NANYANG TECHNOLOGICAL UNIVERSITY

SEMESTER 1 EXAMINATION 2010-2011

CV3301 – FOUNDATION ENGINEERING

December 2010 | Time Allowed: 2½ hours

INSTRUCTIONS

1. This paper contains FIVE (5) questions and comprises NINE (9) pages.
2. Answer ALL questions.
3. An Appendix of THREE (3) pages is attached to the Question Paper.
4. Assume unit weight of water is 10 kN/m^3.
5. This is a Closed-Book Examination.
6. The questions do not carry equal marks.

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1. Eight apartment blocks, each 30-storey high, are to be constructed on a site measuring 250 m by 150 m. A warehouse formerly occupied the site. The only construction record that can be retrieved from the factory construction is a cross section of the soil profile at the site (Figure Q1). It is proposed that a soil investigation programme comprising 12 boreholes combined with relevant in-situ and laboratory tests should be carried out.

 (a) What are the primary objectives of a soil investigation programme? (3 Marks)

 (b) State the two key principles in the design of a foundation system. What foundation system would you recommend for the apartment blocks? Justify your recommendation. (4 Marks)

 (c) The elevation of the sandstone layer given in Figure Q1 may not be reliable. How deep should the proposed boreholes be? (2 Marks)

Note: Question No.1 continues on page 2.

 (d) Suggest suitable in-situ tests for the various soil deposits encountered in the borehole during drilling. What useful information would these in-situ tests provide? (3 Marks)

 (e) What sampling methods and laboratory tests will you recommend for the clay, sand and sandstone that will provide the essential soil parameters for the foundation design? (3 Marks)

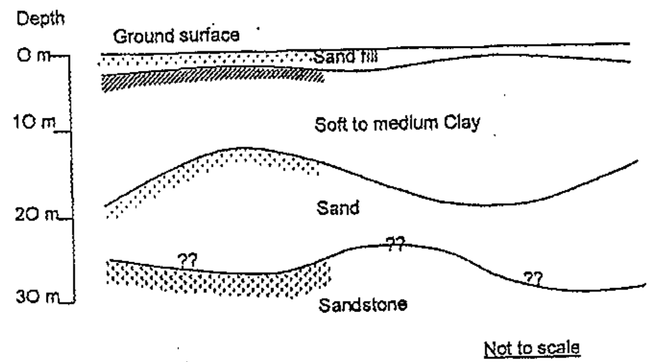


Figure Q1

2. Details of a reinforced concrete cantilever retaining wall are shown in Figure Q2. The angle of friction between the base of the wall and the foundation soil is 28°. ϕ γ The unit weight of the reinforced concrete is 24 kN/m^3. The force due to active earth pressure on the vertical plane above the heel of the wall is inclined at an angle of 30° to the horizontal and has a magnitude of 230 kN per metre run of the wall. It is required that the factor of safety (F_s) against all potential failure modes must be at least 1.5. $F_s = 1.5$

(a) By ignoring the passive pressure in front of the wall, find the factors of safety against the following failure modes:

(i) horizontal sliding;

(ii) rotational failure about the toe of the wall; and

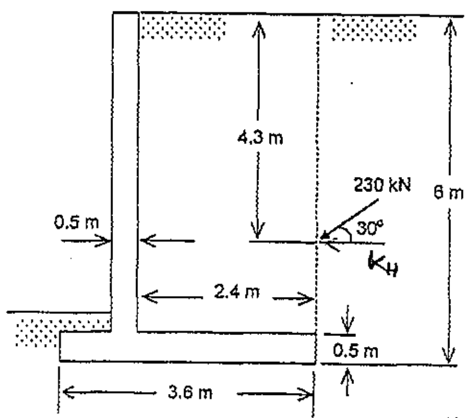
(iii) comment on the adequacy of the F_s for parts (i) and (ii).

(11 Marks)

(b) Find the location of the resultant force at the base of the wall. (2 Marks)

(c) How would including the passive pressure in the calculation affect the F_s computed in (i) and (ii)?

