CV3013 2022/23 Sem 1

by Renardi M. (@pardirenardi)

1. (a) ***Phases of Site Investigation***

1. Project assessment and desk study
2. Field reconnaissance/site visit
3. Planning of SI task
4. Subsurface exploration (drill, sample, in-situ test)
5. Soil sampling and lab testing
6. Ground water exploration and monitoring
7. Synthesis of data and interpretation

(b) Undisturbed soil sample

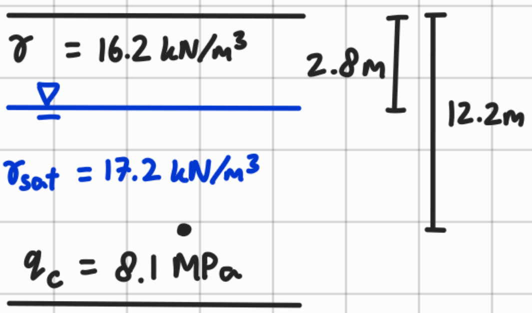
- Soil structure and water content preserved
- Used for shear strength, consolidation, permeability

Disturbed soil sample

- Soil structure significantly damaged
- Retains composition and grain size
- Suitable for classification and compaction

Lab tests	Disturbed	Undisturbed
water content	✓	✓
γ		✓
LL, PL, PI	✓	✓
Size distribution	✓	✓
Permeability		✓
Proctor Compaction test	✓	✓
Consolidation test		✓
CU test		✓

(c)



$$\sigma'_{v0} = (16.2 \times 2.8) + (17.2 - 10) \times (12.2 - 2.8)$$
$$= 113.04 \text{ kPa}$$

$$I_D = \left[-1.21 + 0.584\left(\frac{8.1 \times 10^3}{(113.04)^{0.5}}\right)\right] \times 100\%$$
$$= 47.3\%$$

$$\phi'_{max} = 6.6 + 11 \log\left(\frac{8.1 \times 10^3}{(113.04)^{0.5}}\right)$$
$$= 38.3^\circ$$

2\. (a)

$$\sum \gamma_A Q \leq \frac{R\left(\frac{X}{\gamma_x}\right)}{\gamma_R} \qquad (\text{All } \gamma \geq 1.0)$$

To allow for margin of error, the actions (load) Q is multiplied by γ_Q, while the material strength properties X is reduced by γ_x. The corresponding resistance R (which is dependent on X) is also reduced by γ_R.

(b)

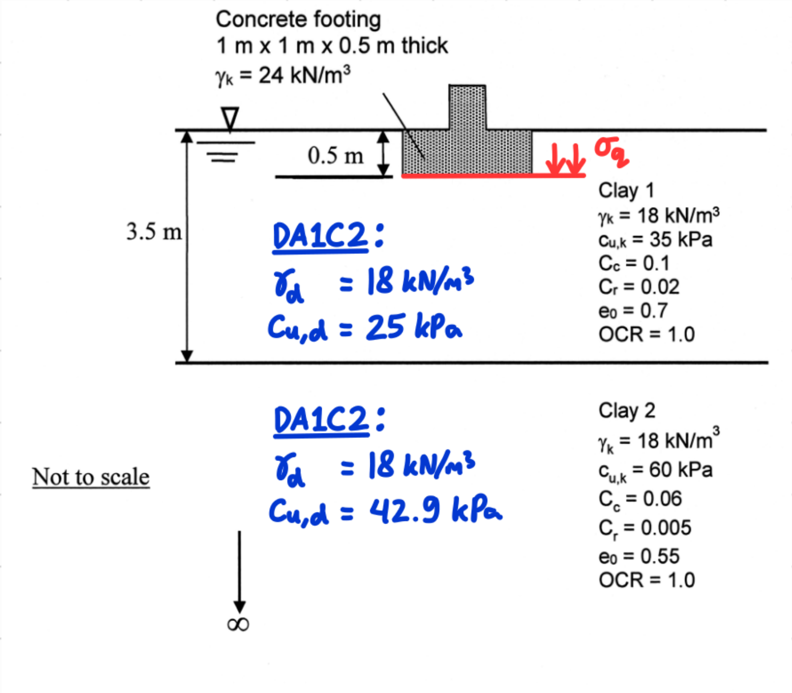


Figure Q2

(i) DA1C2

$$Q_d = 1.0\left[110 + \underbrace{(24 \times 1 \times 1 \times 0.5)}_{\text{weight of foundation}}\right] + 1.3(30) = 161 \text{ kN}$$

