University Of Technology
Building and Construction Eng. Dept.
Final Exam 1st Attempt –2015/2016

Subject : Foundation Eng. Class: 4th Year
Branch : All Branches. Time : 3.0 Hours
Examiner :Foundation Eng. Committee Date : 21/5/ 2016

Note: Attempt (4) Questions only

Q1- For the structure of the grain silo shown in figure (1), two proposals for its foundation are presented.
For a F.S =3 and an allowable settlement of 20mm.Check the suitability of foundation in terms of bearing capacity and settlement for proposal (1). (25 Marks)

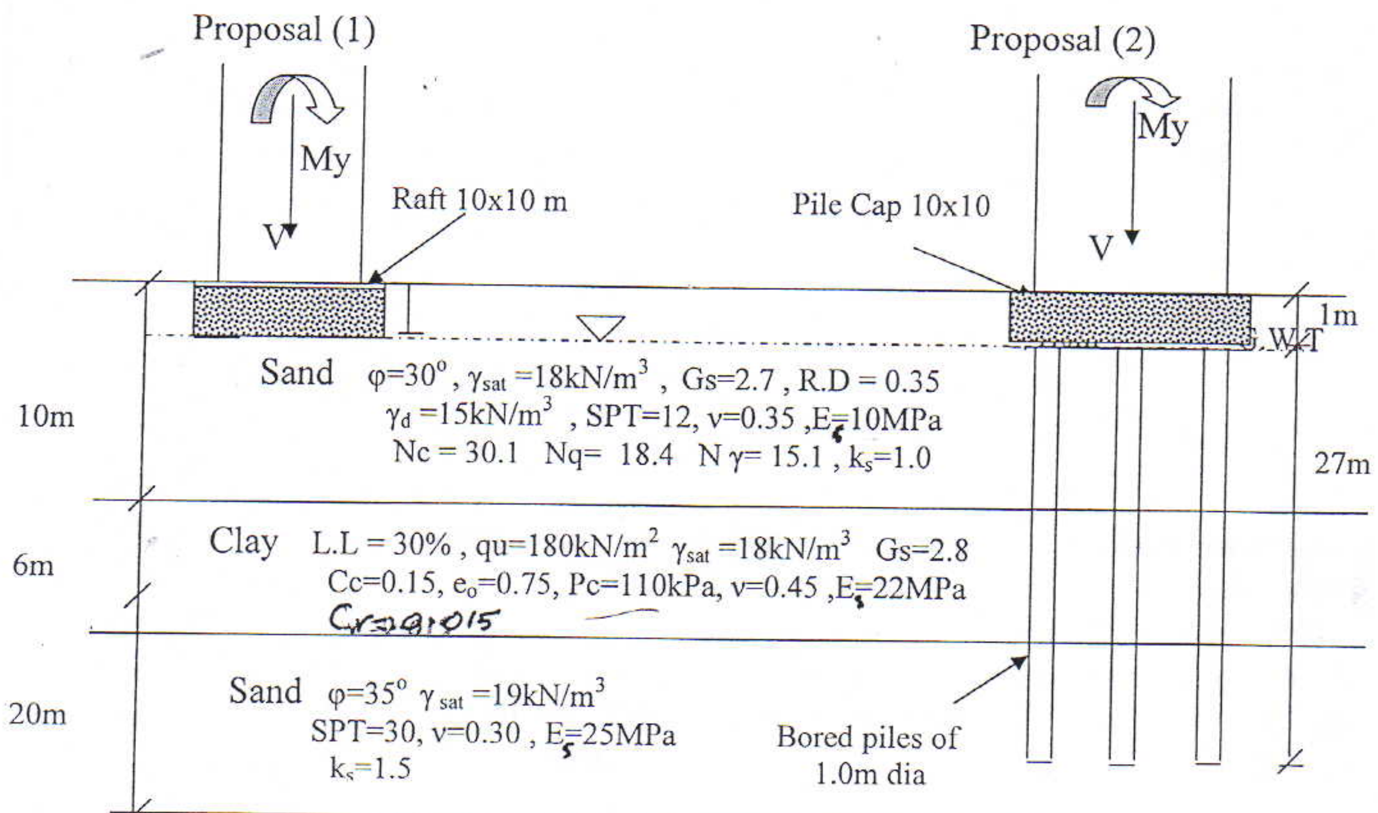
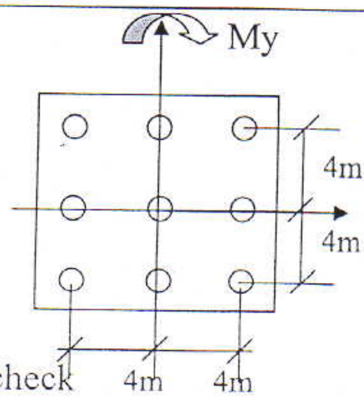


Fig (1)

If V = 15000 kN
My = 30000 kN.m
Note : Weight of foundation or cap neglected



Q2- For the Pile foundation of proposal (2) of fig(1) with a F.S=3.0 and allowable settlement of 20mm, check it's suitability in terms of bearing capacity and settlement only. (25 Marks)
If E_p =21 x10⁶ kPa.