(2 Marks)



Assume $\gamma_{soil} = 16 kN/m^3$

Not to scale

Figure Q2

3. A long cofferdam is to be constructed across a river estuary to allow a barrage construction to be carried out in the dry. It consists of two rows of cantilever sheetpile walls spaced 25 m apart. Figure Q3 shows a half-section of the cofferdam. After the sheetpiles are installed, 2 m of sandfill will be placed within the cofferdam before the start of dewatering. The water level in the cofferdam is maintained at the top of the sandfill by water pumps. The properties of the various soil layers are shown in Figure Q3.

High and low tides are at RL 105 m and RL 103 m. The water level of a piezometric standpipe installed in the sand deposit shows that the pore pressure regime in the lower sand deposit is unaffected by the fluctuation in tidal levels and may be assumed to be at RL 104 m throughout the construction period.

(a) If the wall rotates about point 'o', construct the total lateral pressure diagrams acting on both sides of the wall above point 'o'. Assume Rankine earth pressure theory applies. It is not necessary to construct the pressure diagrams below point 'o'.

(14 Marks)

(b) Calculate the factor of safety against rotational failure. (2 Marks)

(c) In addition to the rotational failure mode of the sheetpile wall, what other potential failure scenarios must be considered in design?

(4 Marks)

Note: Question No.3 continues on page 5.

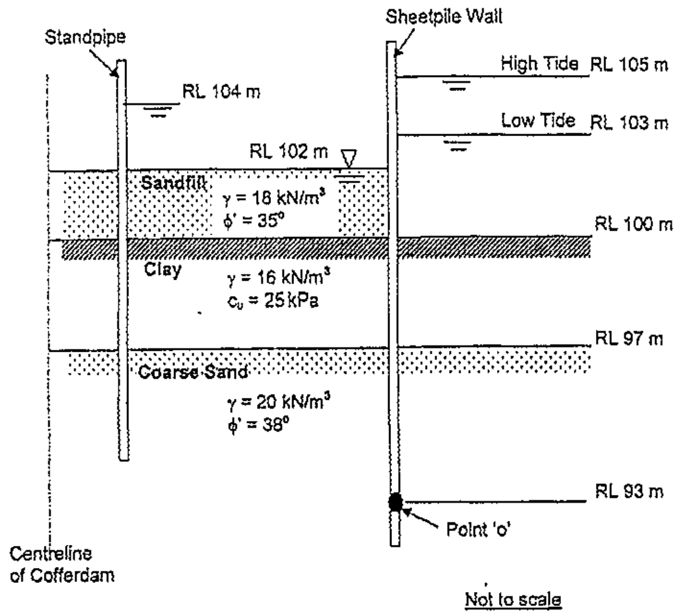


Figure Q3

4. (a) What is the purpose of foundations and why are they required? (3 Marks)

(b) The soil profile of a site is shown in Figure Q4. A single-storey warehouse on square spread footings (1.5 m x 1.5 m) at a depth of 1 m is to be built on the site. The groundwater table is at a depth of 1 m.

(i) Determine the allowable load (kN) on the footing using a factor of safety of 3 against bearing capacity failure.

(ii) Determine the immediate, consolidation and total settlement of the footings under the allowable load computed in part (i).

State the assumptions used in your computation.

(16 Marks)

(c) Raft foundation has some advantages over spread footings. List three of these advantages.

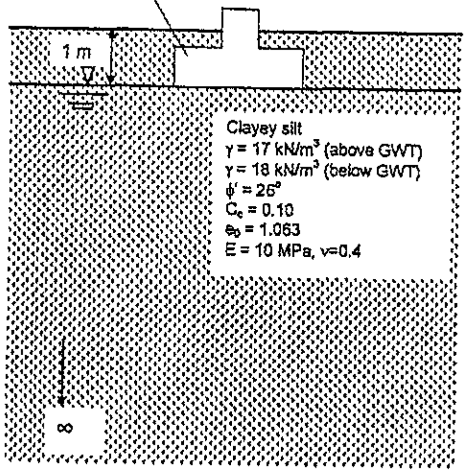
(3 Marks)

(d) A fully-compensated concrete raft foundation is considered for the site shown in Figure Q4. If the raft foundation (16 m x 12 m x 0.5 m thick) supports a building whose weight is 7.5 MN, determine the depth of the raft foundation. Assume unit weight of concrete as 24 kN/m^3.

(3 Marks)

Note: Question No.4 continues on page 7.

