CV3013 2021/22 Sem 1

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1. (a) **Borehole Depth**

1. Include all significantly affected strata
2. Extend below all soft/weak strata
3. If rock encountered, extend >3m to confirm bedrock
4. Check locations of utilities, services, tunnels

(b)

Parameter	SPT	FVT	PMT	CPT	DMT	PLT
Consolidation characteristics: m_v, C_c						
Stiffness properties: G, G_0			G	G_0* (SCPTU)	G, G_0*	YES
Drained strength properties: ϕ', c'	YES		YES	YES	YES	YES
Undrained strength properties: c_u (in-situ)	YES	YES	YES	YES	YES	YES
Soil state properties: I_D, OCR, K_0	I_D		K_0 (via σ_{h0})	I_D, OCR (K_0)	ALL	
Permeability: k				YES[†]		YES[‡]

Notes: *Using a seismic instrument (i.e. SCPTU or SDMT). [†]Via a dissipation test on a piezocone (CPTU or SCPTU) – i.e. stopping penetration and measuring decay of u_2 (see Further Reading). [‡]By stopping DMT expansion and measuring decay of cavity pressure (see Further Reading).

CPT is the most reliable.

- Provides continuous data } unlike SPT
- Strong theoretical basis for interpretation } unlike SPT
- Suitable for various types of soil (unlike PMT which is better for cohesive soils)

(c) Area ratio, C_a. The ratio between the volume of displaced soil to the volume of sample. $C_a = \frac{V \text{ of displaced soil}}{V \text{ of sample}}$

(d) $C_a = \frac{d_1^2 - d_3^2}{d_3^2} = \frac{48^2 - 46^2}{46^2} \times 100\% = 8.9\% < 10\%$ OK!

∴ OK for undisturbed samples.

(e) $C_a = \frac{d_4^2 - d_5^2}{d_5^2} = \frac{52^2 - 45^2}{45^2} \times 100\% = 33.5\% > 10\%$ NOT OK!

∴ Not OK for undisturbed samples.

② (a) Limit States are the criteria a structure must satisfy to ensure its performance.

- Ultimate Limit State (ULS) → deals with forces that may cause failure (bearing capacity, shaft resistance, stability, etc.)
- Serviceability Limit State (SLS) → deals with displacements, etc. that may affect the functionality of the structure.

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

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.

The partial factor γ depends on the limit state considered.

(b)

$g_k = 5$ kN/m
$q_k = 10$ kN/m

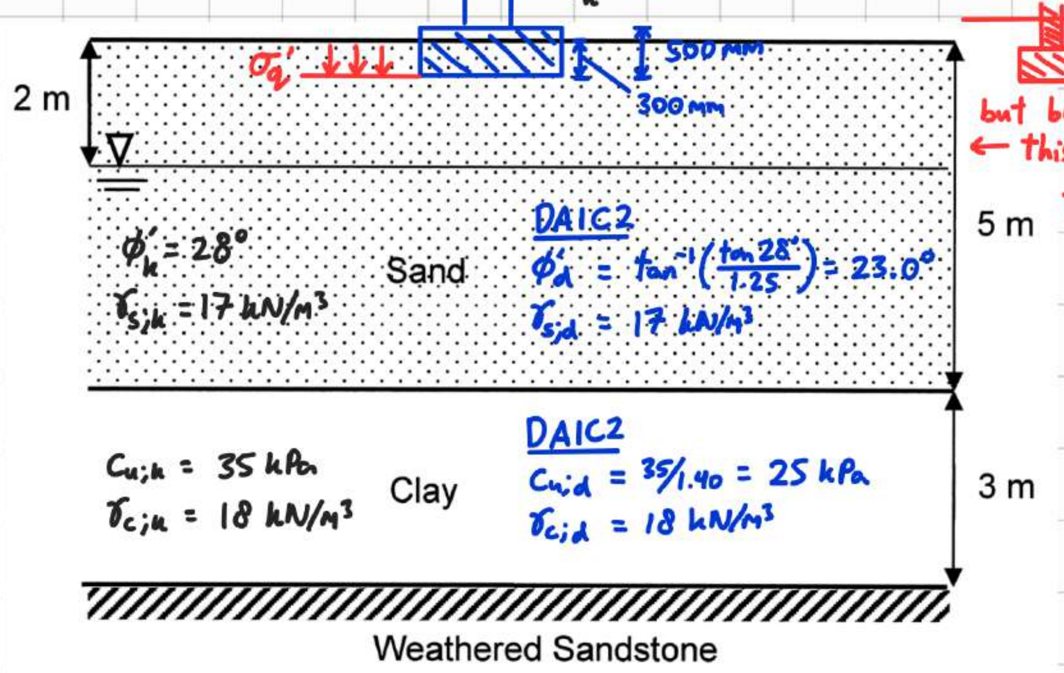
Strip footing *
b = 500 mm
γ_k = 23.5 kN/m³

