Module 4D5
Foundation Engineering

May 9th 2011 9am

Crib

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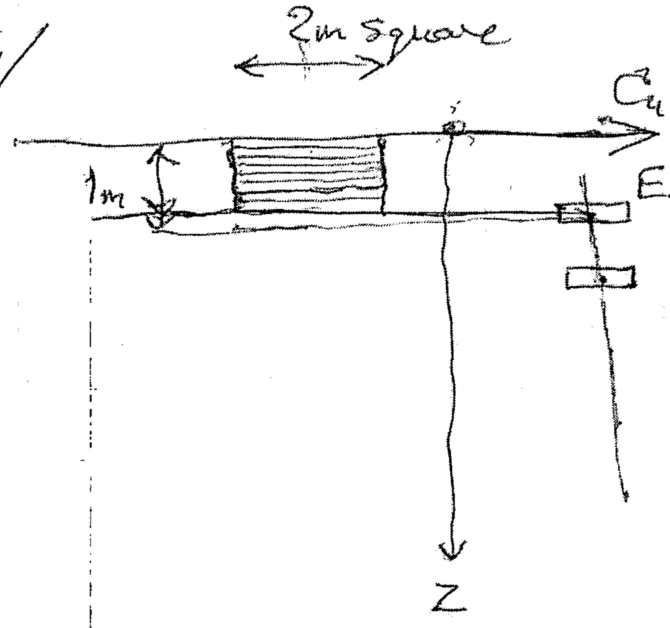


$C_u = 80 + 10Z \pm 20$ kPa

$E_o = 10000 \pm 2000$ kPa at $Z = 1$m

a) $\sigma_{v,net} = 800/4 = 200$ kPa

$\sigma_{v,ult,net} = S_c d_c N_c S_u = 1.18 \times 1 \times 5.14 \times 80$
$= 6.05 \times 80$
$= 484$ kPa

ignoring the increase of C_u with depth.

According to Osman $C_{u,effective} \simeq 80 + 0.3 \times 2 \times 10 \simeq 86$ kPa

in which case $\sigma_{ult} \simeq 522$ kPa

Then $F \doteq 522/200 \simeq 2.6$

Using Osman: $\gamma_{average} = 1.3 \dfrac{w_a}{D} = \dfrac{\gamma_u}{F^2}$

$\therefore \quad w_a = \dfrac{2m}{1.3} \times \dfrac{0.04}{6.8} = 9.0$mm

This was the "best estimate".

Range: $\sigma_{ult} = 400$ to 643 kPa
$F = 2$ to 3.2
$w_u = 15.4$mm to 6.0mm

4 b) $\sigma_v = q\left[1 - \frac{1}{(1+(a/z)^2)^{3/2}}\right]$

$\pi a^2 = 4$
$\therefore a = 1.13\,m$

Z below base	σ_v/q
0.5	0.934
1.5	0.490
2.5	0.243
3.5	0.138
4.5	0.088

Then if $E_o = 10000$ kPa

$$W_{oed} = \frac{200}{10000}\left[0.934 + 0.490 + 0.243 + 0.138 + 0.088 \text{ etc}\right]$$

$$\simeq \frac{200}{10000} \times (1.89 + \ldots)$$

$\simeq$ **38 mm** ranging **32 mm to 47 mm**

c) $$W_{punch} = \frac{\pi}{2}\frac{(1-\nu)}{G}\frac{200}{2}\sqrt{4} \times 0.9$$

$$= 180\,\frac{(1-\nu)}{G}$$

Now at $F = 2.5$, $C_{mob} = \frac{85 \times (0.38)}{[illegible]}$
$= 33$ kPa

and $\gamma_{mob} \simeq \frac{0.04}{2.5^2} = 6.4 \times 10^{-3}$

So $G \simeq 5{,}000$ kPa

and $\nu_{undrained} = 0.5 \rightarrow w_u = 18$ mm

$\nu_{drained} = 0.2 \rightarrow w_d = 28$ mm

1 d) a: Assumed parabola – we need data from DSS test (e.g.) to confirm.

b: Used linear elasticity to provide stress distribution for a non-linear material.

c: Assumed Poisson's ratio – we need data from triaxial test (e.g.) to confirm.

