CV3013 Foundation Eng PYP 16-17 .

1(a) Characteristic values of geotechnical parameters shall be based on results and derived values from laboratory and field tests, complemented by well-estabished experience .

→ selected as cautious estimate

→ can also defined as haven't multiply partial safety of factor.

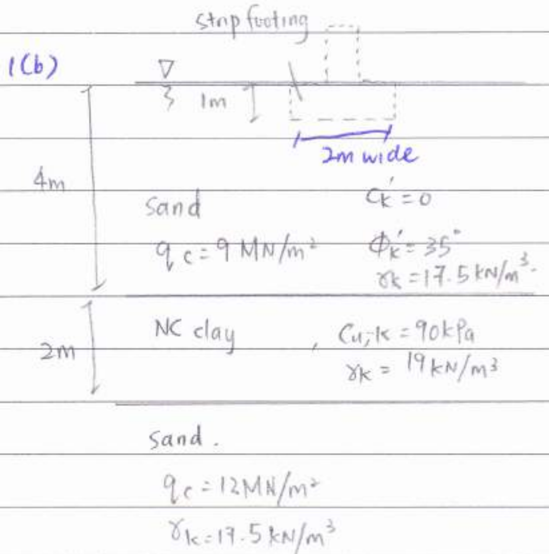
Design values of geotechnical parameters is based on 3 types which are action / force , materials' parameter and resistance. ~~according~~ Each types of design values have their own favourable or unfavourable action / resistance.

→ Favourable action is contribute to resistance of the mechanism and hence to prevent fail.

→ Unfavourable action is lead to failure of the mechanism.

→ can also defined as multiply / divide partial safety of factor ( depends on action, parameter and resistance) .

1(b)




i) According to EC7 DA1b,

Action : $\gamma_{G,f} = 1.0$    Material : $\gamma_{c'} = \gamma_{\phi'} = 1.25$

$\gamma_{G,u} = 1.0$    $\gamma_{cu} = 1.4$

$\gamma_{Q,u} = 1.3$    $\gamma_{\gamma'} = 1.0$

Design values :

For sand : $\gamma_{c,des} = 0$

$\gamma_{\phi,des} = \tan^{-1}\left(\frac{\tan 35°}{1.0}\right) = 28°$

$\gamma_{\gamma,des} = \frac{17.5}{1.0} = 17.5kN/m^3$

For NC clay ; $\gamma_{cu,} = \frac{90}{1.4} = 64.286kPa$

$\gamma_{\gamma,des} = \frac{19}{1.0} = 19\ kN/m^3$.

~~Qtn $q_f = S_cN_cC_u + \sigma_q$~~

~~$= (1)(2+\pi)($~~

~~For strip footing,~~

~~$S_c = 1 + 0.2(0)$~~

~~$= 1.0$~~

~~$N_c = (2+\pi)$~~

For sand : $q_f = S_cN_c\cancel{c'}^{\,0} + S_qN_q\sigma_q' + 0.5\,\gamma B S_\gamma N_\gamma$

$S_c$ = (no need cal)    $N_c$ = (no need cal).

$S_q = 1 + \frac{B}{L}\sin\phi'$    $N_q = \left(\frac{1+\sin\phi'}{1-\sin\phi'}\right)e^{\pi\tan\phi'}$

$= 1.0$ (strip footing)

$= \frac{1+\sin 28°}{1-\sin 28°}\,e^{\pi\tan 28°}$

$= 14.7198$.

$S_\gamma = 1.0$ ( strip footing )    $N_\gamma = 2(N_q - 1)\tan\phi'$

$= 2(14.7198 - 1)\tan 28°$

$= 14.58998$.

$\therefore\ q_f = 1.0(14.7198)(17.5-9.81)(1) + 0.5(17.5-9.81)(2m)(1.0)(14.58998)$

$= 225.392\ kPa$ .

