$= 17.98\,kN/m^3$

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* A one-dimensional consolidation test is carried out on a standard 19mm thick clay sample. The oedometer's deflection gauge indicates a reading of 2.1mm, just before removal of the load, without allowing any swelling. The void ratio is 0.62 at this stage. Calculate the initial void ratio.

Soln:

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

initial void ratio (CT 22)

$$\Rightarrow \frac{2.1}{19} = \frac{e_0 - 0.62}{1+e_0}$$

$$\Rightarrow 0.11 = \frac{e_0 - 0.62}{1+e_0}$$

$$\Rightarrow 0.11 + 0.11e_0 = e_0 - 0.62$$

$$\Rightarrow (1-0.11)e_0 = 0.11 + 0.62$$

$$\therefore e_0 = 0.82 \quad \underline{A}$$

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$$R_d = \frac{17.986}{19} \times 100 \approx 95\% \text{ (ok)}$$

(25) A project requires fill to be compacted to 95% relative density with relation to the standard proctor. Laboratory results for the standard proctor indicated that the soil has a max^m dry density (MDD) of $19 \, KN/m^3$ and an optimum moisture content (OMC) of 8.9%.

After compaction, the compacted fill soils have an insitu unit weight of $19.54 \, KN/m^3$ & moisture content of 7.5%. Does the compacted soil meet project requirements? DMTCL'22

Sol^n: dry density of compacted soil,

$$\gamma_d = \frac{\gamma_{moist}}{1+w}$$

$$= \frac{19.54}{1+0.075} = 18.177 \, KN/m^3$$

$$R_d = \frac{18.177}{19} \times 100 = 95.6\% > 95\% \text{ (ok) Ans}$$

* OMC এর কোনো use নাই প্রশ্নের

** In a standard proctor test, 1.8kg of moist soil was filling the mould (volume = 944cc) after compaction. A soil sample weighting 23g was taken from the mould and ovendried for 24hrs at a temperature of 110°c. Weight of the dry sample was found to be 20g. Specific gravity of soil solids = 2.7. calculate the theoritical maximum dry unit weight of the soil at that water content.

Soln:

$$w = \frac{23-20}{20} = 0.15$$

$$Se = wG_s \quad \therefore e_{min} = \frac{wG_s}{S} = \frac{0.15 \times 2.7}{1} = 0.405$$

$$\therefore \gamma_{d(max)} = \frac{G_s \gamma_w}{1+e_{min}} = \frac{2.7 \times 9.81}{1+0.405}$$

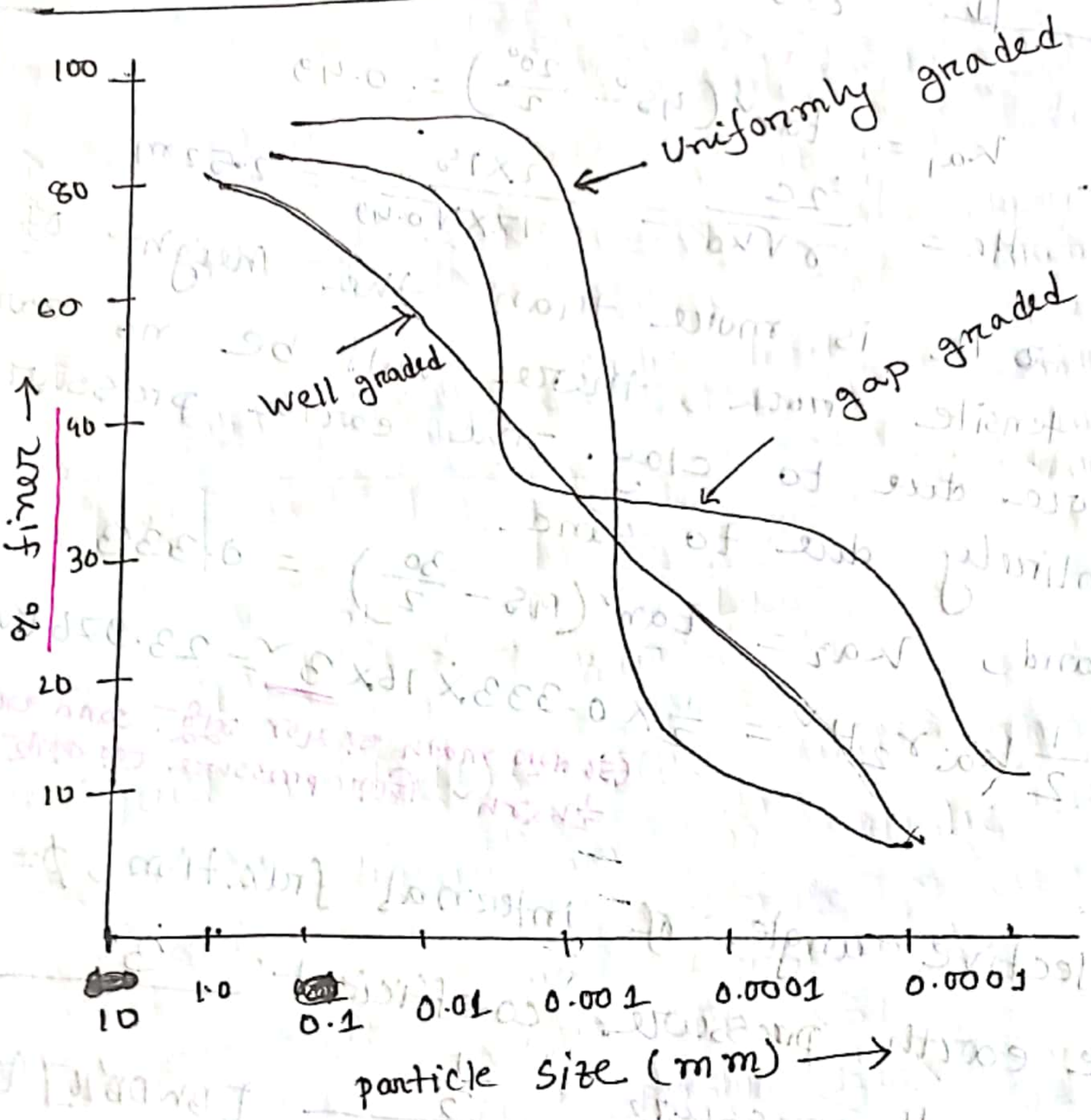
$$= 18.85 \, kN/m^3 \; Ans$$

Soil classification based on grain/particle size:

Gravel = > 4.75mm
Sand = 4.75 to 0.075 mm
Silt & clay = < 0.075mm

} based on USCS = Unified Soil classification system

Particle Size Distribution Curve: [DNCC'2020] [Titas'14]



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Grading Characteristics:

D_{60} = Dia corresponding to 60% finer

D_{75} = Dia corresponding to 75% finer

D_{10} = Dia corresponding to 10% finer

D_{30} = Dia corresponding to 30% finer

* D_{10} = Effective size

*Why D10 is called effective diameter?:

As per the numerous experiments done by Allen Hazen on soil properties, he found the D10 diameter can be related to most of the soil properties such as liquid limit, plasticity index, unconfined compressive strength, permeability etc hence it is called effective diameter.

* Uniformity coefficient, $C_u = \frac{D_{60}}{D_{10}}$

*Curvature coefficient/coefficient of gradation, $C_c = \frac{D_{30}^2}{D_{10} \times D_{60}}$

~~* Sorting coefficient, $S_0 = \sqrt{\frac{D_{75}}{D_{25}}}$~~

For well graded soil, $C_c = 1 \sim 3$

$C_u > 4$ for sand

$C_u > 6$ for gravel

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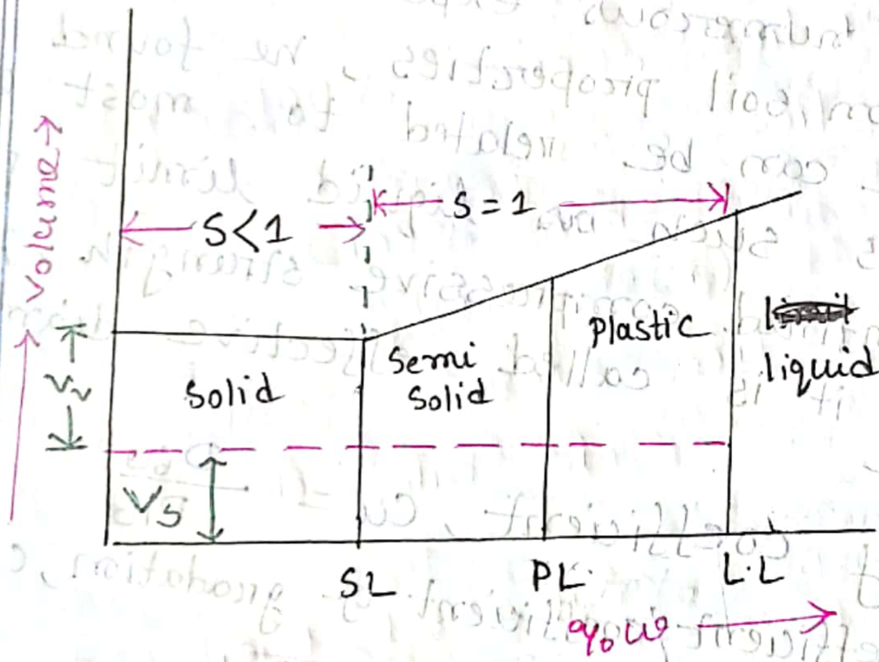
Consistency limit / Atterberg limit :

The water contents at which the soil changes from one state to the other are known as consistency limits or Atterberg limits.

* Atterberg was a swedish agriculture engineer who correlated this



volume of solid fixed কিন্তু water content বাড়ছে সাথে সাথে volume of void বাড়ছে

* **Liquid Limit (LL) :** যেটুকু water content হলে plastic state এ থাকা- অবস্থা soil liquid state থেকে plastic ~~[illegible]~~ plastic থেকে ~~[illegible]~~ ওপরের যেটুকু water থাকলে liquid এ আসতে শুরু করে Liquid Limit

Plastic Limit (PL) : ~~[illegible] PL ⟺ SL~~

w% plastic $\overset{w\%}{\Longleftrightarrow}$ semi-solid

Shrinkage limit (SL) : Semi solid $\overset{w\%}{\Longleftrightarrow}$ Solid

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Flow Index, I_F: The flow index is the rate at which a soil mass loses its shear strength with an increase in water content.

$$I_F = \frac{w_1 - w_2}{\log\left(\frac{N_2}{N_1}\right)}$$ | N = No of blows

Liquid Limit, LL:

Laboratory test formula

$$LL = w \times \left(\frac{N}{25}\right)^{0.121}$$ | N = no of blows to obtain 12.7mm contact length of soil

Plasiticity Index, I_P: It is the range in moisture content when the soil exhibits its plastic behavior.

$I_P = LL - PL$

Liquidity Index, I_L: Liquidity index দিয়ে বোঝায় soil এ কত percent water আছে ~~add~~, ~~তাহলে এর~~ liquid state এর ~~liquid state এ পরিণত হবে।~~

$$I_L = \frac{w - PL}{I_P} \times 100\% = \frac{w - PL}{LL - PL} \times 100\%$$ | w = in-situ moisture content in soil

যেমন I_L যদি 20% হয় তাহলে liquid state এর 20% water আছে আর 80% water add করলে পুরো liquid এ পরিণত হবে।

* When the soil is at its liquid limit, its liquidity index is 100% and it behave as a liquid. When the

P.T.O

soil is at plastic limit, its liquidity index is zero. 100% পানি add করলেও এখন এটির liquid state এ যাবে।

<u>**Consistency Index, I_c:**</u> I_c দিয়ে soil এ Plastic limit এর চেয়ে কত percent water content আছে তা বুঝায়।

$$I_c = \frac{LL - w}{I_p} \times 100\%$$

$I_c = 100\%$ means soil is in plastic state

$I_c = 0\%$ means soil is in liquid state

(100% পানি কমলে তবে plastic state এ আসতে চাইলে)

$I_c > 100\%$ means soil is in semi-solid or solid state

$$\boxed{I_L + I_c = 100\%}$$ always

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* Gap filling:

(i) PL = 30%, LL = 60%, ω = ~~…~~ 40%, I_p = ? I_L = ? [MSC'19][NHA'19] [JGTDSL'21]

Soln: $I_p = LL - PL = 60 - 30 = 30\%$

$$I_L = \left(\frac{\omega - PL}{I_p}\right) \times 100\%$$

$$= \left(\frac{40-30}{30}\right) \times 100\% = 33.33\%$$

(ii) $I_p = 50\%$, $PL = 20\%$, $LL = ?$ [BWDB'18] [NHA'19] [GTCL'16] [BIWTA'19] [SGCL'20] [~~…~~] [Petrobangla'22]

Soln: $I_p = LL - PL$

$\therefore LL = I_p + PL = 50 + 20 = 70\%$

(iii) Liquidity Index of a sand having equal natural water content & plastic limit is zero [SGFL'17] [BIWTA'19]

(iv) The effective size of a soil is D_{10} [BPDB'2013]

(v) The soil transported by wind is called Aeolian soil [BPSC]

(vi) $D_{10} = 0.1$, $D_{60} = 0.3$, uniformity coefficient, ~~…~~ [DNCC'16] [GTCL'16] [NHA'19] [BIWTA'19]

$C_u = \frac{0.3}{0.1} = 3$ [Petrobangla'22]

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

Solⁿ: For 10 blows,

$$LL_1 = w_1\left(\frac{N_1}{25}\right)^{0.121}$$
$$= 70\times\left(\frac{10}{25}\right)^{0.121}$$
$$= 62.7\%$$

For 100 blows,

$$LL_2 = w_2\left(\frac{N_2}{25}\right)^{0.121}$$
$$= 20\times\left(\frac{100}{25}\right)^{0.121}$$
$$= 23.7\%$$

$\therefore LL = (62.7 - 23.7)\% = 39\%$

[61] Grain size distribution of a soil are given bellow: Find. Cu, Cc. Is the soil well graded? [PGCB'19]

particle size (mm)	% finer
0.05	10%
0.3	30%
0.4	60%

Solⁿ: $C_u = \frac{D_{60}}{D_{10}} = \frac{0.4}{0.05} = 8$

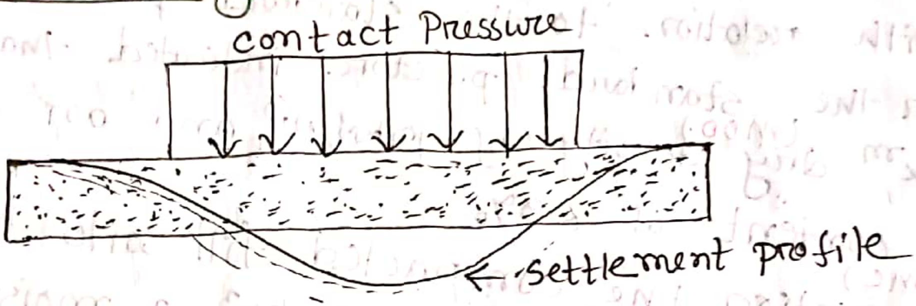
$C_c = \frac{D_{30}^2}{D_{10}\times D_{60}} = \frac{(0.3)^2}{0.05\times 0.4} = 4.5$

It is not well graded soil. For well graded soil $C_u > 6$, $1 < C_c < 3$

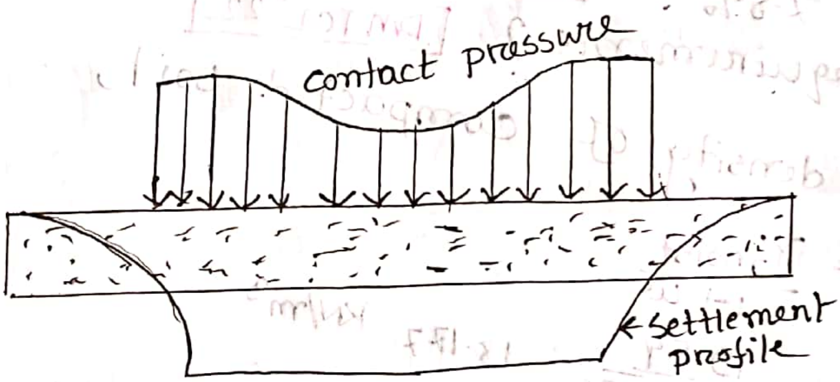
Sub : Consolidation & Settlement

Elastic settlement Profile in soil

For Clay soil :