Q3 : A- The spread footings shown in Figure (2) are subjected to a centric total load of 4000 kN and moment about y-axis of 1000 kN.m. Determine the depth of boring. (15 Marks)

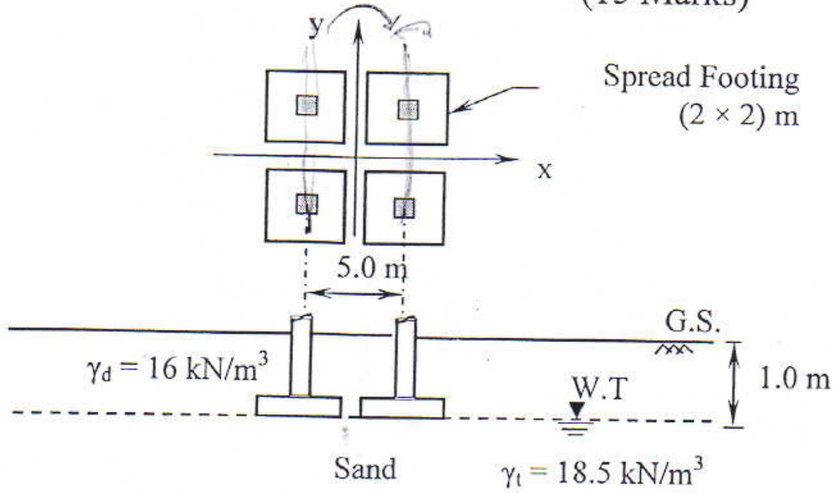
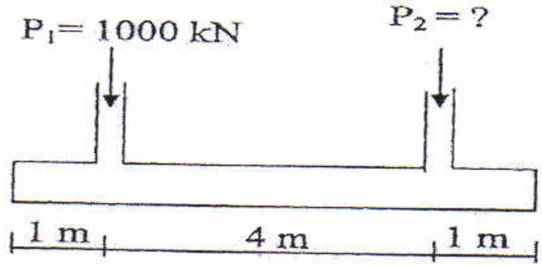


Fig.(2)

B- For the rectangular footing (6×3)m shown in figure (3), find the maximum value of load on column (P_2) if soil pressure is uniform and the allowable bearing capacity is 140 kPa. (10 Marks)



Fig(3)

Q4- A- (0.3× 0.3) m reinforced concrete pile of 20 m length is driven through loose sand and then into gravel to a final set of 3 mm/blow using 30 kN single acting hammer with a stroke of 1.5 m. Determine the ultimate driving resistance (Ru) of the pile if it is fixed with a helmet, plastic dolly and 50 mm of packing on the top of the pile. The weight of the (**Pile + helmet and dolly**) is 78 kN. (k = 0.9 and e = 0.4 for single acting hammer. (15 Marks)

B- For the pile group shown in fig(4) if pile no.2 was broken what will be the load on pile no.5. If the total load on pile cap 3000kN, and pile dia. 0.4m installed with in clay layer with spacing S of (3D). (10 Marks)

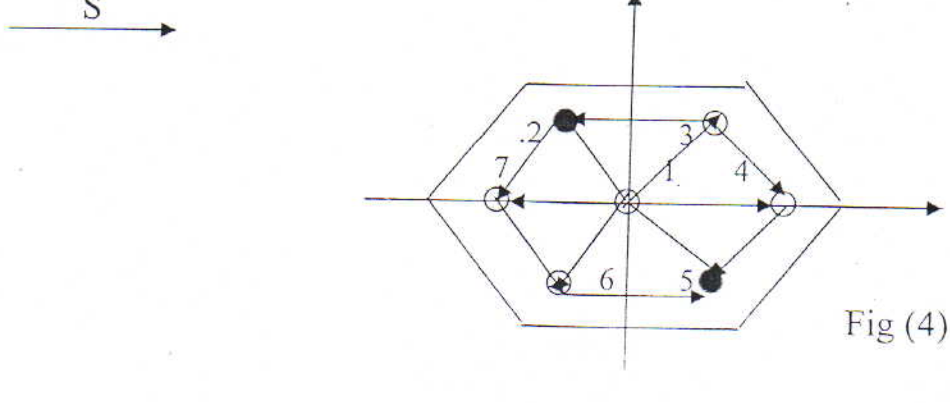
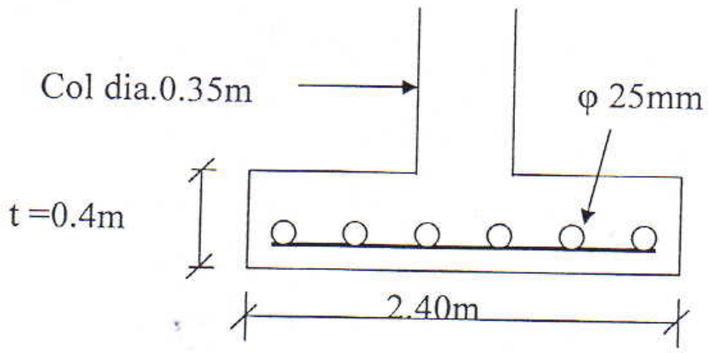


Fig (4)

Q5- A- Check the adequacy of footing thikness *only* for the square footing shown in fig (5) for the following conditions:
D.L= 320kN , L.L = 310kN, fy = 350MPa , Fc' =20MPa, q_{all} =120kN/m^2
(10 Marks)

Fig (5)



b- For the sheet pile of 9.5m length and soil profile shown in Figure (6) check the suitability of penetration depth (15 Marks)

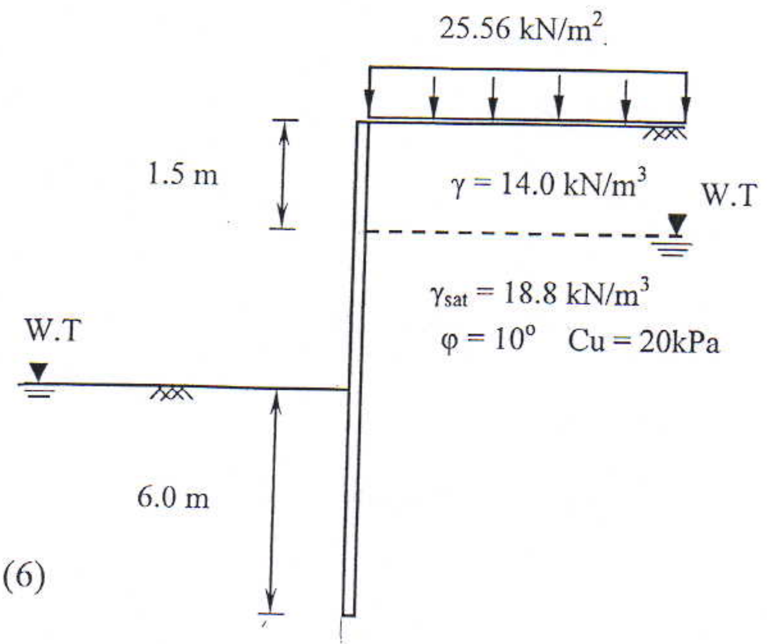


Fig. (6)