Take (a) as best estimate of immediate, undrained settlement. Then differential settlement should not exceed $15.4 - 6.00 = \underline{8.6 \text{ mm}}$

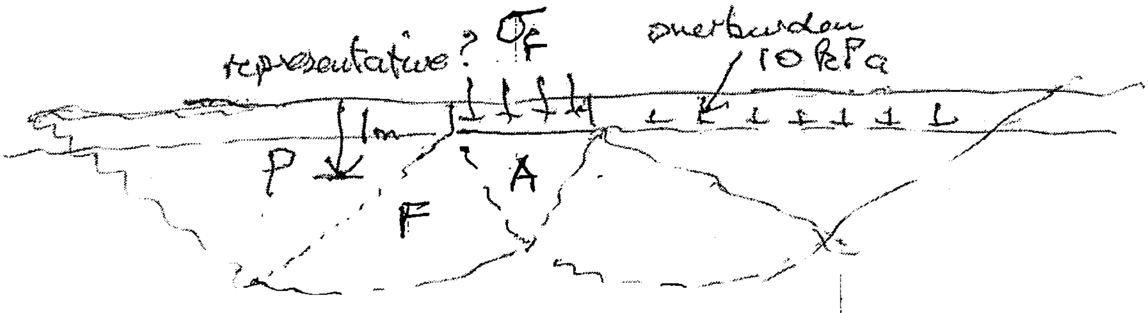
Take (c) with $\nu' = 0.2$ as best estimate of long-term settlement. Its typical value is 26 mm, with a range 44 mm to 17 mm, so differential settlement should not exceed 27 mm, which may be just-tolerable.

[Note that $w_{(b)} \simeq w_{(a)} + w_{(c)}$]

This question was attempted by most candidates, with varying success. Many candidates were obviously running out of time and only produced minimal attempts, hence the low average mark. Most candidates could calculate the bearing capacity of the foundation and estimate elastic settlements, but differences between undrained and drained analysis, use of parabolic stress-strain curves etc were in general not well answered in the time available.

2. a) In axisymmetry $\phi_{max} = \phi_{crit} + 3 I_R$

where $I_R = I_D \ln \frac{20000}{p'} - 1$ for quartz sand



Assume a representative depth of 1m at which

in Zone A, $\sigma'_{v,a}$ increases from 20 to 320 kPa.

in Zone P, $\sigma'_{v,p}$ remains at 20 kPa.

Estimate the average $p' \simeq \sqrt{20 \times 320}$ in

a working state, ie $p' \simeq 80$ kPa

If the load factor to collapse happened to be 3, the effective stresses at failure would range from 20 kPa to 920 kPa with $p'_f \simeq 136$ kPa.

Then $I_R = 0.8 \ln \frac{20000}{136} - 1 = 3$

So $\phi_{max} = 43°$

Then $s_q = 1 + \sin\phi = 1.68$; $s_\gamma = 0.7$

$N_q = 5.29 \times 18.7 = 99$; $N_\gamma = 2(N_q - 1)\tan\phi = 183$

So $q_{f,net} = 1.68 \times 99 \times 10 + 0.7 \times 183 \times 20 \times 0.5$

$= 1663 + 1281$

$= 2944$ kPa

Load factor to collapse $= 2944/300 = 9.8$ or so.

But we should correct for the much increased p'.

2 a) cont.

Now average $p' \simeq 80\sqrt{9.8} \simeq 250$ kPa

So $\phi_{max} \simeq 34^\circ + 3\times 2.5^\circ \simeq 41.5^\circ$

So $s_q \to 1.66$, $N_q \to 4.93 \times 16.1 \simeq 79$

$N_\gamma \to 138$

So $q_{ult} \to 1311 + 966 \to \underline{2277 \text{ kPa}}$

So a better (and conservative) estimate of load factor to collapse is $2277/300 = \underline{7.6}$

Check $\phi_{mob} = 30^\circ$:

$s_q = 1.5$, $N_q = 18.4$, $N_\gamma = 20.1$

So $q_{mob} = 276 + 141 = 417$ kPa

This is marginally above 300 kPa, so ϕ_{mob} is marginally below 30°, $\underline{\text{eg } 28^\circ}$.

b) In its working state, $p'_{average} \simeq 80$ kPa.