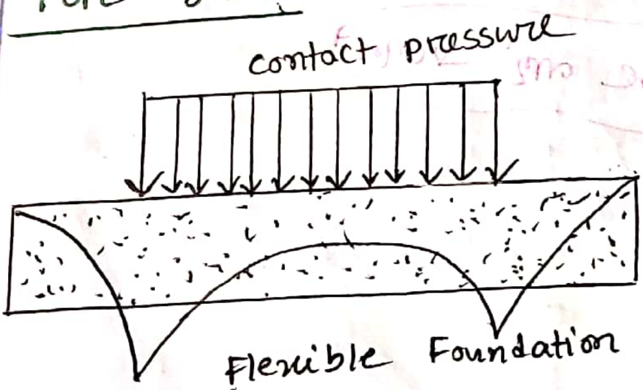


Flexible Foundation

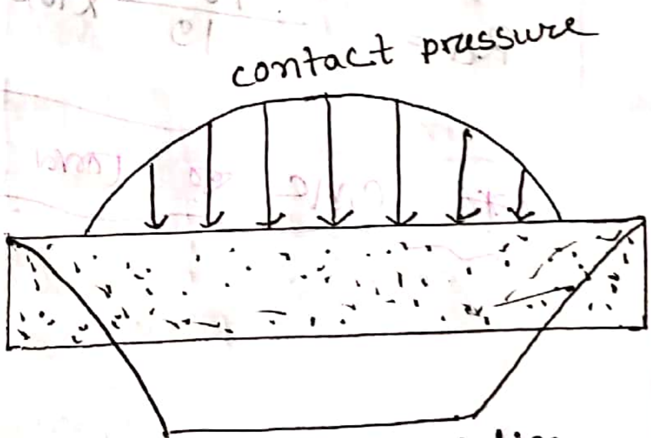


Rigid ~~pavement~~ Foundation

For Sand



Flexible Foundation

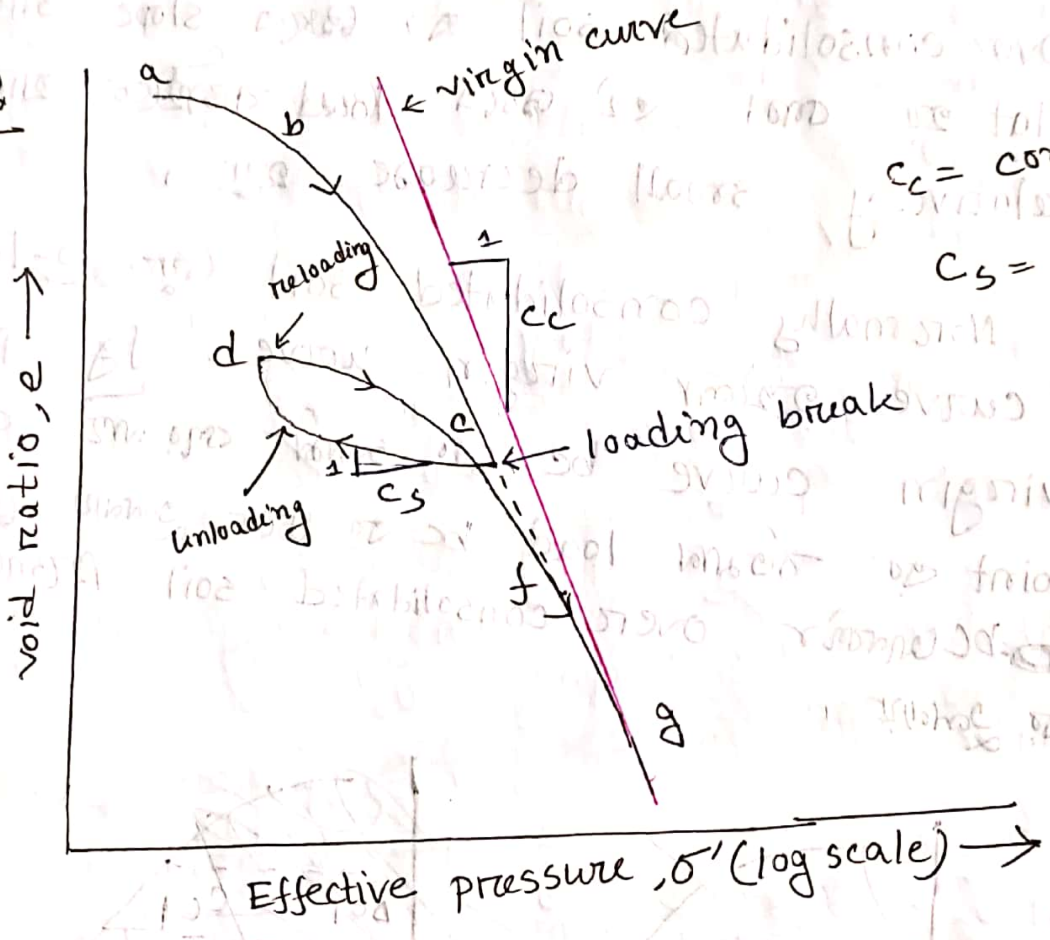


rigid Foundation

(25) Draw e-log σ' curve for loading & unloading

[NESCO'21] [COXDA'19] [BEPZA'16] [BMA]

<u>Soln:</u>



C_c = compression Index

C_s = swelling index

<u>Normally consolidated Clay:</u>

A soil is NC if the present effective pressure to which it is subjected is the max^m pressure the soil has ever been subjected to.

The branches bc & fg are NC clay

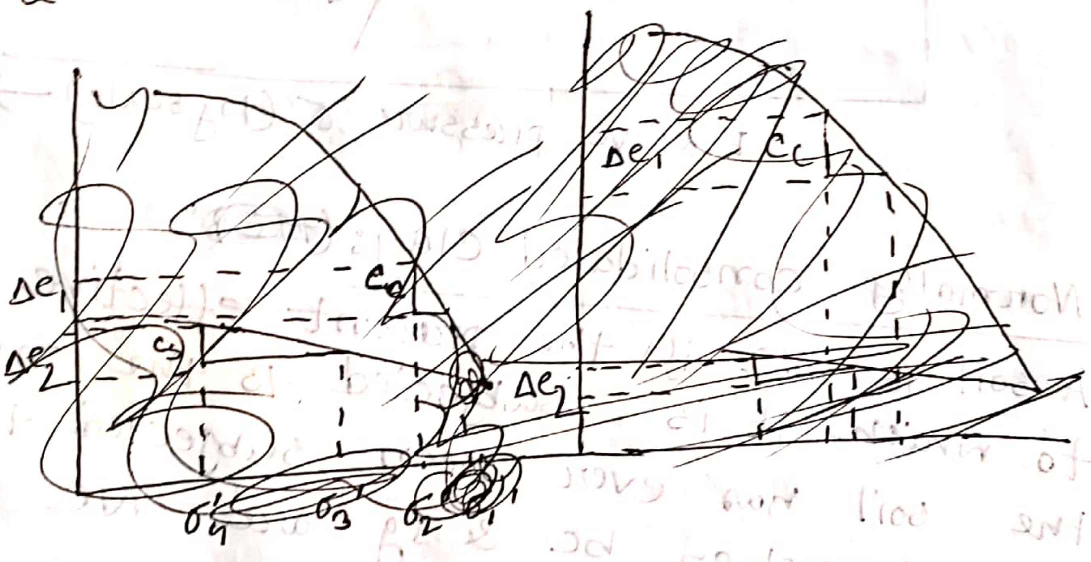
<u>Over consolidate Clay:</u>

A soil is OC if the present effective pressure to which it is subjected to is less than the max^m pressure to which the soil was subjected to in the

past.
The soil branches ab, cd, df are the OC state of a soil.

* Over consolidated soil এর ক্ষেত্রে slope সাধারণত flat হয় কারণ এই ক্ষেত্রে load বাড়াইলে আপেক্ষাও e relatively small decrease হয়।

* Normally consolidated soil এর e-$\log \sigma'$ curve হইলো virgin curve. fg point এর virgin curve bc এর সাথে যুক্ত নাই কারণ fg point এর বর্তমান load bc এর চেয়ে অনেক কম। সেজন্য fg point over consolidated soil হিসেবে ধরা হয় bc অপেক্ষা এর তুলনায়।



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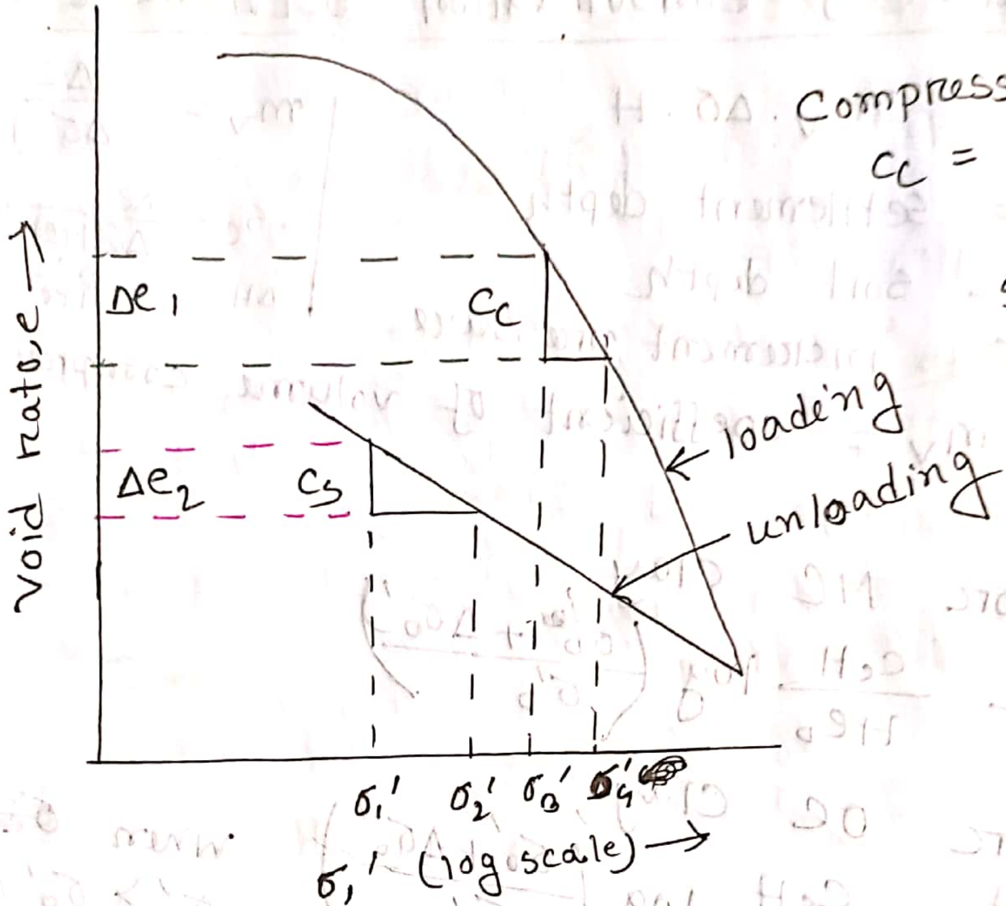

Compression Index,

$$C_c = \frac{\Delta e_1}{\log \frac{\sigma_4'}{\sigma_3'}}$$

Swelling Index

$$C_s = \frac{\Delta e_2}{\log \frac{\sigma_2'}{\sigma_1'}}$$

* The max^m effective past pressure is called the preconsolidation pressure.

* Over consolidation ratio (OCR) = $\frac{\text{preconsolidation stress}}{\text{current stress}}$

OCR = 1 ⇒ NC clay

OCR > 1 ⇒ OC clay

* Effective stress decreases with the increase in void ratio [BCIC'16]

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Calculation of Consolidation Settlement :

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

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

$\therefore S_c = \frac{\Delta e}{\Delta\sigma(1+e)} * \Delta\sigma * H$

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

* ΔH = settlement depth

H = soil depth

$\Delta\sigma$ = increment pressure

m_v = coefficient of volume compressibility

(2) For NC clay

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

(3) For OC clay,

(i) $\Delta H = \frac{C_s H}{1+e_0} \log\left(\frac{\sigma_0' + \Delta\sigma_0'}{\sigma_0'}\right)$ when $\sigma_c' \geq \sigma_0' + \Delta\sigma_0'$

(ii) $\Delta H = \frac{C_s H}{1+e_0} \log\frac{\sigma_c'}{\sigma_0'} + \frac{C_c H}{1+e_0} \log\left(\frac{\sigma_0' + \Delta\sigma_0'}{\sigma_c'}\right)$

when, $\sigma_c' < \sigma_0' + \Delta\sigma_0'$

* σ_c' = preconsolidation pressure

We know, settlement = ΔH

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

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

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

* Time required to reach specific degree of consolidation,

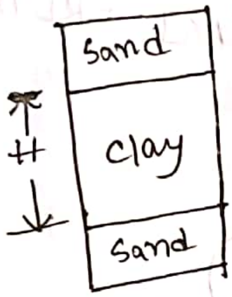
$$t = \frac{\frac{\pi}{4} * H_e^2 * \cancel{\ } D_c^2}{C_v}$$

D_c = Degree of consolidation

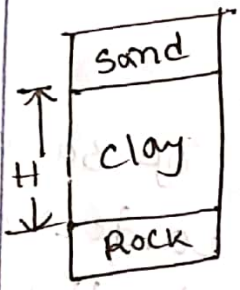
C_v = consolidation rate coefficient

~~H = Drainage path~~

H_e = effective drainage path



$\Rightarrow H_e = \frac{H}{2}$



$\Rightarrow H_e = H$

* One method of reducing differential settlement of a building on sand is to decrease the size of the smallest footing.

size ছোট করলে area কমবে, pressure বাড়বে। ছোট বড় ফুটিং এর সমান হবে। তখন differential settlement কমবে।

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(26) compute consolidation settlement of a 2.5m thick clay layer due to an increase of 30KN/m² pressure at the mid height of the layer. ~~the~~ The vertical stress at the mid height of the layer is 40KN/m², initial void ratio is 0.7 & compression index is 0.28. [NHA'19] [WRGCL'19] [GTCL'16] [BWDB'18] [ERL'17] [SGFL'17] [PGCB'15] [DNCC'15]

Solⁿ:

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

$$= \frac{0.28 \times 2.5}{1+0.7} \log\left(\frac{40+30}{40}\right)$$

$$= 0.100\,m = 10\,cm \quad \underline{Ans}$$

(27) A 3m layer double drainage of saturated clay under surcharge loading underwent 90% consolidation in 75 days. Find the coefficient of consolidation? [PGCL'17]

Solⁿ:

$$t = \frac{\frac{\pi}{4} * H_e^2 * D_c^2}{C_v}$$

double drainage

$\therefore H_e = \frac{3}{2} \times 100\,cm = 150\,cm$

$\Rightarrow$ ~~Cv = ...~~

$$\Rightarrow C_v = \frac{\pi/4 * (150)^2 * 0.9^2}{75 \times 24 \times 60}$$

$= \cancel{0.0022\,cm^2/sec}$

$= 0.1325\,cm^2/min$

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DMTCL-2022 [Non-25 → DMTCL-5 No math; Depth - 5X13 = 65]

* Given, initial void ratio and effective stresses are 0.8 & 200 KN/m³. If compressibility index, $C_c = 0.33$ & final stress is 400 KN/m³. Find final void ratio.

[DMTCL'22]

Soln:

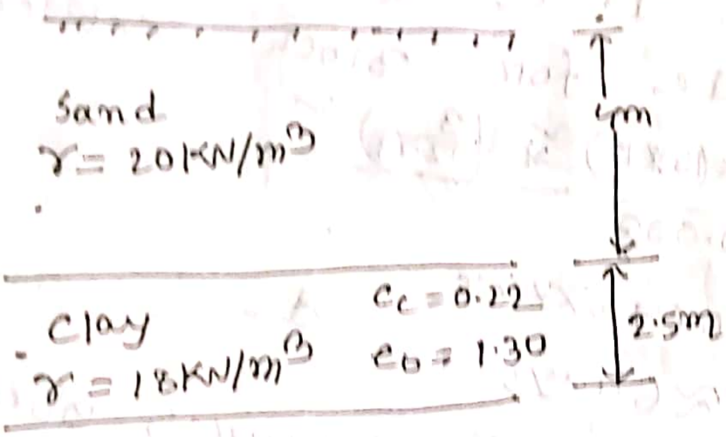
$$C_c = \frac{e_1 - e_2}{\log \frac{\sigma_2'}{\sigma_1'}}$$

$$\Rightarrow 0.33 = \frac{0.8 - e_2}{\log \frac{400}{200}}$$

$$\Rightarrow 0.099 = 0.8 - e_2$$

$$\therefore e_2 = 0.8 - 0.099 = 0.7 \quad \text{Ans}$$

(35) calculate the final settlement of the clay layer shown in figure due to an increasing pressure of $30 kN/m^2$. Also calculate the settlement when the water table is at ground surface.



* distance সবসময় clay soil এর mid থেকে নিতে হবে।
* settlement হয় clay soil এ, sand এ নয়।

Sol^n
$$\sigma_0 = 4\times 20 + \frac{2.5}{2}\times 18 = 102.5\ kN/m^2$$

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

$$= \frac{0.22\times 2.5}{1+1.30}\log\frac{102.5+30}{102.5}$$

$$= 0.0266\ m = 2.66\ cm$$

* (settlement হচ্ছে clay soil এ তাই height ও হবে clay soil এরই উচ্চতা)

When the water table rises to the ground surface,

$$\sigma_0 = 4\times(\gamma_{sat}-\gamma_w) + \frac{2.5}{2}(\gamma_{sat}-\gamma_w)$$

Submerged unit weight

$\boxed{\text{Taking } \gamma_w = 10\ kN/m^2}$

$$= 4\times(20-10) + \frac{2.5}{2}(18-10) = 50\ kN/m^2$$

$$\therefore S_c = \frac{0.22\times 2.5}{1+1.30}\log\left(\frac{50+30}{50}\right) = 0.0488\ m = 4.88\ cm$$

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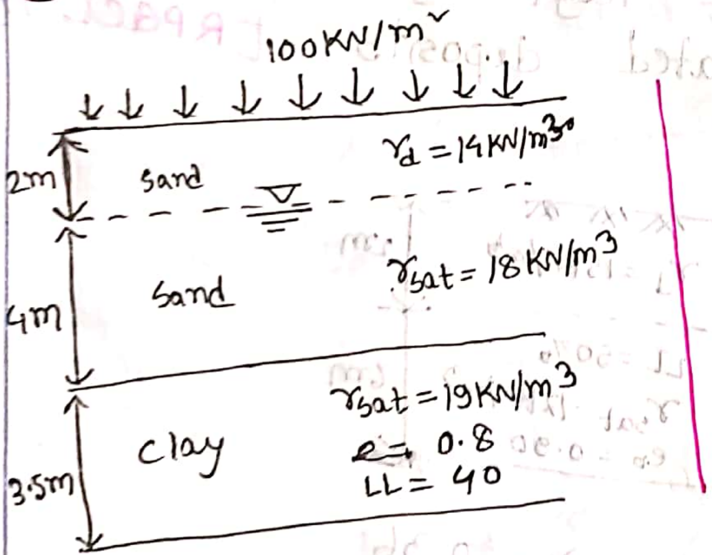
(36) A soil profile is shown in below figure. If a uniformly distributed load $\Delta\sigma$, is applied at the ground surface what is the settlement of the clay layer caused by primary consolidation if [PGCB'18]

(a) The clay is normally consolidated

(b) The preconsolidation pressure $\sigma_c' = 200 KN/m^2$ [BWDB'19]