Concrete footing
1.5 m x 1.5 m x 0.5 m



Not to scale

Figure Q4

5\. (a) The main difference between shallow and deep foundations is the consideration of side shear resistance in deep foundation. Briefly describe two methods to estimate unit side shear resistance in deep foundations from shear strength parameters of soils.

(6 Marks)

(b) Figure Q5 shows a precast concrete pile driven into a deep clay deposit.

(i) Estimate the allowable structural capacity (kN) of the pile using a factor of safety of 4.

(ii) Estimate the allowable downward geotechnical capacity (kN) of the pile using a factor of safety of 3.

(iii) Estimate the allowable upward load (kN) the pile can resist using a factor of safety of 5.

(14 Marks)

(c) Pile driving formulas such as the Engineering News Formula below are sometimes used to estimate pile capacity.

$$P_a = \frac{W_r h}{F(s+c)}$$

where P_a = allowable pile capacity (kN), W_r = weight of hammer (kN), h = height of hammer drop (m), F = factor of safety (normally taken as 6), s = set i.e. amount of pile penetration per blow (m/blow) and c = factor to account for difference between theoretical and actual set (0.025 m for drop hammer and 0.0025 m for steam hammer)

(i) Illustrate the principle for the derivation of the pile driving formulas.

(ii) Using the Engineering News Formula determine the capacity of a steel pile given that the pile was driven with a drop hammer with a manufacturer's hammer energy rating of 40 kNm and the average set s for the last few hammer blows was 6 mm/blow.

(5 Marks)

Note: Question No.5 continues on page 9.

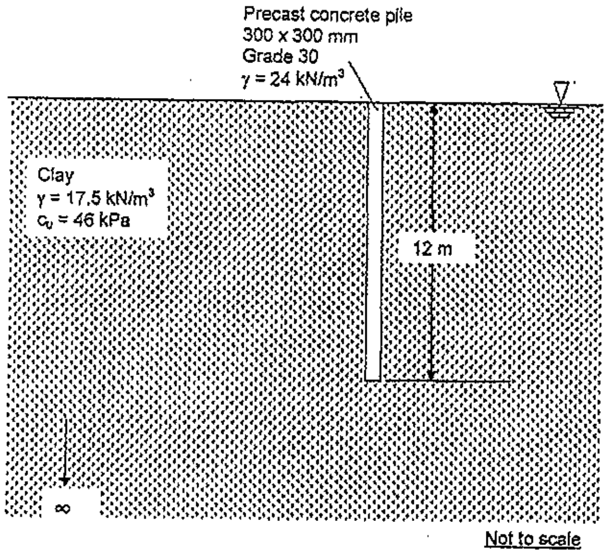


Figure Q5

END OF PAPER

Shallow Foundations

Vesić's bearing capacity equation:

$$q_{ult} = cN_c s_c d_c i_c b_c g_c + \sigma_{zD} N_q s_q d_q i_q b_q g_q + 0.5\gamma B N_\gamma s_\gamma d_\gamma i_\gamma b_\gamma g_\gamma$$

$$N_q = e^{\pi \tan\phi} \tan^2\left(45° + \frac{\phi}{2}\right)$$

$$N_c = 5.14 \qquad \text{for } \phi = 0$$

$$N_c = \frac{N_q - 1}{\tan\phi} \qquad \text{for } \phi > 0$$

$$N_\gamma = 2(N_q + 1)\tan\phi$$

Shape Factors	Depth Factors	Load Inclination Factors
$s_c = 1 + \left(\frac{B}{L}\right)\left(\frac{N_q}{N_c}\right)$ $s_q = 1 + \left(\frac{B}{L}\right)\tan\phi$ $s_\gamma = 1 - 0.4\left(\frac{B}{L}\right)$	$d_c = 1 + 0.4k$ $d_q = 1 + 2k\tan\phi(1 - \sin\phi)^2$ $d_\gamma = 1$ $k = \frac{D}{B}$ for $\frac{D}{B} \le 1$ $k = \tan^{-1}\left(\frac{D}{B}\right)$ for $\frac{D}{B} > 1$	$i_c = 1 - \frac{mV}{AcN_c} \ge 0$ $i_q = \left[1 - \frac{V}{P + \frac{Ac}{\tan\phi}}\right]^m \ge 0$ $i_\gamma = \left[1 - \frac{V}{P + \frac{Ac}{\tan\phi}}\right]^{m+1} \ge 0$ For loads inclined in B direction : $m = \frac{2 + B/L}{1 + B/L}$ For loads inclined in L direction : $m = \frac{2 + L/B}{1 + L/B}$
	Base Inclination Factors	**Ground Inclination Factors**
	$b_c = 1 - \frac{\alpha}{147°}$ $b_q = b_\gamma = \left(1 - \frac{\alpha\tan\phi}{57°}\right)^2$	$g_c = 1 - \frac{\beta}{147°}$ $g_q = g_\gamma = [1 - \tan\beta]^2$