(ii). Schmertmann method ( $E_s = 3.5 q_c$ )

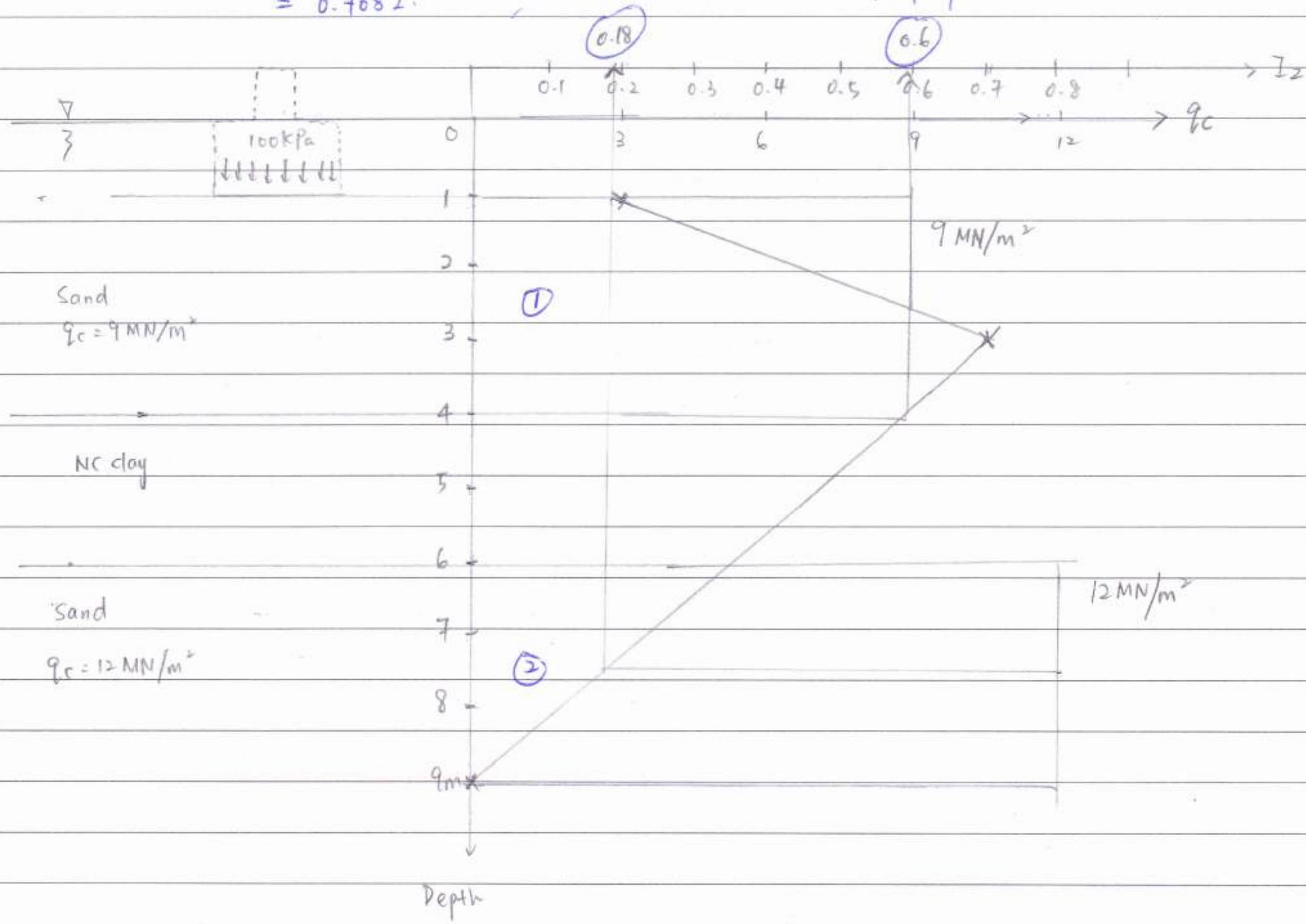
For strip footing, $z_{I,p} = B = 2m$, $z_{f_0} = 4B = 4(2) = 8m$.

$$I_{zp} = 0.5 + 0.1 \left(\frac{q_n}{\sigma_p'}\right)^{0.5}$$

$$= 0.5 + 0.1 \left(\frac{100}{(17.5-9.81)(3)}\right)^{0.5}$$

$$= 0.7082.$$

Idealised CPT profile.




