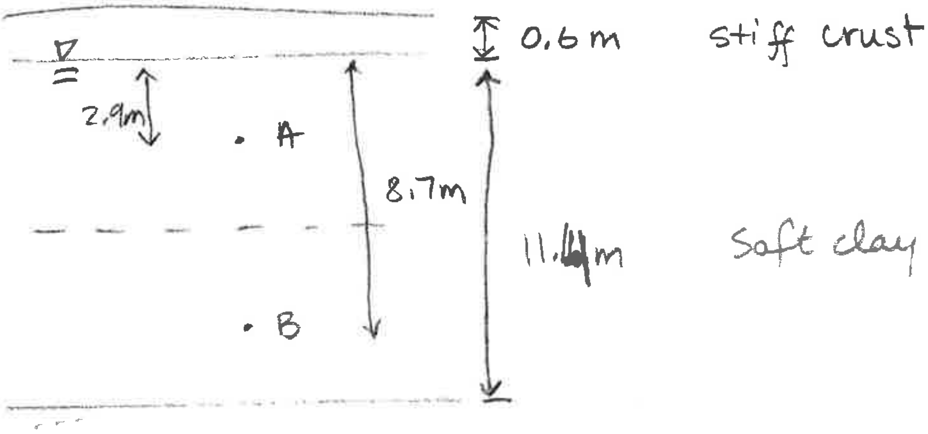
QUESTION 1 20 Points



Side footings

$V = 300kN$
$B = 1.5m$
$L = 2.5m$

$$q_{avg} = \frac{300kN}{(1.5m)(2.5m)} = 80kPa$$

(a) (i) Use elastic solution for finite layer $w_{avg} = \mu_0 \mu_1 \frac{qB}{E}$ (μ_0 = 1.0)
2pts (2pts)

$E_{sand} = 2G(1+\nu) = 2(6MPa)(1+0.3) = 15.6MPa$

Layer	E (MPa)	H/B	μ_1	w_{below} (mm)	H/B	μ_1	w_{above}
1	15.6	0.6/1.5 = 0.4	0.1	0.77			
2	12	12/1.5 = 8	0.7	7	0.4	0.1	1.0

$w_{avg} = 0.77 + 7.0 - 1.0 = 6.77$ mm

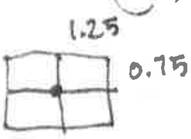
(ii) Assume $\gamma_{sand} = 19 kN/m^3$ $\gamma_{clay} = 17 kN/m^3$
(6points)

$\sigma'_{vA} = (0.6)(19kN/m^3) + (2.85m)(17-9.8) = 32.4 kPa$ (1)

$\sigma'_{vB} = (0.6)(19) + (8.6)(17-9.8) = 73.3 kPa$

Assume bearing pressure is applied at the top of the clay layer

@ A $z = 2.85m$ $m = \frac{1.25}{2.85} = 0.44$, $n = \frac{0.75}{2.85} = 0.26$ } from chart (page 9) $I_r = 0.045$ (2)



@ B $z = 8.7$

$m = \frac{1.25}{8.55} = \cancel{0.44}\ 0.15$

$n = \frac{1.507.5}{8.55} = \cancel{0.17}\ 0.09$

$I_r = \cancel{0.35}\ 0.085$

$\Delta\sigma_A = (0.045)(4)(80\,kPa) = \cancel{37}\ 14.4\ kPa$

$\Delta\sigma_B = (0.085)(4)(80\,kPa) = 1.6\ kPa$

0.6

$V_0 = 1 + (0.39)(2.7) = 2.053$

$\Delta V_A = \lambda \ln\left(\frac{\sigma'_{vA} + \Delta\sigma_A}{\sigma'_{vA}}\right) = (0.41)\ln\left(\frac{37.4 + \cancel{37}\ 14.4}{37.4}\right) = \cancel{0.312}\ 0.154$

1

$\Delta V_B = (0.41)\ln\left(\frac{743 + 1.6}{743}\right) = \cancel{0.062}\ 0.009$