(c) $\sigma_c' = 150 KN/m^2$; Take $C_s = \frac{1}{5} C_c$



$C_c = 0.009(LL-10)$
$= 0.009(40-10) = 0.27$

Solⁿ: $\sigma_0' = 2\times14 + 4\times(18-9.81) + \frac{3.5}{2}(19-9.81) = 76.84 \ KN/m^2$

$$S_c = \frac{0.27\times3.5}{1+0.8}\log\left(\frac{76.84+100}{76.84}\right) = 0.19m = 19cm$$

(b) $\sigma_0' + \Delta\sigma' = 100 + 76.84 = 176.84 < \sigma_c'$

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

$$= \frac{\frac{1}{5}\times0.27\times3.5}{1+e_0}\log\left(\frac{176.84}{76.84}\right) = 3.8cm$$

P.T.O

Sub :

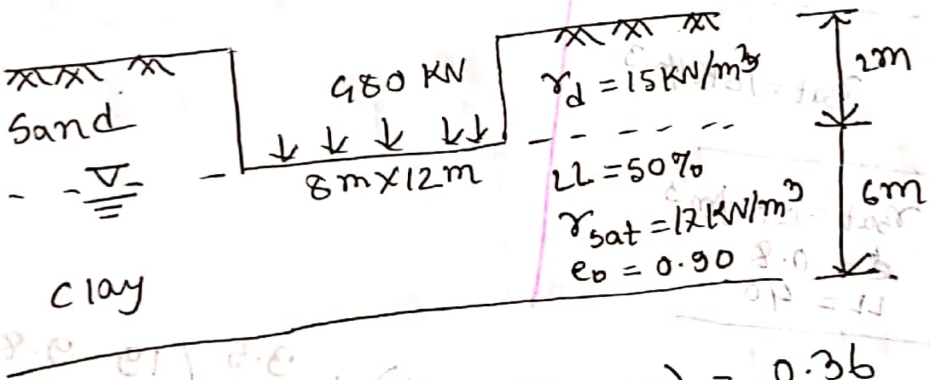
(c) $\Delta\sigma_0' + \Delta\sigma_0 = 176.84 > \sigma_c' = 150 \text{ kN/m}^2$

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

$$= \frac{\frac{1}{5} * 0.27 * 3.5}{1+0.8} \log \frac{150}{76.84} + \frac{0.27 * 3.5}{1+0.8} \log \left(\frac{176.84}{150} \right)$$

$$= 0.068 \text{ m} = 6.8 \text{ cm} \quad A$$

(37) Determine primary consolidation settlement at centre of the raft foundation. Assume normally consolidated deposits. [RPGCL'17]



Solⁿ:

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

$$\sigma_0' = 2 \times 15 + \frac{6}{2} * (17 - 9.81) = 51.57 \text{ kN/m}^2$$

$$\Delta\sigma' = \frac{480}{8 \times 12} = 5 \text{ kN/m}^2$$

$$\therefore S_c = \frac{0.36 \times 6}{1+0.9} \log \left(\frac{51.57 + 5}{51.57} \right) = 0.0456 \text{ m} = 4.56 \text{ cm} \quad A$$

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(28) Determine the coefficient of volume compressibility if the settlement is 5cm for 7.5m clay layer having increasing pressure of 8 kg/cm^2 ~~8 kg/cm~~ [BPDB'16]

Soln: $S_c = m_v \Delta\sigma' H$

$5 = m_v \times 8 \times (7.5 \times 100)$

$\Rightarrow m_v = 8.33 \times 10^{-4}$ cm^2/kg

(29) Calculate the total settlement of compressible soil stratum 2m depth and co-efficient volume compressibility 0.2cm^2/kg under a pressure increment of 2kg/cm^2.

Soln: $S_c = 0.2 * 2 * (2 \times 100) = 80$ cm Ans

*** (30) A clay stratum 5m thick has the initial void ratio of 1.5 & the effective overburden pressure of 120 kN/m^2. what is the increased pressure when void ratio decreases to 1.44. Value of co-efficient of volume compressibility is $2*10^{-4}$ m^2/KN. Also calculate the final settlement of the stratum. [CPGCBL-18] [RPGCL-17]

Soln: We know,

$m_v = \frac{\Delta e}{\Delta\sigma(1+e_0)}$ $\Rightarrow \Delta\sigma = \frac{\Delta e}{m_v(1+e_0)} = \frac{(1.5-1.44)}{2\times10^{-4}\times(1+1.5)}$

$= 120$ kN/m^2

now, $S_c = m_v \times \Delta\sigma \times H$

$= 2\times10^{-4} \times 120 \times 5$

$= 0.12$ m $= 12$ cm Ans

CamScanner

(32) For 80% consolidation of 2m thick soil, time taken is 5 years. Determine the time for 5m thickness soil for the same amount of consolidation? [COXDA '2019]

Solⁿ: $t \propto H^2$ for same % of consolidation

$$\Rightarrow \frac{t_1}{t_2} = \frac{H_1^2}{H_2^2}$$

$$\Rightarrow t_1 \times \frac{H_2^2}{H_1^2} = t_2$$

$$\Rightarrow 5 \times \frac{25}{4} = t_2$$

$$\Rightarrow t_2 = 31.25 \text{ years}$$

* **Hydraulic gradient:** The loss of hydraulic head per unit length of flow may be expressed as hydraulic gradient, $i = \frac{\Delta h}{L}$

Discharge through soil: , $Q = kiA$

k = hydraulic conductivity / permeability coefficient
unit = cm/sec
A = X-sectional area

Relation between k & e (void ratio)

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

$$\therefore \frac{k_1}{k_2} = \frac{\frac{e_1^3}{1+e_1}}{\frac{e_2^3}{1+e_2}}$$

Quicksand (চোরাবালি), $i = \frac{\gamma_{submerged}}{\gamma_w}$

in quick sand condition,

$$\therefore i = \frac{\gamma'}{\gamma_w} = \frac{G_s - 1}{1+e}$$

cohesionless soil i.e. sand এর সম্পূর্ণরূপে shear strength & bearing capacity নষ্ট হয়ে গিয়ে তরলের মত- আচরণ করে তখন, quicksand condition বলে।

Sub :

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(38) A field sample of an unconfined aquifer is packed in a test cylinder. The length and diameter of the cylinder are 50cm and 6cm respectively. The field sample tested for a period of 3min under a constant head difference of 16.3 cm. As a result, 45.2cm³ of water is collected at the outlet. Determine hydraulic conductivity of the aquifer sample [SGFL 21]

Solⁿ: $Q = KiA$

$$\Rightarrow K = \frac{Q}{iA}$$

$$= \frac{\frac{45.2}{3}\ cm^3/min}{\frac{16.3}{50} \times \frac{\pi}{4} \times 6^2} = 1.6346\ cm/min$$

(39) Calculate the hydraulic head that would produce a quick condition in a sand stratum of thickness 1.47m. Given, $\gamma_d = 1.65\ gm/cc$, sp. gr = 2.67 [RPGCL'17] [BPDB'14]

Solⁿ: We know, $\gamma_d = \frac{G_s \gamma_w}{1+e}$

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

$$= \frac{2.67 * 1}{1.65} - 1 = 0.62$$

$$i = \frac{G_s - 1}{1+e} = \frac{2.67 - 1}{1 + 0.62} = 1.03$$

again, $i = \frac{\Delta h}{L}$ $\therefore \Delta h = i \times L = 1.03 \times 1.47 = 1.5141\ m$

A

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(40) A falling head permeability test was performed in a ~~perame~~ permeameter with an inside diameter of 6cm. The inside diameter of standpipe is 3mm. The sample had a length of 12cm. During a period of 8minutes the head of sample decreased from 120 cm to 60cm. Compute the coefficient of permeability of the sample [Titas'18]

Solⁿ:

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

$$= \frac{0.0707\times 12}{28.27\times 8}\ln\left(\frac{120}{60}\right)$$

$$= 2.6\times 10^{-3}\ \text{cm/min}$$

A

Pipe area, $a = \frac{\pi}{4}\times(0.3)^2$
$= 0.0707\ \text{cm}^2$

sample area, $A = \frac{\pi}{4}\times 6^2$
$= 28.27\ \text{cm}^2$

(inside ধরে or soil sample থাকে।)

Length of sample, $L = 12\ \text{cm}$

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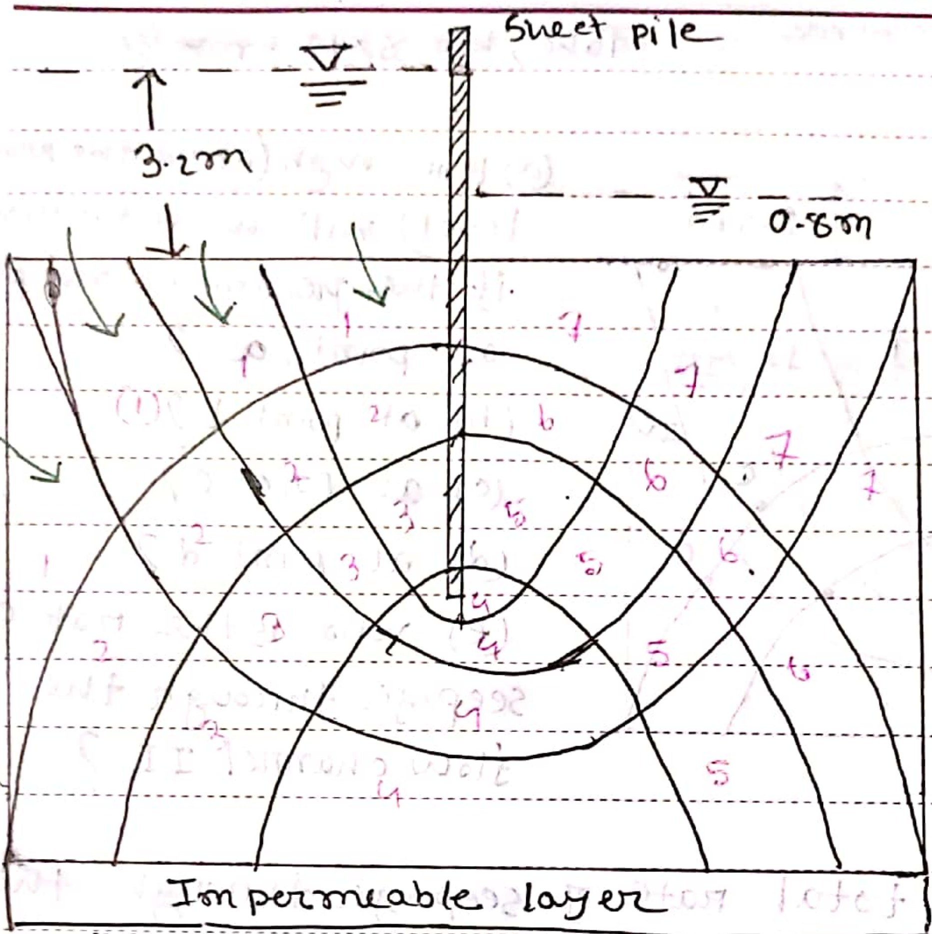


Calculate the ~~spe~~ seepage loss per meter length of the sheet pile.
Take, $k = 6.3 \times 10^{-4}$ cm/sec

Soln: No of flow channel, $N_F = 4$

No of equipotential drop, $N_d = 7$

$\therefore$ Seepage loss, $Q = k * \Delta H * \frac{N_f}{N_d}$

$$= 6.3 \times 10^{-6} \frac{m}{sec} \times (3.2 - 0.8)m \times \frac{4}{7}$$

$$= 8.64 \times 10^{-6} \; m^3/sec/m \text{ length}$$

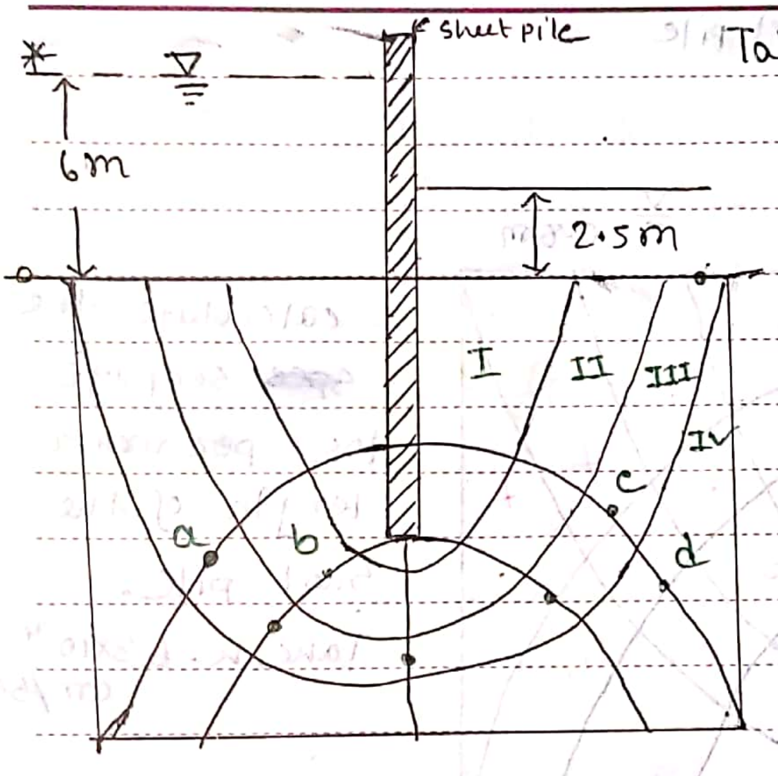
Note:

No of Flow line $= 4 + 1 = 5$ (3 টা line + sheet পাইলের ছুঁয়ে যাওয়া + Impermeable layer ছুঁয়ে যাওয়া)

No of equipotential line $= 7 + 1 = 8$

Date: / /

Sun | Mon | Tue | Wed | Thu | Fri | Sat



Take, $k = 8 \times 10^{-5}$ m/sec

(a) How high (above the ground level) will the water rise if the piezometer are placed at point a?

(b) at point b?

(c) at point c?

(d) at point d?

(e) what is the rate of seepage through the flow channel II?

(f) what is the total rate of seepage through the permeable layer per unit width?

drop গুলোর গুণতে point ধরে, 6m থেকে 6টি drop হয়ে 2.5m হয়েছে।

Solⁿ: No of flow channel, $N_F = 4$

No of equipotential drop, $N_d = 6$

~~Piezometric height at any point~~

Potential drop at any point $= \left[\dfrac{H_1 - H_2}{N_d}\right] \times n$

$= \left(\dfrac{6 - 2.5}{6}\right) n$

$= \dfrac{7}{12} n$

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(a) Piezometric height at point a, $= 6 - \frac{7}{12}n$

$= 6 - \frac{7}{12} \times 1$

$= 5.4167\,m$

(b) Piezometric height at point b, $= 6 - \frac{7}{12} \times 2$

$= 4.83\,m$

(c) ~~at~~ Piezometric height at point c $= 6 - \frac{7}{12} \times 5$

$= 3.083\,m$

(d) Piezometric height at point d $= 6 - \frac{7}{12} \times 5$

$= 3.083\,m$

(e) rate of seepage through the flow channel II

$$Q_2 = K * \Delta H * \frac{1}{N_d}$$

$\rightarrow N_f = 1$ কারণ মাত্র 1টা channel এর জন্য।

$= 8 \times 10^{-5} \times (6 - 2.5) \times \frac{1}{6}$

$= 0.000046\ m^3/s/m$

(f) total rate of seepage through the ~~channel~~ permeable layer $= k * \Delta H * \frac{N_f}{N_d}$

$= 8 \times 10^{-5} \times (6 - 2.5) \times \frac{4}{6}$

$= 0.0001867\ m^3/s/m$

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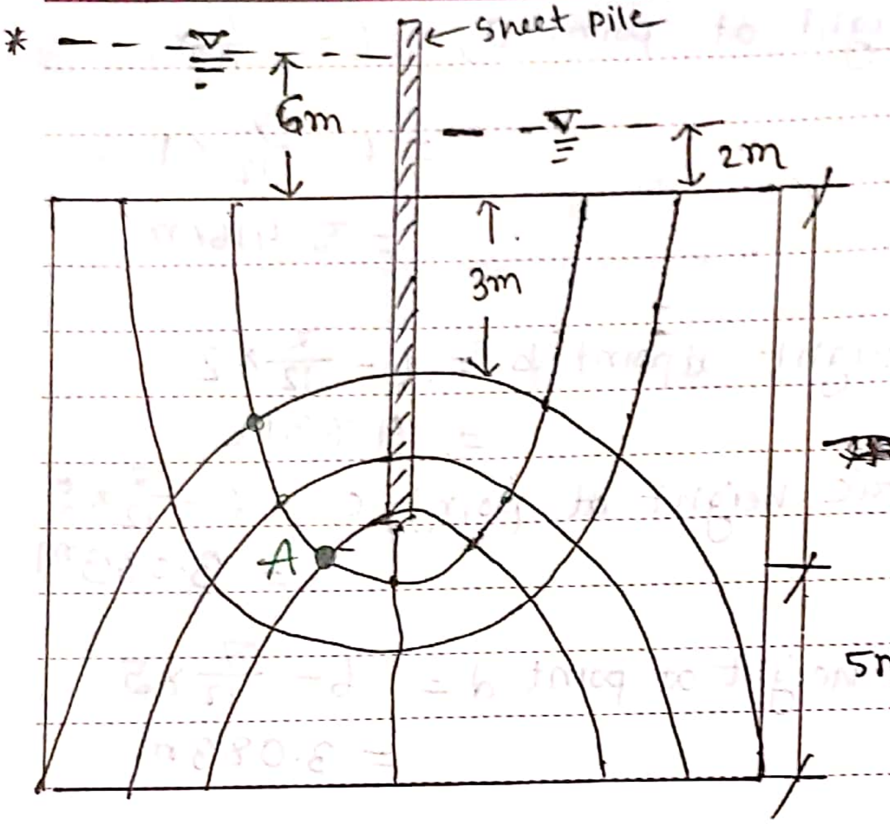

7.5m

5m

Saturated unit weight of the soil is 24 KN/m³.

(a) calculate vertical effective stress at A.