$$\sigma_q = 18 \times 0.5 = 9 \text{ kN/m}^2$$

$$s_c = 1 + 0.2\left(\frac{1}{1}\right) = 1.2$$

$$q_f = s_c N_c c_u + \sigma_q = (1.2)(2+\pi)(25) + 9 = 163.2 \text{ kPa}$$

$$R_d = q_f \times A = 163.2 \times (1 \times 1) = 163.2 \text{ kN} > Q_d = 161 \text{ kN}$$

OK!

(ii)

Recommended layers for manual computation of s_c:

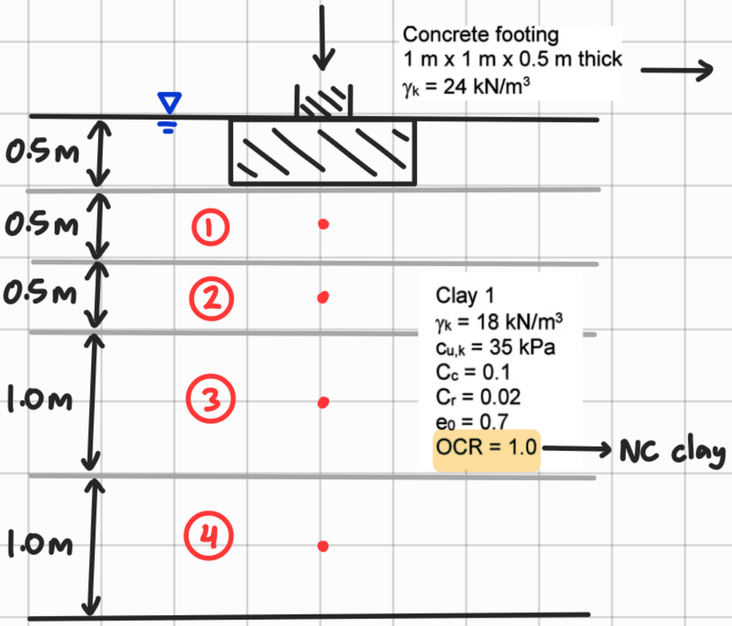
Layer number	Approximate layer thickness	
	Square footing	Continuous footing
1	B/2	B
2	B/2	B
3	B	2B
4	B	2B

For square footing, find equivalent diameter i.e.

$$B^2 = \frac{\pi D^2}{4} \rightarrow D = \sqrt{\frac{4B^2}{\pi}}$$

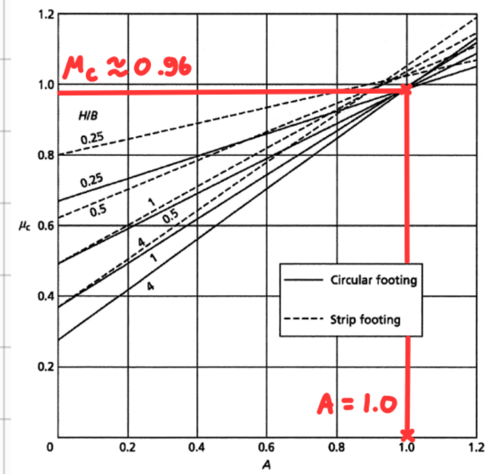
Note: Use middle of layer for z

$Q = 110 + 30 + (24 \times 1 \times 1 \times 0.5) = 152\ kN$



$B = 1m$, square

$D = \sqrt{\frac{4(1)^2}{\pi}} = 1.128$



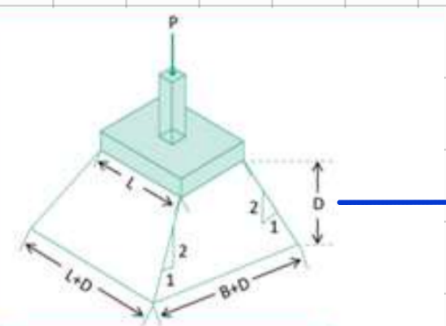
Q	152	kN
γ	18	kN/m³
e_0	0.7	
C_c	0.1	
C_r	0.02	
OCR	1	