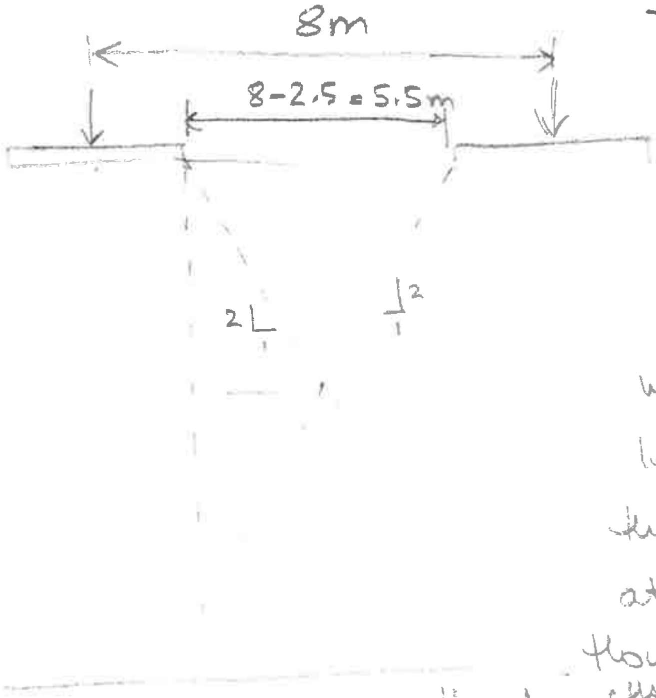
$\rho_A = \frac{\Delta V_A}{V_0}(5.8\,m) = \frac{0.154}{2.053}(5.8) = 0.43\,m$

1

$\rho_B = \frac{\Delta V_B}{V_0}(5.8\,m) = \frac{0.062}{2.053}(5.8) = 0.025\,m$

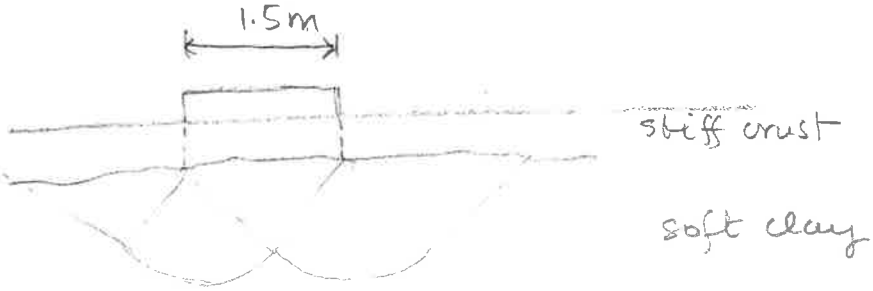
$\rho_{TOT} = 0.455\,m$

(b)
(3 points)



Not very worried about load interaction at depth. With a rough estimate of load spreading at a 2:1 slope the interaction would start at 5.5 m between two footings however interaction below the edge of the foundation would occur at 11 m depth

(c) (i)
(2pts)



The stiff crust would likely be punctured as the clay settles. Therefore, we could assume that the failure occurs through the clay and the stiff crust would act as an overburden

(ii)
(3 points) $q_{bf} = s_c N_c S_u + \gamma_{sand} h = (1.11)(5.14)(30\,kPa) + (19\,\frac{kN}{m^3})(0.6) =$

$= 182\ kPa$

$V_{ult} = (182)(2.5)(1.5) = 682\,kN$

(d) (i)
(2pts) We can size the corner footings in such a way that the bearing pressure would be similar to the bearing pressure of the other footings. The size of the footing will affect somewhat the increase in stress at depth from the elastic solutions, but the effect will be small.

(ii)
(2pts) The drained settlement of the clay is rather large and may easily result in differential settlements which will exceed allowable values.

QUESTION 2 (20 points)

(a) $q_{bf} = (1.18)(5.14)(7\,kPa) = 42\,kPa$ (-2) s_c missing

(3 points)

(b) $V_{ult} = (42\,kPa)(15\,m)^2 = 9.5\,MN$

(4 points)

$V/V_{ult} = 6.5/9.5 = 0.68 > 0.5$ (-1) for skipping this step

$H/H_{ult} = 1 - \left(2\frac{V}{V_{ult}} - 1\right)^2 = 1 - \left[(2)(0.68) - 1\right]^2 = 0.87$

$H_{ult} = A\,S_u = (15\,m)^2(7\,kPa) = 1.58\,MN$ (-1.5) for using B instead of A

$H = (0.87)(1.58\,MN) = 1.37\,MN$

(c)

(i)

(3 points)

When the horizontal load is applied above the plane of the foundation it generates a moment

When a moment is added to the vertical and horizontal loads, then the capacity for these decreases.