Calculate $I_z \Delta z / E$ for sand layers ( $E = 3.5 q_c$ ):

| Layer | $\Delta z$ (m) | $q_c$ (MPa) | E (MPa) | $I_z$ | $I_z \Delta z / E$ |
|---|---|---|---|---|---|
| 1 | 3 | 9 | 31.5 | 0.6 | 0.05714 |
| 2 | 3 | 12 | 42 | 0.18 | 0.01286 |

$\Sigma = 0.069997.$

calculate correction factor.

$$C_1 = 1 - 0.5 \frac{\sigma_q'}{q_n} = 1 - 0.5 \frac{(17.5-9.81)(1)}{100} = 0.96155.$$

$$C_2 = 1 + 0.2 \log\left(\frac{t}{0.1}\right) = 1 + 0.2 \log\left(\frac{15}{0.1}\right) = 1.43522.$$

$$\therefore s = C_1 C_2 q_n \Sigma \frac{I_z \Delta z}{E}$$

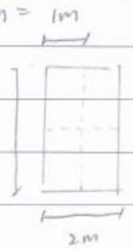
$$= 0.96155 (1.43522)(100)(0.069997)$$

$$= 9.66 mm.$$

(iii) Immediate settlement is negligible.

Consolidation settlement:

use Fadum Chart :




$C_c = 0.4$

$e_0 = 0.95$

$\Delta \sigma' = q\, I_{qr} = 4 \times 100 \times I_{qr}$ (kPa)

$S_{oed} = \frac{H}{1+e_0}\left[ C_c \log\left(\frac{\sigma_f'}{\sigma_p'}\right)\right]$

| Layer | z (m) | m | n | $I_{qr}$ | $\Delta\sigma'$ (kPa) | H (m) | $\sigma_0'$ (kPa) | $S_{oed}$ |
|---|---|---|---|---|---|---|---|---|
| clay | $\cancel{5}$ 4 | $\cancel{0.5}$ 1 | x | 0.215 | 86 | 2 | (19−9.81)(5) = 45.95 | $\frac{2}{1+0.95}\left[0.4 \log\left(\frac{45.95+86}{45.95}\right)\right]$ |
| | | | | | | | | = 0.188 mm |

using Scott's chart:

given A = 0.45.

$\frac{H}{B} = \frac{\cancel{0}2}{2} = \cancel{0}1 \rightarrow$ from chart, $\mu_c = 0.75$.

$\therefore$ consolidation settlement : $S_c = \mu_c S_{oed} = 0.75(0.188)$

$= 0.141\text{mm}$ ✗.

(iv). Total settlement $= 9.66 + 0.141$

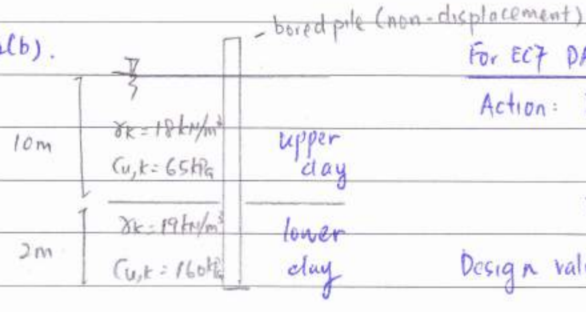
$= 9.801\text{mm}$ ✗.

2(a). 3 examples of displacement pile:

① driven pile ③ steel tubular pile.

② precast RC pile

2(b).




For EC7 DA1b:

Action: $\gamma_{G,f} = 1.0$ Material: $\gamma_{c'} = \gamma_{\phi'} = 1.0$ ← Be careful for pile, DA1b is all 1.0 for ~~material.~~

$\gamma_{G,u} = 1.0$ $\gamma_{cu} = 1.0$

$\gamma_{Q,u} = 1.3$

Design values: For upper clay : $\gamma_{des} = \frac{18}{1.0} = 18\text{kN/m}^3$

$C_{u,des} = \frac{\cancel{6}5}{1.0} = \cancel{6}5\text{kPa}$

For lower clay : $\gamma_{des} = \frac{\cancel{2}\,19}{1.0} = 19\text{kN/m}^3$

$C_{u,des} = \frac{160}{1.0} = 160\text{kPa}$.