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

layer thickness

Layer	d from GL (m)	H (m)	σ'_0 (kPa)	$\Delta\sigma'$ (kPa)	$\sigma'_f = \sigma'_0 + \Delta\sigma'$ (kPa)	s_{oed} (mm)	μ_c	$s_c = \mu_c * s_{oed}$ (mm)
1	0.75	0.5	6	97.28	103.28	36.35	0.96	34.90
2	1.25	0.5	10	49.63	59.63	22.81	0.96	21.90
3	2	1	16	24.32	40.32	23.61	0.96	22.67
4	3	1	24	12.41	36.41	10.65	0.96	10.22
						Total s_c (mm) =		89.68

$\sigma_o' = \gamma' \times d$



$$\boxed{\Delta\sigma'_D = \frac{P}{A} = \frac{P}{(B+D)\times(L+D)}}$$

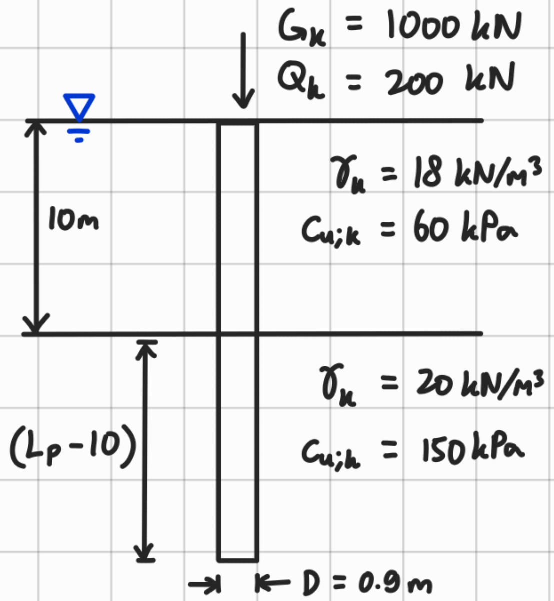
P = the load applied on the foundation (KN)
A = the area of the stress distribution at **depth (D)**

Note the "D" here is the depth from the bottom of foundation (not from Ground Level)

3. (a)

- Weak upper soil
- Foundation is located near a slope

(b)



Use M1 if no negative skin friction

↑

DA1C2 (M1)

$\gamma_d = 18$ kN/m³

$c_{u;d} = 60$ kPa

$\gamma_d = 20$ kN/m³

$c_{u;d} = 150$ kPa

$D = 0.9$ m

$\gamma_{k;\,concrete} = 24$ kN/m³

$\xi = 1.55$

Load Condition

$$Q_d = 1.0\left(1000 + 24 \times \frac{\pi}{4}(0.9)^2 \times L_p\right) + 1.3(200) = 15.268\, L_p + 1260$$

End Bearing

$$\sigma_q = (18 \times 10) + (20 \times (L_p - 10)) = 20L_p - 20$$

Assume $L_p \gg D \rightarrow N_c = 9.0$

$$Q_{b;k} = \left(\frac{\pi}{4} \times 0.9^2\right) \times \left[9.0 \times 150 + (20L_p - 20)\right]$$

$$= 12.723\, L_p + 846.109$$

$$Q_{b;k} = \frac{Q_{b;u}}{\xi} = \frac{12.723\, L_p + 846.109}{1.55} = 8.208\, L_p + 545.88$$

Shaft Resistance

Bored Pile → non-displacement pile

Clay 1

$30 < c_u = 60 < 150$ kPa

$$\alpha = 1.16 - \left(\frac{60}{185}\right) = 0.836 \Rightarrow \tau_{int} = 0.836 \times 60 = 50.14 \text{ kPa}$$

Clay 2

$C_u = 150 \geq 150$ kPa

$\alpha = 0.35 \Rightarrow \tau_{int} = 0.35 \times 150 = 52.5$ kPa

$$Q_{s;k} = \frac{Q_{s;u}}{\xi} = \frac{50.14 \times (\pi \times 0.9) \times 10 + 52.5 \times (\pi \times 0.9) \times (L_p - 10)}{1.55}$$

$$= 95.77\, L_p - 43.05$$

Find pile length

$$\frac{Q_{b;k} + Q_{s;k}}{\gamma_t} \geq Q_d$$

$$\frac{(8.208\, L_p + 545.88) + (95.77\, L_p - 43.05)}{2.0} \geq 15.268\, L_p + 1260$$

