DR I. Thusyanthan

4D5 Foundation Engineering
Crib

Question 1: Lower bound solution for V-H loading of a rectangular footing

Attempts: 7. Mean: 9.3/15. Max: 15/15. Min: 6.75/15

This question was not popular as only 7 attempted it. Some candidates found it difficult to relate the principle stresses in active, passive regions to Mohr's circles. Most candidates did not do well in obtaining the lower bound solution for the bearing capacity, even through one candidate provided good answer to all parts and got full marks for the question.

Question 2: Undrained and drained settlement of a shallow foundation

Attempts: 26. Mean: 9.9/15. Max: 14.25/15. Min: 6/15

Part (a) of the question, calculation of undrained settlement, was well attempted by most of the candidates. However, some candidates found it difficult to use superposition of settlements from appropriate rectangular areas to make up the overall loading. The second part of the question, calculation of drained settlement, was well done by candidates who used a tabular system to calculate the increase in effective stress using Fadum's chart in databook.

Question 3: Axial loading of piles in sand and clay

Attempts:26. Mean: 10.8/15. Max: 15/15. Min: 6/15

Overall this was the best done question in the paper. The first part of the question, calculation of vertical pile capacity in sand, was well done by all the candidates: Few candidates found it difficult to form the correct quadratic equation that was required for second part of the question (required pile length in clay). Description of pile load tests and sketch of typical result was poorly done by many candiates. Very few candidates described the pile load tests well and produced the correct typical graph of pile head load versus pile head settlement.

Question 4: Lateral capacity of pile

Attempts: 25. Mean: 10.1/15. Max: 14.25/15. Min: 5.25/15

Almost all candidates calculated the equivalent axial stiffness and the plastic moment capacity of the pile correctly. Most candidates used the databook charts efficiently to calculate of the pile length required for the axial load and lateral load. Some candidates were unable to calculated the pile settlement under given vertical load. Most candidates stated the commonly used anchors in offshore but nnly few candidates provided a full description as requested in the question.

(Note- Part 4(a)(i) required calculation of plastic moment (bending) capacity, the yield stress of steel was not given in the question as candidates were expected to use any typical value from the databook. During the examination two candidates asked what value to use for yield stress of steel (databook has a range), Hence in order to avoid any confusion among other students, it was announced that 250 MPa can be used as yield stress of steel for question 4(a). Students who used any other typical value for yield stress was given full credit as well. The rest of question 4 was unaffected by this.)

May 2008

4D5 Foundation Engineering 2008

Crib

1/

a/

(i) $\dfrac{V_{uH}}{B} = (2+\pi) S_u + \sigma_{v0}'$

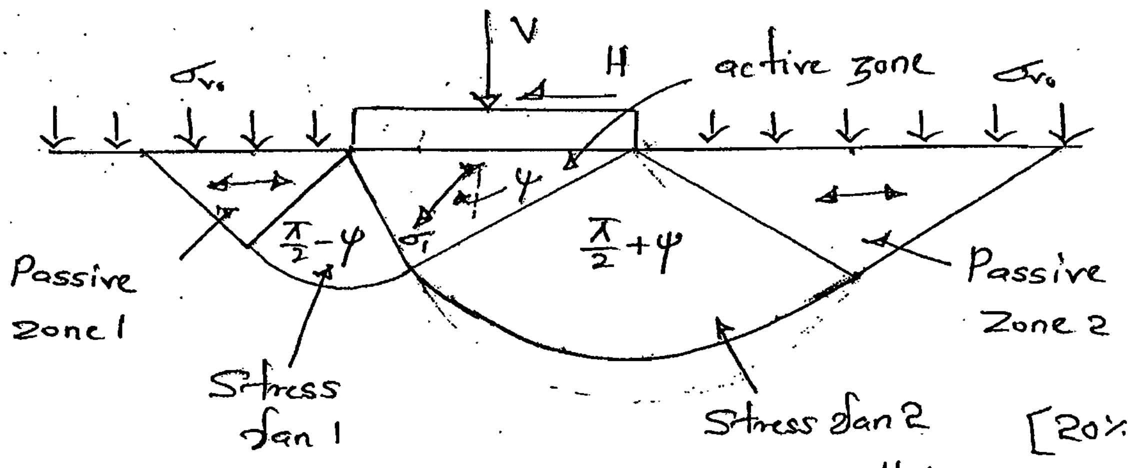
$$V_{uH} = (2+\pi) S_u B + B\sigma_{v0}'$$

[10%]