Settlement

Correction factor r

Foundation rigidity	Correction factor r
Perfectly flexible	1.00
Intermediate	0.85 – 1.00
Perfectly rigid	0.85

Elastic settlement, $\delta_D = C_s q b \dfrac{(1-\nu_s^2)}{E_s}$

C_s for infinite depth

Shape & Rigidity	Centre	Corner	Edge/Middle of long side	Average
Circle (flexible)	1.0		0.64	0.85
Circle (rigid)	0.79		0.79	0.79
Square (flexible)	1.12	0.56	0.76	0.95
Square (rigid)	0.82	0.82	0.82	0.82
Rectangle (flexible) Length/width				
2	1.53	0.76	1.12	1.30
5	2.10	1.05	1.68	1.82
10	2.56	1.28	2.10	2.24

C_s for finite depth

H/B	Centre of rigid circular area	Corner of Flexible Rectangular Area				
ν = 0.50	(diameter =B)	L/B = 1	L/B = 2	L/B = 5	L/B = 10	L/B = ∞
0	0.0	0.0	0.0	0.0	0.0	0.0
0.5	0.14	0.05	0.04	0.04	0.04	0.04
1.0	0.35	0.15	0.12	0.10	0.10	0.10
1.5	0.48	0.23	0.22	0.18	0.18	0.18
2.0	0.54	0.29	0.29	0.27	0.26	0.26
3.0	0.62	0.36	0.40	0.39	0.38	0.37
5.0	0.69	0.44	0.52	0.55	0.54	0.52
10.0	0.74	0.48	0.64	0.76	0.77	0.73
ν = 0.33						
0	0.0	0.0	0.0	0.0	0.0	0.0
0.5	0.20	0.09	0.08	0.08	0.08	0.08
1.0	0.40	0.19	0.18	0.16	0.16	0.16
1.5	0.51	0.27	0.28	0.25	0.25	0.25
2.0	0.57	0.32	0.34	0.34	0.34	0.34
3.0	0.64	0.38	0.44	0.46	0.45	0.45
5.0	0.70	0.46	0.56	0.60	0.61	0.61
10.0	0.74	0.49	0.66	0.80	0.82	0.81

Consolidation settlement

$$\delta_c = \frac{C_r}{1+e_0} H \log\left(\frac{\sigma'_p}{\sigma'_{z0}}\right) + \frac{C_c}{1+e_0} H \log\left(\frac{\sigma'_{zf}}{\sigma'_p}\right)$$

Deep Foundations

α-Method

For $c_u < 25\,\text{kPa}$: $\alpha = 1.0$

For $25\,\text{kPa} \le c_u < 75\,\text{kPa}$: $\alpha = 1.0 - 0.5\left(\dfrac{c_u - 25\,\text{kPa}}{50\,\text{kPa}}\right)$

For $c_u \ge 75\,\text{kPa}$: $\alpha = 0.5$

Toe bearing resistance in clays

For $c_u < 250\,\text{kPa}$,

$$q_t' = c_u N_c^*$$

where :

$N_c^* = 6.5$ for $c_u = 25\,\text{kPa}$

$N_c^* = 8.0$ for $c_u = 50\,\text{kPa}$

$N_c^* = 9.0$ for $c_u \ge 100\,\text{kPa}$

c_u is undrained shear strength between toe and 2 times pile base diameter below toe

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1. a). To evaluate surface site conditions.
To assess variability of subsoil conditions
To identify potential geotechnical hazards
To establish soil profile for design
To determine design soil parameters.

b). No catastrophic failure
No excessive settlement.
Since the building is 30-storey high, which is very heavy and the soil contains a large portion of soft to medium clay, which ~~so~~ will cause excessive settlement, deep foundation such as bored piles ~~and~~ or drilled shaft can be used.

c). The depth should be where $N \geq 100$.

d). for sand fill — SPT — N, consistency. D_r, ϕ'
soft to medium clay — CPT — q_c, f_s, u, consistency, continuous soil profile
c_u, σ_p'
FVT — c_u, σ_p'

e). Clay — ~~Thin-wall sampler~~ density test. water content test. Atterberg limits test
Consolidation test. UU. CU, permeability.
Sand — SPT split-barrel sampler Sieve analysis, Fines content tests.
sandstone — Triple tube core barrel.
Cone test.