* "embedded 300 mm below the ground surface" may be ambiguous as it might also mean:

300 mm
500 mm

but based on numerical solution ← this is the one meant.

Try clarify with prof :)



(i) DA1C1

$$N_q = \frac{(1+\sin 28°)}{(1-\sin 28°)} e^{\pi \tan(28°)} = 14.72$$

$$N_c = \frac{14.72-1}{\tan 28°} = 25.8$$

$$N_\gamma = 2(14.72-1)\tan 28° = 14.6$$

$s_q = 1$; $s_c = \frac{1(14.72) - 1}{14.72 - 1} = 1$; $s_\gamma = 1$

Case	For $0.5\gamma' B s_\gamma N_\gamma$
Case 1: $d_w \le d$	$\gamma' = \gamma - \gamma_w$
Case 2: $d < d_w < d+B$	$\gamma' = \gamma - \gamma_w \left[1 - \left(\frac{d_w - d}{B}\right)\right]$
Case 3: $d + B \le d_w$	$\gamma' = \gamma$ (d_w = depth of GWT)

$d = 0.3\ m$ $\quad d_w = 2m \ge d_w + B \rightarrow \gamma' = \gamma$

$d+B = 0.8\ m$

$\sigma_q' = 17 \times 0.3 = 5.1\ kPa$

$q_f = (1)(25.8)(0) + (1)(14.72)(5.1) + 0.5(17)(0.5)(1)(14.6) = 137.1\ kPa$

$R_d = q_f \times B = 137.1 \times 0.5 = 68.6\ kN/m$ //

$Q_d = 1.35(5 + 23.5 \times 0.5 \times 0.5) + 1.5(10) = 29.7\ kN/m < R_d$ OK! //

DA1C2

$N_q = \frac{(1+\sin 23°)}{(1-\sin 23°)} e^{\pi \tan(23°)} = 8.7$

$N_c = \frac{8.7 - 1}{\tan 23°} = 18.1$

$N_\gamma = 2(8.7-1)\tan 23° = 6.55$

$s_q = 1$; $s_c = \frac{1(8.7) - 1}{8.7 - 1} = 1$; $s_\gamma = 1$

$\sigma_q' = 17 \times 0.3 = 5.1\ kPa$

$q_f = (1)(18.05)(0) + (1)(8.7)(5.1) + 0.5(17)(0.5)(1)(6.55) = 72.2\ kPa$

$R_d = q_f \times B = 72.2 \times 0.5 = 36.1\ kN/m$ //

$Q_d = 1.0(5 + 23.5 \times 0.5 \times 0.5) + 1.3(10) = 23.9\ kN/m < R_d$ // OK!

(ii) $q = (5 + 23.5 \times 0.5 \times 0.5) = 10.9 \text{ kN/m}$

Sand

Let $z_I \approx 1 \text{ m}$,

$$\bar{N}_{60} = \frac{10}{1} = 10$$

$$I_c = \frac{1.71}{(10)^{1.4}} \times 100\% = 6.81\%$$

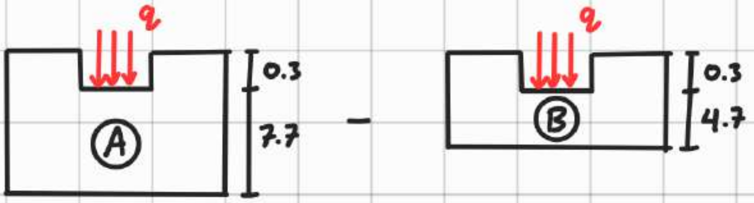
$$S = 10.9 \times (0.5)^{0.7} \times 6.81\% = 0.457 \text{ m}$$