(ii) $H_{uH} = B s_u$

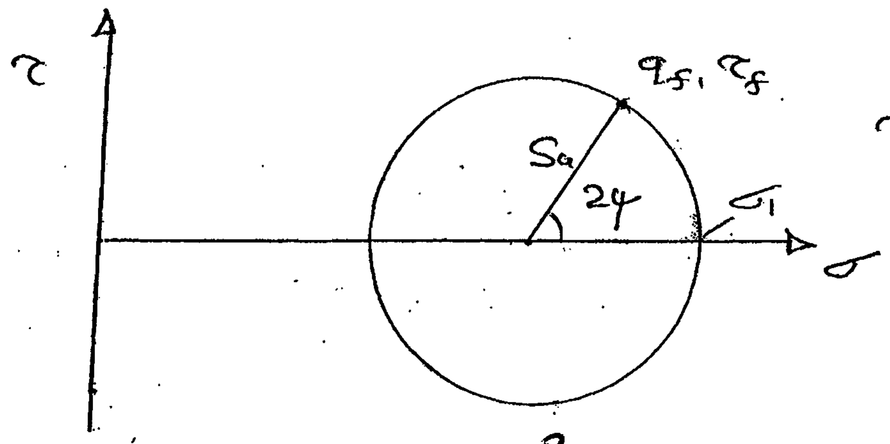
H_{uH} can be increased by embedding the foundation, or having skirts. [10%]

(iii)



[20%]

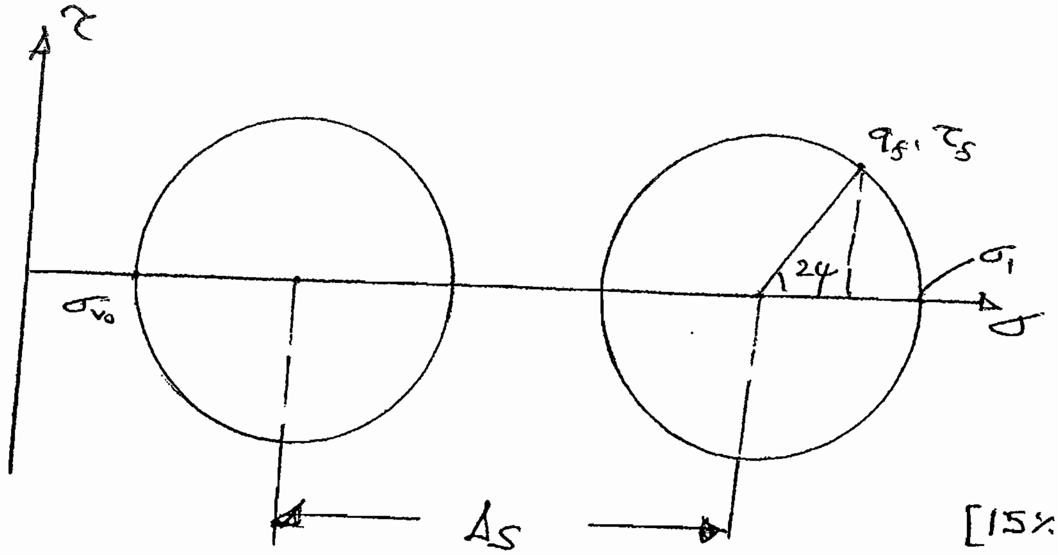
$$q_f = V/B \qquad \tau_f = H/B$$



$$\tau_f = S_u \sin 2\psi$$

$$\psi = \frac{1}{2}\sin^{-1}\left[\frac{H}{B s_u}\right]$$

Working from Passive zone 1

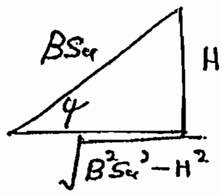


[15%]

$$\Delta s = 2S_u\left(\frac{\pi}{2}-\psi\right)$$

$$\frac{V}{B} = q_f = \sigma_{v0}' + S_u + 2S_u\left(\frac{\pi}{2}-\psi\right) + S_u \cos 2\psi$$

$$\frac{H}{B} = \tau_f = S_u \sin 2\psi$$



$$\frac{V}{BS_u} = \frac{\sigma_{v0}}{S_u} + 1 + 2\left[\frac{\pi}{2}-\psi\right] + \cos 2\psi$$

$$= \frac{\sigma_{v0}'}{S_u} + 1 + \pi - \sin^{-1}\left[\frac{H}{BS_u}\right] + \sqrt{1-\left(\frac{H}{BS_u}\right)^2}$$

Note: shear strength S_u is not fully mobilised in stress fan 2.