So a mean value during loading $\simeq \frac{1}{2}(20+80) \simeq 50$ kPa

This will control $G_0 = \dfrac{5760 \times 100 \times 0.5^{0.5}}{(1.55)^3} = 109$ MPa

Modified hyperbola: $\dfrac{G}{G_0} = \dfrac{1}{\left[1 + \left(\frac{\gamma - \gamma_e}{\gamma_r}\right)^a\right]}$ with $a = U_c^{-0.075}$

So $a = 4^{-0.075} = 0.9$

$\gamma_r = 10^{-2}\left[10^{-2} U_c^{-0.3} \frac{p'}{p_a} + 0.08\, e I_D\right] = 3.85 \times 10^{-4}$

$\gamma_e = 2 \times 10^{-6} + 0.012\, \gamma_r = 6.6 \times 10^{-6}$

So $G = \dfrac{109 \times 10^3}{\left[1 + \left[\frac{\gamma - 6.6\times 10^{-6}}{3.84 \times 10^{-4}}\right]^{0.9}\right]}$ kPa

2 b) cont.

But the loaded foundation mobilises, typically,

$$\tau_{max} = p'_{average} \sin\phi \simeq 80 \sin 28^\circ \simeq 38\text{kPa}$$

$$\text{So } 38 \simeq G\,\gamma_{mob} \simeq \frac{109 \times 10^3\,\gamma}{\left[1 + \left(\frac{\gamma - 6.6\times10^{-6}}{3.84\times10^{-4}}\right)^{0.9}\right]}$$

$$\text{So } 1 + \left(\frac{\gamma - 6.6\times10^{-6}}{3.84\times10^{-4}}\right)^{0.9} \simeq 2.87\times10^3\,\gamma$$

If $0.9 \rightarrow 1$ then $3.84\times10^{-4} + \gamma - 6.6\times10^{-6} \simeq 1.10\gamma$

So $\gamma = 3.77\times10^{-3}$

So $G \simeq \frac{38}{3.8\times10^{-3}}$

$\underline{G \simeq 10000\text{kPa}}$

Dilatancy causes volume to expand as γ rises.

c) With $G = 10000$kPa and $\nu = 0.2$

$$w \simeq \frac{(1-\nu)}{G}\,\frac{q}{2}\,\sqrt{BL}\,I$$

$$\simeq \frac{0.8}{10000} \cdot \frac{300}{2} \cdot 1 \cdot 0.9$$

So $\underline{w \simeq 11 \text{ mm}}$

d) The conventional footing at 300kPa should be safe (load factor 7.6) and serviceable (settlement < 25 mm) for most simple buildings. Extensions may require smaller bearing pressures to reduce cracking at connection. Check for loose spots & consistency of calcs by plate bearing tests

Q2 examiner's comment:

This question was attempted by few candidates, with varying success. Many candidates were obviously running out of time and only produced minimal attempts, hence the low average mark. Part a of the question was in general well-answered, though the rider of allowing for p' increasing beyond working load was in general ignored. In part b, the equation of the hyperbolic stress-strain curve was handled well but the equivalent linear value was not. Part c was in general well answered by those who attempted it, whereas part d was almost entirely unanswered by many candidates, probably due to lack of time.

3. a) i) Static Load testing

- uses dead load to measure short & long-term response of pile.
- requires a large reaction mass (canteledge) equal to pile capacity, so an expensive & slow test to do.
- Can give both long & short term behaviour depending on length of time for which loading is maintained.

ii) Constant Rate of Penetration (CRP) test

Jacks pile into ground at a constant rate.

- quicker than static loading & hence cheaper but still requires reaction mass
- only gives short-term behaviour, especially in clay soils.

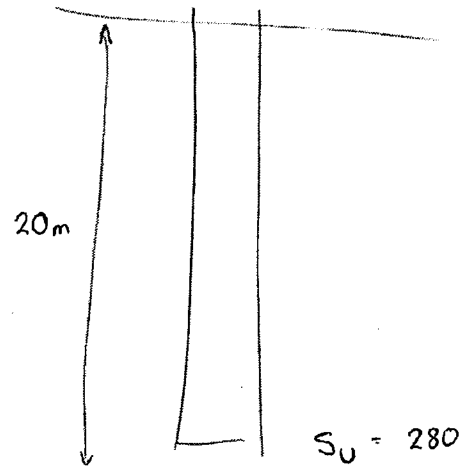
iii) Dynamic Testing

- Top of pile is struck with a hammer & acceleration is measured. "Spring-stiffnesses" at pile base & along shaft are then back-analysed from accel trace.
- Cheap & quick
- only measures small displacement behaviour of pile.

iv) Statnamic testing.