Soln: No of potential drops, $N_d = 8$

pressure drop at point A, $h_A = \left(\frac{6-2}{8}\right) \times 3$ m

$= 1.5$ m

Total head at point A, H_A = ~~6+1.5 = 7.5m~~

$= (6 + 7.5 - 1.5)$ m

= ~~11.5m~~

$= 12$ m

Total vertical pressure at point A, $\sigma = 6 \times 9.81 + 7.5 \times 24$ (soil)

$= 238.86$ KN/m²

Pore water pressure at point A, $U = 12 \times 9.81$

$= 117.72$ KN/m²

∴ Effective stress, $\sigma' = \sigma - U = (238.86 - 117.72) = 121.14$ KN/m²

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(c) From previous question -

if, specific gravity & void ratio of soil are 2.65 & 0.72 respectively, then calculate the Factor of safety against the occurrence of piping failure.

Soln: $i_{exit} = \dfrac{\Delta h}{\text{length of last flow element}}$

$= \dfrac{\left(\frac{6-2}{8}\right)}{3} = 0.167$

$i_{critical} = \dfrac{G_s - 1}{1+e} = \dfrac{2.65-1}{1+0.72} = 0.96$

$\therefore FS = \dfrac{i_{cr}}{i_{exit}} = \dfrac{0.96}{0.167} = 5.75$ Ans

Sub: In Situ Stresses of Soil

Day

Time: Date: / /

Soil mass এর total stress/pressure কে দুই ভাগে ভাগ করা যায়।

(i) Pore water pressure, u
(Water table থাকলে এর জন্যে)

(ii) Effective stress
(water effect বাদ দিয়ে যা থাকে তা)

* water table ~~এর~~ থাকলে effective stresses এর জন্য pressure
হবে submerged unit weight

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

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

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

---

(41) Find the vertical effective stress and total stress in a depth of 8m where the unit weight of saturated soil is 18 KN/m³ and unit weight of water 9.81 KN/m³? [BWDB'13]

Sol^n: Pore water stress $= 8 \times 9.81 = 78.48$ KN/m²

~~total effective stress =~~

effective stress $= (18 \times 8) - 78.48 = 65.52$ KN/m²

Total stress $= 78.48 + 65.52 = 144$ KN/m²

A

Sub:

Day | Time: | Date: / /

(42) Determine the total stress and effective stress at a depth of 5m below the top of water level at swimming pool. [CPGCBL'18]

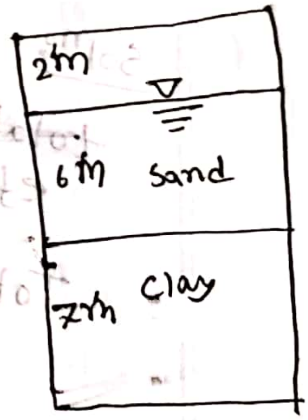
Solⁿ: Total stress = $9.81 \times 5 = 49.05$ KN/m^2

Effective stress = Total stress − pore water stress

$= 49.05 - 49.05$

$= 0$ A

(43) A sand layer has 8m depth. water table is at 2m depth below GL. The sand layer is overlying a clay layer of large depth. The moist (above WT) 16KN/m^3 and saturated unit weight of sand is 20KN/m^3 and saturated unit weight of clay is 18KN/m^3. Find the total & effective pressure at 15m depth. [GTCL'16] [DNCC'16]



Solⁿ: effective pressure,

$\sigma' = 2\times16 + 6\times(20-9.81) + 7\times(18-9.81)$

$= 150.47$ KN/m^2

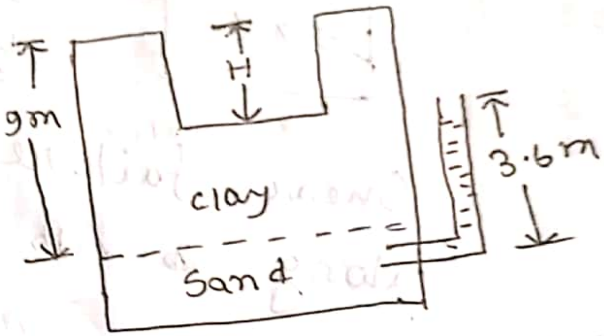
Total pressure $= 150.47 + 13\times9.81 = 278$ KN/m^2

A

Sub : Day Time : Date : / /

(44) A 9m thick layer of stiff saturated clay is underlained by a layer of sand. The sand is under artesian pressure of 3.6m water head. Calculate the max^m depth of cut that can be made in the clay. [BD-china'16]

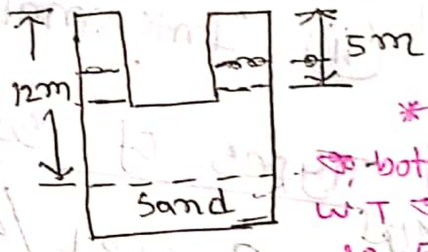
unit weight of clay = 18 kN/m^3



Sol^n: $H = 9 - \frac{3.6 * 9.81}{18}$

$= 7.04\ m$

(45) A saturated clay layer of 12m underlined by a sand layer. water table is 5m below the ground surface. Determine the max^m depth of cut of clay layer. The saturated unit weight of clay is 18.9 kN/m^3. [DWASA'21]



Sol^n: ∴ $H_{max} = 12 - \frac{7 * 9.81}{18.9}$

$= 8.369\ m$ Ans

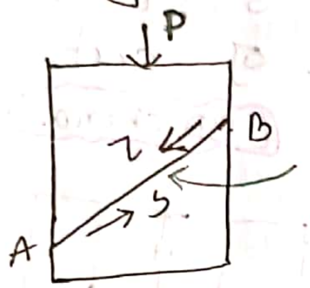
* Clay layer bottom থেকে W.T এর height 12 − 5 = 7 m

Sub: Sear Strength of Soil

Day

Time: Date: / /

Shear failure means sliding or slipping along a plane.



Typical plane of failure

τ = Applied shear stress due to load P

s = shear resistance

Shear failure occurs when $\tau > s$ by sliding along ~~a~~ AB

Factors Affecting shear stress of soils:

(viva)

(i) Frictional reistance (ϕ)

(ii) Intermolecular attraction / Cohesion (c)

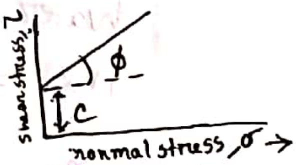
Types of soils based on shear properties:

(viva)

[BCIC '17]

* Gravel / sand $\Rightarrow$ ϕ (frictional soil)
* Silty soil $\Rightarrow$ $c-\phi$ ($c-\phi$ soil)
* clay $\Rightarrow$ c (pure cohesive soil / frictionless soil)

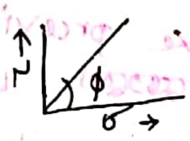
Shear strength of soil, $\tau = c + \sigma' \tan\phi$



$c-\phi$ soil

* σ' = effective shear stress

$$\boxed{\begin{array}{l}\text{For clay, } \phi = 0 \text{ (cohesive soil)} \\ \text{For sand, } c = 0 \text{ (frictional soil)}\end{array}}$$



frictional soil ($c=0$)

cohesive soil ($\phi=0$)

Sub:

Day

Time: Date: / /

*** Undrained shear strength, c_u

$$= \frac{\text{unconfined compressive strength}}{2}$$

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

---

(45) A sample of dry sand was tested in direct shear apparatus under a normal load of 36kg. The sample failed under a shearing load of 58lb. The sample is 2"X2". Find out the internal friction. [BWDB'18] [NHA'19] [PGCB'15] [Petrobangla 22] [GTCL'16] [CPGCBL'21] [NESCO'21]

Solⁿ: Shear stress, $\tau = \frac{58}{2\times 2} = 14.5 \text{psi}$

Normal stress, $\sigma = \frac{36\times 2.205}{2\times 2} = 19.84 \text{psi}$

We know, $\tau = c + \sigma \tan\phi$ [$\because$ for sand $c=0$]

$\Rightarrow 14.5 = 0 + 19.84 * \tan\phi$

$\therefore \phi = 36.16^\circ$ Ans.

(46) The unconfined compressive strength 120 KN/m². Find undrained shear strength? [APSCL'20] [EGCB'20] [DWASA'17]

Solⁿ: $c_u = \frac{120}{2} = 60 \text{ KN/m}^2$ Ans.

* $\sigma = 39$ KPa, $\tau = 56$ kPa, $c = 25$ kPa, $\phi = ?$ [NESCO'21]

Solⁿ: $\tau = c + \sigma\tan\phi$ $\Rightarrow \phi = \tan^{-1}\left(\frac{\tau - c}{\sigma}\right) = \tan^{-1}\left(\frac{56-25}{39}\right) = 38.48^\circ$ Ans.

Sub: ______ Day: ______ Time: Date: / /

(46) In an unconfined compression test sample of clay 8cm long & 4cm dia fails at a load 12kg at 7% strain. Find undrained shear strength of clay taking in account that the effect of change in x-section of sample. [BPDB'15]

Solⁿ:

Area, $A_0 = \frac{\pi}{4} * 4^2 = 12.57 \text{ in}^2$

~~Actual~~ Final area, $A = \frac{A_0}{1-\varepsilon} = \frac{12.57}{1-0.07} = 13.5 \text{ inch}^2$

প্রশ্নে শুধুমাত্র length এর strain দেওয়া থাকলেও এই formula. No change হবে

$q_u = \frac{12 \times 2.205}{13.5} = 1.96 \text{ psi}$

$\therefore C_u = \frac{1.96}{2} = 0.98 \text{ psi}$ (A)

(47) A sand layer having properties, $\gamma_d = 18 KN/m^3$, ~~$\gamma_{sat} = 16 KN/m^3$~~, $G_s = 2.65$, $\phi = 35°$. Water table is 2.4m below the ground level. Determine shear strength at a depth of 5m from ground level. [BPDB'18]

Solⁿ: $\gamma_d = \frac{G_s \gamma_w}{1+e}$

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

$= \frac{2.65 \times 9.81}{18} - 1$

$= 0.44$

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

$= \frac{(2.65+0.44) \times 9.81}{1+0.44}$

$= 21 \text{ KN/m}^3$

$\sigma' = 2.4 \times 18 + 2.6 \times (21 - 9.81) = 72.3 \text{ KN/m}^2$

$\therefore$ shear stress, $\tau = C + \sigma \tan\phi$ [sand $\therefore c = 0$]

$= 0 + 72.3 \tan 35°$

$= 50.62 \text{ KN/m}^2$

Sub : Day Time : Date : / /

Soil sensitivity

$$\text{Sensitivity, } S_t = \frac{q_u(\text{undistubed})}{q_u(\text{remoulded})}$$

* $q_u = 50\,kPa$ (undisturbed),
$= 25\,kPa$ (remoulded)
calculate sensitivity ? [BWDB'18] [NHA'19] [BITWA'19]

Sol^n: $S_t = \frac{50}{25} = 2$ [CPGCBL'22]

* sensitivity of a soil indicates its weakening due to remouldeing. If sensitivity is 1, the soil is insensitive.

Liquefaction: [MSC'13] [48BMA]

When saturated sandy soil is subjected to earth quakes loads, the pore water pressure suddenly increases and thus decreases the shear strength of soil and it may become zero also. The soil momentarily liquefies and behaves as a dense fluid. This phenomena when sand loses it is shear strength due to oscillatory motion is called Liquefaction of sand.

Soils most susceptible to liquefaction are saturated ~~fin~~ sand.

Shear strength $\tau = 0$ হলে Liquid

Sub : Lateral Earth Pressure   Day   Time :   Date : / /

Types of Lateral earth pressure :

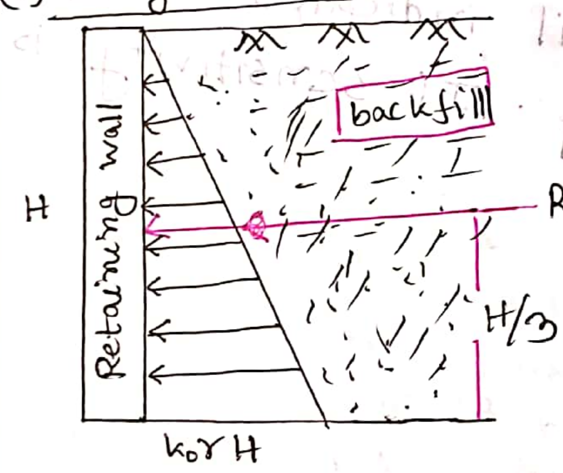
(i) Earth pressure at rest
(ii) Active earth pressure
(iii) Passive earth pressure

Estimation of earth pressure at rest :

(i) only backfill

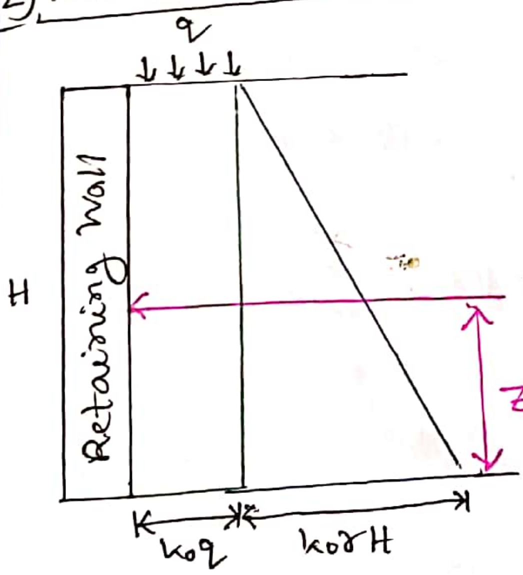
$k_o = 1 - \sin\phi$

earth pressure coefficient at rest $= k_o$



Resultant force, $P_o = \frac{1}{2} k_o \gamma H^2$ (area of triangle)

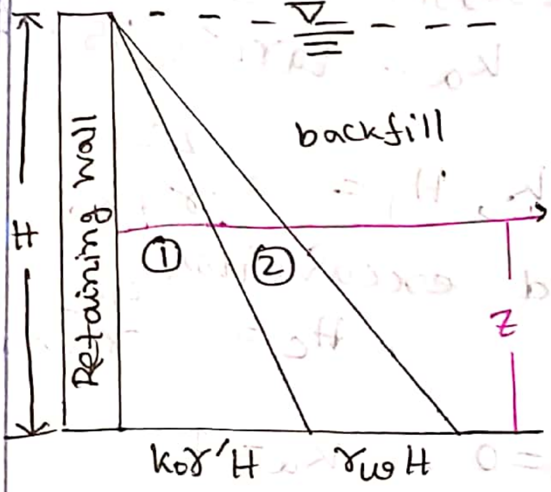
(2) With surcharge, q



$P_o = P_1 + P_2 = k_o q H + \frac{1}{2} k_o \gamma H^2$

$$Z = \frac{\frac{P_1 H}{2} + \frac{P_2 H}{3}}{P_o}$$

(3) Water table on Top :



$$P = \frac{1}{2}k_o\gamma' H^2 + \frac{1}{2}\gamma_w H^2$$

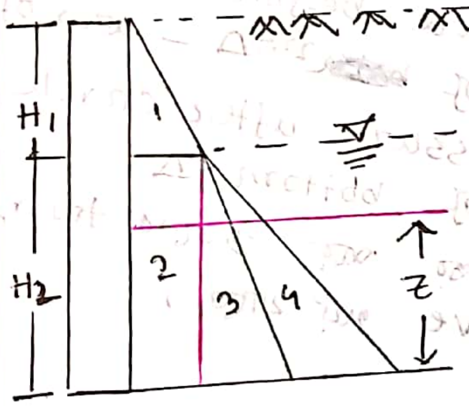
$\frac{1}{2}k_o\gamma' H^2$ → due to submerged soil

$\frac{1}{2}\gamma_w H^2$ → due to water table

water pressure এর জন্যে K_o আসবে না

$$Z = \frac{P_1 * \frac{H}{3} + P_2 * \frac{H}{3}}{P}$$

(4) Water table at some distance below Ground level :



$$P = \underset{①}{\frac{1}{2}k_o\gamma h_1^2} + \underset{②}{k_o\gamma h_1 h_2} + \frac{1}{2}k_o\gamma' H_2^2 + \frac{1}{2}\gamma_w H_2^2$$

P_1

উপরের ত্রিভুজটার অংশকে surcharge হিসেবে ধরে নিয়েছি with load = $k_o\gamma h_1$

Estimation of Active Earth Pressure:

Active earth pressure coefficient

$$k_a = \tan^2\left(45 - \frac{\phi}{2}\right)$$

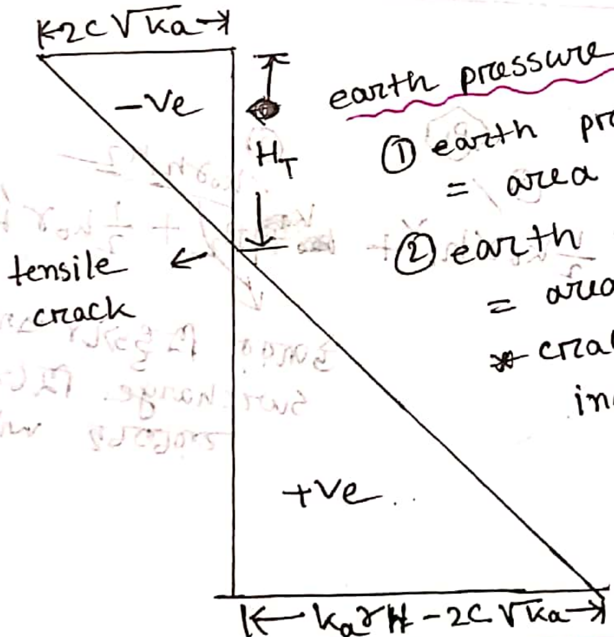
***depth of tensile crack, $H_T = \dfrac{2c}{\gamma\sqrt{k_a}}$