2. a). $\gamma_c = 24 kN/m^3$ $P = 230 kN$ $30°$ required $F_S = 1.5$. $\delta = 28°$

$W_{c_1} = 24 \times 5.5 \times 0.5 = 66 \; kN/m$. $l_1 = 0.95m$

$W_{c_2} = 24 \times 0.5 \times 3.6 = 43.2 \; kN/m$ $l_2 = 1.8m$

$P_H = 230 \times \cos 30° = 199.2 \; kN/m$ $l_3 = 1.7m$

$P_V = 230 \times \sin 30° = 115 \; kN/m$ $l_4 = 3.6m$.

Assume $\gamma_s = 16 kN/m^3$ $W_s = 2.4 \times 5.5 \times 16 = 211.2 \; kN/m$. $l_5 = 2.4m$

i). $F_{S_1} = \dfrac{(W + P_V) \cdot \tan 28°}{P_H} = \dfrac{(66 + \cancel{43.2} + 211.2 + 115) \times \tan 28°}{199.2} = \cancel{0.96}\ 1.16$ ※

ii). $F_{S_2} = \dfrac{66 \times 0.95 + 43.2 \times 1.8 + 211.2 \times \cancel{3.6}\,2.4 + 115 \times 3.6}{199.2 \times 1.7} = 3.13$ ※

iii). $F_{S_1} \approx 1.16 < 1.5 \Rightarrow$ not adequate

$F_{S_2} = 3.13 > 1.5 \Rightarrow$ adequate.

b). $x \cdot (66 + 43.2 + 211.2 + 115) = 66 \times 0.95 + 43.2 \times 1.8 + 211.2 \times 2.4 + 115 \times 3.6 - 199.2 \times 1$

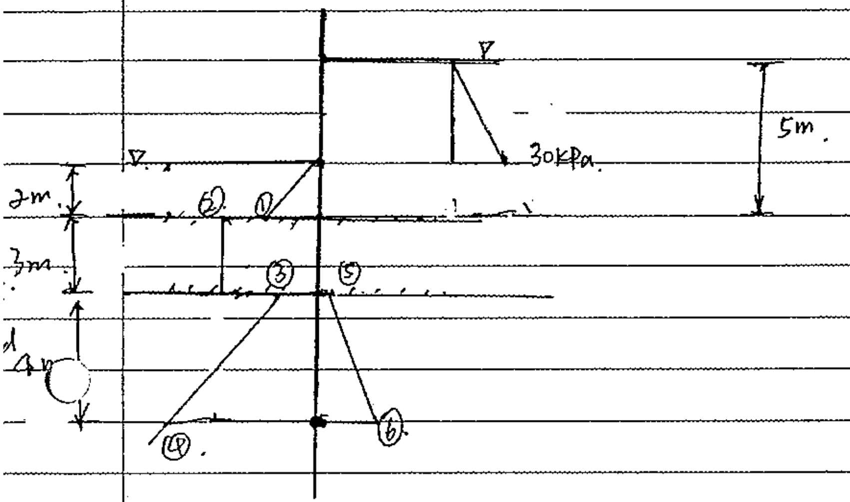
$x = 1.06m$. ※

c). F_{S_1} will increase

F_{S_2} will increase.

3.

3. a). During high tide.



sand fill:

$K_{A_1} = \frac{1-\sin 35^\circ}{1+\sin 35^\circ} = 0.27.$ $\quad K_{p_1} = 3.69.$

①. $\sigma_z' = 3.69 \times 2 \times (18-10) = 59.04$ kPa

clay: ②. $\sigma_p - \sigma_A = 5.29\, C_{um} - q_n = 5.29 \times 25/1.5 + 18 \times 2 - 5 \times 10 = 74.17$ kPa.

sand: $K_{A_2} = \frac{1-\sin 38^\circ}{1+\sin 38^\circ} = 0.24$ $\quad K_{p_2} = 4.20.$

③. $\sigma_z' = 4.20 \times (2 \times 18 + 16 \times 3 - 7 \times 10) = 58.8$ kPa.