(-1.5) for not saying explicitly that bearing capacities (~~[illegible]~~ V_{ult}) are reduced

(-0.5) for not ~~[illegible]~~ mentioning H_{ult}

(-0.5) Meyerhof (reduced width)

(-1) No moment mentioned

(ii) $H = 0.8\,MN$

(6 points) $h = 5\,m$

$$e = \frac{(0.8\,MN)(5\,m)}{(6.5\,MN)} = 0.615\,m \quad 0.5$$

$$B' = B - 2e = 15 - (2)(0.615) = 13.77\,m \quad 1$$

We need to recalculate ultimate capacities based on the reduced width:

$$S_c = 1 + 0.18\,\frac{B}{L} = 1 + 0.18\,\frac{13.77}{15} = 1.165 \quad 0.5$$

$$q_f = (1.165)(5.14)(7\,kPa) = 41.9\,kPa$$

$$V_{ult} = BB'q_f = (15)(13.77)(41.9\,kPa) = 8.65\,MN \quad 1$$

$$V/V_{ult} = \frac{6.5}{8.65} = 0.75 > 0.5$$

0.5   1

Vertical capacity still ok ✓

$$H_{ult} = BB'S_u = (15)(13.77)(7\,kPa) = 1.44\,MN \quad 1$$

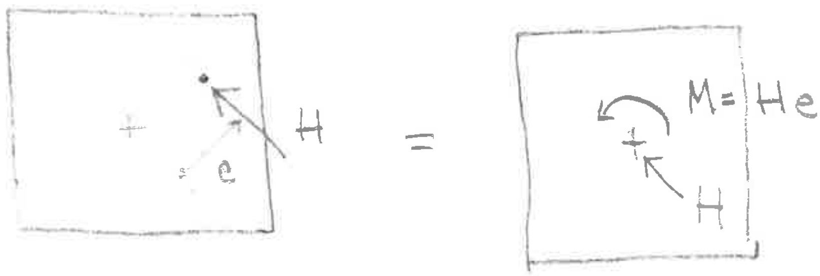
$$\frac{H}{H_{ult}} = 1 - \left(2\frac{V}{V_{ult}} - 1\right)^2 = 1 - \left[(2)(0.75) - 1\right]^2 = 0.75$$

$$H = (0.75)(1.44) = 1.08\,MN > 0.8\,MN \quad 0.5$$

$$FS_H = \frac{1.08}{0.8} = 1.35$$

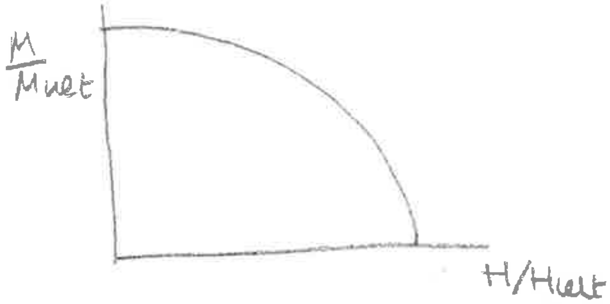
Horizontal capacity still ok ✓

(d)
(4 points)



We would have combined translation and rotation around the vertical axis.

Therefore we would need to find Hnet and Mnet and then the interaction



3 a) Shallow foundations are generally cheaper than deep foundations + don't require specialist equipment to construct

Deep foundations cause more noise + vibration during construction

Deep foundations are preferred where there are soft surficial soils relative to working loads + allowable settlements and where scour is a concern.

Deep foundations also tend to be more efficient at carrying H & M loads.

b) i) $V = A s_c d_c N_c s_u$

$$A = \frac{10{,}000}{50 \times 5.14 \times 1.2} = 32.4 \text{ m}^2$$

side length = 5.7m

$$\omega = \frac{1-\nu}{G} \frac{V}{A} \frac{B}{2} \times 0.9$$

$$= \frac{0.5}{4000} \times \frac{4000}{32.4} \times \frac{5.7}{2} \times 0.9 = \underline{\underline{40 \text{ mm}}}$$

ii) $V = 2.5\ \text{MN}$ per pile

$$V_b = 9 s_u A$$

$$= 88.4\ \text{kN}$$

$$V_s = 2412\ \text{kN}$$

$$\sigma'_{vo} = s_u \ @ \ \frac{50}{6} = 8.5\text{m} \ \text{depth.}$$

$$V_s = \pi \times 0.5 \times \frac{50}{2} \left[\int_0^{8.5} \left(\frac{6z}{50}\right)^{1/4} dz + \int_{8.5}^{L} \left(\frac{6z}{50}\right)^{1/2} dz \right]$$