The principal stress directions in Passive zone 2 and active zone differ by an angle $(\pi/2 + \psi)$

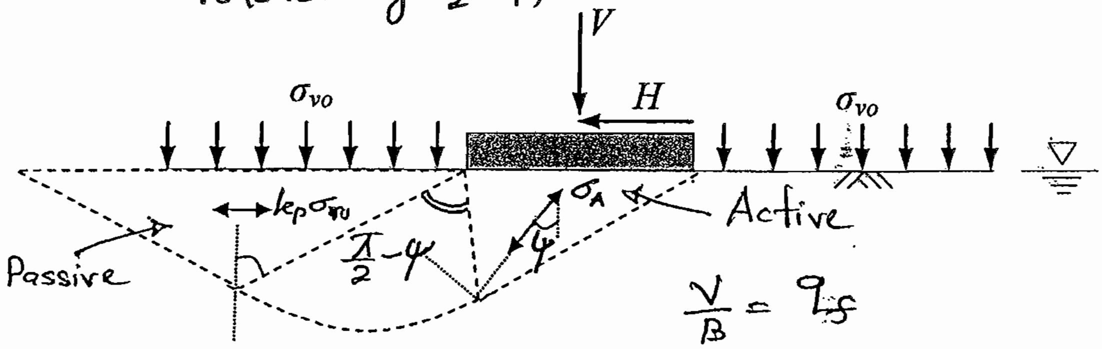
Principal stresses differ by $\Delta s_{fan\,2} = (\pi - 2\psi) S_u$

$\therefore$ strength mobilised in fan 2 is $= \dfrac{S_u(\pi - 2\psi)}{(\pi + 2\psi)}$

[40%]

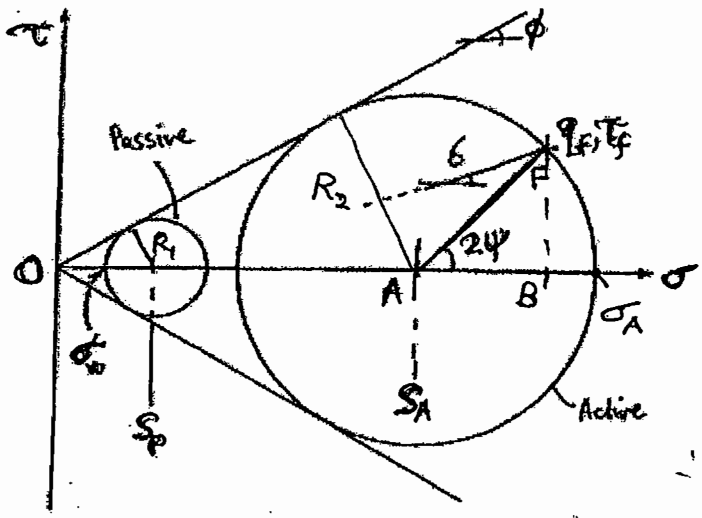
1/ b/

Principal stress in active zone needs to be rotated by $(\frac{\pi}{2}-\psi)$ in the stress plan.



$\frac{V}{B}=q_f$

$\frac{H}{B}=\tau_f$



$$\frac{S_A}{S_p}=\exp 2\left(\frac{\pi}{2}-\psi\right)\tan\phi$$

$R_1 = S_p - \sigma_{vo}$

$S_p \sin\phi = R_1$

$\therefore\ S_p = \frac{\sigma_{vo}}{1-\sin\phi}$

$AB = R_2 \cos 2\psi$

$$\begin{aligned} R_2 &= S_A \sin\phi \\ &= S_p \exp 2\left(\frac{\pi}{2}-\psi\right)\tan\phi \sin\phi \\ &= \frac{\sigma_{vo}}{1-\sin\phi}\exp 2\left(\frac{\pi}{2}-\psi\right)\tan\phi\sin\phi \end{aligned}$$

$$\begin{aligned} \frac{V}{B}=q_f &= S_A + AB \\ &= S_A + S_A\sin\phi\cos 2\psi \\ &= (1+\sin\phi\cos 2\psi)\, S_A \end{aligned}$$

$$\frac{V}{B\sigma'_{v0}} = \frac{q_f}{\sigma'_{v0}} = \frac{(1+\sin\phi\cos 2\psi)}{(1-\sin\phi)}\exp\left[2\left(\frac{\pi}{2}-\psi\right)\tan\phi\right]$$

[30%]

1. (c)

Drained vertical capacity arises from a combination of the soil weight and the surcharge. Drained vertical capacity due to self weight of the soil is accounted by N_γ factor. The surcharge is accounted by N_q factor.