$F_s = 1.0$

$F_l = 1.0$

$F_t = 1.0$ (immediate) $\qquad S_{sand} = 0.457\text{m}$

Clay



$E_{u;k} = 6 \text{ MPa}$

Ⓐ: $\frac{d}{B} = \frac{0.3}{0.5} = 0.6$

$\frac{H}{B} = \frac{7.7}{0.5} = 15.4$

$\frac{l}{B} = \infty$

$\mu_0 = 0.95$

$\mu_1 = 1.55$

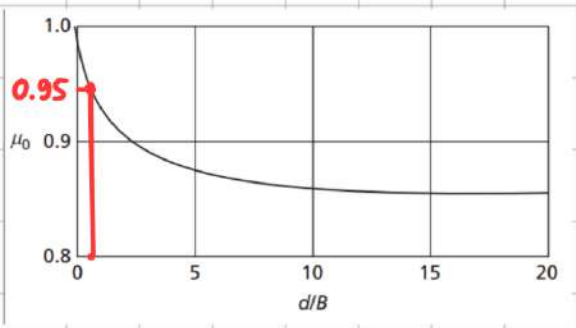
Ⓑ: $\frac{d}{B} = \frac{0.3}{0.5} = 0.6$

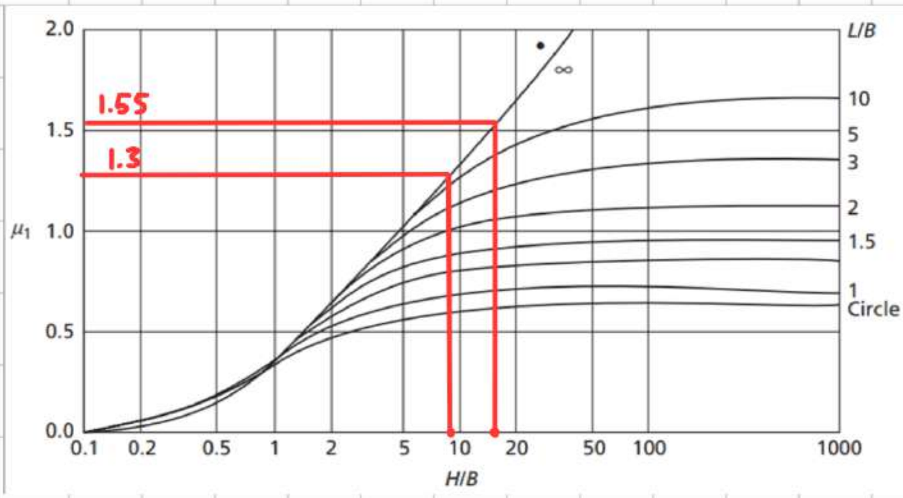
$\frac{H}{B} = \frac{4.7}{0.5} = 9.4$

$\frac{l}{B} = \infty$

$\mu_0 = 0.95$

$\mu_1 = 1.3$





$$S_A = 0.95 \times 1.55 \times \frac{10.9 \times 0.5}{6} = 1.338$$

$$S_B = 0.95 \times 1.3 \times \frac{10.9 \times 0.5}{6} = 1.122$$

$$S_{clay} = S_A - S_B = 1.338 - 1.122 = 0.216 \text{ m}$$

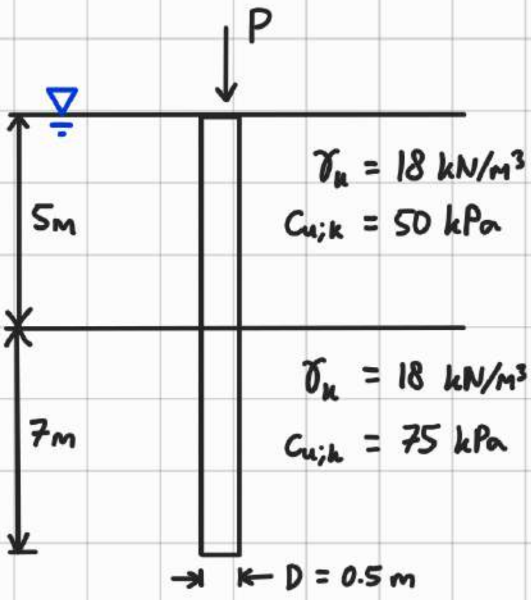
$\therefore$ Immediate settlement $= S_{sand} + S_{clay}$

$= 0.457 + 0.216 = 0.673 \text{ m} > 125 \text{ mm}$ **NOT OK!**

Possible solutions:

- Increase width of foundation to reduce q
- Perform ground improvement works on the soil (temporary surcharge, drains, compaction, etc.)
- Replace soft soil with strong fill materials

3. (a)



Use M1 if no negative skin friction

↑

DA1C2 (M1)

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

$c_{u;d} = 50\ kPa$

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

$c_{u;d} = 75\ kPa$

$D = 0.5\ m$

$\gamma_{k;\,concrete} = 23.5\ kN/m^3$

$\xi = 1.55$

End Bearing

$$\sigma_q = (18 \times 5) + (18 \times 7) = 216\ kPa$$

$$N_c = (2+\pi)\left(1 + 0.27\sqrt{\frac{7}{0.5}}\right) = 10.3 > 9.0 \Rightarrow \text{use } 9.0$$

$$Q_{b;u} = \left(\frac{\pi}{4} \times 0.5^2\right) \times \left[9.0 \times 75 + (18 \times 12)\right] = 174.9\ kN$$

$$Q_{b;k} = \frac{Q_{b;u}}{\xi} = \frac{174.9}{1.55} = 112.8\ kN$$

Shaft Resistance

Bored Pile → non-displacement pile

Clay 1

$30 < c_u = 50 < 150\ kPa$

$$\alpha = 1.16 - \left(\frac{50}{185}\right) = 0.890 \Rightarrow \tau_{int} = 0.890 \times 50 = 44.49\ kPa$$

Clay 2

$30 < c_u = 75 < 150\ kPa$

$$\alpha = 1.16 - \left(\frac{75}{185}\right) = 0.755 \Rightarrow \tau_{int} = 0.755 \times 75 = 56.63\ kPa$$

$$Q_{s;k} = \frac{Q_{s;u}}{\xi} = \frac{44.49 \times (\pi \times 0.5 \times 5) + 56.63 \times (\pi \times 0.5 \times 7)}{1.55} = 627.2\ kN$$

Find pile resistance

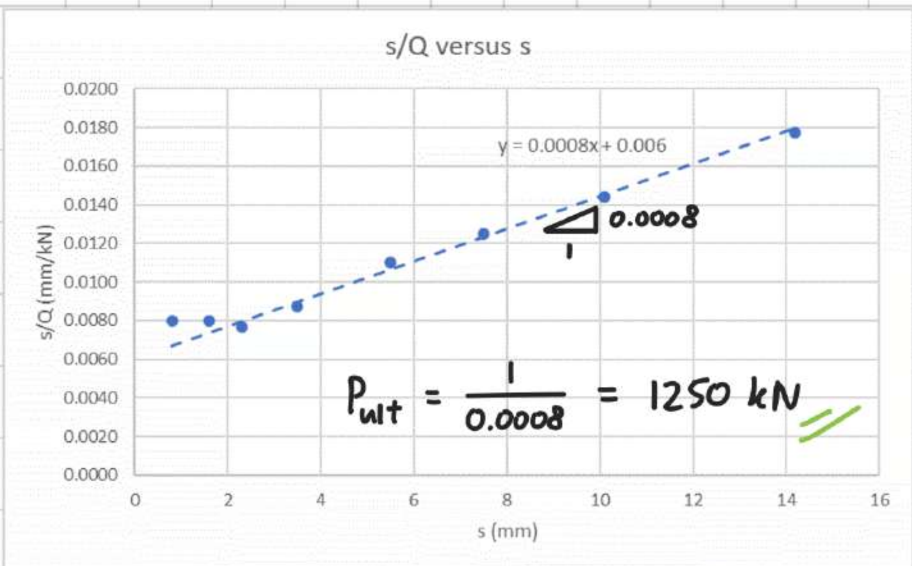
$$R_d = \frac{Q_{b;k} + Q_{s;k}}{\gamma_t} = \frac{112.8 + 627.2}{2} = 370 \text{ kN}$$

(b) • Single pile failure (Mode 1): one or more piles may fail and hence reduces the foundation's load bearing capacity.

• Block failure (Mode 2): the whole block of soil beneath the pile cap and enclosed by the piles fail as a single pier.

