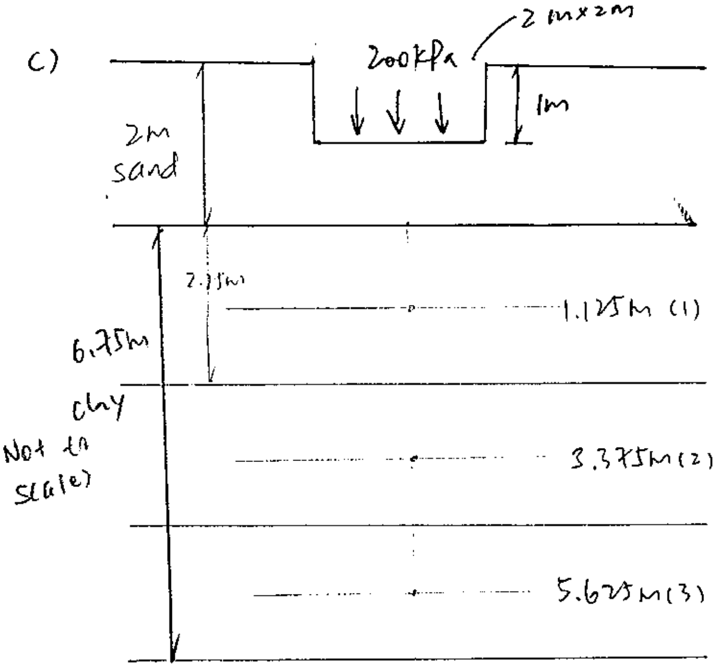
CV3013 - Foundation Engineering . AY 2013-2014 Sem 1

1). ① Strength - such that its capacity or resistance is sufficient to support the loads applied (i.e. so that it doesn't collapse)
② Serviceability - to avoid excessive deformation under these loads, which might damage the supported structure or lead to a loss of function.
other considerations: Cost and constructibility.

b) $q_f = S_c N_c C_u + \sigma_q$
for strip footing, $S_c = 1$.
$\sigma_q = \gamma \cdot d = 18 \times 1 = 18\ kN/m^2$. $C_u = 20kPa$
$q_f = 1 \times (2+\pi) \times 20 + 18 = 120.8\ kPa$.

c)

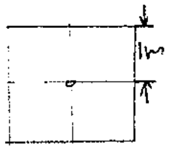


① immediate settlement:
$B = L = 2m$ $d = 1m$ $H = 2+6.75-1 = 7.75m$ $q = 200\ kPa$
using principle of superposition:
① $\frac{d}{B} = \frac{1}{2} = 0.5 \rightarrow \mu_0 = 0.94$. $\frac{L}{B} = 1$, $\frac{H}{B} = \frac{7.75}{2} = 3.875 \rightarrow \mu_1 = 0.62$
$S_{i1} = \mu_0 \mu_1 \cdot \frac{qB}{E} = 0.94 \times 0.62 \times \frac{200 \times 2}{(200 \times 30)} = 38.85mm$
② $\frac{d}{B} = \frac{1}{2} = 0.5 \rightarrow \mu_0 = 0.94$ $\frac{L}{B} = 1$, $\frac{H}{B} = \frac{2-1}{2} = 0.5 \rightarrow \mu_1 = 0.17$
$S_{i2} = \mu_0 \mu_1 \cdot \frac{qC}{E} = 0.94 \times 0.17 \times \frac{200 \times 2}{(200 \times 30)} = 10.65mm$
By superposition, $S_i = S_{i1} - S_{i2} = 38.85 - 10.65 = 28.2mm$.

② consolidation settlement:

divide the clay layer into 3 equal layers and use Fadum's chart:

$\frac{6.75}{3} = 2.25\,m$



$\Delta\sigma' = q \cdot I_{qr} = 4 \times 200 \times I_{qr}$ (kPa)

$\Delta S_{oed} = m_v \cdot \Delta\sigma' h = 0.2 \cdot \Delta\sigma' \cdot 2.25 = 0.45\,\Delta\sigma'$ (mm)

Layer	z (m)	m, n	I_{qr}	$\Delta\sigma'$ (kPa)	S_{oed} (mm)
1	1.125	0.889	0.16	128	57.6
2	3.375	0.296	0.035	28	12.6
3	5.625	0.178	0.015	12	5.4
				ΣS_{oed}	75.6

Using Scott's chart (1963).

equating area $B^2 = \frac{\pi D^2}{4} \rightarrow D = \sqrt{\frac{4B^2}{\pi}} = \sqrt{\frac{4\times4}{\pi}} = 2.257\,m$

$\frac{H}{D} = \frac{6.75}{2.257} = 2.99 \rightarrow \mu_c = 0.7$

(A = ab)

consolidation settlement, $S_c = \mu_c S_{oed} = 0.7 \times 75.6 = 52.92\,mm$

2.

a) Both methods can be used to find shaft resistance of piles.