$$51.989\, L_p + 251.415 \geq 15.268\, L_p + 1260$$

$$36.721\, L_p \geq 1008.59$$

$$L_p \geq 27.5 \text{ m}$$

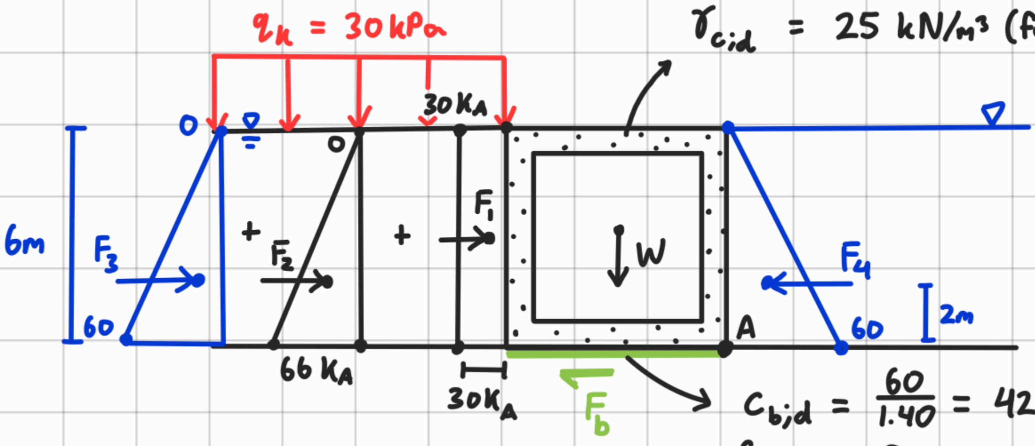
(c)
- Pile load test : to verify that the installed pile's load capacity is greater than the loading condition used in design.
- Trial/test piles : Piles constructed solely for the purpose of load testing, usually before main piling works start.

4. (a) (i) & (ii)

Case 1: High tide level, empty tank

$V_{concrete} = (6 \times 6 - 5.2 \times 5.2) = 8.96 \; m^3/m$

$\gamma_{c;d} = 25 \; kN/m^3$ (favorable)



$C_{b;d} = \dfrac{60}{1.40} = 42.86 \; kPa$

$\delta_{b;d} = 0$

Sand

$\gamma_d = 21 \; kN/m^3$

$\gamma_d' = 11 \; kN/m^3$

$\phi_d' = \tan^{-1}\left(\dfrac{\tan(34^\circ)}{1.25}\right) = 28.4^\circ$

$K_A = \dfrac{1 - \sin\phi'}{1 + \sin\phi'} = 0.355$

Clay

$\gamma_d = 18 \; kN/m^3$

$C_{u;d} = \dfrac{84}{1.40} = 60 \; kPa$

$F_1 = 0.355(30 \times 6) \times 1.30 = 83.07 \; kN$	$M_1 = 83.07 \times 3 = 249.21 \; kNm$
$F_2 = 0.355\left(\frac{1}{2} \times 66 \times 6\right) \times 1 = 70.29 \; kN$	$M_2 = 70.29 \times 2 = 140.58 \; kNm$
$F_3 = \left(\frac{1}{2} \times 60 \times 6\right) \times 1 = 180 \; kN$	$M_3 = 180 \times 3 = 540 \; kNm$
$F_4 = F_3 = 180 \; kN$	$M_4 = 180 \times 3 = 540 \; kNm$
$F_b = 42.86 \times 6 \times 1 = 257.16 \; kN$	$M_w = (25 \times 8.96) \times 3 = 672 \; kNm$

