PROBLEM ①

$V = 225\,kN$

$M = 150\,kN$

(a) $q_{ult} = s_c d_c N_c s_u + \gamma h$

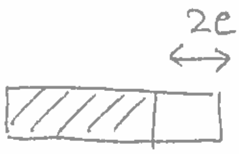
$s_c = 1 + 0.2\frac{B}{L} = 1.2$

$d_c = 1 + 0.33\tan^{-1}(h/D) = ?$ since $d_c > 1$ it is conservative to use $d_c = 1$

$N_c = 2 + \pi = 5.14$

$q_{ult} = (1.2)(1.0)(5.14)(30\,kPa) + (18\,kN/m^3)(1\,m) = 200\,kPa$

(b) Use Meyerhof's method



$e = \frac{M}{V} = \frac{150\,kN\,m}{225\,kN} = 0.67\,m$

$B' = B - 2e$

Using EUROCODE factors

Factor S_u $\quad S_{u_{all}} = \frac{30\,kPa}{1.4} = 21.4\,kPa$

$q_{all} = (1.2)(5.14)(21.4) + 18 = 150\,kPa$

$B' = \sqrt{\frac{225\,kN}{150\,kPa}} = 1.22\,m$

Factor load

$$B' = \sqrt{\frac{(225\,kN)(1.35)}{200\,kPa}} = 1.23\ m$$

Therefore use $B' = 1.23\,m$ $\Rightarrow$ $B = 1.23 + (0.67)(2) = 2.57\,m$

Use $B = 2.6\ m$

One could argue for a footing of size $2.6\,m \times 1.3\,m$

Check if q_{all} is reduced

$$s_c = 1 + 0.2\left(\frac{1.3}{2.6}\right) = 1.1$$

$$q_{all} = (1.1)(5.14)(21.4) + 18 = 139\ kPa$$

$$B' = \sqrt{\frac{225\,kN}{139\,kPa}} = 1.27\,m \qquad B = 2.61\,m$$

Select size : $\boxed{2.6 \times 1.3\,m}$

(c) Derive interaction diagram

$$\frac{V}{L(B-2e)} = (2+\pi)\,S_u$$

$$\frac{V}{L(2+\pi)S_u} = B - \frac{2M}{V}$$

$$M = \frac{BV}{2} - \frac{V^2}{2L(2+\pi)S_u}$$

$$M = \frac{BV}{2}\left[1 - \frac{V}{\underbrace{BL(2+\pi)S_u}_{V_{ult}}}\right]$$

$$M = \frac{BV}{2}\left[1 - \frac{V}{V_{ult}}\right]$$

For V alone, $M = 150\text{kNm}$; $V_{ult} = \left[(1.1)(5.14)(30\text{kPa}) + 18\text{kPa}\right](2.6)(1.3)$

$= (188\text{ kPa})(2.6)(1.3) = 634\text{ kN}$

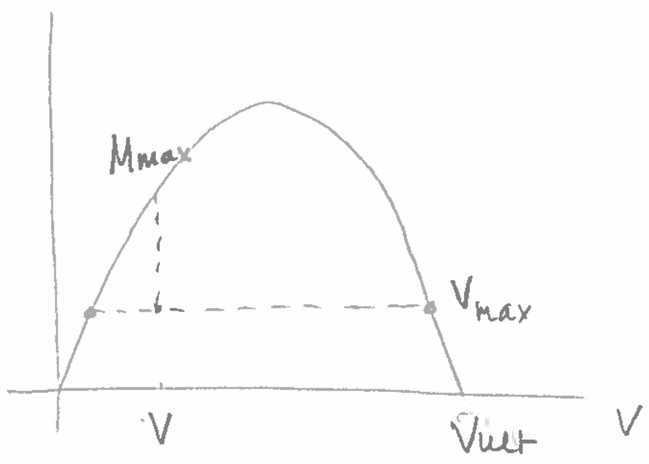
$$M = \frac{B V_{max}}{2}\left[1 - \frac{V_{max}}{V_{ult}}\right]$$

$$\left(\frac{B}{2V_{ult}}\right)V_{max}^2 - \frac{B}{2} V_{max} + M = 0$$

$$V_{max}^2 - V_{ult} V_{max} + \frac{2M V_{ult}}{B} = 0$$

$$V_{max}^2 - (634\text{kN}) V_{max} + \underbrace{\frac{(2)(150\text{kNm})(634\text{kN})}{(2.6\text{m})}}_{73,154} = 0$$

$$V_{max} = \frac{634 \pm \sqrt{(634)^2 - 4(73,154)}}{2} = \frac{634 \pm 331}{2} = 482\text{kN}$$



$$FS = \frac{482}{225} = 2.14$$

For M alone

$$M_{max} = \frac{B(225\text{kN})}{2}\left[1 - \frac{(225\text{kN})}{(634\text{kN})}\right] = 189\text{ kN m}$$