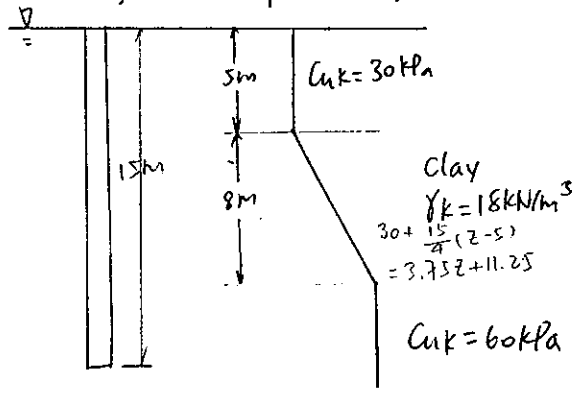
α method: (or Total Stress Method).

$\tau_{int} = \alpha c_u$, where c_u is the undrained shear strength.

β method: (or Effective Stress Method).

$\tau_{int} = \beta \sigma_z'$ → used in the case of Drained Soil.

b) 0.8 m ϕ bored pile → non-displacement pile. $\gamma_k = 24\,kN/m^3$. EC7 DA1b.



For EC7 DA1b, design material parameters are:

$\gamma_{cu} = 1.0 \quad \gamma_\gamma = 1.0$

$c_{u1,des} = \left(\frac{c_{u1}}{\gamma_{cu}}\right) = 30\,kPa$

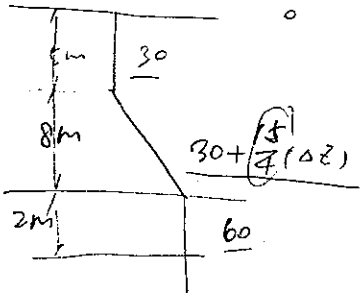
$\gamma_{k,des} = \frac{18}{\gamma_\gamma} = 18\,kN/m^3$

$c_{u2,des} = \left(\frac{c_{u2}}{\gamma_{cu}}\right) = 3.75z + 11.25$

$c_{u3,des} = \left(\frac{c_{u3}}{\gamma_{cu}}\right) = 60\,kPa$

End bearing:

① upper layer: (type 2) undrained shear strength profile.



$q_{bu} = S_c\left[(2+\pi)\cdot c_u + \frac{CB}{4}\right]F_z + \sigma_q$.

$S_c = 1+0.2\frac{B}{L} = 1+0.2\times1 = 1.2$

$\frac{CB}{c_{u0}} = \frac{0.8\times\frac{15}{4}}{30} = 0.1$. from chart, $F_z \approx 1$.

$c_u = 60$ $\quad CB = c_{u0}\cdot 0.1 = 3$.

$q_{bu} = 1.2\left[(2+\pi)\times60+\frac{3}{4}\right]\times1+(5+8)\times18$

$= 370.98 + 234$

$= 605$ kPa

② lower layer. uniform undrain shear strength.

$d = 15-(5+8) = 2m$

$d/B = 2/0.8 = 2.5$, $B/L = 1$ from Skempton's chart, $N_c = 8.6$

$S_c = 1.2$ $\quad S_c\cdot N_c = 10.32 \rightarrow$ limit to 9

$q_{bu} = S_c N_c \cdot c_u + \sigma_q = 9\times60+2\times18 = 576$ kPa

$\therefore Q_{bu,des} = \frac{A_p\cdot(q_{bu1}+q_{bu2})}{\gamma_{Rb}}$

$= \frac{\pi\times\frac{0.8^2}{4}\times(605+576)}{2} = 296.7$ kN

Shaft resistance:

for non-displacement piles,

① 0-5m: $c_u = 30$ kPa, $\alpha_1 = 1$

② 5-13m: $c_u = 30+\frac{15}{4}(z-5) = 11.25+3.75z$

$\alpha_2 = 1.16 - \frac{c_u}{185} = 1.16 - 0.06081 - 0.02027z$

$= 1.1 - 0.02z$.