$$= 39.27 \left[\underbrace{\left[\frac{4 \times 6^{1/4} z^{5/4}}{5 \times 50^{1/4}} \right]_0^{8.5}} + \left[\frac{2}{3} \frac{6^{1/2} z^{3/2}}{50^{1/2}} \right]_{8.5}^{L} \right]$$

$$\cancel{L^{3/2} - 8.5^{3/2} = 121.24}$$

$$L = \cancel{27m} \quad \underline{\underline{40.9\text{m}}}$$

$$\eta = 1 \qquad \frac{L}{D} = \cancel{54.4}\ 81.8 \qquad \rho = 1 \qquad \xi = 1 \qquad \zeta \simeq 4$$

$$\omega = \frac{V}{\cancel{88.8}\ 132.5\ DG} = \frac{1000}{\cancel{88.8}\ 132.5 \times 0.5 \times 4000} = \underline{\underline{\cancel{5.5}\ 3.7\ \text{mm}}}$$

iii) 6×6 raft $A = 36 m^2$

$$\omega = \frac{0.5}{4600} \times \frac{V}{36} \times \frac{6}{2} \times 0.9 = 9.375 \times 10^{-6} V_r \text{ m}$$

for piles $\omega = \frac{\overset{3.7}{\cancel{5.5}} \times 10^{-3} V_p}{1000}$ m per pile

Assume ~~A~~ ω is same for raft & piles

$$\frac{\overset{3.7}{\cancel{5.5}} \times 10^{-3} V_p}{1000} = 9.375 \times 10^{-6} V_r$$

$$\frac{V_p}{V_r} = \frac{9.375}{\underset{3.7}{\cancel{5.5}}} = \cancel{1.7}\ 2.53$$

4 piles $4V_p + V_r = 4000$ kN

$$11.12\ \cancel{7.8}\ V_p = 4000$$

$$V_p = \overset{360}{\cancel{513}} \text{ kN}$$

$$\omega = \underline{\underline{\overset{1.3}{\cancel{2.8}} \text{ mm}}}$$

This assumes no interaction between mechanisms for raft + pile. Downdrag on soil due to piles <u>will</u> influence raft settlement over a radius of ~ 4D (2m). May have a substantial effect on raft stiffness making this an underestimate of settlement.

4. Displacement piles

Precast concrete or steel piles are hammered, ~~or~~ jacked or vibrated into the ground. This results in locked in stresses in the ground giving a high pile stiffness. The process, however tends to cause lots of noise + vibration (except jacking) and is hence often unacceptable in built-up areas.

Non-displacement piles.

A hole is drilled into the ground using an auger with the hole being either unsupported (in clays) or supported by a casing or drilling mud in sands. A reinforcing cage is dropped into the hole & concrete added from the bottom displacing the drilling mud. This process often leaves a soft toe due to spoil in the hol and relieves in-situ horizontal stresses leading to softer pile response. It is, however much less intrusive on the built environment due to low noise + vibration.

b) i) $H = 1\ MN/pile$

$n = 9 \times 5 = 45\ kPa/m$

restrained at pile cap

Short pile => $\frac{H}{nD^3} = 22.2$ $\quad \frac{L}{D} > 6.5$

$L > 6.5m$

if $\frac{M_p}{nD^4} > 100 \rightarrow M_p > 4500$ kNm

Long pile $\frac{M_p}{nD^4} = \cancel{50}\ 45$

$M_p > \cancel{2025}\ 2025\ kNm^2$

not limiting

ii) Piles must also be designed to carry the vertical loads and loads due to overturning moments.

c) i) $\frac{e}{D} = \frac{50}{4} = 12.5$

$$\frac{H}{nD^3} = \frac{4000}{45 \times 64} = 1.39$$

short pile $\frac{L}{D} \sim 7$ $\quad L > \underline{28\,m}$

long pile $\frac{M_p}{nD^4} \sim 20$ $\quad M_p > 230\ MN\,m$

ii) Monopile must also carry vertical loads. Scour may erode surface sediments, so extra length may be required as height of horizontal load above mudline is increased & effective length is reduced.