$$\frac{V_{ult}}{B} \; due \; to \; selfweight = N_\gamma \frac{\gamma' B}{2}$$

There are two alternative techniques for selecting $N\gamma$ as a function of friction angle ϕ:

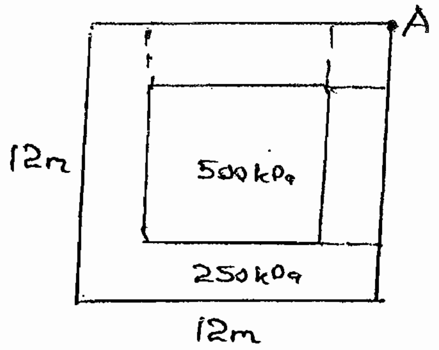
1. Numerically
2. Empirical relationships for N_γ using a link to N_q

   The Eurocode (2004) suggests the following relationship between N_q and N_γ $\rightarrow$ $N_\gamma = 2\,(N_q - 1) \tan \phi$

[10%]

2/

a/

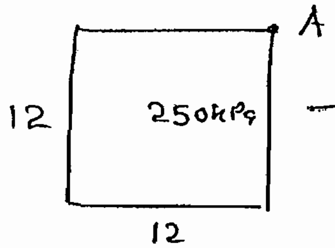


$$\omega_c = \frac{(1-\upsilon)}{G}\frac{qB}{2} I_{rec}$$

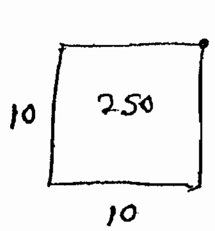
Assumptions.

1/ Ignore stiffness of building & foundation

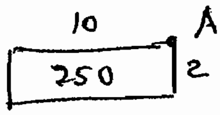
2/ Isotropic, homogenous, elastic half-space



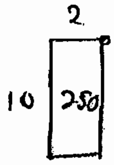
$\longrightarrow \omega_1$ $B = 12$ $L/B = 1$ $I_{rec\,1} = 0.561$



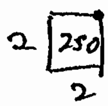
$A \longrightarrow \omega_2$ $B = 10$ $L/B = 1$ $I_{rec2} = 0.561$



$A \longrightarrow \omega_3$ $B = 2$ $L/B = 5$ $I_{rec_3} = 1.052$



$A \longrightarrow \omega_4$ $B = 2$ $L/B = 5$ $I_{rec_4} = 1.052$



$A \longrightarrow \omega_5$ $B = 2$ $L/B = 1$ $I_{rec\,5} = 0.561$

Undrained ω at $A = \omega_1 + \omega_2 - \omega_3 - \omega_4 + \omega_5$

$$\therefore\ \omega = \frac{0.5}{G}\,\frac{q}{2}\left[12\times I_1 + 10\,I_2 - 2I_3 - 2I_4 + 2I_5\right.$$

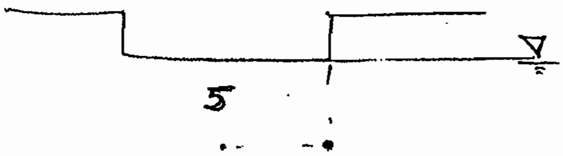
$$= \frac{0.5\times 250\times 10^3}{5\times 10^6\times 2}\left[12\times 0.561 + 10\times 0.561 - \cancel{2}4\times\cancel{2}1.052\right.$$
$$\left. + 2\times 0.561\right]$$

$$= \frac{12.5}{10^3}\left[13.464 - 4.208\right]$$

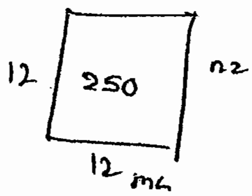
$$= 115.7\,\text{mm}$$

Assumptions – ignore stiffness of building & foundation

2 b/ (i)

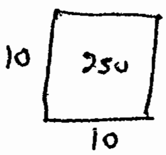


$B = 12\,m$

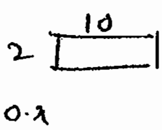


$I_r \approx 0.245$

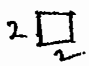
$n = m = 6$



$I_r \approx 0.23$



$I_r = 0.11$



$I_r = 0.06$

$\therefore$ $\Delta\sigma$ at $A = 250 \times 10^{3} \times [0.245 + 0.23 - 0.11 - 0.11 + 0.06]$