$q_{ult} = CNc\ Sc\ dc + q\ Nq\ Sq\ dq + 0.5\ \gamma\ B\ N\gamma\ S\gamma\ d\gamma$, $H = 0.5 B \tan(45 + \varphi/2)$

$Sc = 1 + (Nq/Nc)(B'/L')$, $Sq = 1 + (B'/L')\sin\varphi$, $S\gamma = 1 - 0.4\ B'/L'$

$dc = 1 + 0.4D/B$, $dq = 1 + 2\tan\varphi(1-\sin\varphi)^2 D/B$, $d\gamma = 1$

for $\varphi = 0$, $q_{ult} = 5.14\ Cu(1 + Sc' + dc') + q'$ $Sc' = 0.2\ B/L$, $dc' = 0.4\ D/B$

$$Sc = \frac{Cc}{1+e_o} Ho \log \frac{P_o' + \Delta P}{P_o'} \text{ For Nc.c , } Si = \frac{q\,B\,(1-\mu^2)}{Es} I\,p \text{ for } H > 2B$$

$$\delta = S_1 = \frac{(Q_{wb} + 0.5\,Q_{ws})\,L}{A_p\ E_p} , S_2 = \frac{Q_{wb}\,B\,(1-\nu^2)}{A_b\ E_s} I_p , S_3 = \frac{Q_{ws}\,B\,(1-\nu^2)}{A_s\ E_s} I_w$$

$I_w = (2 + 0.35\sqrt{D/B})$ $S_g = \sqrt{(W_g / B\ S_{total})}$

$$Sc = \frac{C_r}{1+e_o} Ho \log \frac{P_c}{P_o'} + \frac{C_c}{1+e_o} Ho \log \frac{P_o' + \Delta P}{P_c} \text{ For O.c.c If } P_o' < P_c < P_o' + \Delta P$$

$$Si = \frac{H}{Cs} \text{Log}_e \frac{P_o' + \Delta P}{P_o'} , Cs = \frac{1.5\ Cr}{P_o'} , Si = \frac{q\,B\,(1-\mu^2)\,\mu_o\,\mu_1}{Es} \text{ for } H < 2B$$

$$T_v = \frac{Cv\,.\,t}{d^2} , U = \sqrt{\frac{4.\,Tv}{\pi}} \text{ , For } U < 60\% , Tv = \pi/4\ (Uz/100)^2 \text{ For } U > 60\%$$

$$, Ss = C\alpha\ H \log \frac{t_1 + \Delta t}{t_1} , Cc = 0.009(L.L - 10)$$

$$Si = C1.C2.\ \Delta P \sum (Iz/2Cr)\,\Delta z , C1 = 1 - 0.5 \frac{Po'}{\Delta P} , C2 = 1 + 0.2 \log \frac{t}{0.1}$$

$$Xc = \frac{L}{3}\left(\frac{2b_1 + b_2}{b_1 + b_2}\right) , q_{max} = \frac{\Sigma V}{L.B}\left(1 + \frac{6e_x}{L} + \frac{6e_y}{B}\right) \text{ for } e < \frac{B}{6}$$

$$q_{max} = \frac{2\Sigma V}{L.B} \text{ for } e = \frac{B}{6} , q_{max} = \frac{4\,Q}{3L\,(B - 2e)} \text{ for } e > \frac{B}{2}$$

$B' = B - 2e_y$ $L' = L - 2e_x$

$$Z_1 = \frac{1}{2}\sqrt{\frac{v_2 - v_1}{v_2 + v_1}} \cdot Xc \ , \quad Z_2 = \frac{1}{2}\left[T_{i2} - 2Z_1 \frac{\sqrt{v_3^2 - v_1^2}}{v_3 \cdot v_1}\right] \frac{v_3 \cdot v_2}{\sqrt{v_3^2 - v_2^2}}$$

$Q_{ult} = C_u \cdot N_c A_b + C_a \cdot A_s$ For piles in Cohesive Soils

$Q_{ult} = \sigma'_v \, Nq \, A_b + Ks \, \sigma_{v(av)} \tan\delta \, As$ For piles in Cohesionless soils

$$\epsilon g = 1 - \frac{\theta}{90}\left[\frac{(n-1)m + (m-1)n}{mn}\right] \ , \ \theta = \tan^{-1}\frac{B}{S}$$

$Q_{ult(group)} = Q_{ult\,(Single)} \times n \times \epsilon g$,

$Q_{ult(group)Block} = 2(Bg + Lg) \times L \times Cu' + Cu \times Nc \times Bg \times Lg$

$$Q_{pile} = \frac{\Sigma V}{n} + \frac{Mx \cdot Y}{\Sigma y^2} + \frac{My \cdot X}{\Sigma x^2}$$

$P_a = Ka\,\gamma\,H - 2c\sqrt{Ka}$, $P_p = Kp\,\gamma\,H + 2c\sqrt{Kp}$

$$F.S\ (Sliding) = \frac{\Sigma P_p}{\Sigma P_a}, \qquad F.S\ (\text{overturining}) = \frac{\Sigma M_p}{\Sigma M_a}$$

$$Ru = \frac{W \times h \times \acute{\eta}}{S + C/2} \qquad \gamma_{sat} = \frac{Gs + S \cdot e}{1 + e}\gamma_w \ , \quad \gamma_d = \frac{Gs \cdot \gamma_w}{1 + e}$$