(c)

Q (kN)	s (mm)	s/Q (mm/kN)
100	0.8	0.0080
200	1.6	0.0080
300	2.3	0.0077
400	3.5	0.0088
500	5.5	0.0110
600	7.5	0.0125
700	10.1	0.0144
800	14.2	0.0178

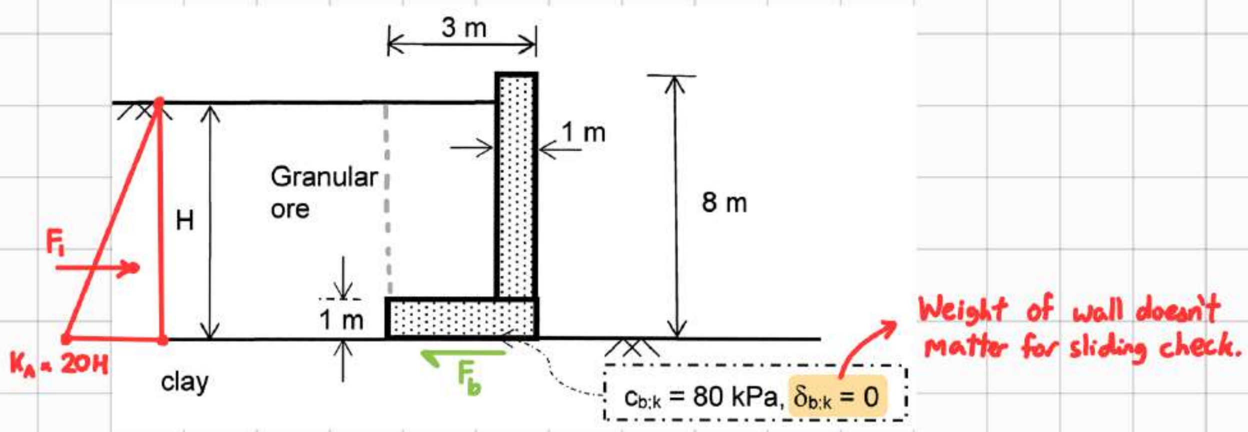


$$P_{ult} = \frac{1}{0.0008} = 1250 \text{ kN}$$

④ (a) (i) **DA1C1**

$\gamma_{ore;d} = 20\ kN/m^3$ $\gamma_{c;k} = 25\ kN/m^3$

$\phi'_{ore;d} = 36^\circ$ $K_A = \frac{1 - \sin 36^\circ}{1 + \sin 36^\circ} = 0.260$



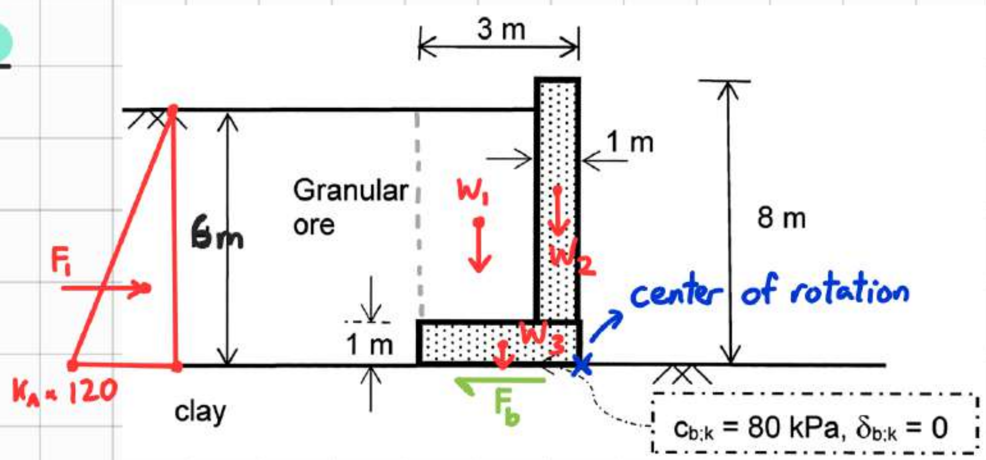
$F_1 = 0.260 \times \left(\frac{1}{2} \times 20H \times H\right) = 2.6\ H^2\ kN$

$F_b = 80 \times 3 = 240\ kN$

$ODF = \frac{R_d}{E_d} = \frac{240}{1.35 \times 2.6H^2} = 1.2 \Rightarrow 240 = 4.212\ H^2$

