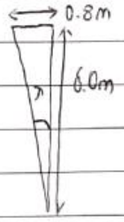
CV4110 Excavation and Retaining Walls
Semester 1 Examination 2019-2020
Suggested Solutions by Lim Kai Jian

Q1

Q1 $H = 6.0\,m$ Date: No:

Coulomb Method since $\delta = 26^\circ$

(a) (i)



$\lambda = \tan^{-1}\left(\frac{0.8}{6.0}\right) = 7.595^\circ$

$\alpha = 90^\circ + 7.595^\circ = 97.6^\circ$

$\phi' = 35^\circ$

$\delta = 26^\circ$

$\beta = 23^\circ$

(ii)

$$K_a = \left[\frac{\dfrac{\sin(97.6-35)}{\sin 97.6}}{\sqrt{\sin(97.6+26)} + \sqrt{\dfrac{\sin(35^\circ+26^\circ)\sin(35-23)}{\sin(97.6-23)}}}\right]^2$$

$= 0.442$

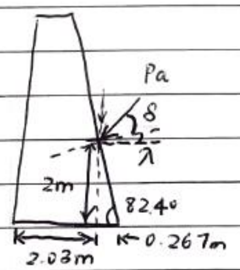
(b) (i) DA1 C2 (DA1 b)

Variable Unfavourable $\rightarrow \times 1.30$

$P_a = \frac{1}{2}(6.0)^2(17)(0.442) = 135.252\ kN/m$

$P_{ah} = 135.252 \cos(26 + 7.595) = 112.66\ kN/m$

$P_{av} = 135.252 \sin(26 + 7.595) = 74.84\ kN/m$



Weight of concrete Retaining Wall

$= 0.5(0.7+2.3)(6.0) \times 24 = 216\ kN/m$

~~ODF for sliding~~ $= \dfrac{\cancel{(216)\tan}\ (216 + 74.84)\tan 26^\circ}{112.66}$ ~~$= 1.259 > 1 \Rightarrow$ OK!~~

(ii)

Force Component	Magnitude (kN/m)	Lever Arm to Toe (m)	Moment (kNm/m)
P_{ah}	112.66	2.0	225.32
P_{av}	74.84	2.033	152.15
Weight of Wall	216	1.15	248.4

~~ODF for overturning~~ $= \dfrac{248.4 + 152.15}{225.32}$ ~~$= 1.778 > 1 \Rightarrow$ OK!~~

(c) Rankine $K_a = \dfrac{1 - \sin\phi'}{1 + \sin\phi'}$

$\Rightarrow$ When there is no interface friction between retained soil and the wall (i.e. $\delta = 0^\circ$)

When the slope of the backfill is 0° (i.e. $\beta = 0^\circ$)

When the wall back is vertical (i.e. $\alpha = 90^\circ$)

(d)

For Kerisel and Absi, the Ka values are almost equivalent to that of Coulomb's method, but it is slightly higher, meaning the active earth pressures will be more conservative as compared to Coulomb's method. On the other hand, the Kp values for Kerisel and Absi are lower than that of Coulomb's method. In general, this means that Kerisel and Absi method is more conservative since it overestimates the active earth pressures and underestimates passive earth pressures which are usually resistive for a retaining wall design.

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Q2

Q2 (a)(i) Net water pressure at G = $\gamma_w \times 3 = 30\,kPa$ // $24.71\,kN/m^2$

(ii) $i_{avg} = 3/(2\times7+3) = 0.1765$

$\gamma_a' = (19-10) + 0.1765(10) = 10.765\,kN/m^3 = \gamma_{eR}'$

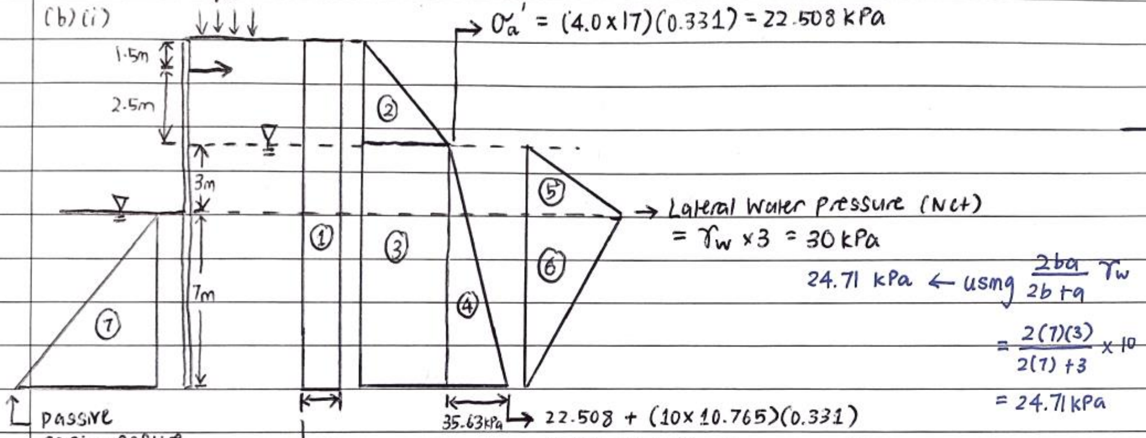
$\gamma_p' = (19-10) - 0.1765(10) = 7.235\,kN/m^3 = \gamma_{eL}'$