$FS = 189/150 = 1.26$ LOW!

Q1 Shallow foundation with V-M load

12 attempts, Average mark 10.71/20 (50.5%), Maximum 19, Minimum 5.5.

Most students could solve easily the first part of the problem, calculating capacity and selecting an appropriate dimension for the foundation. Students could apply code partial factors correctly. Most difficulties were encountered in deriving the V-M failure surface, although a good number of students made good progress.

a)

$L = 20m$ $\quad \frac{L}{D} = 20$ $\quad n = \gamma' K_p^2 = 90 \ kN/m^3$

$D = 1m$

$e = 0$

Short pile:

$$\frac{H}{nD^3} \sim 50$$

Long pile:

If $\frac{H}{nD^3} = 50$ $\quad \frac{M_p}{nD^4} \gtrsim 330$

$$H_{ult} = 50 \times 90 = 4500 \ kN$$

$$M_p > 330 \times 90 = 29,700 \ kNm$$

$$M_p = \sigma_y \left[\frac{d_o^3 - d_i^3}{6} \right]$$

$\sigma_y = 250 \ MPa$

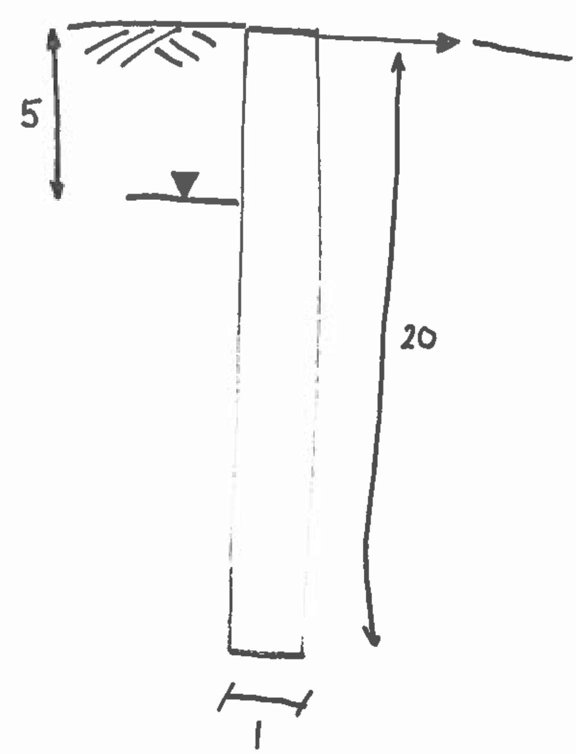
$d_o = 1m$

$d_i^3 = 0.2872$

$d_i = 0.66m$

$t = \underline{\underline{170mm}}$

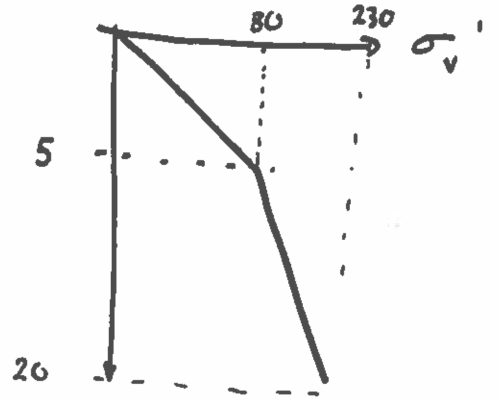
b)