End bearing resistance

$Q_{bu,k} = \frac{A_b q_u}{\xi}$

$= \dfrac{\frac{\pi(0.4)^2}{4}(S_c N_c C_u + \sigma_q)}{1.4}$

$N_c = (2+\pi)\left(1+0.2\sqrt{\frac{2}{0.4}}\right) \le 9.0$

$= 8.246$

$= 0.08976\left(1.0(8.246)(160) + 18\times 10 + 2\times 19\right)$

$= 137.99$

$Q_{bu,d} = \frac{137.99}{\gamma_{Rb}} = \frac{137.99}{2.0} = 68.99\text{ kN}.$

2 (b) Shaft resistance $Q_{su,d} = \dfrac{A_s \tau_s}{\xi \gamma_{RS}}$

For non displacement pile: Upper clay

$30 < c_{u,des} = 65\text{kPa} < 150$

$\therefore \alpha = 1.16 - \dfrac{65}{185}$

$= 0.8086$

Lower clay

$c_{u,des} = 160\text{kPa} > 150$

$\alpha = 0.35$

$\therefore Q_{su,upper}$

$= \dfrac{\pi(0.4)(10)(0.8086)(65)}{1.4(1.60)}$

$= 294.855\text{kN}$

$Q_{su,lower}$

$= \dfrac{\pi(0.4)(2)(0.35)(160)}{1.4(1.60)}$

$= 62.832\text{kN}.$

$\therefore$ Total $Q_{su,d} = 294.855\text{kN} + 62.832\text{kN}.$

$= 357.687\text{kN}$

$\therefore$ Total geotechnical resistance of pile $= Q_{bu,des} + Q_{su,des}$

$= 68.99 + 357.687$

$= 426.677\text{kN}$ ✗.

2(c) Purpose of pile load test:

① The uncertainty associated with using empirical properties in calculations is reduced.

② It can be verified that the proposed construction technique is acceptable and allows the integrity of the cast-in-place piles formed using the proposed method to be checked.

③ It can be verified that the ULS and SLS will be met by proposed design.

Trial piles are constructed solely for the purposes of load testing, usually before the main piling work commence. If sufficient load can be applied, these piles can be tested to the ULS to verify the pile capacity.

Working piles are piles that will be part of foundation, and as such are not tested to failure. A typical max load in such a test would be 150% of working load that the pile will ultimately carry, allowing for the SLS to be verified with an allowance for possible redistribution from other piles within the foundation.

2(d) $R_k = \min\left[\dfrac{R_{avg}}{\xi_1}, \dfrac{R_{min}}{\xi_2}\right]$

$R_{avg} = \dfrac{1300 + 1250 + 1460 + 1320 + 1410}{5} = 1348\text{kN}$

$R_{min} = 1250\text{kN}$

$= \min\left[\dfrac{1348}{1.35}, \dfrac{1250}{1.08}\right]$

$= \min[998.52, 1157.41]$

$= 998.52\text{ kN}$ ✗.

3. No need to consider WT as GWT is located deep below the base.

(a) For EC7 DA1b:

Step 1: Action: $\gamma_{G,f} = 1.0$ Material: $\gamma_{c'} = \gamma_{\phi'} = 1.25$ Resistance:

$\gamma_{G,u} = 1.0$ $\gamma_{\gamma} = 1.0$ $\gamma_{R} = 1.0$

$\gamma_{Q,f} = 0$ $\gamma_{cu} = 1.4$

$\gamma_{Q,u} = 1.3$

Step 2: Design values: For sandfill, $\phi_{des} = \tan^{-1}\left(\frac{\tan 38}{1.25}\right)$ For clay: $C_{u,d} = 50kPa$

$= 32°$ $q_{u,d} = 200kPa$

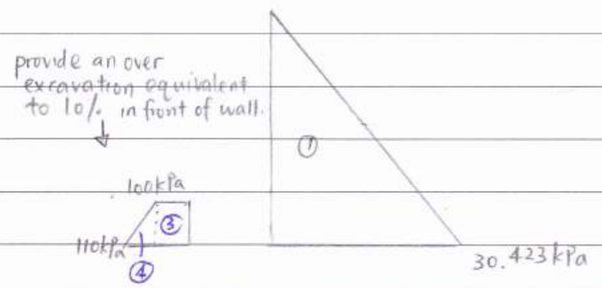
$\gamma_{des} = \frac{18kN/m^3}{1.0} = 18kN/m^3$ $\gamma_{des} = 20kN/m^3$.

