MIDDLE EAST TECHNICAL UNIVERSITY
Civil Engineering Department

FOUNDATION ENGINEERING II
CE462

TERM PROJECT

Group No: **8**

Group Members:

List of Symbols

ρ : Deflection

k_v : Vertical Spring Constant

k_H : Lateral Spring Constant

γ : Soil Unit Weight

φ : Angle of Friction

M : Moment

Q : Vertical Load

I : Moment of Inertia

σ' : Effective Vertical Pressure

α : Interaction factor

C_U : Undrained Shear Strength

E_S : Modulus of Elasticity of Soil

E_C : Modulus of Elasticity of Concrete

S : Distance between piles along x-x and y-y directions

B : Diameter of one pile

K_P : Coefficient of Passive Resistance

ν : Poisson' s Ratio of Soil

Contents

Introduction

As a term project, we are asked to design a pile foundation in order to carry comparatively excessive loads of superstructure. For our group, the soil profile is given as follows:

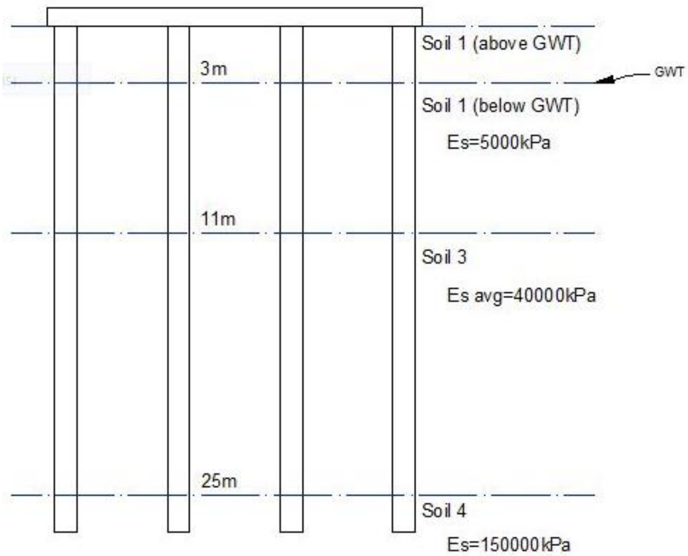


Figure 1: Given Soil Profile

However, we took weighted average of modulus of elasticity values and considered a modified soil profile while conducting in lateral load and deflection calculations.

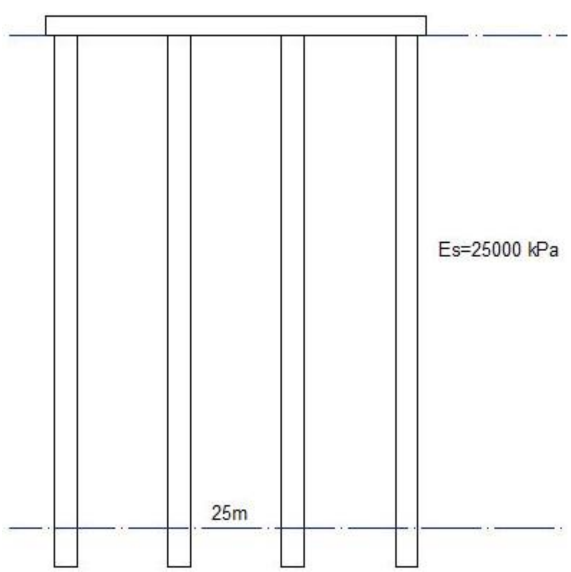


Figure 2: Modified Soil Profile to be used in Lateral Load & Deflection Calculations

4x4 pile grid and a square pilecap is used in design. Diameter of one pile is 1.2 m. One side of the cap is 20 m. long. Distances between piles are 6 m. along x-x and y-y directions.

