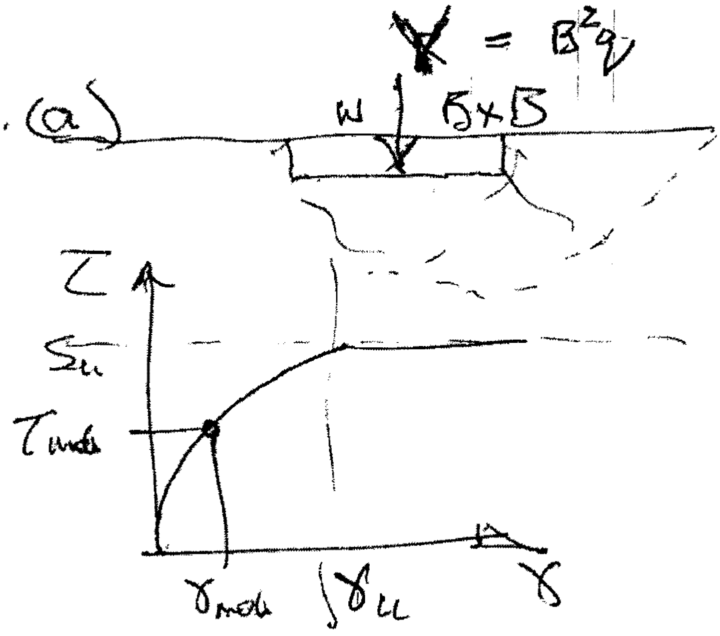
1. (a)



From plasticity theory, $\tau_{mob} = q/N_c$

From MSD, $\gamma_{mob} = 1.35 \frac{w}{B}$

Using a parabolic stress-strain curve:

$$\frac{\tau_{mob}}{s_u} = \left(\frac{\gamma_{mob}}{\gamma_u}\right)^{0.5}$$

$$\therefore \frac{1.35 w}{\gamma_u B} = \left(\frac{q}{N_c s_u}\right)^2$$

$$\therefore w = \frac{0.74 \gamma_u B}{F^2}$$

where $F = \frac{N_c s_u}{q}$

This is approximate because:

(i) the Osman mechanism is approximate

(ii) stress concentration under the edge of the footing cause w/B to exceed the prediction as $F \to 1$

1 (b) (i) Since $w \propto B . \dfrac{V^2}{B^4} \propto \dfrac{V^2}{B^3}$

it follows that if w is to be constant

$$B \propto V^{2/3}$$

So $\dfrac{B_{outer}}{B_{inner}} = 2^{2/3} = 1.59$

(ii) If s_u varies from 60 kPa to 80 kPa from footing to footing, w will vary accordingly.

$$w = 0.74\, \gamma_u B \left(\frac{q}{N_c s_u}\right)^2$$

And $q = \dfrac{V}{B^2}$, $N_c = 5.9$, $\gamma_u = 4\times 1.5 \times 2\% = 4\times 3\%$

So for outer footings with $V = 600$ kN:

$$w_{outer} = 0.74 \times 4\times 3 \times 10^{-2} B_{outer} \left(\frac{600}{B_{outer}^2\, 5.9\, s_u}\right)^2$$

$$\therefore\ w_{outer} = \frac{230 \times 4}{s_u^2 B_{outer}^3}$$

So on a softer spot with $s_u = 60$ kPa

$$w_{outer}^{+} = \frac{0.064}{B_{outer}^3} \times 4$$

And on a stiffer spot with $s_u = 80$ kPa

$$w_{outer}^{-} = \frac{0.036}{B_{outer}^3} \times 4$$

So the maximum relative settlement

$$\Delta = w^{+} - w^{-} = \frac{0.028}{B_{outer}^3} \times 4$$

1 (b) (ii) cont.