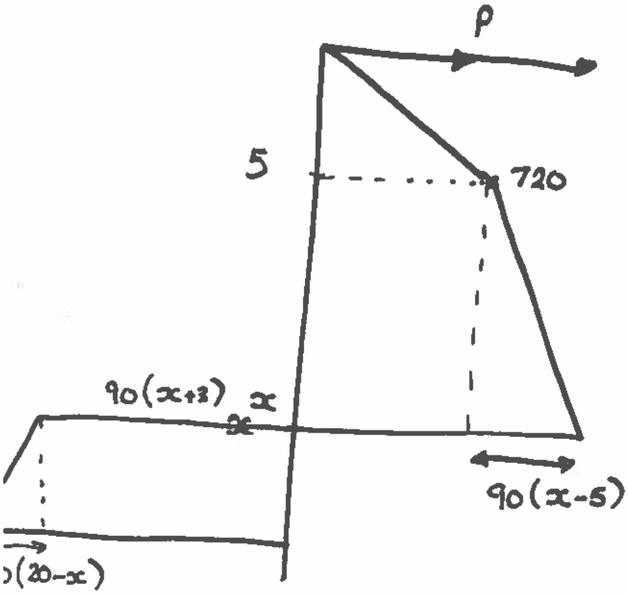


$\gamma_{dry} = 16 \text{ kN m}^{-3}$ $\quad K_p = 3$

$\gamma' = 10 \text{ kN m}^{-3}$

Resistance per unit length $= D\, K_p^2\, \sigma_v'$





(TOP)

$$\frac{5\times720}{2}\times\frac{2\times5}{3} + (x-5)\times720\times\frac{x+5}{2} + \frac{90(x-5)^2}{2}\left(5+\frac{2}{3}(x-5)\right)$$

$$=$$

$$90(x+3)(20-x)\frac{(20+x)}{2} + (20-x)^2\times\frac{90}{2}\times\left(20-\frac{20-x}{3}\right)$$

$$6000 + 360(x^2-25) + 225(x^2-10x+25) + 30(x^3-15x^2+75x-125)$$

$$=$$

$$45\underbrace{(x+3)(400-x^2)}_{(1200+400x-3x^2-x^3)} + 900(400-40x+x^2) - 15(8000-1200x+60x^2-x^3)$$

$$(20-x)^3 = (400-40x+x^2)(20-x) = 8000-1200x+60x^2-x^3$$

$$-295{,}125 + 270x^2 + 60x^3 = 0$$

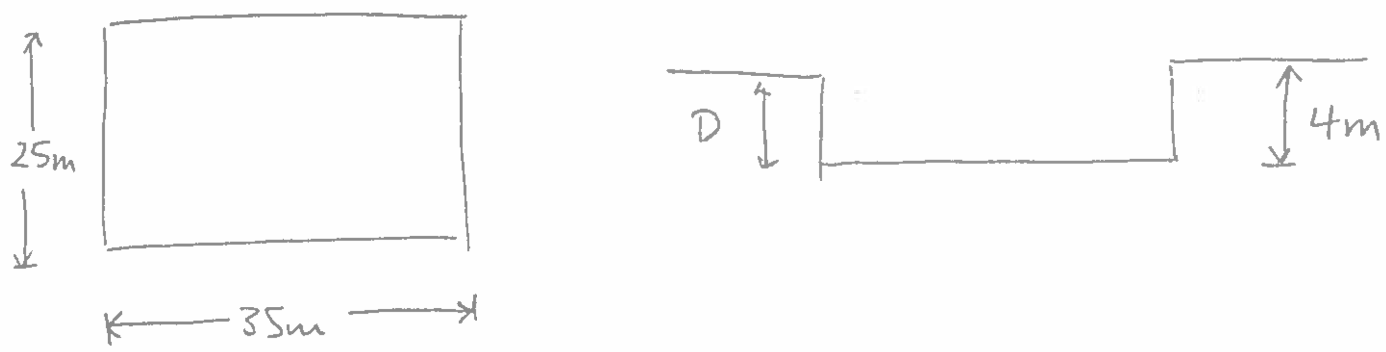
$$x = \underline{\underline{15.683\ \text{m}}}$$

$$P = \frac{720 \times 5}{2} + 720 \times 10.683 + 45 \times 10.683^2 - 90 \times 18.683 \times 4.37 - 45 \times 4.37^2$$

$$= \underline{\underline{6{,}355\ \text{kN}}}$$

Q2 Steel pile design 19 attempts, Average mark 13.7/20 (%), Maximum 18, Minimum 10. Almost all students could design the pile using the standard method in part a, though many used thin-walled Mp calculations despite the thick-walled answer that results in part ii without comment. In part b most students realised that a first-principles calculation was required but with varying success. The main error was in translating rather than rotating the pile and thus not having the opposing loads near the pile base.