No shear stress along the virtual back, ∴ Rankine theory applies.

$$K_a = \frac{1-\sin\phi'}{1+\sin\phi'} = \frac{1-\sin 32°}{1+\sin 32°} = 0.3073$$

Step 3: Plot Active Earth Pressure Diagram behind virtual back:

Due to self weight of soil Due to surcharge





Step 4: Horizontal Force (Active side) (destabilizing).

| Ref | Horizontal Active Pressure | Force | Lever Arm | Moment (kNm) |
|---|---|---|---|---|
| ① | $\frac{1}{2}(30.423)(5.5)(1.0)$ | 83.663 | 5.5/3 | 153.382 |
| ② | 3.073 (5.5)(1.3) | 21.972 | 5.5/2 | 60.423 |
| | | Σ = 105.635 | | Σ = 213.805 |

Vertical Force + Passive side

| Ref | Area | Force | Lever Arm | Moment (stabilizing) |
|---|---|---|---|---|
| Stem | 0.5 × 5 × 25 × 1.0 | 62.5 | 1.25 | 78.125 |
| base | 0.5 × 5 × 25 × 1.0 | 62.5 | 2.5 | 156.25 |
| soil above heel | 3.5 × 5 × $\cancel{25}$ 18 × 1.0 | 315 | 3.25 | 1023.75 |
| ③ | 100 (0.5)(1.0) | 50 | 0.5/2 | 12.5 |
| ④ | $\frac{1}{2}$(10)(0.5)(1.0) | 2.5 | 0.5/3 | 0.467 |
| | | ~~Σ = 492.5~~. | | Σ = 1271.042 |

sliding ULS: (for sliding ULS, ignore the variable load in front of virtual back)

Design horizontal force = 105.635 kN.

Design sliding resistance = $C_u \times B + 50 + 2.5$

= 50 × 5 + 52.5

= 302.5 kN > 105.635 kN, ∴ sliding ULS is o.k!

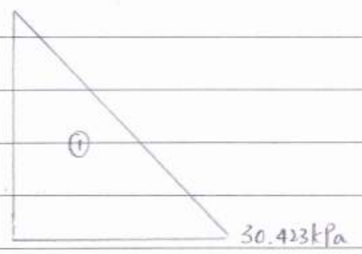
3(b). Step 1: same as part (a)

Step 2: same as part (a)

Step 3. Plot Active Earth pressure Diagram. (Ignore the soil in front of wall).

Due to self weight of soil

Due to surcharge





Step 4: Active side (Retained side).

same as part (a) table.

Passive side + vertical force

| Ref | Vertical Force | Force | Lever Arm | Moment. (stabilizing moment) |
|---|---|---|---|---|
| Stem | $0.5 \times 5 \times 25 \times 1.0$ | 62.5 | 1.25 | 78.125 |
| base | $0.5 \times 5 \times 25 \times 1.0$ | 62.5 | 2.5 | 156.25 |
| Soil on heel | $3.5 \times 5 \times 18 \times 1.0$ | 315 | 3.25 | 1023.75 |
| Transient load | $10 \times \cancel{1.0}\ 1.3$ | $\cancel{10}$ 13 | 1.25 | $\cancel{12.5}$ 16.25 |
| uniform load | $10/3.5 \times \cancel{1.0}\ 1.3$ | 3.7143 | 3.25 | 12.0715 |
| | | $\Sigma = 456.7143$ | | $\Sigma = 1286.45$ kNm. |

Bearing ULS:

$$\text{Lever Arm of } x, = \frac{\Sigma M}{V} = \frac{1286.45 - 213.805}{456.7143} = 2.35\text{m}$$

$$e = B/2 - x$$

$$= 5/2 - 2.35 = 0.15 < \frac{B}{6} = \frac{5}{6} = 0.83 \text{ (within middle third).}$$

$$P = \frac{V}{B}\left(1 + \frac{6e}{B}\right)$$

$$= \frac{456.7143}{5}\left(1 + \frac{6(\cancel{0.83}\ 0.15)}{5}\right)$$

$$= 107.7846 \text{ kPa}$$

Since $P_{max} = 107.7846\text{kPa} < 200\text{kPa}$.