Then if $\frac{\Delta}{L} = 1.5 \times 10^{-3}$ with $L = 10$m

$\Delta = 15 \times 10^{-3}$ m

$\therefore B^3_{outer} = \frac{0.028 \times 4}{15 \times 10^{-3}} = 1.87 \times 4 = 7.48$

So $B_{outer} = \cancel{1.23\text{m}}$ 1.95 m

$\therefore$ $\underline{B_{inner} = \cancel{0.77\text{m}}\ 1.23\text{ m}}$

Note ~~that~~ $q_{inner} = \frac{300}{\cancel{0.77}^2}$ (1.23) $= \cancel{500}$ 198 kPa

whereas $q_{ult} = 5.9 \times 60$ if soft

$= 354$ kPa: so OK

(iii) Treating the soil as quasi-elastic

$w \propto (1 - \nu) B \frac{q}{G}$

so $\frac{w_{drained}}{w_{undrained}} = \frac{(1 - 0.25)}{(1 - 0.5)} = 1.5$

Then Δ would increase by 1.5, so

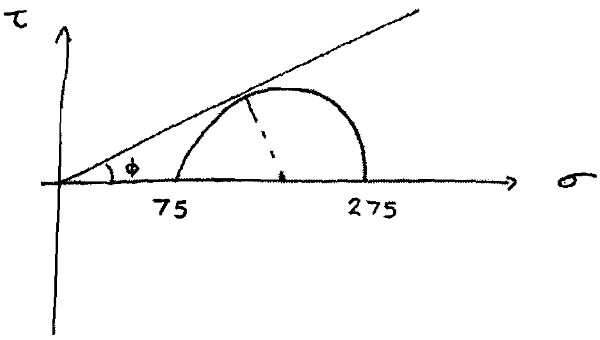
B values should increase by $1.5^{1/3}$ or 1.14

So $B_{outer} \rightarrow 2.22$ m

$B_{inner} \rightarrow 1.40$ m

Examiner's comment: Candidates in general managed to calculate the settlement formula in part a, but struggled to understand its implications in part b.

2. a) $q_u = 200\,kPa \rightarrow S_u = 100\,kPa$



$\sin\phi = \frac{100}{175} \qquad \phi = 35°$

b) $V = 400 \cos 20 = 376 \ kN/m$

$H = 400 \sin 20 = 167 \ kN/m$

$V_{ult} = (2+\pi)\,B\,S_u = 514\,B \ kN/m$

$H_{ult} = B\,S_u = 100\,B \ kN/m$

p.6 of databook:

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

$$\frac{1.37}{B} = \frac{2.92}{B} - \frac{2.14}{B^2}$$

$B = 1.38\,m$ with no safety factor.

Reduce V_{ult} & H_{ult} by factor 2 $\rightarrow$ double B

$\Rightarrow B = 2.75\,m$

Check

$\frac{V}{V_{ult}} > 0.5$

c) Surcharge $\sigma'_{vo} = 20$ kPa

$\gamma' = 10$ kPa

$\phi = 35^\circ \Rightarrow N_q = 33 \quad N_\gamma = 45$

$$V_{ult} = 20 \times 33B + \frac{1}{2} \times 10 \times 45B^2$$

$$= 660B + 225B^2 \quad \text{kN/m}$$

If $B = 2.75$ m as before

$$V_{ult} = 3516 \text{ kN/m}$$

$$\frac{V}{V_{ult}} = 0.1 < 0.5$$

So potential failure is in slip

$$\frac{H}{V} = \tan 20 < \tan 35$$

so OK.

Examiner's comments: Most candidates managed to calculate cu and phi, though some did this in very roundabout ways. Several candidates produced excellent solutions for parts b and c, though some tried to derive results from stress fans with varying success, rather than using interaction formulae from the databook.