(iii) Using DA1C2, $\varphi'_d = \tan^{-1}(\tan 36°/1.25) = 30.17°$

Variable Unfavourable → × 1.30

$K_a = \frac{1-\sin 30.17°}{1+\sin 30.17°} = 0.331$, $K_p = 1/K_a = 3.02$

(b)(i)



Passive earth pressure $= (7\times7.235)(3.02)$ = 152.9 kPa

→ lateral pressure from surcharge $= 1.3\times17\times0.331$ = 7.32 kPa

$22.508 + (10\times10.765)(0.331) = 22.508 + 35.63 = 58.14\,kPa$

(b)(ii)

Component	Force (kN/m)	Lever Arm to Tie Rod (m)	Moment (kNm/m)
①	(7.32)(14) = 102.48	5.5	563.64
②	0.5(22.508)(4) = 45.02	$(2/3\times4) - 1.5 = 1.167$	52.54
③	(22.508)(3+7) = 225.08	2.5+3+2 = 7.5	1688.1
④	0.5(35.63)(3+7) = 178.15	9.167	1633.1
⑤	0.5(30)(3) = 45 37.065	$(\frac{2}{3}\times3)+2.5 = 4.5$	166.8 202.5
⑥	0.5(30)(7) = 105 86.485	7.833	677.44 822.47
⑦	0.5(152.9)(7) = 535.15	10.167	5440.87

~~ODF for overturning~~ $= \frac{\Sigma M_R}{\Sigma M_O} = \frac{5440.87}{4962.35}$ ~~= 1.096 > 1 ⇒ OK!~~

(c) Force in each tie rod = 1.5 × (102.48 + 45.02 + 225.08 + 178.15 + 45 + 105 − 535.15)

= 1.5 × (165.58)

= 248.37

Ans:

F = 207.4 kN

T = 138.28 kN/m

F = 139.13 (1.5) = 208.7 kN

T = 102.48 + 45.02 + 225.08 + 178.15 + 37.065 + 86.485 − 535.15 = 139.13 kN/m

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Note: There are two methods to estimate the net water pressure. The worked solution in black is using a more conservative method which underestimates the factor of safety and overestimates the force in each tie rod. The official numerical answers provided suggests that the net water pressure is worked by using the following formula: $\left(\frac{2ba}{2b+a}\right)\gamma_w$, which is also worked out in blue.

Both solutions are acceptable.

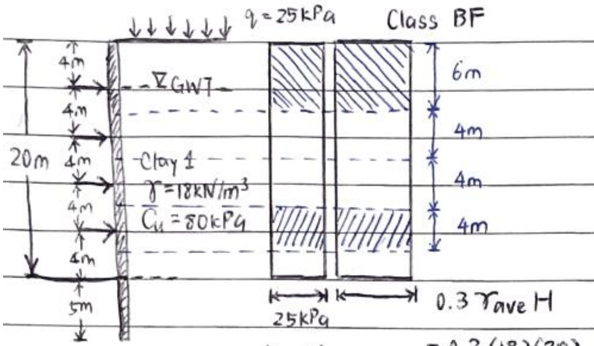
Q3

Date: No:

$H = 20m$, $B = 40m$, Surcharge $q = 25kPa$

(a) Allowable Strut Load = 3000kN

Determine the maximum horizontal strut spacing



$= 0.3(18)(20)$
$= 108kPa$

$K_a = 1$
$\varphi_u = 0$

Maximum Force (kN/m) which occurs at the 1st level strut

$= (25+108)(6) = 798\ kN/m$

$798 \times \text{Strut Spacing} \leq 3000\ kN$

$\text{Strut Spacing} \leq \frac{3000}{798}$

$= 3.759\ m$

For other levels:

$\text{Strut Spacing} \leq 3000/(25+108)(4)$

$= 5.64m$

Maximum Wall Bending Moment

$\approx \frac{p(d_{max})^2}{10}$

$= (25+108)(4)^2 \times \frac{1}{10} = 212.8\ kNm/m$

(b) Wall Stiffness $EI = 4.7 \times 10^4\ kNm^2/m$

$\gamma_w = 10\ kN/m^3$

Terzaghi Method FS for Basal Heave

$= \frac{5.7\, C_{ub}\, B_1}{\gamma H B_1 + q B_1 - C_{uh} H}$

$B_1 = 0.7B = 0.7(40) = 28m$

$C_{uh} = 80kPa$, $C_{ub} = 150kPa$

$H = 20m$, $\gamma = 18\ kN/m^3$

$= \frac{5.7(150)(28)}{(18)(20)(28) + (25)(28) - (80)(20)}$

$= 2.608$

From Clough et al. chart,

for System Stiffness $= (4.7 \times 10^4)/(10 \times 4^4) = 18.4$

$(\delta_{h\,max} / He) = 0.5\%$

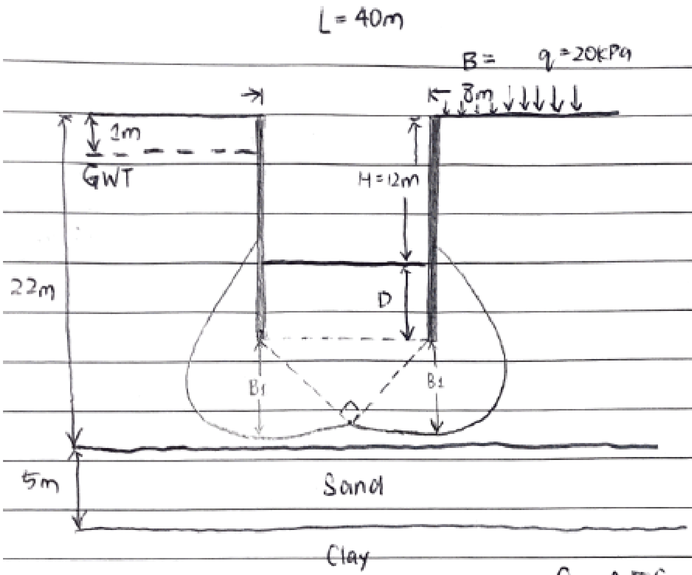
$\delta_{h\,max} = 0.5\% \times 20 = 0.1\ m = 100mm$

(c) Scheme B is likely to lead to larger wall deflections since the excavation depth is always deeper as compared to scheme A. The largest wall deflections generally occur near the formation level, thus based on this idea, a larger excavation depth with the same level of strut restraint will result in greater wall deflection and ground settlement.

(d) Alternative Design : 1.2m thick diaphragm wall

Diaphragm wall is made of concrete and hence it is stiffer than the flexible sheet pile wall. As such, the strut force will increase and hence max strut spacing reduces. On the other hand, since the system stiffness increases, following the general trend of the Clough et. al. chart, we would expect deflection to reduce, for a same Basal Heave factor of safety.

Q4



(a) $H > B = 8m$

$\Rightarrow$ Deep Excavation

$B_1 = 0.7(8) = 5.6m$

(b) For $D = 4m$,

Eide et al. Basal Heave Fs

$$= \frac{C_{ub} N_c + 2 C_A D \left(1 + \frac{B}{L}\right)\left(\frac{1}{B}\right)}{\gamma H + q}$$

$$= \frac{(35)(7.28) + 2(17.5)(4)\left(1 + \frac{8}{40}\right)\left(\frac{1}{8}\right)}{(20)(12) + 20}$$

$$= 1.06$$

$C_A = 0.5\,C_u$

$= 0.5(35)$

$= 17.5\ kPa$

$C_u = 35kPa$

$\gamma = 20kN/m^3$

$$(H+D)/B = \frac{12+4}{8} = 2 < 2.5$$

$$N_c = 5\left(1 + 0.2 \times \frac{8}{40}\right)\left(1 + 0.2 \times \frac{16}{8}\right) 1.0$$

$$= 7.28$$

(c) FS for Blowout Failure $= \frac{\text{Resistance}}{\text{Driving Force}}$

$$= \frac{\text{Weight of Soil} + \text{Frictional Resistance (Soil-Soil)} + \text{(Soil-Wall)}}{\text{Uplift Force}}$$

$$= \frac{(10 \times 8 \times 20) + 2(17.5)x + 2(10 - x)(35)}{(8)(22+2)(10)}$$

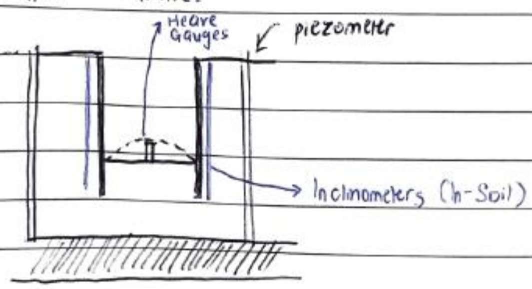
$$\geq 1.10$$

$1600 + 35x + 700 - 70x \geq 2112$

$188 \geq 35x$

$5.37 \geq x$

(d) Heave Gauges or Deep Extensometers to measure the height of the basal heave at the formation level of the excavation.
Piezometer to measure the water pressure of the underlain sand layer that gives rise to the uplift pressure on the block of soil that results in uplift failure.



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