$= 78.75\ kPa$

ii/

$v = 1.3 - 0.02 \ln \sigma_v'$

Initial v_0 at 5m below A

$v_0 = 1.3 - 0.02 \ln [20 + 50]$

$= 1.3 - 0.0849$

$= 1.215$ //

$\Delta v = 0.02 \ln\left(\dfrac{70 + 78.75}{70}\right)$

$= 0.015$

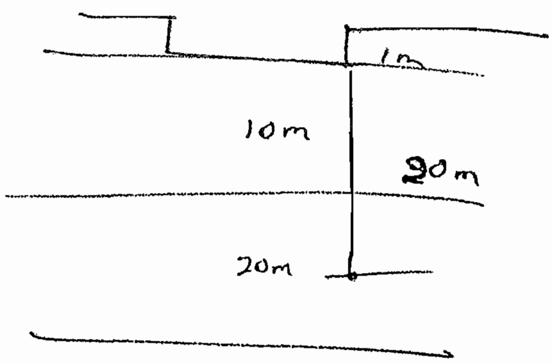
$\therefore \Delta H_1 = H \dfrac{\Delta v}{v_0}$

$= 10 \times \dfrac{0.015}{1.215}$

$= 0.123\ m$ //

b/

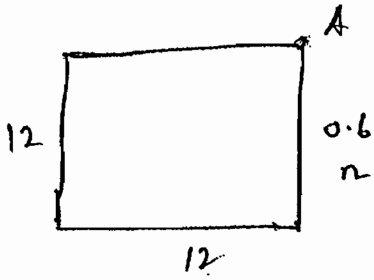
ii/



σ'_{v0} at 30m below A $= 20 + 200$

$= 220$ kPa

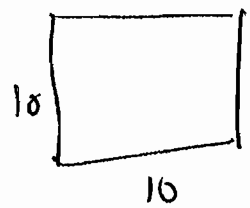
∴ due to foundation



0.6

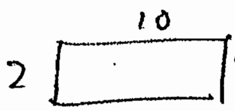
$n = m = 0.6$

$I_r = 0.104$.



0.5

$I_r = 0.082$



$n = 0.1$

$m = 0.5$

$I_r = 0.02$

$n = 0.1$ $I_r = 0.02/5$

Increase in $\Delta\sigma = 250 \times 10^3 (0.104 + 0.082 - 0.02 - 0.02 + 0.02/5)$

$= 37.5 \times 10^3$ Pa

Initial v_0 at 20m below = 220 kPa

$$v_0 = 1.3 - 0.02 \ln 220$$
$$= 1.3 - 0.1$$
$$= 1.2$$

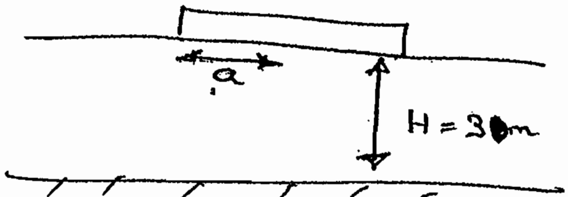
$$\therefore \Delta v = k \ln \frac{257.5}{220}$$
$$= 3.14 \times 10^{-3}$$

$$\therefore \Delta H_2 = 20 \times \frac{3.14 \times 10^{-3}}{1.2}$$
$$= 0.052 \text{ m}$$

$\therefore$ total settlement $= 0.123 + 0.052$
$= 0.175$ m

2 b ii) Covert the foundation area to a circle radius "a"

C_h value of the clay is required
estimate the time for drained settlement, using



$$T_v = \frac{C_h t}{a^2} \approx 10$$

for Drained settlement $T_v \geq 10$

3

a) $D = 0.5m$ $L = 25m$

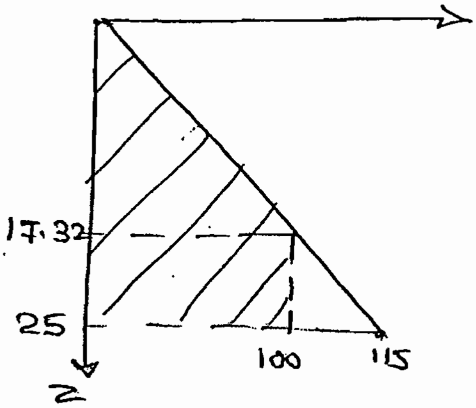
$\phi = 35^\circ$ $\delta = 30^\circ$

$N_q = 40$ $q_b \lim = 9.6 MPa$ } Data book

$\tau_s \lim = 100 kPa$

Closed-ended $\rightarrow K = 1$

$\therefore$ Shaft resistance $\tau = K \sigma_{v0}' \tan\delta$



At 25m

$\tau_s = 25 \times 10 \times \tan 30^\circ$

$= 115.47 kPa \geq 100$

So limit to 100

$x \times 100 \times \tan 30 = 100$

$x = 17.32$ [10%]