3.

a) First a hole is excavated using an auger. Spoil must be excavated from the hole either using a continuous flight auger to bring the soil to the surface or by bringing a short auger to the surface and spinning the spoil off. During the process of excavation the hole must be kept open as in sand it would not be stable. This is achieved using a heavy drilling mud such as bentonite that has a unit weight equal to that of the soil. Following excavation, a reinforcing cage is placed in the hole and concrete is added from the base up, displacing the drilling mud. The concrete then sets to form the pile.

b) Initially the stresses around a pile are geostatic, with horizontal stresses typically being a fraction of vertical stresses. When the hole is excavated, the lateral stresses exerted by the drilling mud are equal to the vertical stresses, resulting in an increased horizontal stress. Sands drain quickly, so this horizontal stress increase will rapidly result in increased effective stresses and slight compression of the sand. Once the concrete is poured and sets within the hole, these stresses are locked in place. The lateral earth pressure coefficient for design is hence often taken as the average of K0 and unity. At the base of the pile, the soil is not compressed during pile installation and loose spoil may still be present. This results in a very soft toe compared to a driven pile. Base capacity is often hence taken as being zero for a bored pile.

c) The limit on maximum allowable shaft friction implies that pile shaft capacity is not quadratic with increasing length. This is as a result of friction fatigue during pile installation in which cyclic movement of the pile relative to the soil results in densification of soil and hence a relaxation of lateral stresses and a fall in skin friction. Longer piles will have a greater tendency for this to have occurred during installation. As the API code relates skin friction to initial vertical stresses, this fall in shaft friction is taken account of indirectly by limiting the value of the maximum shaft friction.

d) Pile plugging occurs due to arching of soil within a tubular pile which allows very high stresses to be carried on the base of a soil plug. As the skin friction on the inner surface of the pile for a frictional material is related to the vertical stress, this gives an exponential increase in vertical stress inside the pile. The stresses exerted on the pile base may be insufficient to case the plug to be forced up the tube. The pile then fails as effectively a closed-base pile. In design a comparison must be made between the failure loads in plugged and unplugged manners, the lower of which will dominate. This implies that the inner shaft friction must be greater than the base capacity minus the plug weight for plugged failure to occur.

4. i) $M_p = D^2 t \sigma_y$

$= 320$ MNm

ii) $n = 9 \times 1.5 = 13.5$

$\frac{e}{D} = 0$ $\frac{L}{D} = 7.5$ $\frac{H_{ult}}{nD^3} \sim 7$ (short pile)

$H_{ult} \sim \underline{\underline{6 \text{ MN}}}$

$\frac{M_p}{nD^4} = 92.6$ $\frac{H_{ult}}{nD^3} \sim 23$ (long pile)

– doesn't limit

$\frac{e}{D} = 4$ $\frac{H_{ult}}{nD^3} \simeq 4$

$H_{ult} = \underline{\underline{3.5 \text{ MN}}}$

iii) $\frac{\sigma'_{vo}}{s_u} = 4 \Rightarrow \alpha = 1$

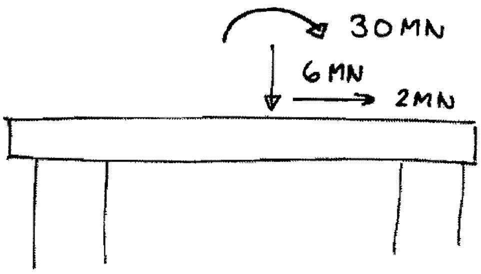
$V_{max} = 2 \left[\pi D L \alpha s_{u_{avg}} \right]$

$= 2 \left[\pi \times 4 \times 30 \times 1 \times (15 \times 1.5) \right]$

$= 17$ MN

b)

b) i)



Worst case on diagonal - pile spacing = $8\sqrt{2}$.

$$V_{max} = 4.16 \text{ MN} = \frac{6}{4} + \frac{30}{8\sqrt{2}}$$

Ignoring base capacity

$$2\pi D L \frac{k_{su} L}{2} = 4.16 \text{ MN}$$

$$\Rightarrow \quad L = \underline{\underline{29.7 \text{m}}}$$

ii) $\frac{L}{D} = 33.5$ $\quad H = 500 \text{ kN}$

$M_p = 10 \text{ MNm}$

$$\frac{M_p}{n D^4} = 741 \qquad \frac{H_{ult}}{n D^3} \sim 150$$

$H_{ult} \sim 2 \text{MN}$

Piles are OK

Examiner's comment:

This was a very popular question, being answered by all candidates. It was well answered by all candidates, though the additional vertical loads due to overturning moments were missed by some.