④. $\sigma_z' = 4.20 \times (2 \times 18 + 16 \times 3 + 4 \times 20 - 11 \times 10) = 226.8$ kPa

⑤. $\sigma_z' = 0.24 \times (5 \times 10 + 16 \times 3 - 7 \times 10) = 6.72$ kPa

⑥. $\sigma_z' = 0.24 \times (5 \times 10 + 16 \times 3 + 4 \times 20 - 11 \times 10) = 16.32$ kPa.

b). $F_s = \dfrac{\frac{1}{2} \times 59.04 \times 7\frac{2}{3} + 74.17 \times 7.5 + 58.8 \times 2 + \frac{1}{2} \times 226.8 \times \frac{4}{3}}{\frac{1}{2} \times 30 \times 10 + 6.72 \times 2 + 16.32 \times \frac{1}{2} \times \frac{4}{3}} = 6.1$

c). Yielding of wall due to insufficient bending moment capacity.

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4. a) Foundations transmit building loads to the soil safely. The ~~soil~~ strength of soil is ~~so~~ much lower than steel and concrete, thus cannot directly support the building load. Foundations made of steel and concrete are required.

b) i) Using Vesic's Bearing Capacity

$q'_{ult} = \sigma'_{z0} N_q S_q d_q + 0.5\gamma' B N_\gamma S_\gamma d_\gamma$

$\sigma'_{z0} = 1 \times 17 = 17\ kPa$

$\gamma' = 18 - 10 = 8\ kPa$

$N_q = e^{\pi \tan 26^\circ} \tan^2(45^\circ + 26^\circ/2) = 11.85$

$N_\gamma = 2(N_q + 1)\tan 26^\circ = 12.54$

$S_q = 1 + 1.5/1.5 \times \tan 26^\circ = 1.48$

$S_\gamma = 1 - 0.4(1.5/1.5) = 0.6$

$\because D/B = 1/1.5 < 1 \quad k = 1/1.5 = 0.67$

$d_q = 1 + 2 \times 0.67 \times \tan 26^\circ \times (1 - \sin 26^\circ)^2 = 1.21$

$d_\gamma = 1$

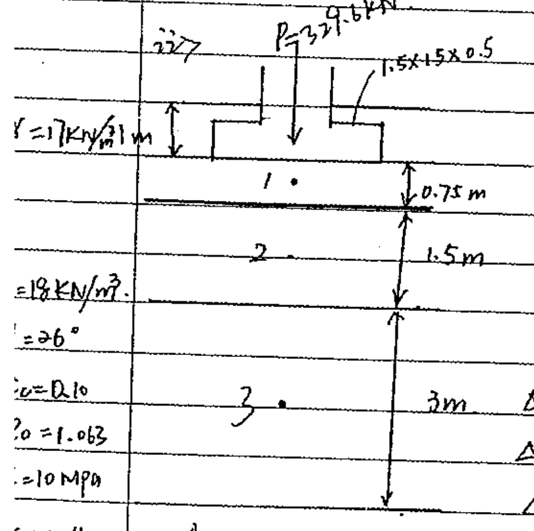
$\Rightarrow q'_{ult} = 17 \times 11.85 \times 1.48 \times 1.21 + 0.5 \times 8 \times 1.5 \times 12.54 \times 0.6 \times 1 = 405.4\ kPa$

$q_{ult} = 405.4 + 0 = 405.4\ kPa$

$F = \dfrac{q_{ult} - \sigma_{z0}}{q_a - \sigma_{z0}} = \dfrac{405.4 - 17}{q_a - 17} = 3 \Rightarrow q_a = 146.5\ kPa$

$P_a = q_a \times 1.5^2 = 329.6\ kN$ ※

ii)



layer	H	z_f	σ'_{z0}	σ'_{zf}
1	0.75	0.375	20	102.88
2	1.5	1.5	29	61.38
3	3	3.75	47	57.57