$Pu = 1.4\ D.L + 1.7\ L.L$, $Vc = 0.08\,\varphi\left(2 + \frac{4}{B_c}\right)\sqrt{fc'} < 0.34\,\varphi\sqrt{fc'}$ for two way shear

$$d^2\left(Vc + \frac{q_u}{4}\right) + d\left(Vc + \frac{q_u}{2}\right)a = \frac{B^2 - a^2}{4} q_u$$

$Vc = 0.17\,\varphi\sqrt{fc'}$ for one way shear

$$Vc\,b\,d = q_u\left(\frac{B}{2} - \frac{a}{2} - d\right)B$$

Table Influence factors (I_r) for vertical displacement due to elastic compression

Shape	*Flexible** *Centre*	*Corner*	*Average*	*Rigid*†
Circle	1.00	0.64	0.85	0.79
Rectangle				
$\frac{L}{B}$ 1.0	1.122	0.561	0.946	0.82
1.5	1.358	0.679	1.148	1.06
2.0	1.532	0.766	1.300	1.20
3.0	1.783	0.892	1.527	1.42
4.0	1.964	0.982	1.694	1.58
5.0	2.105	1.052	1.826	1.70
10.0	2.540	1.270	2.246	2.10
100.0	4.010	2.005	3.693	3.47

* After Giroud (1968)
† After Skempton (1951)

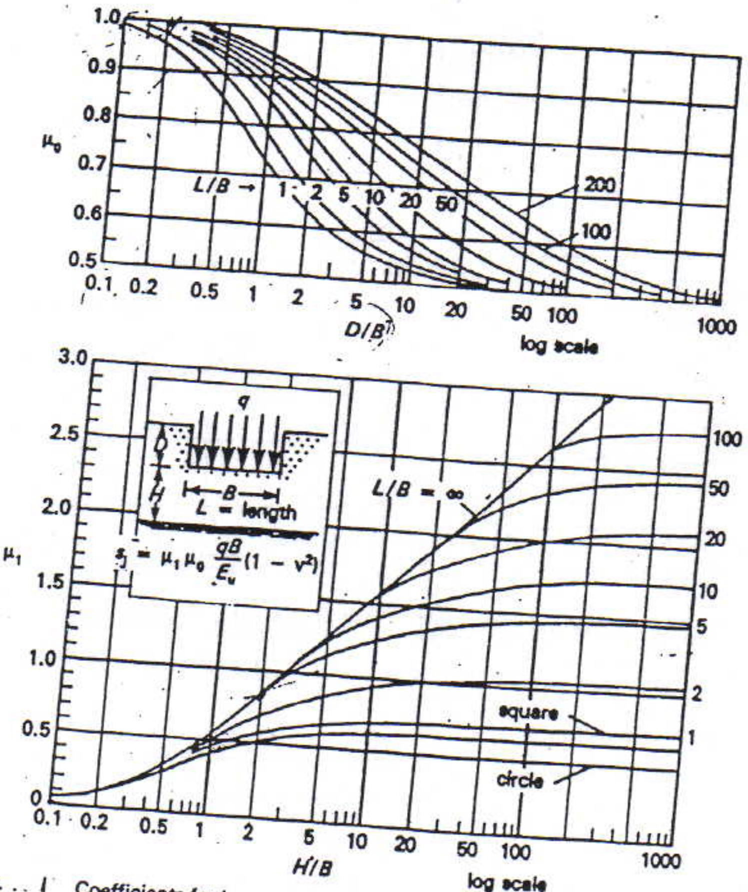


Fig. 1 Coefficients for immediate settlement under a flexible foundation (After Janbu *et al.* 1956)

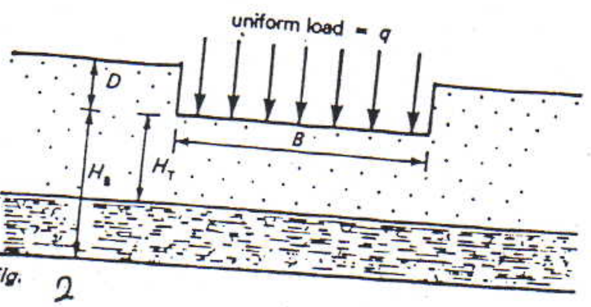
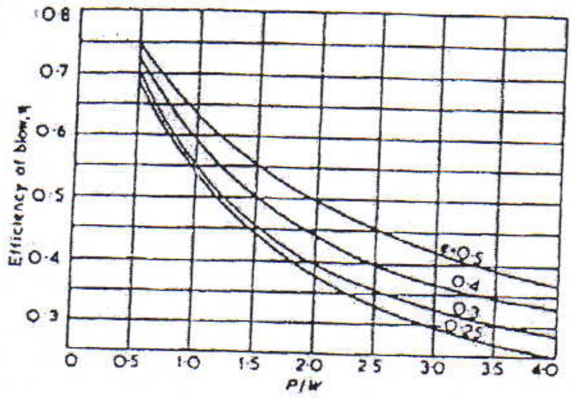
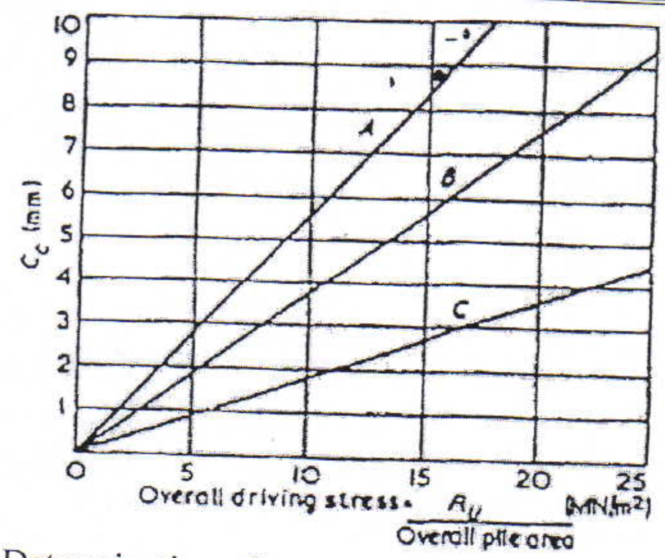