PROBLEM (3)



$q_{build} = 200\,kPa$

(a)

$q_{net} = 200\,kPa - \gamma_{clay} D = 200 - \left(20\frac{kN}{m^3}\right)(4\,m) = 120\,kPa$

(b) Assume foundation is fully flexible.

From data book $\quad w_c = \frac{(1-\nu)}{G}\,\frac{q_b B}{2}\,I_{rect}$

$\frac{L}{B} = \frac{35}{25} = 1.4 \quad \Rightarrow \quad I_{rect} = 0.658$

$$w_c = \frac{(1-0.5)}{10\,MPa}\,\frac{(120\,kPa)(25\,m)}{2}\,(0.658) = 0.05\,m$$

The settlement at the centre can be calculated from superposition of 4 smaller foundations of size $\frac{25}{2}m \times \frac{35}{2}m \quad \Rightarrow L/B = 1.4$

$I_{rect} = 0.658$

$$w_{centre} = 4\,w_c = \frac{(1-0.5)}{10\,MPa}\,\frac{(120\,kPa)(25m/2)}{2}\,(0.658) = 0.1\,m$$

Deflection ratio $\frac{\Delta}{L} = \frac{0.1 - 0.05}{\frac{1}{2}\sqrt{25^2+35^2}} = 2.3 \times 10^{-3}$

This deflection ratio may cause slight damage to stiff masonry infills.

(c) Settlement for rigid rectangle from data book

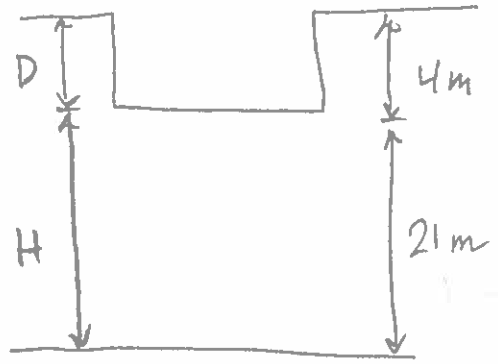
$$w_r = \frac{(1-\nu)}{G} \frac{q_{avg}\sqrt{BL}}{2} I_{rgd}$$

$I_{rgd} \approx 0.89$

$$w_r = \frac{(1-0.5)}{10\,MPa} \frac{(120\,kPa)\sqrt{(25m)(35m)}}{2} (0.89) = 0.08\,m$$

An actual foundation is generally between the two limiting conditions of full flexibility and full rigidity. In this case, a rather large mat foundation at depth, the situation is more likely to be closer to the rigid result.

(d)



From data book:

$$w_{avg} = \mu_0 \mu_1 \frac{qB}{E}$$

$E = 2G(1+\nu) = (2)(10MPa)(1+0.5) =$
$= 30\,MPa$

$D/B = \frac{4m}{25m} = 0.16 \Rightarrow \mu_0 = 1.0$

$\frac{H}{B} = \frac{21m}{25m} = 0.84 \quad \Rightarrow \mu_1 = 0.3$ 3/

$L/B = \frac{35m}{25m} = 1.4$

$$W_{avg} = (1.0)(0.3)\frac{(120kPa)(25m)}{30MPa} = 0.03\ m$$

(c) The stiffness estimated from a triaxial test is likely to be smaller than the actual stiffness in the field. In addition, stiffness is proportional to confining stress and the building pressure will increase stress under the foundation

Q3 Settlement of a raft

15 attempts, Average mark 12.5/20 (62.7%), Maximum 16, Minimum 7. Most students could calculate the various estimates of settlements for a raft foundation, with a few using the incorrect dimensions when applying superposition of elastic solutions. There was some confusion on net pressure and very few could show clear understanding of issues in measuring stiffness.