***Max^m depth of unsupported excavation

$$H_c = \frac{4c}{\gamma\sqrt{k_a}}$$

*clay soil এর ক্ষেত্রে $\phi = 0$ $\therefore$ $k_a = 1$

Force distribution of active earth pressure:



earth pressure = area of triangle

earth pressure before crack

① earth pressure = area of ~~b~~ 2nd Δ − area of 1st Δ

② earth pressure after crack = area of bottom Δ

* crack এর পর উপরের triangle inactive হয়ে যায়।

* ~~Force at any point~~,

* In case of passive earth pressure

$$k_p = \tan^2\left(45 + \frac{\phi}{2}\right)$$

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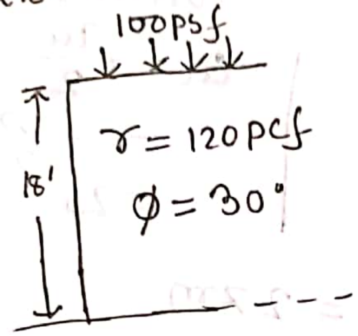
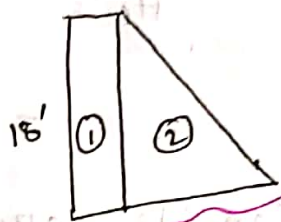
(51) Determine the magnitude of the resultant force per unit length at rest condition and also calculate location of that magnitude. [BCIC'17][SGCL'17]





active pressure হিসেবে
$k_a = \tan^2(45 - \frac{30}{2})$
$= 0.333$ হতো

Solⁿ: $K_o = 1 - \sin 30° = 0.5$

$P = P_1 + P_2$

$= K_o q H + \frac{1}{2} K_o \gamma H^2$

$= 0.5 \times 100 \times 18 + \frac{1}{2} \times \frac{1}{2} \times 120 \times 18^2$

$= 10620 \; lb/ft$

location of resultant force $= \dfrac{900 \times \frac{18}{2} + 9720 \times \frac{18}{3}}{10620}$

$= 6.25 \, ft$ from bottom (Ans)

(52) A pipe is to be laid in a purely cohesive soil having undrained cohesion $C_u = 30kPa$. Maximum depth of excavation without lateral support? [DPDC'15]

Take, $\gamma = 18.5 kPa$

Solⁿ: $H_c = \dfrac{4C_u}{\gamma} = \dfrac{4 \times 30}{18.5} = 6.5m$ (Ans)

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(53) The unconfined compression of the soil is 50kPa. Determine the depth of excavation without any lateral support [BWDB'14]
[BWDB'15]

Solⁿ:

$$H_c = \frac{4 \times 25}{18.5}$$

$$= 5.4m$$

$$c_u = \frac{q_u}{2} = \frac{50}{2} = 25$$

Tensile crack, $H_T = \frac{2 \times 25}{18.5} = 2.7m$

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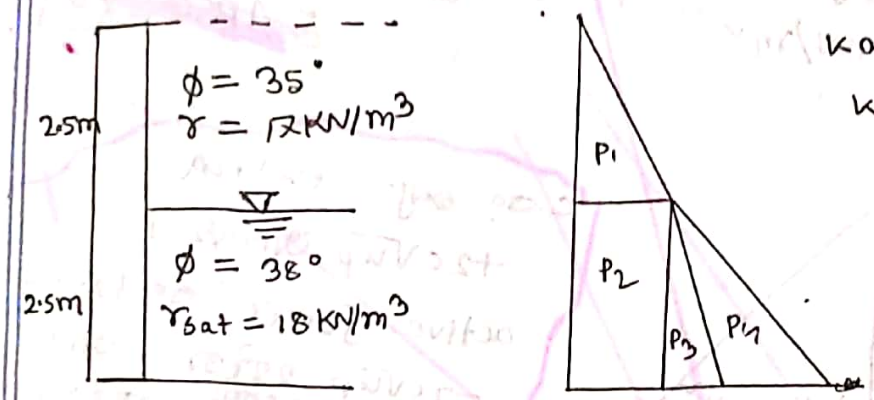
(54) Determine the active pressure on the retaining wall shown in figure. Take, $\gamma_w = 10\ kN/m^3$ [DESCO'19] [PGCL'21] [PRI'21]



$k_{a_1} = \tan^2\left(45 - \frac{35}{2}\right) = 0.271$

$k_{a_2} = \tan^2\left(45 - \frac{38}{2}\right) = 0.238$

C ও নাই তাই
crack ও
tension ও নাই

Soln:

$P_1 = \frac{1}{2} \times 0.271 \times 17 \times 2.5^2 = 14.4\ KN$

~~$P_2 = k_{a_2} \times P_1 \times H_2 =$~~

$P_2 = k_2 \gamma h_1 h_2 = 0.238 \times 17 \times 2.5 \times 2.5 = 25.3\ KN$

$P_3 = \frac{1}{2} \times 0.238 \times (18-10) \times 2.5^2 = 6\ KN$

$P_4 = \frac{1}{2} \times \gamma_w \times h_2^2 = 0.5 \times 10 \times 2.5^2 = 31.3\ KN$

$\therefore\ P = P_1 + P_2 + P_3 + P_4 = 77\ KN$

resultant, $z = \dfrac{14.4 \times \left(2.5 + \frac{2.5}{3}\right) + 25.3 \times \frac{2.5}{2} + 6 \times \frac{2.5}{3} + 31.3 \times \frac{2.5}{3}}{77}$

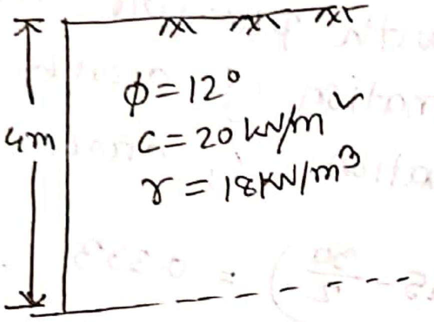
$= 1.44\ m$

(56) Find the active earth pressure of the sandy soil at a depth 4.5 where $\phi = 32^\circ$, $\gamma = 16.5\ kN/m^3$ [DESCO'15] [PGCB'19] [EGCB'15] [DPDC'16]

<u>Sol^n:</u> $K_a = \tan^2\left(45 - \frac{32}{2}\right) = 0.248$

$P_a = \frac{1}{2} K_a \gamma H^2 = \frac{1}{2} \times 0.248 \times 16.5 \times (4.5)^2 \simeq 41.43\ KN/m$

(57) Determine the stresses at the top and the bottom of the cut shown in figure. Also determine the max^m depth of crack and max^m depth of unsupported excavation.

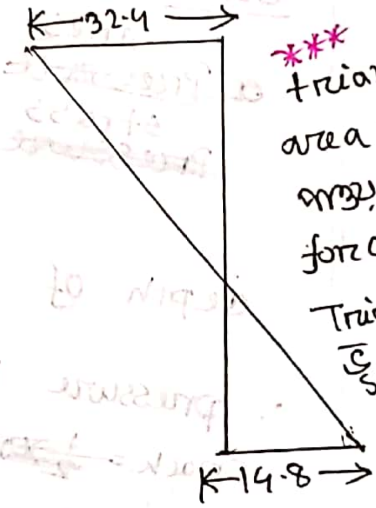


Sol^n:

$$k_a = \tan^2\left(45^\circ - \frac{12^\circ}{2}\right) = 0.656$$

at top, $P_t = -2c\sqrt{k_a}$
$= -2 \times 20\sqrt{0.656}$
$= -32.4\ kN/m^2$

at bottom, $P_b = k_a \gamma H - 2c\sqrt{k_a}$
$= 0.656 \times 18 \times 4 - 32.4$
$= 14.8\ kN/m^2$



*** triangle এর area বের করলে পাওয়া যাবে force.
Triangle এর দুই দিক হলো stress

depth of crack, $H_t = \dfrac{2c}{\gamma\sqrt{k_a}} = \dfrac{2 \times 20}{18 \times \sqrt{0.656}} = 2.745\ m$

$\therefore$ depth of max^m unsupported excavation,
$H_c = 2 \times H_t$
$= 5.49\ m$ Ans

একই কথা
* Pressure আর force/unit length
* আর stress (or stress) ই kN/m^2

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(58)

$\phi = 30^\circ$
$c = 5 \, kN/m^2$
$\gamma = 17.5 \, kN/m^3$

Determine active earth pressure on the wall
(a) before the formation of crack
(b) after the formation of crack [BOWDB'20]

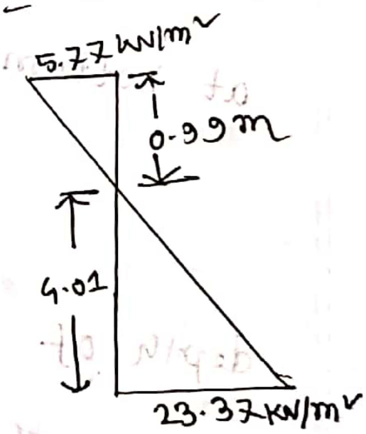
Solⁿ: $k_a = \tan^2\left(45 - \frac{30}{2}\right) = 0.333$

~~Pressure~~ stress at top $= -2 \times 5 \sqrt{0.333} = -5.77 \, kN/m^2$

~~Pressure~~ stress at bottom $= 0.333 \times 17.5 \times 5 - 5.77$
$= 23.37 \, kN/m^2$

depth of tensile crack, $h_t = \frac{2 \times 5}{17.5 \times \sqrt{0.333}}$
$= 0.99 \, m$

∴ pressure before tensile crack $=$ ~~$\frac{1}{2} \times 0.99 \times 5.77 + \frac{1}{2} \times 23.37 \times 4.01$~~
$= \frac{1}{2} \times 23.37 \times 4.01 - \frac{1}{2} \times 0.99 \times 5.77$
$= 44 \, kN/m$



~~Pressure after tensile crack~~
$=$ ~~$\frac{1}{2}$~~ $\times$

Resultant force distance from bottom → Small Δ

$= \frac{46.857 \times \frac{4.01}{3} - 2.856 \times \left(4.01 + \frac{0.99 \times 2}{3}\right)}{44}$

$= 1.12 \, m$

P.T.O

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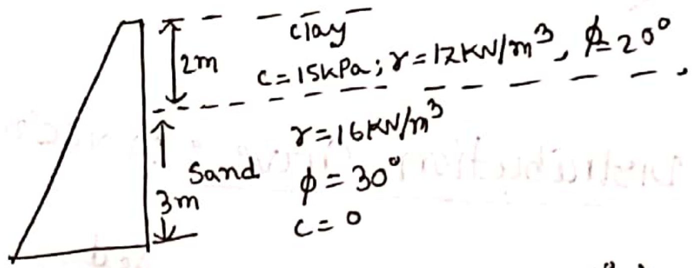
Pressure after formation of crack,

$$P_a = \frac{1}{2} \times 23.37 \times 4.01 = 46.857 \text{ KN/m}$$

Line of action from bottom $= \frac{4.01}{3} = 1.336\text{m}$ Ans

*** (59)



Find the active earth pressure after the occurrence of tensile crack

Sol^{n}:

$$k_{a_1} = \tan^2\left(45° - \frac{20°}{2}\right) = 0.49$$

$$H_c = \frac{2c}{\gamma\sqrt{k_a}} = \frac{2 \times 15}{17 \times \sqrt{0.49}} = 2.52\text{m}$$

Since Hc is more than the height of clay (2m) earth after tensile crack, there will be no earth pressure due to clay. The earth pressure will be entirely due to sand.

For sand, $k_{a_2} = \tan^2\left(45 - \frac{30}{2}\right) = 0.333$

$$\therefore \frac{1}{2} k_a \gamma_2 H^2 = \frac{1}{2} \times 0.333 \times 16 \times 3^2 = 23.976 \text{ KN/m}$$ Ans

(56 নম্বর math এর মত নয় শুধু sand আসলে কেবল $\frac{1}{2}k_a\gamma h^2$ দিয়ে pressure বের করতে হবে।)

Effective angle of internal friction, $\phi = 30°$.

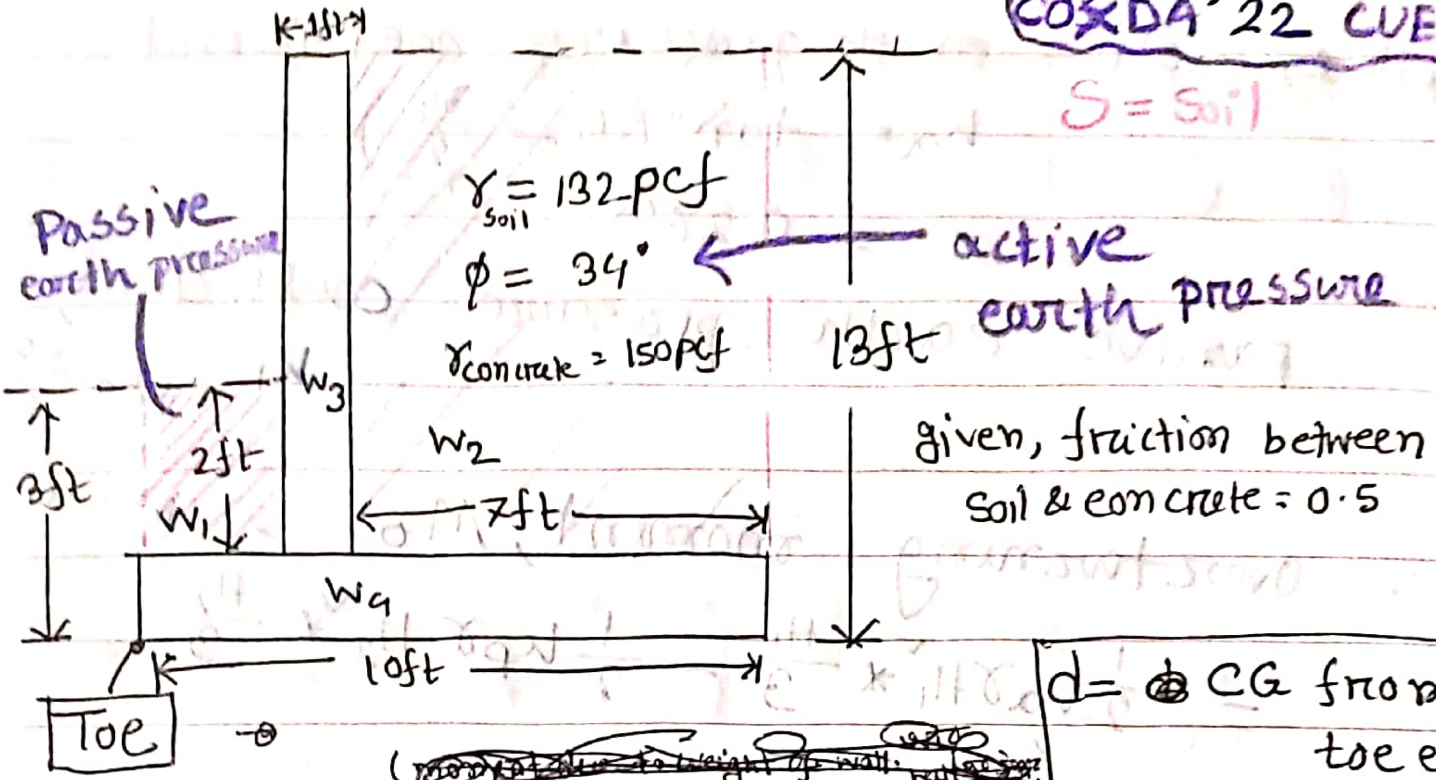
* Effective angle of internal friction, $\phi = 30°$. passive earth pressure coefficient $\underline{\;3\;}$ [N.HA'19]

active earth pressure $\underline{\;\frac{1}{3}\;}$ [BWDB'16] [GTCL'16]

$$K_a = \tan^2\left(45° - \frac{30°}{2}\right) = \frac{1}{3}, \quad K_p = \tan^2\left(45° + \frac{30°}{2}\right) = 3$$

CO❌DA'22 CUET

S = Soil



d = CG from toe end

Resisting moment calculation:

Taking 1ft width,

$$M_R = W_1 * d_1 + W_2 * d_2 + W_3 * d_3 + W_4 * d_4$$

$$= (132*2*2)*1 + (132*7*12)*(3+\tfrac{7}{2})$$

$$+ (150*12*1)*(2+\tfrac{1}{2}) + (150 \times 10 \times 1)*5$$

$$= 84600\ lbs\text{-}ft$$

~~Passive earth pressure is also force resisting.~~

∴ ~~$K_p = \tan^2(45+\frac{34}{2}) = 3.5$~~

Resisting moment হইলো wall আর wall এর উপরের মাটির weight এর জন্য moment আর overturning moment হইলো earth pressure এর জন্য moment.