$Q_{sf} = \cancel{2}\pi \frac{D}{\cancel{2}} [\frac{1}{\cancel{2}} \times \overset{50}{\cancel{100}} \times 17.32 + 100 \times 7.68]$

$= 2566.68 \, kN$ [5%]

q_b at 25m $= 400 \times 250 = 100 MPa > 9.6 MPa$

So limit to 9.6 MPa

$Q_{bd} = \pi \frac{D^2}{4} \times N_q \sigma_{v0}'$

$= \pi \frac{0.5^2}{4} \times 9.6 \times 10^6 = 1884.9 \, kN$

[5%]

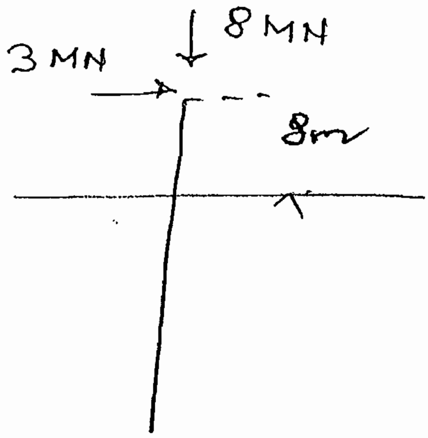
$Q_{Total} = 2.566 + 1.884 = 4.45 MN$

3/

b/ $D = 4m$ $\quad$ $S_u = 2.5z$ kPa

$t = 40$ mm $\quad$ $\gamma' = 7$ kN/m^3

$c_h = 20$ m^2/year



i/ $$\frac{\pi D_{eq}^2}{4} = \pi D t \quad (t \ll D)$$

$$D_{eq} = 2\sqrt{Dt} = 2\sqrt{4 \times 0.04}$$

$$= 0.8\,m$$

After 90% consolidation

$$T_{eq} = \frac{C_h t}{D_{eq}^2} = 10$$

$$t = \frac{10\, D_{eq}^2}{C_h} = \frac{10 \times 0.8^2}{20} = 0.32 \text{ years}$$

$$\approx 117 \text{ days}$$

[10%]

3/ b/ ii/

$$S_u = 2.5z$$

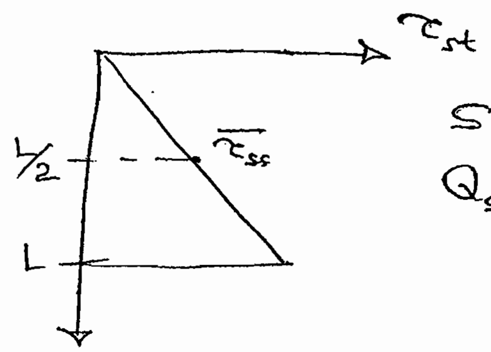
$$\sigma_{v0}' = 7z$$

$$\therefore S_u < \sigma_{v0}' \rightarrow \tau_{sf} = \frac{1}{2}\sqrt{S_u \sigma_{v0}'}$$

$$\alpha = \frac{1}{2}\left[\frac{\sigma_{v0}'}{S_u}\right]^{1/2}$$

$$= \frac{1}{2}\left[\frac{7}{2.5}\right]^{1/2}$$

$$= 0.83666$$



Shaft resistance

$$Q_s = \cancel{2}\pi \times \frac{D}{\cancel{2}} \times L \times \overline{\tau_{sf}}$$

$$= \pi \times \cancel{4}\,2 \times L \times \alpha \times 2.5 \times \frac{L}{\cancel{2}}$$