$\therefore$ Bearing ULS is O.k!

4. Assume using uniform surcharge of ~~10kPa~~ 15kPa in this solution. (You might use ~~15~~ 10 kPa, as long as your concept is correct, prof will not deduct your mark)

Step 1: For EC7 DA1a.

Action: $\gamma_{G,f} = 1.0$ Material: $\gamma_{c'} = \gamma_{\phi'} = 1.0$ Resistance. $\gamma_R = 1.0$

$\gamma_{G,u} = 1.35$ $\gamma_{cu} = 1.0$

$\gamma_{Q,f} = 0$

$\gamma_{Q,u} = 1.5$

Step 2: Design values.

For backfill, $\phi_{des} = \tan^{-1}\left(\frac{\tan 30^\circ}{1.0}\right) = 30^\circ$ $\delta' = 2/3\,(30^\circ) = 20^\circ$ For backfill ($\delta'/\phi' = 0.66$, $\phi'_{des} = 30^\circ$)

$\therefore \delta/\phi' = 0.667$. From log spiral chart,

$\gamma_{des} = 17\ kN/m^3$ $K_{AH} = 0.28$, ~~$K_{PH}$~~

For Sand, $\phi'_{des} = 35^\circ$ $\delta' = 2/3\,(35) = 23.33^\circ$ For sand ($\delta'/\phi' = 0.66$, $\phi'_{des} = 35^\circ$)

$\gamma_{des} = 20\ kN/m^3$ $\therefore \delta/\phi' = 0.667$ From log spiral chart,

$K_{AH} = 0.23$, $K_{PH} = 7.2$.

(a) Assume uniform head loss

(i). Difference in hydraulic head, $\Delta h = $ ~~6.5m~~ 2m

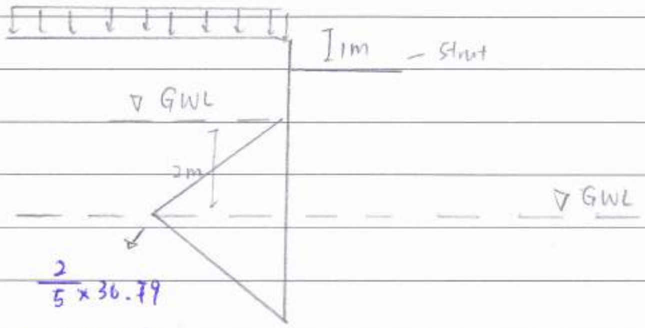
Length of flow path $= 2+3+3 = 8m$.

Loss in head from pile toe to formation level $= 3/8 \times 2 = 0.75m$.

Hydraulic head at wall toe $= 3 + 0.75 = 3.75m$.

$\therefore$ Pore pressure at wall toe $= 3.75 \times 9.81 = 36.7875\ kPa$.

(ii)

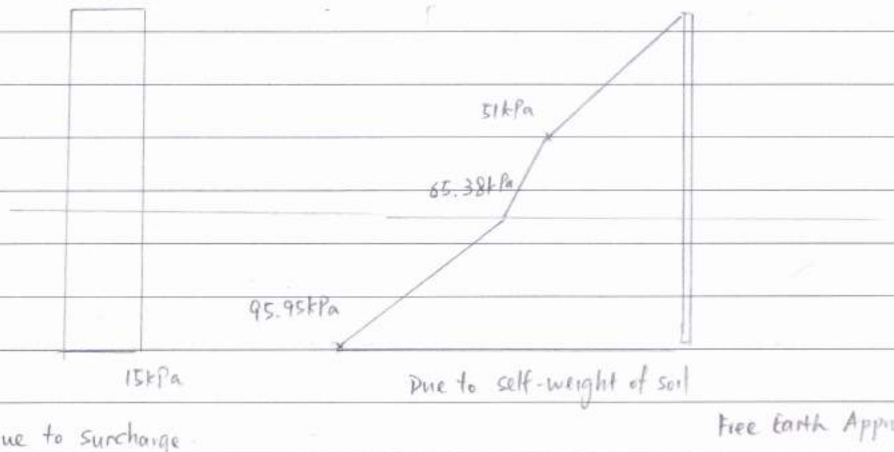



$\frac{2}{5} \times 36.79$

$= 14.716\ kPa$.