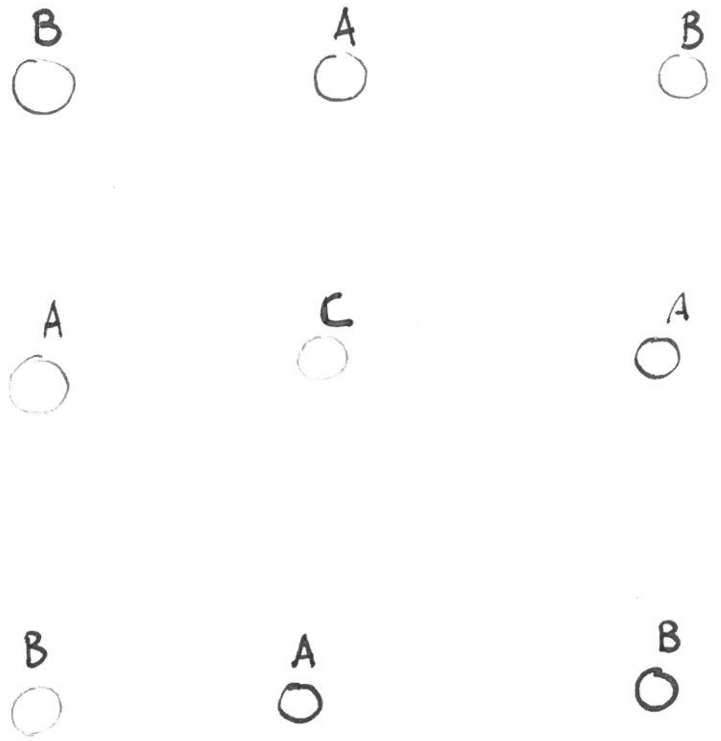
~~2~~ 4

a) i) Pile base is much less stiff than shaft, especially for bored piles in which base is not pre-loaded and may contain disturbed material.

ii) $L = 15m$ $D = 0.3m$

$$\frac{V}{w_{head}} = \frac{2\pi L G_{avg}}{\zeta} = \frac{2\pi \times 15 \times 20 \times 10^6}{4}$$

$$= \underline{\underline{471 \text{ MN/m}}}$$



Pile	5 D	$5\sqrt{2}$ D	10 D	$5\sqrt{5}$ D	$10\sqrt{2}$ D
$\ln \frac{r_m}{r} \div 4$	1.697	1.35	1	0.893	0.658
Settlement 1	0.424	0.3375	0.25	0.223	0.1645

Settlements of individual piles ω_A, ω_B, ω_c

A: $$\omega = \underbrace{(1 + 0.675 + 0.25)}_{1.925}\,\omega_A + \overbrace{(0.848 + 0.446)}^{1.294}\,\omega_B + 0.424\,\omega_c$$

B: $$\omega = \underbrace{(0.828 + 0.446)}_{1.294}\,\omega_A + \underbrace{(1 + 0.5 + 0.1645)}_{1.6645}\,\omega_B + 0.3375\,\omega_c$$

C: $$\omega = (\overset{1.696}{\cancel{0.828 + 0.446}})\,\omega_A + 1.35\,\omega_B + \omega_c$$

$$\begin{bmatrix} \omega_A \\ \omega_B \\ \omega_c \end{bmatrix} = \begin{matrix} 0.233\,\omega \\ 0.409\,\omega \\ 0.053\,\omega \end{matrix}$$

Elastic behaviour loads in proportion to settlements

P_A $4 \times 0.233 + 4 \times 0.409 + 0.053 = 2.5733$

$$P_A = \frac{0.233}{2.573} P = 0.09 P$$

$$P_B = 0.159 P$$

$$P_C = 0.021 P$$

Pile A takes 0.09 of total load and settles by $\frac{\omega_A}{0.233}$

Settlement increases by $\frac{0.09}{0.233} = 0.386$

compared to single pile carrying entire load

So compared to individual pile carrying 1/9 of load

settlement increases by 3.47 ×

efficiency = $\underline{28.7\%}$

stiffness $= \frac{471}{0.386} = \underline{1220}$ MN/m

Q4 Concrete pile group design

14 attempts, Average mark 13.1/20 (%), Maximum 20, Minimum 5.

Most students correctly answered part a and made pleasingly competent attempts at part b which was a difficult question. The major error in part b was in assuming that the piles all had the same settlement if unaffected by their neighbours, which is incorrect owing to the non-uniform distribution of pile loads. I was pleased to see a couple of answers receiving full marks for this question as it is a very complex problem.