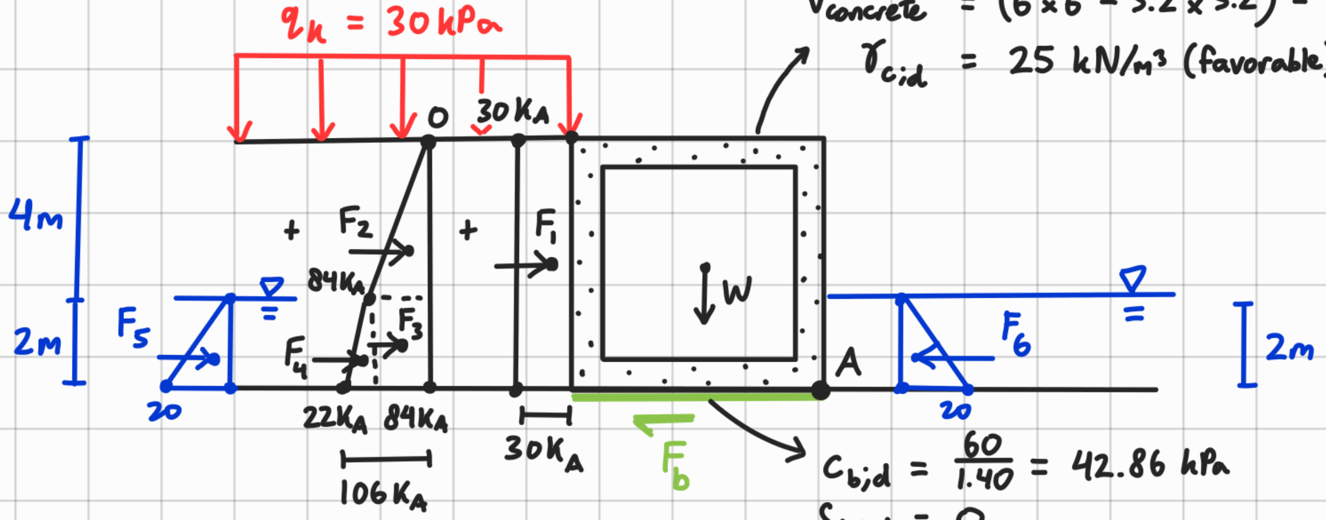
$\gamma_A \Rightarrow$ 1.30 for $\gamma_{Q;dst}$

1.00 for $\gamma_{G;dst}$

$$\text{Sliding ODF} = \frac{R_d}{E_d} = \frac{257.16 + 180}{83.07 + 70.29 + 180} = 1.31 > 1.0 \quad \text{OK!}$$

$$\text{Overturning ODF} = \frac{M_{stb}}{M_{dst}} = \frac{540 + 672}{249.21 + 140.58 + 540} = 1.30 > 1.0 \quad \text{OK!}$$

Case 2 : Low tide level, empty tank



$V_{concrete} = (6\times6 - 5.2\times5.2) = 8.96\ m^3/m$

$\gamma_{c;d} = 25\ kN/m^3$ (favorable)

$c_{b;d} = \frac{60}{1.40} = 42.86\ kPa$

$\delta_{b;d} = 0$

$F_1 = 0.355(30\times6)\times1.30 = 83.07\ kN$	$M_1 = 83.07\times3 = 249.21\ kNm$
$F_2 = 0.355\left(\frac{1}{2}\times84\times4\right)\times1 = 59.64\ kN$	$M_2 = 59.64\times\left(2+\frac{4}{3}\right) = 198.8\ kNm$
$F_3 = 0.355(84\times2)\times1 = 59.64\ kN$	$M_3 = 59.64\times1 = 59.64\ kNm$
$F_4 = 0.355\left(\frac{1}{2}\times22\times2\right)\times1 = 7.81\ kN$	$M_4 = 7.81\times\frac{2}{3} = 5.21\ kNm$
$F_5 = \left(\frac{1}{2}\times20\times2\right)\times1 = 20\ kN$	$M_5 = 20\times\frac{2}{3} = 13.3\ kNm$
$F_6 = F_5 = 20\ kN$	$M_6 = 20\times\frac{2}{3} = 13.3\ kNm$
$F_b = 42.86\times6\times1 = 257.16\ kN$	$M_W = (25\times8.96)\times3 = 672\ kNm$

$$\text{Sliding ODF} = \frac{R_d}{E_d} = \frac{257.16+20}{83.07+59.64+59.64+7.81+20} = 1.20 > 1.0 \text{ ok!}$$

$$\text{Overturning ODF} = \frac{M_{stb}}{M_{dst}} = \frac{13.3+672}{249.21+198.8+59.64+5.21+13.3} = 1.30 > 1.0 \text{ ok!}$$

∴ Adequate for all cases, sliding & overturning. Taking critical cases,

Sliding ODF = 1.20

Overturning ODF = 1.30

(b)
- Sliding ODF increases as the additional friction force acts as a stabilising force towards the left.
- Overturning ODF stays the same as the friction force line of action passes through the center of rotation, creating no stabilising moment.