③ 13-15m: $c_u = 60$ kPa. $\alpha_3 = 1.16 - \frac{60}{185} = 0.836$

$Q_{su,des} = \frac{\pi D_0 L_p \alpha\cdot c_{u,des}}{\gamma_{RS} \rightarrow 1.6}$

①: $\pi\times0.8\times5\times1\times30 = 376.8$

②: $\pi\times0.8\times\int_5^{13}\tau\, dz = \pi\times0.8\times\int_5^{13}(1.1-0.02z)(11.25+3.75z)\,dz$

$= 0.8\pi\times328 = 824$

③: $\pi\times0.8\times2\times60\times0.836 = 252.13$

$Q_{su,des} = \frac{(376.8+824+252.13)}{1.6} = 908.1$ kN

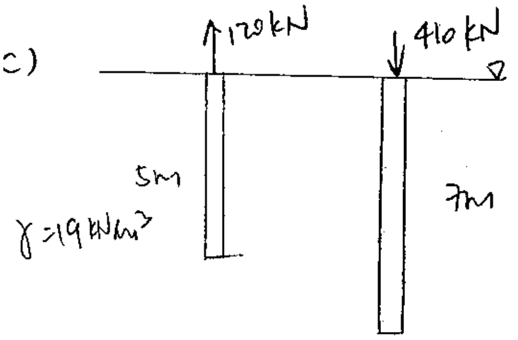
$\therefore R = Q_{bu,des}+Q_{su,des} = 296.7+908.1 = 1204.8$ kN

To satisfy ULS,

Q_A + self-weight of pile $\le R$. $\quad (Q+24\pi\cdot(\frac{0.8^2}{4})\times15)\times1.0 \le 1204.8$ ③

$Q \le 1023.8$ kN

c)



$\square$ 0.2×0.2

for tension pile. $Q_{su} = (4 \times 0.2 \times 5) \times \tau_{int,} = 4 \times \beta \cdot \sigma'_{vo} = 4 \times \beta \times (19-9.81) \times \frac{5}{2}$

$= 91.9\beta = 120kN$

$\therefore \beta = 1.306$

for compression pile, $Q_{su} = 4 \times 0.2 \times 7 \times \tau_{int\,2}$

$70\% \cdot Q_{su,c} = Q_{su,t}$

$\therefore Q_{su,c} = 171.43kN$

$Q_{bu} = 410 - 171.43 = 238.57\ kN$

3. a) EC7 DA1b:

$\gamma_{\phi'} = 1.25$ load: $1.0 \rightarrow \gamma_{G,stb}$

$\gamma_{c'} = 1.25$ $1.3 \rightarrow \gamma_{G,dstb.}$

$\gamma_{cu} = 1.4$ $\phi_d' = \tan^{-1}\left(\frac{\tan\phi_k}{\gamma_\phi}\right) = \tan^{-1}\left(\frac{\tan 38^\circ}{1.25}\right) = 32^\circ$ ← Fill behind the wall

$\gamma_{qu} = 1.4$

$c'_R = 0$

$k_a = \frac{1-\sin\phi_d'}{1+\sin\phi_d'} = 0.3072$

Fill beneath the wall: $c' = 0$

$\delta_d = \tan^{-1}\left(\frac{\tan\delta_k}{1.25}\right) = 23.043^\circ$ $\phi_d' = \tan^{-1}\left(\frac{\tan\phi_k}{\gamma_\phi}\right) = \tan^{-1}\left(\frac{\tan 34^\circ}{1.25}\right) = 28.352^\circ$

$k_a = \frac{1-\sin\phi_d'}{1+\sin\phi_d'} = 0.356$

$G < \begin{matrix} u & 1.0 \\ F & 1.0 \end{matrix}$

$Q < \begin{matrix} u & 1.3 \\ F & 0 \end{matrix}$

b). Thrust from active earth pressure

$P_{ad} = \frac{1}{2} \times 0.3072 \times 20 \times 6.8^2 \times \underbrace{\gamma_{G,dstb}}_{1.0} = 142.05kN$

Thrust from surcharge:

$P_{ad,} = 0.3072 \times 10 \times 6.8 \times \underbrace{\gamma_{Q,dstb}}_{1.3} = 27.1565\ kN$

stem: $(6+0.8) \times 0.4 \times 25 \times 1.0 = 68\ kN$

base: $(5-0.4) \times 0.4 \times 25 \times 1.0 = 46\ kN$

Soil on heel: $4.6 \times 6.4 \times 20 \times 1.0 = 588.8kN$

$R_v = 702.8kN$

design resistance $= R_{v,d} \cdot \tan\delta = 702.8 \times \tan 23.043^\circ = 299\ kN$

for vertical action acting on the base:

assumptions: Surcharge is favourable variable action,

d) Horizontal sliding:

$\Sigma F_H = 142.05 + 27.1565 = 169.2$ kN

Resistance = 299 kN

$\frac{299}{169.2} = 1.767$ satisfied

e) Destablising moment:

$M_{dst} = M_{pa} + M_{pq} = 27.1565 \times 3.4 + 2.767 \times 142.05 = 414.38835$

Stablising moment:

$M_{stb} = 68 \times 0.2 + 46 \times (0.4 + \frac{1}{2} \times 4.6) + 588.8 \times (0.4 + \frac{1}{2} \times 4.6)$

$= 1727.56$ kNm

$x = 1.87$ m $\quad e = \frac{5}{2} - 1.87 = 0.6315$ (within middle 1/3)

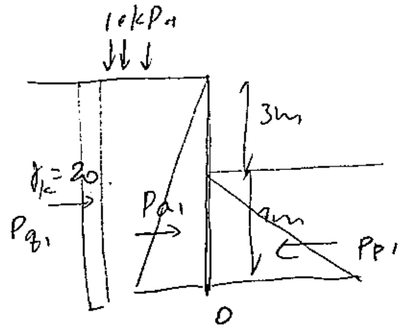
f) bearing failure:

R_v is permanent unfavourable action.

. the position of R is determined by dividing net moment M at any point along the base by V. (Moment usually is taken about toe or centre of the base).

. To ensure that base pressure remains compressive over the entire base width, R must act within middle one third of the base:

i.e. $e < B/6$.



4.(a) $\delta_k = \frac{2}{3}\phi'_k$. EC7 DA1b.

(i) design $\phi' = \tan^{-1}(\frac{\tan 37^\circ}{1.25}) = 31.1^\circ$ $\gamma_{des} = \frac{20}{1} = 20$.

for $\delta/\phi' = 0.66$. from chart,

$K_a = 0.27 \quad K_p = 5.2$.

behind: $P_{a,1} = \frac{1}{2} \times K_a \cdot \gamma \cdot (4+3)^2 = 132.3$ kN

$P_{q,1} = K_a \cdot q \cdot h = 0.27 \times 10 \times 7 = 18.9$ kN

infront of the wall: $P_{p,1} = \frac{1}{2} K_p \gamma h^2 = \frac{1}{2} \times 5.2 \times 20 \times 4^2 = 832$ kN

$\Sigma M_{dstb} = 1.3 \times P_{q1} \times \frac{7}{2} + P_{a1} \times \frac{1}{3} \times 7 = 395$ kNm

$\Sigma M_{stb} = P_{p,1} \times \frac{4}{3} = 1109.3$ kNm permanent, fav.

$\Sigma M_{stb} > \Sigma M_{dstb}$, will not fail by rotation about the toe

(ii) $\Sigma F_{stb,H} = P_{p1} = 832$ kN

$\Sigma F_{dstb,H} = 1.3 \times 18.9 + 1 \times 132.3 = 378$ kN

$F_{stb} > F_{dstb}$, will not fail by lack of horizontal equilibrium.

4. (b)

(i) SPT :- equipment used: hammer, rod, split-spoon, anvil slide

- The split-spoon is driven into the borehole by the hammer tied to the rod. Number of blow count needed to drive the split tube into certain penetration depth will be recorded as SPT-N value.
- measurements obtained: Penetration resistance "N".
  for sand: related to relative density and friction angle.
  for clay: related to undrained shear strength and compressibility

CPT: - equipment used: rod, friction sleeve, load cell, cone

- Cone is pushed into underlying ground at certain rate. Resistance generated by the tip and friction generated by rod string are recorded.
- measurements: Skin friction f_s, Cone resistance q_t, , pore pressure u_s.
  interprete soil behavior type
  for sand: relative density and friction angle
  for Clay & silt: undrained shear strength and preconsolidation pressure.

(ii) From ~~CB~~ CPT, continuous soil profile could be obtained and soil type could be estimated. CPT is superior to SPT due to its versatility and wealth of information on the subsoil.

(iii) The reasons that SPT is still frequently carried out today are:

- it's low cost and simple to carry out
- SPT could be conducted on very stiff clay and very dense sand while CPT could not.

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