$H = 7.55\ m$

(ii) **DA1C1**



$F_1 = 0.260 \times \left(\frac{1}{2} \times 120 \times 6\right) = 93.6\ kN$

$F_b = 80 \times 3 = 240\ kN$

$W_1 = 20 \times (2 \times 5) = 200\ kN$

$W_2 = 25 \times (1 \times 7) = 175\ kN$

$W_3 = 25 \times (3 \times 1) = 75\ kN$

$M_1 = 93.6 \times \left(\frac{1}{3} \times 6\right) = 187.2\ kNm$

$M_b = 0$

$M_{W1} = 200 \times 2 = 400\ kNm$

$M_{W2} = 175 \times 0.5 = 87.5\ kNm$

$M_{W3} = 75 \times 1.5 = 112.5\ kNm$

$ODF = \frac{R_d}{E_d} = \frac{1.0\ (400 + 87.5 + 112.5)}{1.35 \times 187.2} = 2.37 > 1.0$ OK!

(b) DA1C2

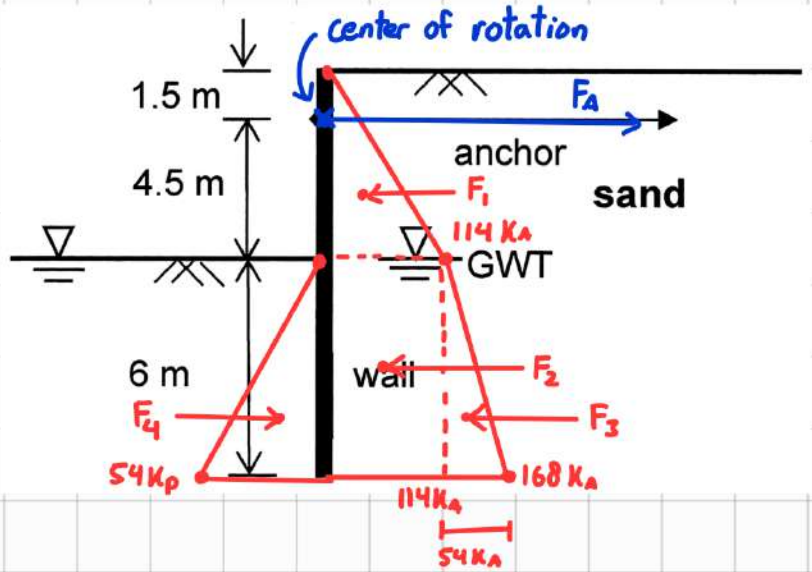
$\gamma_k = 19\ kN/m^3 \rightarrow \gamma_d = 19\ kN/m^3$

$\phi'_k = 30° \rightarrow \phi'_d = \tan^{-1}\left(\frac{\tan 30°}{1.25}\right) = 24.8°$

$K_A = \frac{1-\sin 24.8°}{1+\sin 24.8°} = 0.409$

$K_P = \frac{1+\sin 24.8°}{1-\sin 24.8°} = 2.445$

Free earth support → No force R acting at the bottom of the wall.



Note: water pressure cancels each other due to equal GWT.

$F_1 = 0.409 \times \left(\frac{1}{2} \times 114 \times 6\right) = 139.9\ kN$

$F_2 = 0.409 \times (114 \times 6) = 279.8\ kN$

$F_3 = 0.409 \times \left(\frac{1}{2} \times 54 \times 6\right) = 66.3\ kN$

$F_4 = 2.445 \times \left(\frac{1}{2} \times 54 \times 6\right) = 396.1\ kN$

$F_A = ?$

$M_1 = 139.9 \times \left(4.5 \times \frac{2}{3} - 1.5\right) = 209.9\ kNm$

$M_2 = 279.8 \times \left(4.5 + \frac{6}{2}\right) = 2098.5\ kNm$

$M_3 = 66.3 \times \left(4.5 + 6 \times \frac{2}{3}\right) = 563.6\ kNm$

$M_4 = 396.1 \times \left(4.5 + 6 \times \frac{2}{3}\right) = 3366.9\ kNm$

$M_A = 0$

Overturning ODF $= \frac{R_d}{E_d} = \frac{1.0 \times 3366.9}{1.0 \times (209.9 + 2098.5 + 563.6)}$

$= 1.172 > 1.0$ // OK!

Minimum anchor force required,

$F_{dst} = F_{stb}$

$1.0 \times (139.9 + 279.8 + 66.3) = 1.0 \times (396.1 + F_A)$

$F_A = 89.9\ kN/m$

For 4m spacing, $F_A = 4 \times 89.9 = 359.6\ kN/anchor$ //