- An explosion is used to drive a counterweight upwards, reacting down on pile, pile force & displacement measured.
- Gives quick test of undrained response
- expensive equipment but can load piles to undrained capacity.
- may overestimate capacity if inertial effects are ignored.

3 b) i)



$S_u = 280\ kPa \qquad \sigma' = 140\ kPa$

$$\alpha = 0.5\left(\frac{7z}{14z}\right)^{1/4} = 0.5^{1.25} = \frac{\tau}{S_u}$$

$$F_{shaft} = \int_0^{20} \pi D\ \alpha\ s_u\ dz$$

$$= \pi \times 0.5 \times 0.5^{1.25} \times 14\left[\frac{z^2}{2}\right]_0^{20}$$

$$= \underline{\underline{1849\ kN}}$$

$$F_{base} = 9\,s_u \times \frac{\pi D^2}{4} = 495\ kN$$

ii) As pile is installed in clay soil around the tip is pushed sideways giving large increases in σ_h, the undrained shearing & total stress increases cause u to rise, by more in soft than OC clays

During each hammer cycle soil around shaft is cyclically loaded due to elastic pile behaviour, this leads to a reduction in σ_h & σ_h' (friction fatigue)

After installation, drainage causes pore pressures to return to hydrostatic values resulting in increasing σ_h', this causes the soil to contract so σ_h tends to fall

During loading +ve pore pressures may again be developed around the shaft.

Q3 Examiner's comment:

This was a very popular question, being answered by all candidates. Parts a and bi were well answered by the majority of candidates but the descriptive part bii proved problematic.

4a) $M_{max} = 64\ MNm$

$$M_{el} = \frac{\sigma \pi (r_{out}^4 - r_{in}^4)}{4 r_{out}}$$

$$r_{in}^4 = r_{out}^4 - \frac{4 r_{out} M}{\sigma \pi}$$

$$= 2^4 - \frac{4 \times 2 \times 64}{200 \pi}$$

$$= 1.974$$

$t = \underline{\underline{26\ mm}}$

b) $\frac{e}{D} = 4$

$$Z_p = \frac{D_{out}^3 - D_{in}^3}{6}$$

$$M_p = 200 \times \left(\frac{4^3 - (3.92)^3}{6} \right)$$

$$= 125.5\ MNm$$

$$\frac{H_{ult}}{S_u D^2} = \frac{4000}{40 \times 16} = \frac{100}{16} = 6.25 \Rightarrow \frac{L}{D} > 5.5$$

Short

$\Rightarrow \underline{\underline{L = 22m}}$

Long. pile $\quad \frac{M_p}{S_u D^3} \gtrsim 25$

$$\frac{M_p}{S_u D^3} = \frac{125 \times 500}{40 \times 4^3} = 49$$

OK

c) 6 MN

$$F_{shaft} = 2 \times \pi D \int_0^L \alpha \, s_u \, dz$$

~~Take~~ $\sigma'_{vo} = s_u$ @ 5m depth

$$F_{shaft} = 2\pi D \int_0^5 40 \times \tfrac{1}{2} \times \left(\frac{8z}{40}\right)^{1/4} dz$$

$$+ \, 2\pi D \int_5^L 40 \times \tfrac{1}{2} \times \left(\frac{8z}{40}\right)^{1/2} dz$$

$$= 40\pi \times 4 \left[\left[\frac{4 z^{5/4}}{5 \times 5^{1/4}}\right]_0^5 + \left[\frac{2 z^{3/2}}{3 \times 5^{1/2}}\right]_5^L \right]$$

$$= 160\pi \left[4 + \frac{2}{3\sqrt{5}} L^{3/2} - \frac{10}{3} \right] = 6000$$

$$L^{3/2} = \frac{3\sqrt{5}}{2} \left[\frac{6000}{160\pi} - 4 + \frac{10}{3} \right] = 37.8$$

$L = \underline{\underline{11.3\text{m}}}$

d) - Gravity base - cheap construction cost but can have poor moment resistance unless very large. Not suitable where surficial soils are very soft, e.g. N.C. marine clays

- Multi-pile or multi-suction caisson systems. Better at carrying moment loads than monopiles. Difficulties connecting piles together offshore due to positional tolerances during driving.

This was a very popular question, being answered by all candidates. In part a almost all candidates estimated plastic capacity rather than elastic when avoiding yield. The remainder of the question was in general answered well.