$$= 2\pi \times \alpha \times 2.5 \times L^2$$

$$= 5\pi\alpha L^2$$

Base resistance $Q_b = N_c S_u A_b$

$$= 9 \times L \times 2.5 \times \frac{\pi D^2}{4}$$

$$= 90\pi L$$

weight of soil in pile $= \frac{\pi D^2}{4} \times L \times \gamma'$

$$= 28\pi L$$

3/ b/ ii

$\therefore$

$$V_{Total} = Q_s + Q_b - \text{weight of soil in pile}$$

$$8000 = 5\pi\alpha L^2 + 90\pi L - 28\pi L$$

$$0 = L^2 + \frac{62L}{5\alpha} - \frac{8000}{\pi 5\alpha}$$

$$0 = L^2 + 14.82L - 608.7249$$

$$L = \frac{-14.82L \pm \sqrt{14.82^2 + 4 \times 608.7249}}{2}$$

$$= 18.35 \text{ m}$$

3 b iii/ Databook Chart in Pg 12

$$e/_D = \frac{8}{4} = 2$$

$n \rightarrow$ $S_u = 2.5z$ $k_u = 2.5$

$n = 9k_u$

$= 22.5$

$H = 2.5 \text{ MN} = 2500 \text{ kN}$

$$\frac{H}{nD^3} = \frac{2500}{22.5 \times 4^3} = 1.736$$

From chart $\rightarrow$ $\frac{L}{D} \geqslant 4.5 \therefore L > 18\text{m}$

So 18.35m is just sufficient to withstand the horizontal load.

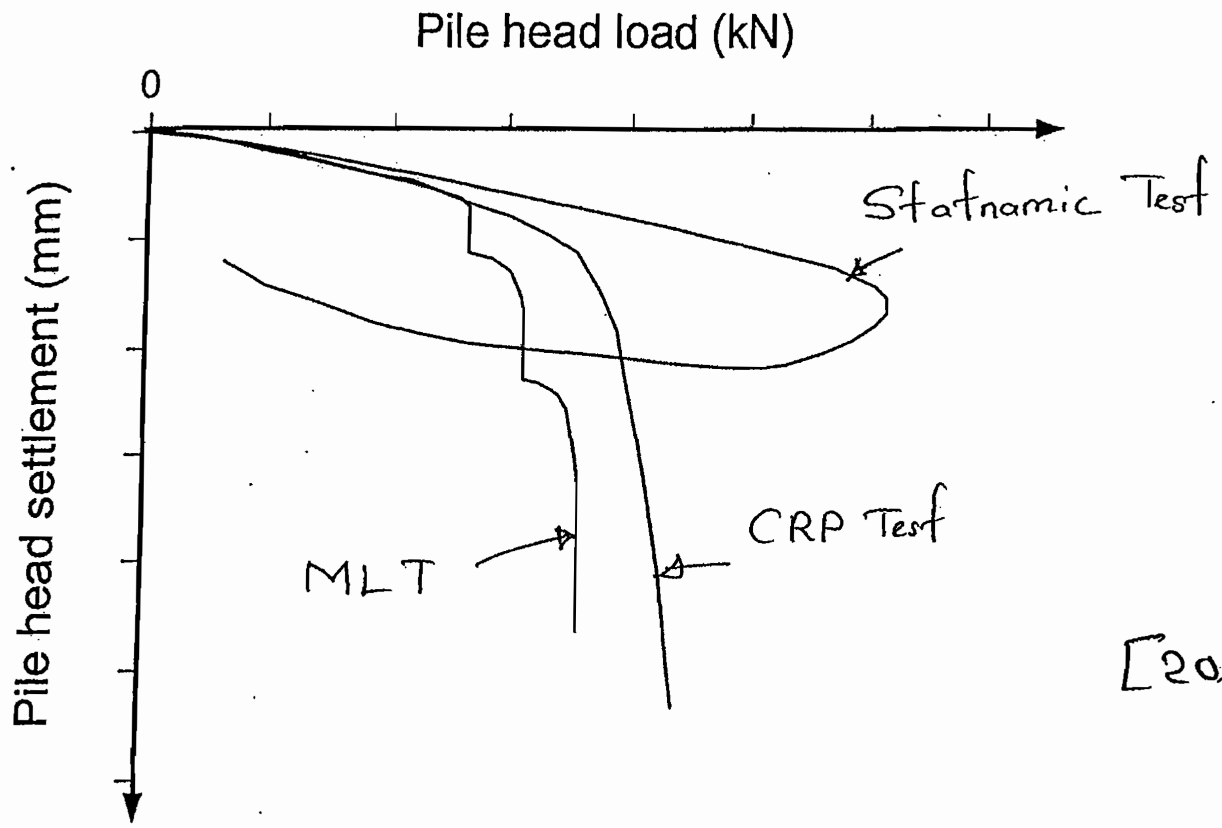
[redacted] 3 c

Types of load tests

1. Maintained load test
During a maintained load test (MLT) static force is applied via a jack to the pile head. Reaction is provided either by kentledge (deadweight) stacked above the pile or from nearby tension piles via a reaction beam. The load is increased in stages, with a pause after each increment to wait for the creep rate to subside. The settlement of the pile is measured relative to a reference beam suspended above the ground.

2. Constant rate of penetration test
During a constant rate of penetration test (CRP), the pile is pushed downwards at a constant velocity whilst the required force is recorded. This has the benefit of reducing the testing period compared to an MLT, but can give a misleadingly low settlement at a given load since the test rate does not permit full consolidation.