Here, active earth pressure coefficient

$$K_a = \tan^2\left(45 - \frac{34}{2}\right)$$

$$= 0.283$$

$\therefore$ passive earth pressure coefficient,

$$K_p = \frac{1}{K_a} = 3.54$$

$\therefore$ Overturning moment, M_o

passive

$$= \frac{1}{2} K_a \gamma H_1^2 * \frac{H_1}{3} - \frac{1}{2} K_p \gamma \cdot H_2^2 * \frac{H_2}{3}$$

active

13673.52

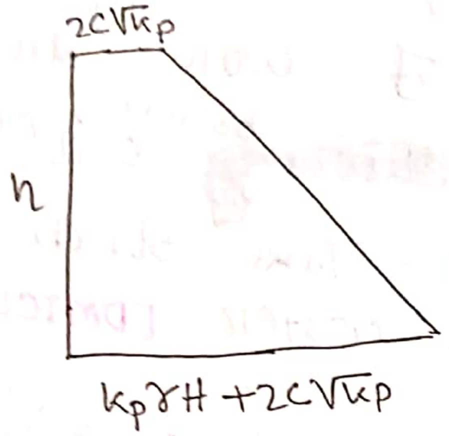
$$= \frac{1}{2} \times 0.283 \times 132 \times 13^2 \times \frac{13}{3} - \frac{1}{2} \times 3.54 \times 132 \times \frac{3^2}{3} \times \frac{3}{3}$$

$$= 11575.762 \text{ lb-ft}$$

$$\therefore FS = \frac{0.5 \times 84600}{11575.762} = 3.65$$

A

Passive Earth Pressure:

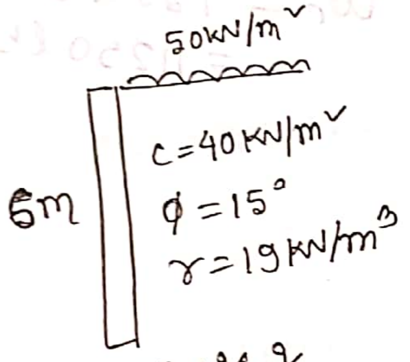


surcharge, q থাকলে উপরে নিচে সবার extra $k_p \cdot q$ অ্যাডিশন দিলেই হবে,

Stress diagram of passive earth pressure

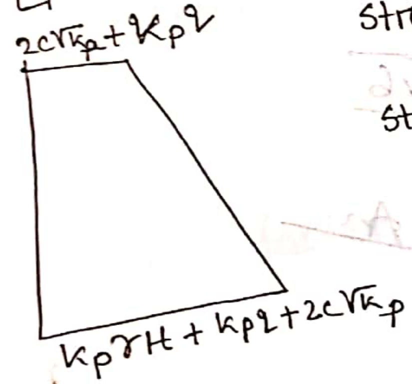
* Force/pressure of passive earth pressure = area of stress diagram

(55)



Find passive earth pressure, [APSCL'16]

Solⁿ:



$k_p = \tan^2(45° + \frac{15°}{2}) = 1.698$

Stress at top, $= 2*40*\sqrt{1.698} + 1.698*50$
$= 189.146\ kN/m^2$

Stress at bottom $= 1.698*19*6 + 1.698*50 + 2*40*\sqrt{1.698}$
$= 382.72\ kN/m^2$

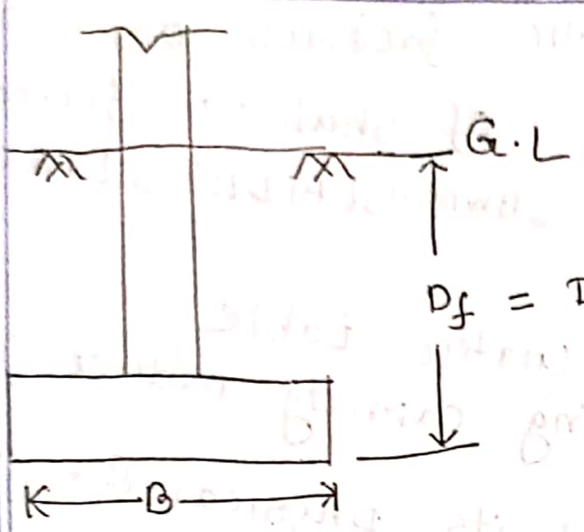
$\therefore$ Total pressure $= \frac{1}{2}*(189.146+382.72)*6$
$= 1715.6\ kN/m$

Point of application of resultant pressure from bottom $= \frac{\text{মোটা} + 2*\text{চিকন}}{\text{মোটা} + \text{চিকন}} * \frac{\text{উচ্চতা}}{3}$

$= \left(\frac{382.72 + 2*189.146}{382.72 + 189.146}\right) * \frac{6}{3}$

$= 2.66\,m$

Sub: Bearing Capacity of Footings



Terzaghi's ultimate bearing capacity eq^ns :

(1) For strip/continuous footing,

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

(2) For square footing, □

$$q_u = 1.3cN_c + qN_q + 0.4\gamma B N_\gamma$$

(3) For circular footing, ○

$$q_u = 1.3cN_c + qN_q + 0.3\gamma B N_\gamma$$

where, N_c, N_q, N_γ are bearing capacity factors

B = width of footing

* For pure cohesive soil, $N_\gamma = 0$, $N_q = 1$, $N_c = 5.7$
* $q = \gamma D_f$
* Allowable bearing capacity, $q_{all} = \frac{q_u}{FS}$
* Net ultimate bearing capacity, $q_{nu} = q_u - \gamma D_f$
* Net allowable bearing capacity, $q_{na} = \frac{q_{nu}}{FS}$

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(64) Write two major factors affecting the bearing capacity of shallow foundation [BWDB'16] [BPDB'15] [PGCL'17]

Soln:

(1) Position of ground water table (GWT rises, bearing capacity reduces)

(2) Types of soils and its physical & engineering properties

(3) types of ~~found~~ foundations (strip, square, circular, raft etc)

(4) Size of foundation

(5) Nature of ground surface (Inclined or horizontal)

(6) Nature of loading (concentric or eccentric)

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(65) Determine the ultimate bearing capacity of a continuous footing having width 1.2m, $\gamma = 17.2 KN/m^3$. Depth of the foundation is 0.3m. Cohesion = $20 KN/m^2$, $N_c = 17.69$, $N_q = 7.44$, $N_\gamma = 3.64$ Water table is ~~not near~~ 8m below (not near) the foundation. [DWASA'17][PGCB'19][EGCB'20][APSCL'20]

Solⁿ: $q_u = CN_c + qN_q + \frac{1}{2}\gamma B N_\gamma$

$\boxed{q = 17.2 * 0.3 = 5.16 KN/m^2}$

$= 20*17.69 + 5.16*7.44 + 0.5*17.2*1.2*3.64$

$= 429.75 KN/m^2$

(66) A square footing is 1.5m×1.5m in plan. Determine the allowable gross load on the footing with FOS=4. Depth of footing is 1m. Given; $c = 15.2 KN/m^2$; $\gamma = 17.8 KN/m^3$, $N_c = 17.69$, $N_q = 7.44$, $N_\gamma = 3.64$ [BCIC'17] [BCPCL'16] [APSCL'1] [DESCO'15]

Solⁿ: $q = 17.8 \times 1 = 17.8 KN/m^2$

$q_u = 1.3CN_c + qN_q + 0.4\gamma B N_\gamma$

$= 1.3 \times 15.2 \times 17.69 + 17.8 \times 7.44 + 0.4 \times 17.8 \times 1.5 \times 3.64$

$= 521 KN/m^2$

$q_{all} = \frac{521}{4} = 130 KN/m^2$

Allowable gross load $= 130 \times 1.5 \times 1.5 = 292.5 KN$

(2) A rectangular footing of 12'x8' is situated under a saturated clay soil at a depth of 5ft, Using Terzaghi's bearing capacity equation. Find the ultimate bearing capacity of the footing considering only shape factor. The unconfind compressive strength of the soil is 2000psf. [SGFL 23]

Soln: ~~Tengaz~~ Terzaghi's bearing capacity equation for rectangular footing –

~~$q_u = (1+0.3\frac{B}{L})CN_c + qN_q$~~

s/o footing হলে
$S_q = 1$

$$q_u = S_c C N_c + S_q q N_q + \frac{1}{2} S_\gamma \gamma B N_\gamma$$

For rectangular footing,
$S_c = 1+0.3\frac{B}{L}$; $S_q = 1$; $S_\gamma = 1-0.2\frac{B}{L}$

Date: ______
Sun | Mon | Tue | Wed | Thu | Fri | Sat

Saturated = γ'

$$q = \gamma' D_f = (130-62.43)\times 5$$

Considering only shape factor,

$$q_{ult} = C S_c + q S_q + \frac{1}{2}\gamma' B S_\gamma$$

$$= \frac{2000}{2}\times\left(1+0.3\times\frac{8}{12}\right) + (130-62.43)\times 5\times 1$$

$$+ \frac{1}{2}\times(130-62.43)\times 8\times\left(1-0.2\times\frac{8}{12}\right)$$

$$= 1773.25 \text{ psf (Ans:)}$$

S = shape factor
N = bearing capacity factor
only shape factor consider করতে হবে

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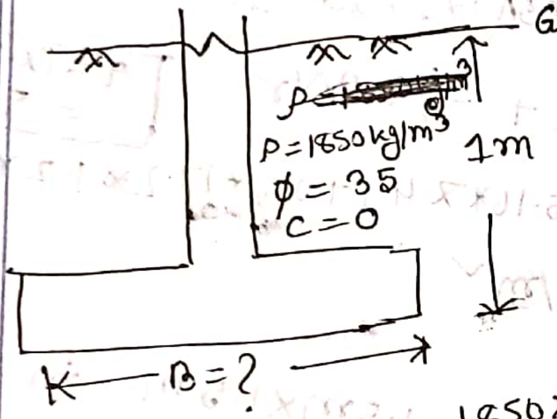
(67) A square footing is shown in figure below. The footing will carry a gross mass of 30,000kg using factor of safety equals to 3. Determine the size of footing.

$N_c = 57.75$, $N_q = 41.44$, $N_\gamma = 45.41$

[BWDB'19]



Solⁿ: $\gamma = \frac{1850 \times 9.81}{1000} = 18.15\ kN/m^3$

total gross load $= \frac{30,000 \times 9.81}{1000} = 294.3\ kN$

We know,

$q_{all} = \frac{1}{3}(1.3cN_c + qN_q + 0.4\gamma BN_\gamma)$

$\Rightarrow \frac{294.3}{B^2} = \frac{1}{3}[0 + 18.15 \times 1 \times 41.44 + 0.4 \times 18.15 \times B \times 45.41]$

$\Rightarrow \frac{294.3}{B^2} = 250.7 + 109.9B$

$\therefore B \approx 0.95m$ Ans

[Gap]

* If FS = 3, then margin of safety = FS − 1 = 3 − 1 = 2 [BUET]

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(68) A strip footing carrying a load of 340 KN/m which is located at EGL (existing ground level). Water table is located at EGL. $\gamma_{sat} = 21\ KN/m^3$, width of footing = 2m and depth of footing = 1m. FS = 3, Ultimate pressure is 530 KN/m². Prove that the size of footing is safe. [Titas'18]

Soln: Net ultimate bearing capacity,

$$q_{nu} = q_u - \gamma' D_f$$
$$= 530 - (21 - 9.81) \times 1$$
$$= 518.81\ KN/m^2$$

Net allowable bearing capacity, $q_{na} = \frac{518.81}{3}$
$= 172.93\ KN/m^2$

Applied pressure $= \frac{\text{Force}}{\text{footing area}}$

$$= \frac{340}{2 * 1} \rightarrow \text{unit length for strip footing}$$

$= 170\ KN/m^2$

$\therefore q_a = 172.93 > 170\ KN/m^2$ (ok)

$\therefore$ Size of footing is safe!

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(71) A square footing ($2.5m * 2.5m$) is to be placed on a homogeneous clay layer at 3m depth. The ~~too~~ mean unconfined compression strength was obtained as $50 kN/m^2$. Estimate the ultimate bearing capacity utilising Terzaghi's concept. [BWDB'14]

Soln: $q_u = 1.3cN_c + qN_q + 0.4\gamma BN_\gamma$

$= 1.3 \times \frac{50}{2} \times 5.7 + q*1$

$\therefore$ Net ultimate bearing capacity,

$q_{nu} = q_u - q$

$= 185.25 \, kN/m^2$

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

for clay, $N_c = 5.7$, $N_q = 1$, $N_\gamma = 0$

* মনে রাখ unit weight দেয়া নাই তাই ultimate bearing capacity এর জন্য net ultimate bearing capacity হবে একই।

Gap:

most suitable method for densification of 20m deep loose sand deposit is ~~vibro~~ dynamic compaction [PGCL'14]

* A square foundation is 2m × 2m in plan. The soil supporting the foundation has a friction angle of 25° and $c' = 20 KN/m^2$. The unit weight of soil, γ is $16.5 KN/m^3$. Depth of foundation is 1.5m. Assume general shear failure occurs in the soil. Take FS = 3, $N_c = 25.13$, $N_q = 12.72$, $N_\gamma = 8.34$. Determine allowable gross load.

Solⁿ: For square footing,

$q = \gamma D_f$

$$q_u = 1.3 C N_c + q N_q + 0.4 \gamma B N_\gamma$$

$$= 1.3 \times 20 \times 25.13 + (16.5 \times 1.5) \times 12.72 + 0.4 \times 16.5 \times 2 \times 8.34$$

$$= 1078.29 \, KN/m^2$$

∴ Allowable gross load per unit area, $q_{all} = \frac{q_u}{FS}$

$$= \frac{1078.29}{3}$$

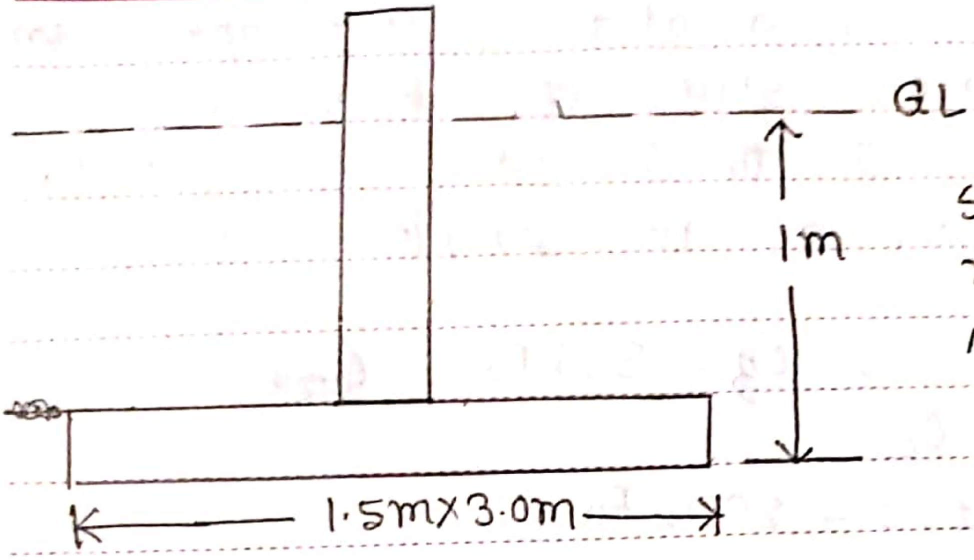
$$= 359.5 \, KN/m^2$$

∴ Total allowable gross load $= 359.5 \times (2 \times 2)$

$$= 1438 \, KN$$

** এখন Net allowable load জানতে চাইলে,

$$q_{nu} = \frac{q_u - \gamma D_f}{FS}$$ হবে,



Sand
$\gamma = 18 KN/m^3$
$N_q = 24$
$N_\gamma = 20$

The water table is at a depth of 10m below the base of the footing. calculate The net ultimate bearing capacity of the footing based on Terzaghi's bearing capacity equation.

Soln:

$$q_u = \left(1+0.3\frac{B}{L}\right)CN_c + \overset{\gamma D_f}{\cancel{\gamma}} q N_q + \frac{1}{2}\gamma B\left(1-0.2\frac{B}{L}\right)N_\gamma$$

For sand, $C = 0$

$$\therefore q_u = 18\times1\times24 + \frac{1}{2}\times18\times1.5\left(1-0.2\times\frac{1.5}{3}\right)\times20$$

$$= 675 KN/m^2$$

$\therefore$ Net ultimate capacity, $q_{nu} = q_u - \gamma D_f$

$$= 675 - 18\times1$$

$$= 657 KN/m^2$$

square cases $B = L$

$\therefore 1.3CN_c$,

$\frac{1}{2}\times0.8 = 0.4$

$0.4\gamma BN_\gamma$

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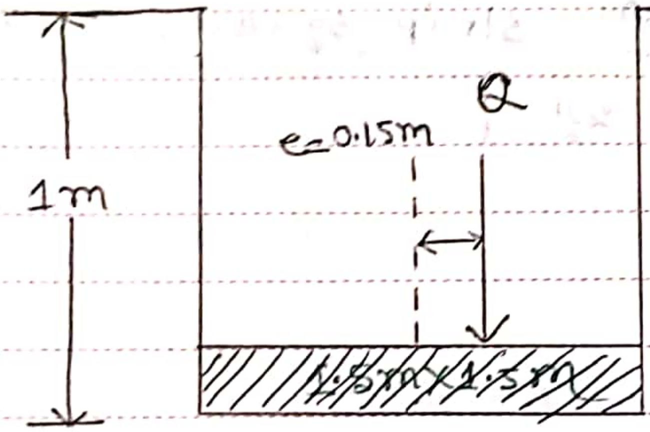
* An eccentrically loaded foundation shown in figure. Use FS of 4 and determine the maximum allowable load that the foundation can carry. Use Meyerhof's eqn.



$\gamma = 17\ KN/m^3$

$C' = 0$

$N_c = 50.59,\ N_q = 37.75,\ N_\gamma = 56.31$

$S_c = 1.597,\ S_q = 1.581,\ S_\gamma = 0.68$

$d_c = 1.267,\ d_q = 1.1646,\ d_\gamma = 1$

$i_c = i_q = i_\gamma = 1$

Solⁿ: $B' = B - 2e = 1.5 - 2\times0.15 = 1.2m$

$L' = L = 1.5m$

$$\therefore q_u = \overset{c=0}{\cancel{CN_cS_c}}d_ci_c + (\gamma D_f)N_qS_qd_qi_q + \frac{1}{2}\gamma B'N_\gamma d_\gamma i_\gamma S_\gamma$$

$$= 0 + 17\times1\times37.75\times1.1646\times1\times1.581 + \frac{1}{2}\times17\times1.2\times56.31\times0.68\times1\times1$$

$$= 1572.17\ KN/m^2$$

$\therefore$ Maximum allowable load per unit area

$$= \frac{q_u}{FS} = \frac{1572.17}{4} = 393.0425\ KN/m^2$$

Net হলে γD_f বিয়োগ হবে

$\therefore$ Maximum allowable load $= 393.0425\times1.2\times1.5$

$= 707.47\ KN$

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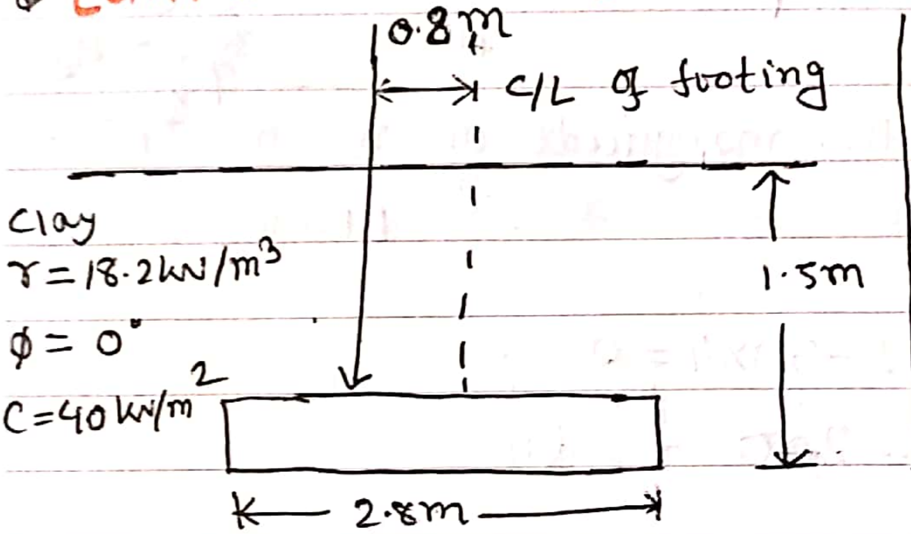
(5) A rectangular footing of size 2.8m × 3.5m is embedded in a clay layer and a vertical load is placed with an eccentricity of 0.8m as shown in the figure. Take bearing capacity factors – $N_c = 5.14$, $N_q = 1.0$ and $N_\gamma = 0.0$; shape factors: $S_c = 1.16$, $S_q = 1.0$ and $S_\gamma = 1.0$; Depth factors: $d_c = 1.1$, $d_q = 1.0$ & $d_\gamma = 1.0$. Inclination factors $i_c = 1.0$, $i_q = 1.0$, $i_\gamma = 1.0$ [DMTCL Line 55 2022]



Using Meyerhoff's method, the load that can be applied on the footing with a factor of safety of 2.5 is ______?