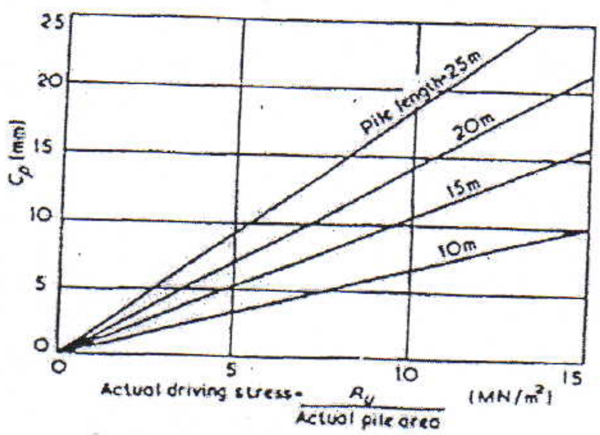
Fig. 2



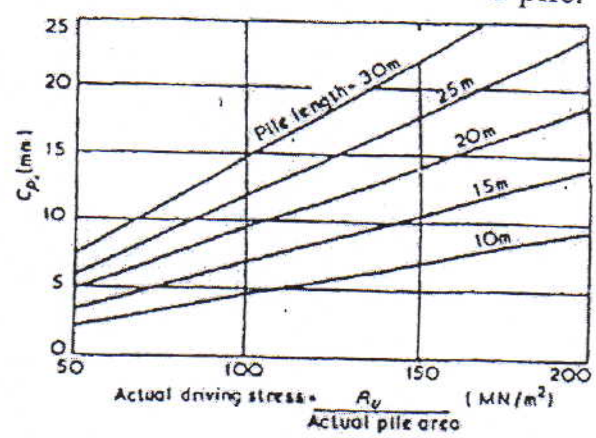
Determination of efficiency factor, η for use in Hiley pile driving formula.



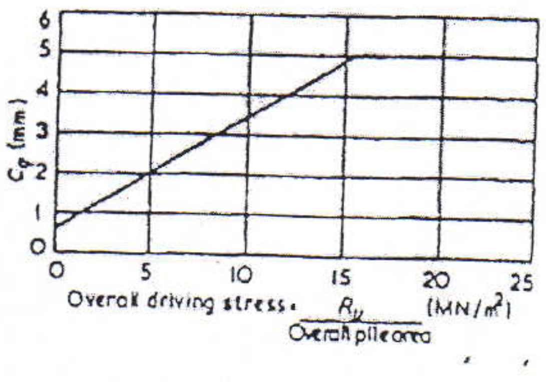
Determination of temporary elastic compression, Cc.
Key: A = concrete pile, 75 mm packing under helmet, B = concrete or steel pile, helmet with dolly or helmet of timber pile. C = 25 m pad only on head of RC pile.



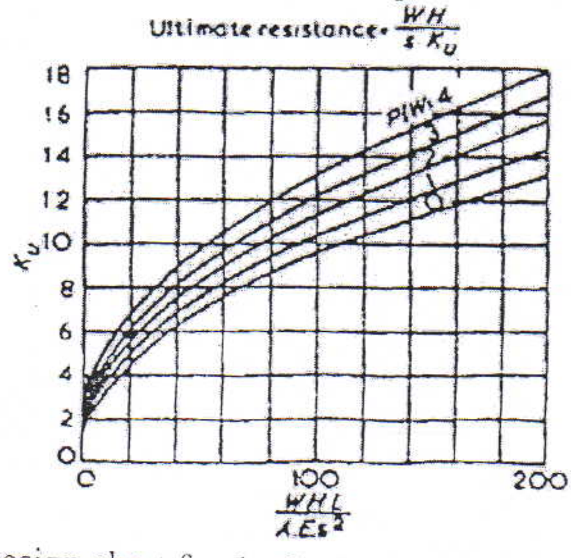
Determination of temporary elastic compression, Cp for concrete piles.



Determination of temporary elastic compression, Cp for steel piles.



Determination of temporary elastic compression, Cq.



Design chart for the Janbu pile driving formula.