$\sigma'_{z0_1} = 17 \times 1 + (18-10) \times 0.375 = 20\ kPa$

$\sigma'_{z0_2} = 17 \times 1 + (18-10) \times 1.5 = 29\ kPa$

$\sigma'_{z0_3} = 17 \times 1 + (18-10) \times 3.75 = 47\ kPa$

$\Delta\sigma'_{z1} = \dfrac{1.5^2}{(1.5+0.375)^2} \times (146.5 - 17) = 82.88\ kPa$

$\Delta\sigma_{z2} = \dfrac{1.5^2}{(1.5+1.5)^2} \times (146.5 - 17) = 32.38\ kPa$

$\Delta\sigma_{z3} = \dfrac{1.5^2}{(1.5+3.75)^2} \times (146.5 - 17) = 10.57\ kPa$

Assume the soil is normally consolidated

$\delta_c = \dfrac{0.10}{1+1.063} \times \left[0.75 \times \log\left(\dfrac{102.88}{20}\right) + 1.5 \times \log\left(\dfrac{61.38}{29}\right) + 3.75 \times \log\left(\dfrac{57.57}{47}\right)\right]$

$= 65.5\ mm$

Assume the foundation is rigid

$\delta_d = 0.82 \times 146.5 \times 1.5 \times \dfrac{(1-0.4^2)}{10 \times 10^3} = 15.1\ mm$

$\delta = 15.1 + 0.85 \times 65.5 = 70.8\ mm$ ※

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c) ① Erratic soil conditions can be easily overcome with the raft "bridging" over these conditions

② The continuity of the raft will reduce the differential settlement caused by uneven load distribution

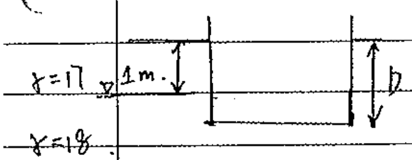
③ The greater weight and continuity of the raft provides resistance against lift.

d) $W_f = 16 \times D \times 0.5 \times 24 = 2304\ kN.$

$q = \frac{P + W_f}{A} = \frac{7500 + 2304}{16 \times 12} = 51.06\ kPa.$

$q = \sigma_{zD} = (D-1) \times 18 + 1 \times 17 = 51.06$

$\Rightarrow D = 2.9m$



5. a) ① Effective stress analysis / β method.

$f_s = \sigma_x' \cdot \tan\phi_f = K_0 (K/K_0) \tan[(\phi_f/\phi') \cdot \phi'] \cdot \sigma_z'$

$\beta = K_0 \cdot (K/K_0) \cdot \tan[(\phi_f/\phi') \cdot \phi']$

② Total stress analysis / α method

$f_s = \alpha \cdot C_u$

b) i) $f_c' = 30\ kN/mm^2$. $F_a = \frac{30}{4} = 7.5\ MPa.$

ii) Toe bearing

$q_t' = C_u \cdot N_c^*$

$C_u = 46\ kPa$. $\frac{N_c^* - 6.5}{46 - 25} = \frac{8 - 6.5}{50 - 25} \Rightarrow N_c^* = 7.76$

$\Rightarrow q_t' = 46 \times 7.76 = 356.94\ kPa$

Side friction

$\alpha = 1.0 - 0.5 \times \left(\frac{46-25}{50}\right) = 0.79$

$f_s = 0.79 \times 46 = 36.34\ kPa$

$\Rightarrow P_a = \frac{356.94 \times 0.3^2 + 36.34 \times 0.3 \times 4 \times 12}{3} = 185.14\ kPa.$ ✱

iii) $(f_s)_{upward} = 0.75 \times f_s = 27.26\ kPa$

$(P_{upword})_a = \frac{27.26 \times 0.3 \times 4 \times 12}{5} = 78.51\ kPa.$ ✱

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c) i) Work done = Energy input - Energy loss.

$$P_{ult} \cdot S = W_r \cdot h - P_{ult} \cdot C$$

$$W_r h = P_{ult}(S+C) = P_a \cdot F(S+C)$$

$$\Rightarrow P_a = \frac{W_r h}{F(S+C)}$$

ii) $S = \cancel{6mm}$ 0.006 /blow $C = 0.025\,m$.

$W_r h = 40\,kN.m$ $F = 6$.

$$\Rightarrow P_a = \frac{40\,kN \cdot m}{6 \times (0.006 + 0.025)\,m} = 215.1\,kN$$ #