Soln:

Ultimate bearing capacity,

$$q_u = cN_cS_cd_ci_c + qN_qS_qd_qi_q + \frac{1}{2}\gamma B' N_\gamma S_\gamma d_\gamma$$

$\frac{1}{2}\gamma B N_\gamma S_\gamma d_\gamma = 0$

$$= 40 \times 5.14 \times 1.16 \times 1.1 \times 1 + 27.3 \times 1 \times 1 \times 1 \times 1 + 0$$

$$= 289.65 \ kN/m^2$$

$q = \gamma D_f$
$= 18.2 \times 1.5$
$= 27.3 \ kN/m^2$

$e = 0.8m$

$\therefore B' = B - 2e$
$= 2.8 - 2 \times 0.8$
$= 1.2m$

$L = 3.5m$

$$q_{nu} = q_u - \gamma D_f$$

$$= 289.65 - 27.3 = 262.345 \ kN/m^2$$

$$q_{ns} = \frac{262.345}{2.5} = 104.94 \ kN/m^2$$

$\therefore$ Net safe load $= 104.94 \times 1.2 \times 3.5$

$= 440.74 \ kN$

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DNCC 2022 MCQs

Minimum depth of footing / foundation ~~for cohesive soil~~ according to Rankine's formula –

$$D_f = \frac{q_u}{\gamma}\left(\frac{1-\sin\phi}{1+\sin\phi}\right)^2$$

D_f = depth of footing ; q_u = ultimate bearing capacity of soil ; γ = unit weight of soil ; ϕ = frictional angle

Example:

Given that, ultimate bearing capacity of soil is 180 kN/m², unit weight of soil is 20 kN/m³ & frictional angle $\phi = 30°$. Determine minimum depth.

Solⁿ: $D_{min} = \frac{180}{20} \times \left(\frac{1-\sin 30°}{1+\sin 30°}\right)^2 = 1m$ A

* Minimum depth of footing for cohesive soil is 1.5m for cohesive soil & 2m for cohesionless soil [DNCC'21]

(70) When combine footing is used? [SGCL'17]

Solⁿ: A combined footing is provided under the following circumstances —

(i) when the columns are very near to each other so that their footings overlap

(ii) when the bearing capacity of the soil is less, requiring more area under ~~individua~~ individual footing. ~~footing~~

(iii) when the end column is near a property line so that its footing cannot spread in that direction.

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(72) Write four methods to improve the bearing capacity of soils [47BMA]

Solⁿ: Methods —

(i) by increasing the depth of foundation

(ii) by draining off water from sub-soil

(iii) by compacting the soil (void decreases, bearing capacity increases)

(iv) by increasing width of foundation

(v) by pumping in cement-grout into the ground

(vi) by solidifying the ground by using $CaCl_2$.

(73) How can we improve ground surface? [WZPDCL'16]

Solⁿ: Ground improvement techniques -

→ Preloading or surcharging with sand, either with or without vertical drains

→ various compaction techniques including vibratory methods

→ Soil removal and replacement, stone columns & geotextile encased sand columns

→ In-situ admixtures like lime, cement and fly ash

Sub: Sub-Soil Exploration

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Standard Penetration Test (SPT): [BEPZA'16] [PGCL'17]

→ The test is used for determining the relative density and the angle of shearing resistance (ϕ) of cohesionless soils.

→ This test is conducted in a bore hole using standard split spoon sampler

→ SPT value (N) means, the number of blows required for 30cm (12 inch) penetration

→ Hammer weight 63.5kg (140 lb)
height of fall 75cm (30inch) [ERLB'22]

→ To avoid setting errors, the blows for the first 15cm (6inch) penetration are not taken into account. Thus, N values for 15cm to 45cm (total 30cm) constitue the SPT value.

[PGCL'17] [BEPZA'18] [BMA]
SPT-N value definition: The SPT-N value provides information regarding the soil strength. SPT(N) value in sandy soil indicates the friction angle in sandy soils and in clay soils it indicate the stiffness of the clay startum.

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Gap:

(i) Weight of hammer is SPT test <u>63.5kg (140lb)</u>

[SGFL'17] [SGCL'20] [Petrobangla'22] [MGMCL'22] [JGIDSL'21] [BIWTA'19] [NAHA'19]

(74) In SPT test for each 15cm penetration needs 8, 10, 12 SPT value. Calculate N value [BGDCL'21] [MSC'11] [EGCB'20] [APSCL'20]

Ans: $N = 2^{nd} + 3^{rd} = 10 + 12 = 22 > 15$ [অতিরিক্ত count তার জন্য error সংশোধন করা]

corrected N value $= 15 + \frac{1}{2}(22 - 15)$

$= 18.5$ **A**

(75) Short note on black cotton soil [Padma Bridge'15]

Soln: Black cotton soil is one of the major soil deposits in Indian subcontinent. They exhibit a very high rate of swelling & shrinkage when moisture content is increased and decreased respectively. Thats why this type of soil gets very sticky is rainy season and gets cracked in dry season. Hence this soil is most troublesome from engineering consideration. Chemically black cotton soil consists of lime, iron, magnesium, alumina and potash. It posseses great water holding capacity.

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(76) When SPT value is rejected? [WRGCL'14]

Soln: SPT value is rejected if →

(i) 50 blows are required for any 150mm (6inch) ~~15cm increment~~ increament

(ii) 100 blows are obtained for 300mm (12inch) penetration

(iii) 10 successive blows produce no advancement

(77) What are the advantages of CPT test over SPT test. Also describe disadvantages of CPT test [MSC'14]

Ans: Advantages of CPT test –

→ It is faster and more accurate

→ It offers digital in situ ~~soil data~~ detailed data of soil

→ It produces real time data

→ Suitable for soft soils

Disadvantages

→ No sample recovered for other geotehnical tests

→ penetration depth limited to 150-200 ft

→ Normally cannot push ~~the~~ through gravels

→ Requires special equipment and skilled operators

(78) Purposes of pile foundation [APSCL'15] [MSc'16]

Ans: Pile foundations are principally used to transfer the loads from superstructures, through weak, compressible strata or water, into stronger, more compact, less compressible and stiffer soil at depth. They are typically used for large structures.

The following conditions makes the construction company use the piles —

(i) when the soil at the construction site ~~the usage of Si~~ is extremely loose or soft, ~~the usage of~~

(ii) when the plan area is not sufficient because of the loads being too hight that the required size of the foundation can't be accommodated.

(iii) when the foundation has large lateral (⇐) loads acting on it.

(iv) when The underground structures causes immense ~~the pile~~ ~~constructed using~~ uplift loads. ~~are~~ ~~foundation~~.

(v) when the foundation has to support inclined loads, eccentric loads and moments, the pile foundation is used.

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(79) Advantage & disadvantages of cast insitu piles and precast piles [HBRI'19]

Soln: Advantages of cast insitu piles

(i) Piles possessing any size or length may be used for the construction at the site.

(ii) Damaged caused because of driving or handling in case of precast piles is eliminated

(iii) In case of water logged area, cast in situ piling with permanent casing is more suitable.

(iv) cast in situ pile can fully utilize the skin friction resistance with the ground during design stages which cannot be achieved in pre cast piles.

(v) Cast in situ piles are connected above ground with a pile cap using a monolithic approach. Due to this monolithic approach, cast in situ pile has an advantage of having more resistance to the earthquake and wind forces.

Disadvantage of cast in situ piles:

(i) A strict quality control and a thorough supervision need to be done for installing the situ piles. This is required to be done for all kinds of materials used for the construction.

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(2) Since sufficient storage space is required for all types of materials required for the construction involved could be quite therefore the method complecated.

(3) The places having a lot of current of the ground water flow or a lot of artesian pressure, the construction in such places can't be done using ~~site~~ in situ piles.

(4) More labor is required

(5) In cast in situ piles more concrete is consumed as at the site ~~there~~ is enough loss of concrete sometimes due to poor workmanship.

Advantages of precast piles:

(i) The main advantage of the precast concrete piles is that these piles compact soil. Thus it increases the bearing capacity the soil.

(ii) They are readily available in ~~[illegible]~~ construction site which saves ~~[illegible]~~ a lot of time

(iii) The position of the reinforced in the pre cast concrete ~~[illegible]~~ piles is not distrubed or changed.

(iv) Pre cast concrete piles are cost effective. These are cheap and suitable to use.

(v) ~~[illegible]~~ They can be driven underwater

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(vi) Precast concrete piles are not affected by the chemical action of the subsoil

(vii) They can be taking load immediately after their casting.

Disadvantages of precast concrete piles :

(i) Being very heavy, transporting, handling and driving theme is very hard.

(ii) Extra steel must be provided near the top and bottom of the piles, in order to avoid the stresses developed during handling and driving.

(iii) The precast concrete piles can not be cast in a correct length unless trail bores are made. Therefore it becomes more complecated.

(iv) It may break without proper handling and driving.

(v) The pile length is also restricted depending on the mode of transportation available.

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Ultimate load bearing capacity of pile in clay:

$Q_u = Q_b + Q_s$

Q_s = skin friction capacity

$= \alpha C_u A_s$

Q_b = end bearing capacity

$= 9 C_u A_b$

$\therefore Q_u = 9 C_u A_b + \alpha C_u A_s$

Here,

$N_c = 9$

Side area, $A_s = \pi d L$

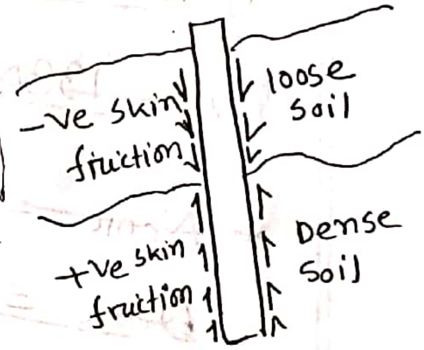
Pile tip area = Area of bearing

$= \frac{\pi}{4} d^2$

Gap: For deep foundation, $N_c = 9$ [BWDB'18] [SGCL'20] [SGFL'17] [NHA'19] [Petrobangla'22] [JGTDSL'21]

Negative skin friction/Downward drag: [DMTCL'19] [S3BMA]

When the soil surrounding a portion of pile shaft settles more than the pile, a down ward drag occurs on the pile. The downward drag is called "Negative skin friction".



→ The net load carrying capacity of the pile decreased due to negative skin friction.

→ occurs in compressible soils, soft and loose soil recently filled up soils, also due to lowering of water table.

→ can be eliminated by providing a protective sleeve for that portion of the pile.

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Pull out capacity of Pile:

The pullout capacity of a pile is the combination of dead weight and skin friction that can be used to resist uplift loads.

End bearing is neglected here since the pile is assumed to move upwards.

∴ Pull out capacity / uplift capacity,

$$P_1 = W_p + \frac{2}{3} A_s f_s$$

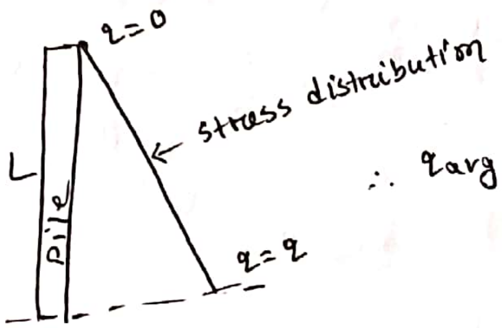
∴ For clay, $P_\phi = W_p + \frac{2}{3} * A_s * \alpha C_u$

For sand, $P = W_p + \frac{2}{3} * A_s * k \tan\delta \cdot q_{avg}$ | $q_{avg} = \frac{\gamma L}{2}$

~~take $\delta = 0.75\phi$~~

For driven / precast pile, $\delta = 0.75\phi$, $k = 1 - \sin\phi$

For bored cast in situ pile, $\delta = \phi$, $k = 1 - \sin\phi$



$$\therefore q_{avg} = \frac{q}{2} = \frac{\gamma L}{2}$$

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(80) A nine pile group constituting of 18inch dia concrete pile was cast insitu in a soil whose unconfined compressive strength is 1000 lb/ft^2. Each pile is 60ft long and the pile spacing is 2.5 times the pile diameter. Calculate allowable skin friction capacity of single pile. Take, $\alpha = 0.5$ and FOS = 3 [BWDB'16] [BIWTA'19] [NHA'19] [PGCB'15] [SGFL'17] [DNCC'16] [GTCL'16] [ERL'17] [PetroBangla'22]

Soln:

Here, $C_u = \frac{q_u}{2} = \frac{1000}{2} = 500$ ~~kN/m²~~ lb/ft^2

$$Q_s = \alpha C_u A_s = \alpha C_u \pi d L$$
$$= 0.5 \times 500 \times \pi \times \frac{18}{12} \times 60$$
$$= 70686 \text{ lb}$$

প্রশ্নেই দিলো :

$\therefore$ Allowable skin friction capacity $= \frac{70686}{3}$ lb
$= 23562$ lb
$= 23.56$ kip

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(81) A pile is embedded in clayey soil having length and depth are 100ft and 24inch respectively. Unconfined compressive strength of clay is 2000 psf Determine the ultimate ~~[illegible]~~ capacity of pile

where, $\alpha = 0.5$ [BWDB'16] [BEPZA'19] [PGCL'17] [PGCB'14]

$$c_u = \frac{2000}{2\times1000} = 1\,ksf$$

Soln:

$$Q_u = Q_s + Q_b$$

$$= \alpha c_u \pi d L + 9 c_u * \frac{\pi}{4} * d$$

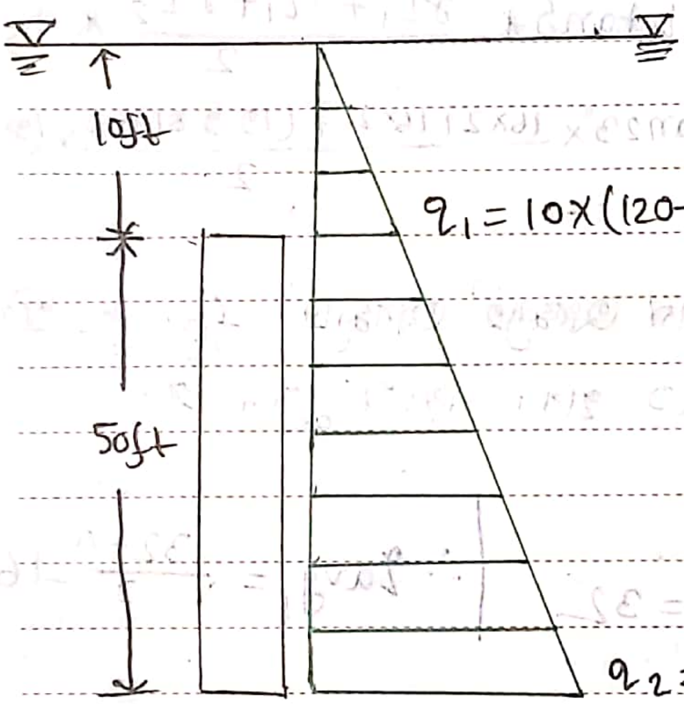
$$= 0.5 * 1 * \pi * \frac{24}{12} * 100 + 9 * 1 * \frac{\pi}{4} * \left(\frac{24}{12}\right)$$

$$= 342.43\ Kip$$

A

CamScanner

* Determine the skin friction capacity of a cast in situ bored RCC pile of dia 20inch and length 50ft in a homogeneous sand layer. The top of the pile is at 10ft below the ground surface. Assume water table is at ground surface. Take, $\gamma_{sat} = 120\,pcf$, $\phi = 30°$ [BWDB'14]



$q_1 = 10 \times (120 - 62.43) = 575.7\ lb/ft^2$

$\therefore q_{avg} = \dfrac{575.7 + 3454.2}{2}$

$= 2014.95\ lb/ft^2$

$q_2 = (10+50) \times (120 - 62.43) = 3454.2$

$k = 1 - \sin\phi = 1 - \sin 30° = 0.5$; $\cancel{\delta = \frac{2}{3} \times 0.5} =$

$\delta = \frac{2}{3}\phi = \frac{2}{3} \times 30° = 22.5°$

$\therefore$ Skin friction capacity,

$Q = k \tan\delta \cdot q_{avg} \cdot A_s$

$= 0.5 \times \tan 22.5° \times 2014.95 \times \pi \times \frac{20}{12} \times 50$

$= 109251.45\ lb$

$= 109.25\ kip$

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(83) A pile of 40cm diameter, passes through a recently filled up material of 4.50m length. If the unconfined compressive strength of clay is 60KN/m², calculate the negative skin friction of the pile.