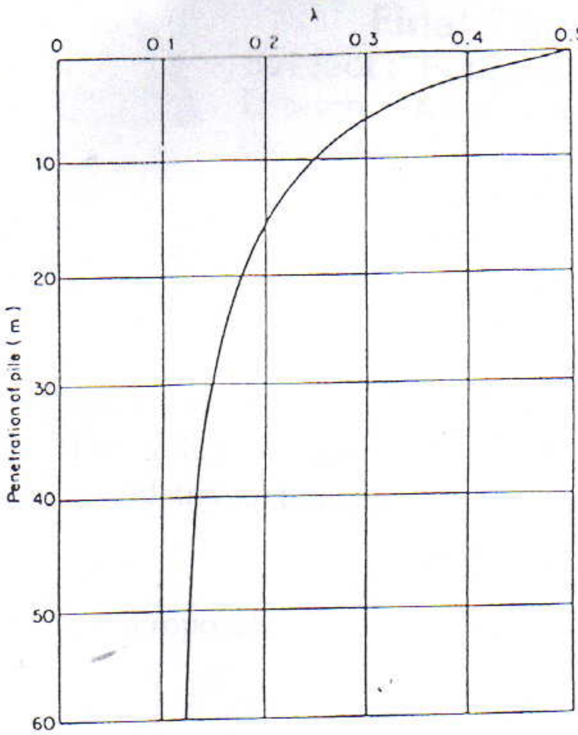
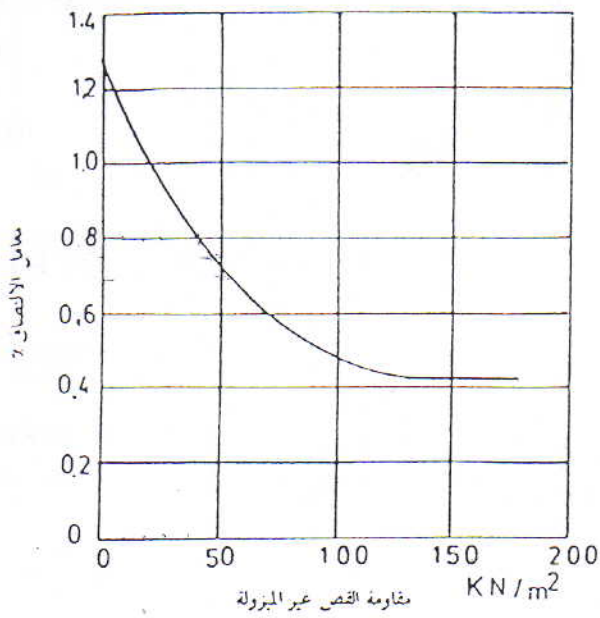


Fig. 2 Values of coefficient λ for various penetration depths of pile driven into cohesive soils (after Vijayvergiya and Focht[7.17]).



الشكل - 1 7العلاقة بين معامل الالتصاق . α . ومقاومة القص غير المزولة للاطيان (توملسون . 69

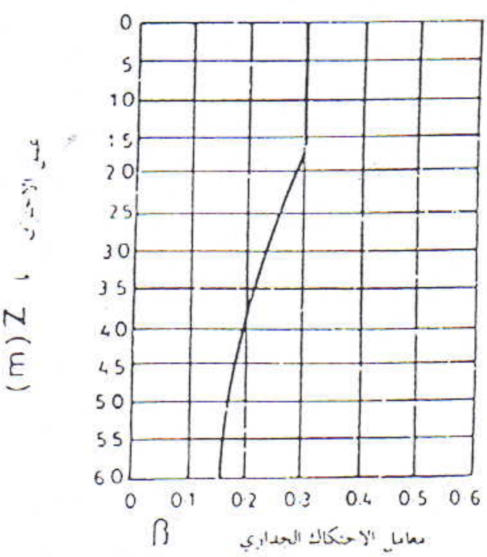


Fig 3 : معامل الاحتكاك الجداري . β . كما في اطيان رخوة ومتوسطة

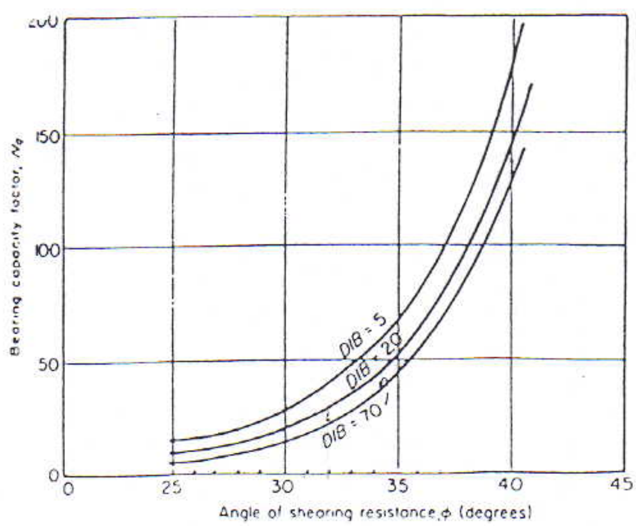


Fig 4 Berezantev's bearing capacity factor, N_q.

حل الأسئلة النموذجية لامتحان مادة
هندسة الأسس للدور الأول للعام ٢٠١٥ - ٢٠١٦

$Q_1)$ $H = 0.5 \times B \tan(45 + \phi/2)$

$= 0.5 \times 10 \tan(45 + 30/2) = 8.66 < 10\,m$

$\therefore$ it one layer

For $C = 0 \to 0$

$q_{ult} = c N_c \cancel{S_c} d_c + q N_q S_q d_q + 0.5 \gamma \bar{B} N_\gamma S_\gamma d_\gamma$

for $\phi = 30$, $N_c = 30.1$, $N_q = 18.4$, $N_\gamma = 15.1$

$q = \gamma D_f = 15 \times 1 = 15\ kN/m^2$

$\bar{B} = B - 2e$

$e_x = \frac{M}{V} = \frac{30000}{15000} = 2m$, $> B/6 = \frac{10}{6} = 1.66\,m$

$\bar{B} = 10 - 2\times 2 = 6\ m$

$L' = 10 - 0 = 10\ m$

$S_q = 1 + (\frac{B'}{L'})\sin\phi = 1 + (\frac{6}{10})\sin 30 = 1.3$

$d_q = 1 + 2\tan\phi(1-\sin\phi)^2 \frac{D}{B} =$

$d_q = 1 + 2\tan 30(1-\sin 30)^2 \frac{1}{10} = 1.02$

$S_\gamma = 1 - .4\frac{B'}{L'} = 1 - .4\frac{6}{10} = 0.76$

$d_\gamma = 1.0$

$q_{ult} = 0 + 15 * 18.4 * 1.3 * 1.02 + 0.5 * 8 \times 6 * 15.1 * .76 \times 1.0$

$= 0 + 365.9 + 275.4 = 541.3\ kN/m^2$

$q_{all} = \frac{541.3}{3} = 180.4 \ kN/m^2$

Since $e > \frac{B}{6}$ $\therefore q_{max} = \frac{4Q}{3L(B-2e)}$

$$q_{max\,(net)} = \frac{4 \times (15000 - Wt. of soil (15 \times 10 \times 10))}{3 \times 10 \ (10 - 2 \times 2)} = 300 \ kN/m^2$$

Since $q_{max} > q_{all}$ $\therefore$ it is not adequate.