3. Statnamic testing
During a Statnamic load test, liquid propellant fuel is exploded within a chamber above the pile head, driving a heavy mass upwards using the pile head as reaction. Typically an acceleration of 10-20 g is achieved, allowing a mass of 5-10% of the desired pile head load to be used. The Statnamic load test is of a longer duration than a simple dynamic test.



[20%]

4/

a/ $D = 0.75 m$ $t = 0.05 m$

i/ $\sigma_y = 250 \times 10^6$ Pa [informed in the examination]

[Any reasonable σ_y for steel was acceptable]

$E = 220 \times 10^9$ Pa

Steel area $= 2\pi r t$

$A_s = \pi D t$

$= \pi \times 0.75 \times 0.05$

$= 0.1178 \; m^2$

Gross Area $A_p = \dfrac{\pi D^2}{4} = 0.4417$

Equivalent Stiffness $E_p = \dfrac{E_{steel} \; A_{stl}}{A_p}$

$= \dfrac{220 \times 10^9 \times 0.1178}{0.4417}$

$= 58.66 \times 10^9$ Pa

$\simeq 59$ GPa

$M_p = D^2 t \sigma_y$

$= 0.75^2 \times 0.05 \times 250 \times 10^6$

$= 7031.25$ kNm

[20%]

4 a/ ii/

$L = 7.5m$

$S_u = 1.5z$ kPa

$K_{su} = 1.5$ $\quad n = 9 \times 1.5$
$= 13.5$ kPa

$\therefore L/D = 10$, $\frac{e}{D} = 0$ [mudline]

From Data book Pg 12
Short pile failure mechanism

$$\frac{H_{ult}}{nD^3} \simeq 12$$

$\therefore H_{ult} \simeq 12 \times 13.5 \times 0.75^3$
$= 68.34$ kN

Check long pile failure

$\frac{M_p}{nD^4} = 50$, $M_p = 213.57$ kNm $< M_p$ of pile
So safe
[10%]

When $e = 3m$
$e/D = 4$

$\frac{H_{ult}}{nD^3} \simeq 9$, $H_{ult} = 9 \times 13.5 \times 0.75^3$
$= 51.25$ kN

Check long pile failure
$\frac{M_p}{nD^4} = 60$, $M_p = 256.28$ kNm $< M_p$ of pile
So safe
[10%]

4

a/

iii/ $L/D = 10$ $e/D = 0$

$\therefore$ Data book Pg 12

$$\frac{H_{ult}}{n D^3} = 50 \text{ [for restrained pile]}$$

$$H_{ult} = 50 \times 13.5 \times 0.75^3$$
$$= 284.76 \text{ kN}$$

Check long pile failure

$$\frac{M_p}{n D^4} \simeq 180$$

$$M_p \simeq 180 \times 13.5 \times 0.75^4$$
$$\simeq 768.86 \text{ kNm} < M_p \text{ of Pile}$$

Safe //

iv/ maximum pile stiffness

$$L \geq 1.5 D \sqrt{\lambda}$$ [Data book Pg 14]

$$\lambda = \frac{E_p}{G_L}$$

$G_L = 150 s_u$
$= 150 \times 1.5 L$
$= 225 L$ kPa

$$\therefore L \geq 1.5 \times 0.75 \sqrt{\frac{59 \times 10^9}{225 L \times 10^3}}$$

$$L^{3/2} \geq 576$$

$$L \geq 69.2 \text{ m}$$ [10%]

4

b/

$$\rho = \frac{G_{avg}}{G_L} = \frac{135}{200} = 0.675$$

$$\zeta = \ln\left(5\rho(1-\nu)\frac{L}{D}\right) = 4.4$$

$$\eta = \frac{D_{base}}{D} = 1$$

$$\xi = \frac{G_L}{G_b} = 1$$

$$\frac{V}{w_{head} D G_L} = \frac{2}{1-\nu}\frac{\eta}{\xi} + \frac{2\pi}{\zeta}\rho\frac{L}{D}$$ [Data book Pg 14]

$$= \frac{2}{0.8} + \frac{2\pi}{4.4} \times 0.675 \frac{14}{0.41}$$

$$\frac{V}{w_{head}} = 439 \text{ kN/mm}$$

when $V = 800\text{kN}$

$$w = 1.82 \text{ mm}$$

c/ Common anchor systems are

1/ Suction anchors
2/ Plate anchors
3/ Drop anchors
4/ Gravity anchors.

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