Soln: $d = 40\text{cm} = 0.4\text{m}$, $C = \frac{60}{2} = 30 \text{ KN/m}^2$

$Q_{nf} = \underbrace{\pi D L_c C}_{C_u A_s} = \pi \times 0.4 \times 4.5 \times 30 = 169.65 \text{ KN}$

(84) Estimate the pull out capacity of a cast in situ bored concrete pile of dia 500mm and length 15m. Pile installed in a homogeneous sand deposit. If $\gamma_{sat} = 18 \text{KN/m}^3$, $\phi = 30°$. water table is at ground surface [DMTCL '19]

Soln:

Pull out capacity, $P = W_p + \frac{2}{3} * A_s * \underbrace{k \tan\delta * q_{avg}}$

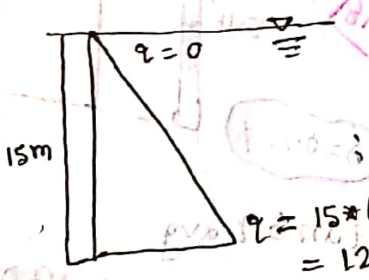
$= 73.63 + \frac{2}{3}(\pi * 0.5 * 15) * \tan \cancel{30}\ 22.5° * \cancel{\ }\ 61.425$ (0.5)

= ~~909.22 KN~~

= ~~352.11 KN~~

$= 273.46 \text{KN}$

for bored pile
$\therefore \phi = \frac{3}{4}\delta \approx 0.75\delta$



$q = 15 * (18 - 9.81)$
$= 122.85 \text{ kN/m}^2$

$q_{avg} = \frac{122.85 + 0}{2}$

$= 61.425 \text{ kN/m}^2$

weight of pile = unit weight * pile volume
$= 25 \text{KN/m}^3 * \pi * \left(\frac{.5}{2}\right)^2 * 15 = 73.63 \text{ KN}$

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Techniques of Soil Improvement: [DBCC'19]

(1) surface compaction

(2) Groundwater drainage

(3) Vibration method: Vibration's and shock waves in loose deposits of sandy soils cause liquefaction followed by densification ~~as~~ due to the dissipation of excess pore water.

(4) Grouting

(5) Chemical stabilization like lime, fly ash, cement to reduce ~~p~~ permeability of cement.

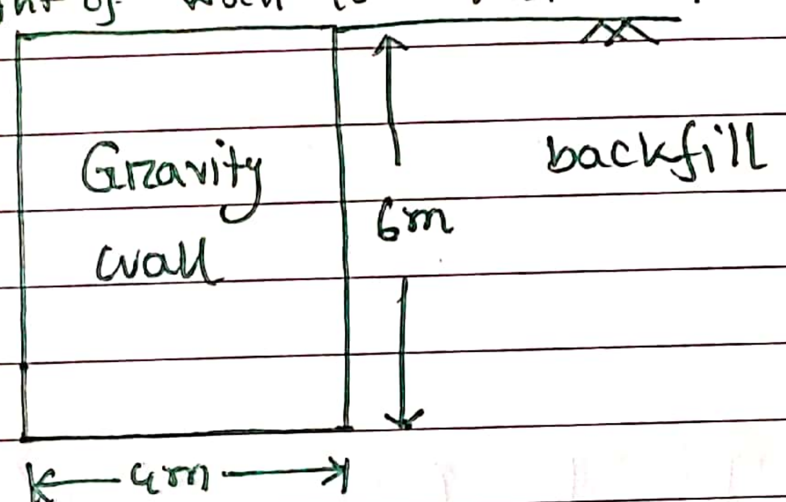
(6) Soil reinforcement

(7) Geotextiles and geomembranes

(8) other methods like thermal methods, moisture barriers, prewetting etc.

(9) Sand compaction pile ~~cone~~ method

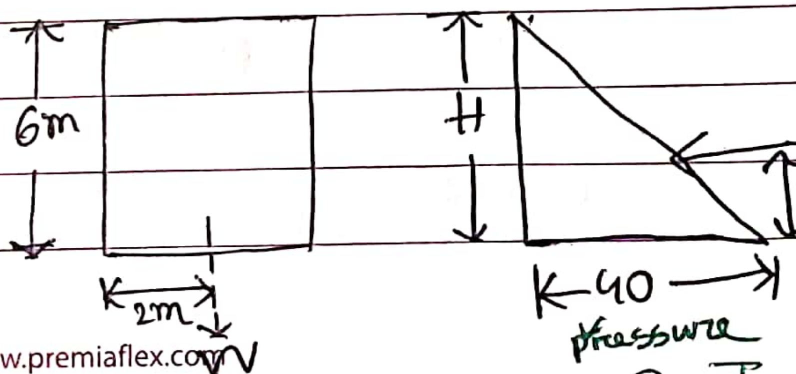
* A homogeneous gravity retaining wall supporting a cohesionless backfill is shown in the figure. The lateral active earth pressure at the bottom of the wall is ~~120~~ 40 kPa. Find the minimum weight of wall to withstand the moment.



Solⁿ: For stability,

$$P_a \times \frac{H}{3} = W \times \frac{B}{2}$$

$$\Rightarrow W = \frac{2 P_a H}{3B} = \frac{2 \times 120 \times 6}{3 \times 4} = 120 \text{ kN/m length}$$



Force = pressure area

$P_a = \frac{1}{2} \times 6 \times 40$

$= 120 \text{ kN}$

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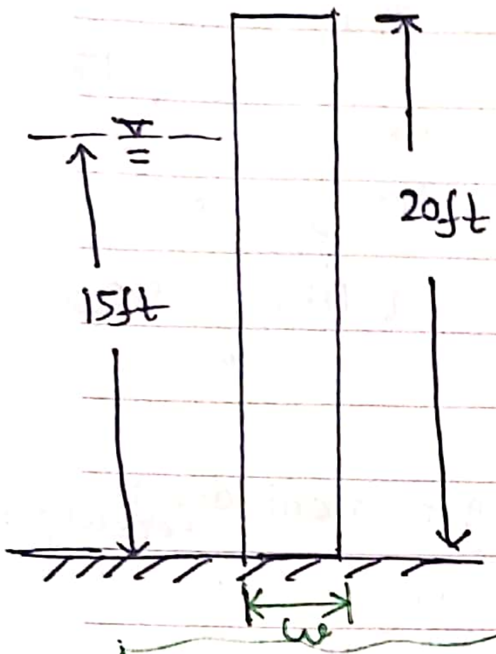

Calculate the width of concrete dam that is necessary to prevent the dam from sliding. Friction coefficient between the base of the dam and the foundation is 0.42. Use 1.5 as the factor of safety against sliding. Will it also be safe against overturning?

Taking 1ft of length of wall

Soln:

$$F.S._{(sliding)} = \frac{\text{Sliding resistance}}{\text{Sliding force}}$$

unit weight of concrete

$$\Rightarrow 1.5 = \frac{0.42 \times (20 \times w \times 1) \times 150}{\frac{1}{2} \times 62.43 \times 15^2}$$

$\boxed{\gamma_c = 150\ lb/ft^3}$

$$\Rightarrow w = 8.36ft$$

$$F.S._{(overturning)} = \frac{0.42 \times (20 \times 8.36 \times 1) \times 150 \times \frac{8.36}{2}}{\frac{1}{2} \times 62.43 \times 15^2 \times \frac{15}{3}}$$

$$= 3 \quad (ok)$$

Therefore it should be safe against overturning.

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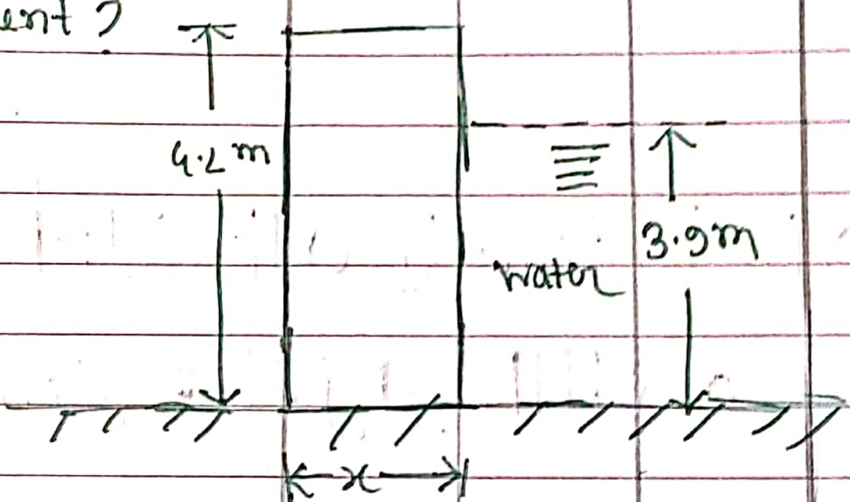
Water is retained by a concrete wall. The weight of the wall resists the overturning moment of water. What width "x" of the wall would ensure the resisting moment is twice the water pressure's overturning moment?



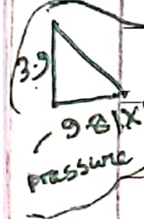
Soln: unit weight of concrete = 25 KN/m^3
unit weight of water = 9.81 KN/m^3

Resisting moment $= (25 \times 4.2 \times x \times 1) \times \frac{x}{2}$
$= 52.5x^2$ KN-m (Consider 1m strip)

~~Overturning moment~~

Force due to water, $F = \gamma \bar{h} A = 9.81 * \left(\frac{3.9+0}{2}\right) * (3.9 * 1)$

$= \frac{1}{2}\gamma h^2$



$= \frac{1}{2} \times 3.9 \times 9.81 \times 3.9$

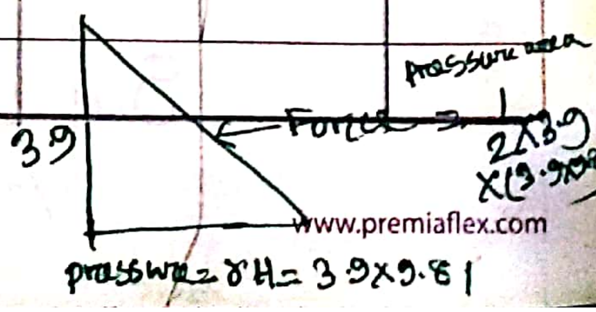
$= 74.61$ KN

Overturning moment $= 74.61 \times \frac{3.9}{3} = 96.986$ KN-m

A.T.Q,

$52.5x^2 = 2 \times 96.986$

$\therefore x = 2m$



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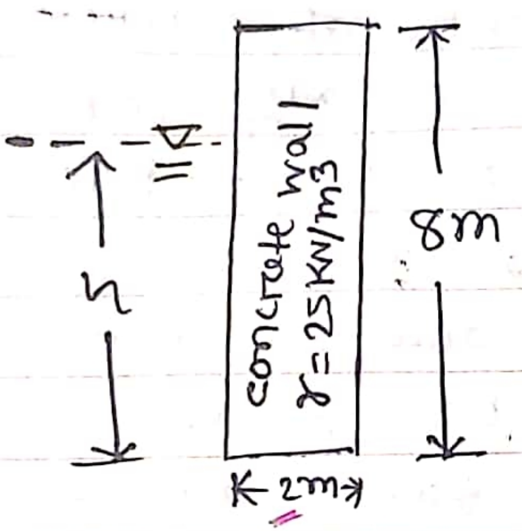


If factor against overturning is 3. Then what is the depth of water, h = ?

[DESCO 22]

[MIST]

Soln:

$$F.S. = \frac{\text{Resisting moment due to self weight}}{\text{Overturning moment due to water force}}$$

Taking 1m ~~width~~ length of wall

$$\Rightarrow 3 = \frac{(2\times 8\times 1)\times 25\times \frac{2}{2}}{\frac{1}{2}\gamma h^2 \times \frac{h}{3}}$$

Moment = Force × distance

$OTM = \frac{1}{2}\gamma h^2 \times h/3$

$RM = \text{self weight} \times \frac{\text{width}}{2}$

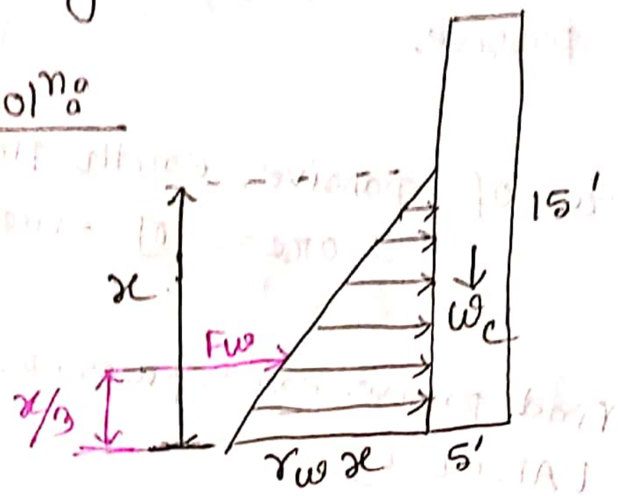
$$\Rightarrow h = \left(\frac{16\times 25\times 1}{\frac{1}{2}\times 9.81\times \frac{1}{3}\times 3}\right)^{1/3}$$

$\therefore h = 4.336m$ A

Sub :

(85) Compute the height of retaining wall if overturning moment of water and soil is equal. length of ~~retaining~~ boundary wall is 15ft and width is 5ft. Take standard unit weight of concrete & water. [DMTCL'19]

Solⁿ:



$F_w = \frac{1}{2} \times (\gamma_w x) \times x \times 1$

$= \frac{\gamma_w x^2}{2}$

Let, 1m length of wall

$W_c = 150 \times 5 \times 15 \times 1$

$= 11250$ lb

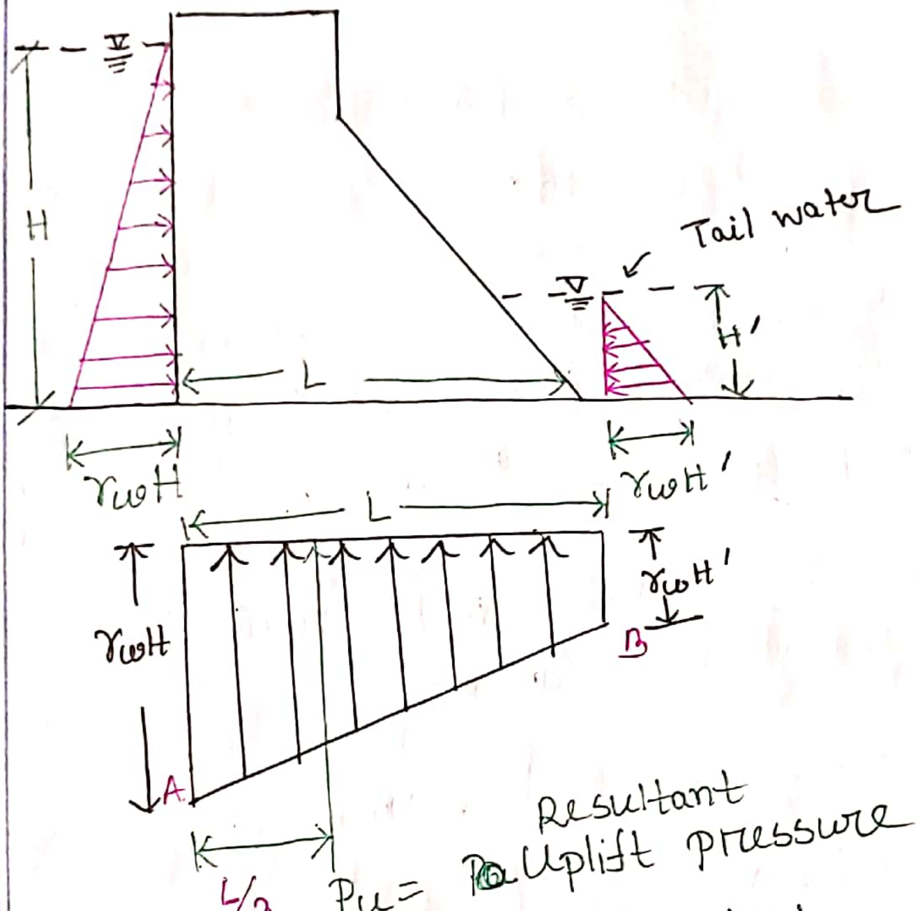
$\therefore F_w \times \frac{x}{3} = 11250 \times 2.5$

$\Rightarrow \frac{\gamma_w x^2}{2} \times \frac{x}{3} = 28125$

$\Rightarrow x = \sqrt[3]{\frac{28125 \times 6}{62.4}}$

$= 13.93$ ft Ans

Uplift pressure of Gravity Dam :



P_u = Resultant Uplift pressure force

$= \frac{1}{2}(A+B) \times L$

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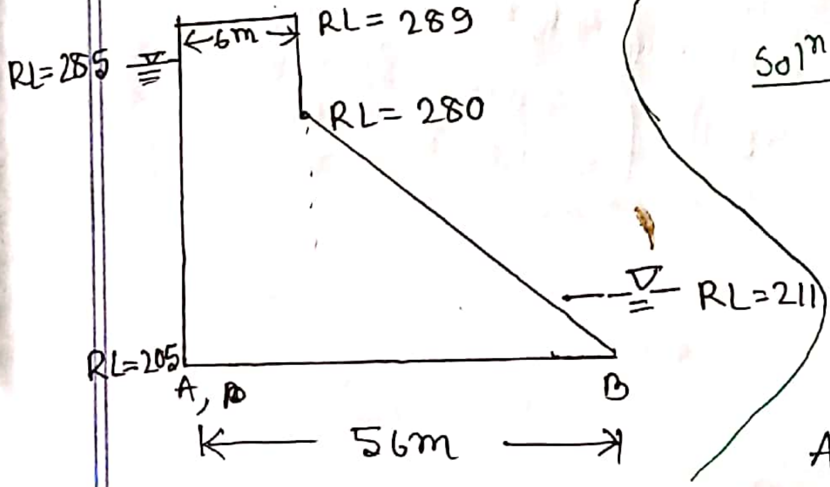
* Figure shows the section of a gravity dam built of concrete. Calculate the uplift pressure. [WARPO 2017]



Sol^n: Uplift pressure at, A

$= (285 - 205) \times 9.81 = 784.8 \text{ kN/m}^2$

Uplift pressure at, B

$= (211 - 205) \times 9.81 = 58.86 \text{ KN/m}^2$

Uplift pressure force $= \frac{1}{2} \times (784.8 + 58.86) \times 56$

$= 23622.48 \text{ KN/m}$

Acting at $\frac{56}{3} = 18.67$m distance from point A