Check Settlement

- First layer

$$S_i = \frac{q_{net} \ B (1 - \nu^2)}{E_s} \ I_p$$

$$S_i = \frac{300 \times 10 \ (1 - 0.35^2)}{10 \times 1000} \ 0.82 = 0.215 \ m$$

$= 215 \ mm > 20 \ mm$

$\therefore$ it is not adequate

Q_2 $Q_{ult\ single} = Q_s + Q_b$

Note δ for bored Pile $= \phi - 3$. N_q for $\phi = 32 = 25$

$$= 1\left(\frac{1+10}{2}\right) * 8 \tan(.75 \times 27) * \pi * 1 \times 9 +$$

$$0.45 * 90 * \pi \times 1 * 6 + 1.0\left(\frac{16+27}{2}\right) * 9 \times \tan(.75 \times 32)$$

$$* \pi * 1 * 9 + 27 * 9 * 25 \frac{\pi \times 1^2}{4} =$$

$$= 375.3 + 763.0 + 2434.6 + 4768.8$$

$$q_{all} = 3572.9 + 1589.6 = 5162.5 \text{ kN}$$

$$E_g = 1 - \frac{\theta}{90}\left(\frac{(n-1)m + (m-1)n}{mn}\right)$$

$$\theta = \tan^{-1}\frac{B}{S} = \tan^{-1}\frac{1}{4} = 14.03$$

$$E_g = 1 - \frac{14.03}{90}\left[\frac{(3-1)3 + (3-1)3}{3 \times 3}\right] = 0.793$$

$$Q_{ult\ group} = 5162.5 * 9 * 0.793 = 36844.1 \text{ kN}$$

$$Q_{max} = \frac{15000}{9} + 0 + \frac{30000 * 4}{6 \times 4^2} = 2916.6 \text{ kN}$$

< 5162.5 kN

$\therefore$ o.k.

Check Elastic Settlement

$$S_T = S_1 + S_2 + S_3$$

$$S_1 = \frac{(1589.6 + 0.5 * 3572.9)\,27}{\frac{\pi \times 1^2}{4} * 21 \times 10^6} = 0.0055 \text{ m}$$

$$S_2 = \frac{1589.6 \times 1(1-0.35^2)}{\frac{\pi \times 1^2}{4} \times 25000} \times 0.79 = 0.0561 m$$

$$S_3 = \frac{3572.9 \times 1(1-0.35^2)}{\pi \times 1 \times 27 \times 25000}\left(2 + .35\sqrt{\frac{27}{1}}\right) = 0.0056 \ m$$

$$S_T = 0.0055 + 0.0561 + 0.0056 = 0.0671$$

$$S_g = \sqrt{\frac{10}{1 \times 0.0671}} = 12.2 \ mm < 20 \ mm \therefore o.k.$$

Q3 - depth of bore hole for single footing = $3 * B$

$= 3 \times 2 = 6 m$

- For heavy structure > 200 kN

The load each Footing from the axil load $= \frac{4000}{4} = 1000$ kN

The load on two footing on each side due to moment

$\frac{1000}{5} = 200$ kN

The on each footing $= \frac{200}{2} = 100$ kN

The total load on a single Footing $= 1000 + 100 = 1100$ kN

$q = \frac{1100}{2 \times 2} = 275$ kN > 200 ∴ heavy structure

Therefor Z for heavy structure $= 2B$

$Z = 2 \times 2 = 4 m$

- For deep Compressible Soil

- assume 2:1 method of stress distribution

$q_{net} * Area = 0.1 \, q_{net} (B + Z)^2$

$2 \times 2 = 0.1 (2 + Z)^2$

$Z = 4.3 m$

$- 0.05 P_o' = \frac{q_{net} * Area}{(B + Z)^2}$

$P_o' = 16 \times 1 + (18.5 - 10) Z = 16 + 8.5 Z$

$0.05 * (16 + 8.5 Z) = \frac{(275 - 16) \times 2 \times 2}{(2 + Z)^2}$

$Z = 11.5 m$

So use $Z = 6 m$ & Total depth $= 6 + 1 = 7 m$.

Q_3 b- For Uniform soil Pressure

$$\frac{P_1 + P_2}{A} = q_{all}$$

$$\frac{1000 + P_2}{6 \times 3} = 140$$

$$P_2 = 1520 \text{ kN}$$

For max value of P_2 of Non Uniform

$$q_{all} = \frac{\Sigma V}{A} \left(1 + \frac{6e}{L}\right)$$

$$e = \frac{\Sigma M}{\Sigma V} = \frac{(P_2 - 1000) \times 2}{P_2 + 1000}$$

$$\therefore 140 = \frac{P_2 + 1000}{6 \times 3} \left(1 + \frac{6\left(\frac{P_2 - 1000)2}{P_2 + 1000}\right)}{6}\right)$$

$$P_2 = 1173 \text{ kN}$$

Q4 A) $\frac{P}{W} = \frac{78}{30} = 2.6$

For $e = .4$ from Fig $\eta = 0.39$

$h = k * H = 0.9 * 1.5 = 1.35\ m$

Assume $R_u = 1000\ kN$

overall driven stress $= \frac{R_u}{A} = \frac{1000}{(0.3^2)} = 11111.1\ kN$
$= 11.1\ MPa$