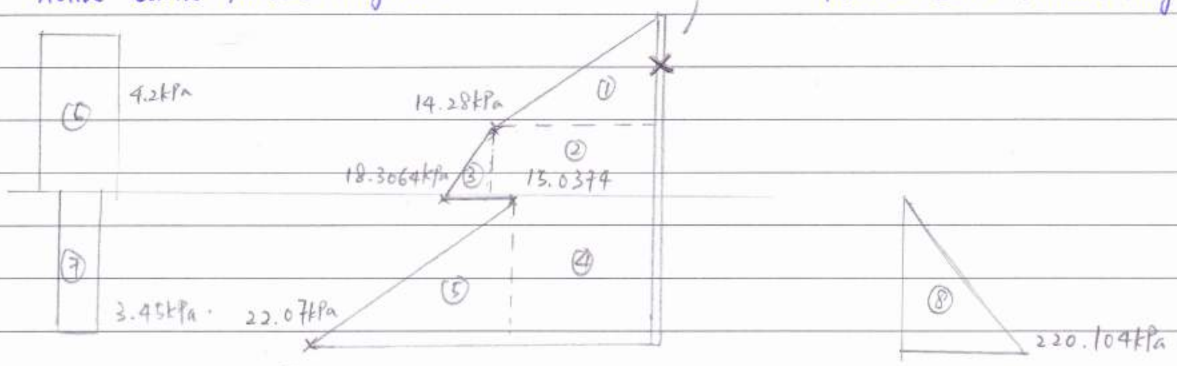
net water pressure.

(b) (i) Plot vertical effective stress diagram:




Free Earth Approach

Active Earth Pressure Diagram: Passive Earth Pressure Diagram.




Step 4: Active side (~~Retained~~)

| Ref | Horiz Act Pressure | Force | Lever Arm | Moment (destabilizing) |
|---|---|---|---|---|
| 1 | $\frac{1}{2}(14.28)(3)(1.35)$ | 28.917 | 1 | 28.917 |
| 2 | $14.28(2)(1.35)$ | 38.556 | 3 | 115.668 |
| 3 | $\frac{1}{2}(18.3064-14.28)(2)(1.35)$ | 5.436 | 3.33 | 18.1019 |
| 4 | $15.0374(3)(1.35)$ | 60.9015 | 5.5 | 334.96 |
| 5 | $\frac{1}{2}(22.07-15.0374)(3)(1.35)$ | 14.241 | 6 | 85.446 |
| 6 | $4.2(5)(1.5)$ | 31.5 | 1.5 | 47.25 |
| 7 | $3.45(3)(1.5)$ | 15.525 | 5.5 | 85.3875 |
| Water pressure | $\frac{1}{2}(5)(14.716)(1.35)$ | 49.6665 | 5.333 | 264.8714 |
| | | 244.743 | | $\Sigma = 980.60$ |

Passive side.

| Ref | Area | Force | Lever Arm | Moment (stabilizing) |
|---|---|---|---|---|
| 8 | $\frac{1}{2}(220.104)(3)(1.35)$ | 445.7106 | 6 | 2674.26 |

(c) Rotational ULS:

destabilizing moment = 980.60 kNm < stabilizing moment = 2674.26 kNm.

$\therefore$ Rotational ULS is O.K!

(d) $\Sigma F = 0$.

Since the passive force = 445.7 kN is large than active force = 244.7 kN, there is no need ~~to~~ additional force for strut.

$\therefore$ min. resistance in strut is 0.

8

Done by:
HP
21/1/17.