From Fig $C_c = \frac{2}{3} \times 6 + 4 = 8\ mm$

$C_p = 15.5\ mm$

$C_q = 3.6\ mm$

Total $C = 27.14\ mm$

$\therefore R_u = \frac{30 * 1.35 * 1000 \times 0.39}{3 + \frac{27.14}{2}} = 953\ kN$

which is close enough to assumed R_u.

b- Since the Pile Configration is symmetry So,
the load on each Pile after Pile No. 2 Proken will be

$$S = 3 \times 0.4 = 1.2 m$$

$$X = 0.6 m$$

$$Y = \sqrt{1.2^2 - 0.6^2} = 1.04 m$$

$$\sum X^2 = 3 \times 0.6^2 + 2(1.2)^2 = 3.96$$

$$\sum Y^2 = 3 * (1.04)^2 = 3.24$$

$$M_y = 500 * 0.6 = 300 \ kN.m$$

$$M_x = 500 * 1.04 = 519.6 \ kN.m$$

$$P_s = \frac{3000}{6} + \frac{519.6(1.04)}{3.24} + \frac{300 \times .6}{3.96} = 712.2 \ kN$$

Q_5 -

$$A_r = \frac{320 + 310}{120} = 5.25\ m^2$$

$$B = \frac{5.25}{2.4} = 2.18\ m$$

Use $B = 2.2\ m$ & $L = 2.4\ m$

$$U = 1.4 * 320 + 1.7 * 310 = 975\ kN$$

$$q_{ult} = \frac{975}{2.2 * 2.4} = 184.66\ kN/m^2$$

Determin (d)

1- By two way action then check by beam action

$$V_c = 0.34 * 0.85 \sqrt{20} = 1.292\ MN/m^2$$
$$= 1292\ kN/m^2$$

$$b_o = (0.35 + d)\pi = 1.1 + 3.14d$$

$$V_c\, b_o\, d = q_{ult} * \text{Critical area}$$

$$V_c\,(1.1 + 3.14d)d = q_u\left[B \times L - (\pi * (a+d)^2 / 4)\right]$$

$$1421.2d + 4056.8d^2 = 184.6\left[2.2 \times 2.4 - (\pi(0.35+d)^2/4\right]$$

$$d^2 + 0.367d = 0.227$$

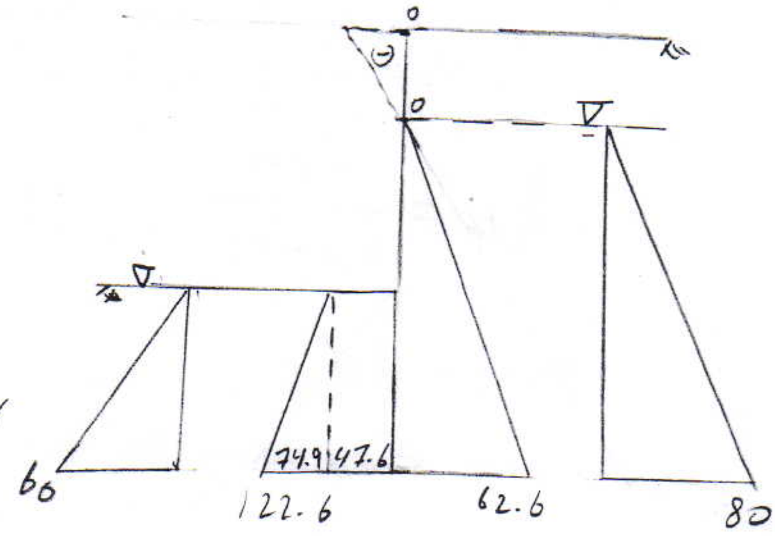
$d = 0.33\ m$ which govern.

$$t = 0.33 + 0.075 + 0.0125 = 0.417$$

Use $t = 0.42\ m$

b) $k_a = \frac{1-\sin 10}{1+\sin 10} = 0.704$

$kp = \frac{1}{k_a} = 1.42$



$P_{a_{9.5}} = k_a \gamma h - 2c\sqrt{k_a}$

$= .704 \times 25.56 - 2 \times 20\sqrt{.704}$

$= -15.56 \ KN/m^2$

$P_{a_8} = (25.56 + (1.5 \times 14)) .704 - 2 \times 20\sqrt{.704}$

$= -0.755 \ kN/m^2$ 33.56

$P_{a_8} = (25.56 + 21 + (8 \times 8.8)) \times .704 - 33.56 = 62.6 \ kN/m^2$

$P_{aw} = 8 \times 10 = 80 \ kN/m^2$

$P_{p_0} = 1.42 \times (18.8 - 10) \times 6 + 2 \times 20\sqrt{1.42} = 122.64 \ kN/m^2$

$P_{pw} = 6 \times 10 = 60 \ kN/m^2$

Total Active Pressure

$\sum A_{act} = 62.6 \times \frac{8}{2} + 80 \times \frac{8}{2} = 570.4$

$\sum A_{passive} = 47.6 \times 6 + 74.9 \times \frac{6}{2} + 60 \times \frac{6}{2} = 452.3$

$$\sum M_{active} = 62.6 \times 4 * \frac{8}{3} + 80 \times 4 \times \frac{8}{3} = 1521.0 \ kN.m$$

$$\sum M_{passive} = 47.6 \times 6 * 3 + 74.9 \times 3 * \frac{6}{3} + 60 \times 3 \times \frac{6}{3} = 1666.2$$

$$F.s_{sliding} = \frac{\sum \text{Area Passive}}{\sum \text{Area Active}} = \frac{452.3}{570.4} = 0.797 < 2 \quad \text{Not ok}$$

$$F.s_{overturing} = \frac{\sum M_{Passive}}{\sum M_{Active}} = \frac{1666.2}{1521.0} = 1.09 < 2 . \quad \text{Not ok.}$$