Calculations

Load Calculations

To calculate total M,

$M_{wind} = 1000 \times 33 = 33000\ kNm$;

$M_{weight} = 12000 \times 2.5 = 30000\ kNm$; $\Longrightarrow$ $M_{total} = 103800\ kNm$

$M_{earthquake} = 24000 \times 17 = 40800\ kNm$; $H_{total} = 3400$ kN

$Q_{total} = 12000 + \underbrace{20 \times 20 \times 25}_{\text{Weight of pile cap}} = 22000\ kN$

Maximum Vertical Loads on Piles

$$Q_{max} = \frac{M * y}{I} + \frac{\sum Q_v}{n}$$

Where,

M = total moment. Moment is taken as it acts on diagonal direction so that the moment arm will have the maximum value.

y= maximum distance of a file from the centroids of the pile group

I=Inertia of the pile group along diagonal direction.

Q_v= total vertical load

n= number of piles

To calculate Inertia,

$$I = 2 * \left(9\sqrt{2}\right)^2 + 4 * \left(6\sqrt{2}\right)^2 + 6 * \left(3\sqrt{2}\right)^2$$

$$I = 720\ m^2$$

$$Q_{max} = \frac{103800 * 9\sqrt{2}}{720} + \frac{22000}{16} \qquad Q_{max} = 3210\ kN$$

Note that due to rock socket, vertical loads carried by each pile is almost equal to each other. Therefore we operated our calculations based on equally distributed load.

Vertical Capacity Calculation

Pile length is taken as 24m to start with;

- Skin friction for sand (1st soil layer, type 1);

$f_{s\ sand} = \beta \times \sigma_v^l = (1 - sin24)\left(tan \frac{3\times 24}{4}\right) \times \sigma_v^l = 14.26\ kPa$

Correction from p.49, (27-3) at 5.5m, 74 kPa

$L_{cr} = 15 \times B = 15 \times 1.2 = 18\ m$

Meyerhoff $\Rightarrow$ $f_{s\ sand} = N_{60} = 6\ kPa$ to be one safe side, $f_{s\ sand} = 8\ kPa$

$f_{s\ sand} = 14.26\ kPa$ $Q_{skin\ sand} = 331.8\ kN$

- Skin friction for clay (2nd soil layer, type 3);

$\alpha \times C_u = 0.573\ kPa \Longrightarrow Q_{skin\ clay} = 3092.6\ kN$

$$\left.\begin{array}{l} \text{p.45, } \alpha = 0.57, \\ \text{p.63, } \alpha = 0.48, \\ \text{p.77, } \alpha = 0.67, \end{array}\right\} \alpha_{avg} = 0.573$$

$$Q_{tip} = 9C_u \times A_p = 9 \times 110 \times \pi \times 0.6^2 = 1119.7\ kN$$

$Q_{all} = \min\left(\left(\frac{Q_{skin}}{1.3} + \frac{Q_{tip}}{1.6}\right)\left(\frac{Q_{skin}+Q_{tip}}{1.5}\right)\right)$ Where;

Q_{skin}=3424.4 kN

Q_{tip}=1119.7 kN

$$Q_{all} = \min\left(\left(\frac{3424.4}{1.3} + \frac{1119.7}{1.6}\right)\left(\frac{3424.4+1119.7}{1.5}\right)\right)$$

$Q_{all} = 3029.4\ kN < 3210$ kN. Pile length is not sufficient, it has to be extended to rock layer. Rock socket calculations are necessary.

To find the length of the rock socket,

$q_{tip} = \left(\sqrt{t} + \sqrt{m * \sqrt{t} + t}\right) * q_u$ Where m=0.34, t=10^{-4} (page 91) and q_u=5 MPa

$q_{tip} = 354.8\ kPa$

$f_s = \alpha * \beta * q_u$

Where

α=0.14 (page 92)

β=0.64 (page 93)

$q_u = 5\ mPa$

$f_s = 448\ kPa$

$$Q = \min\left(\left(\frac{Q_{skin}}{1.3} + \frac{Q_{tip}}{1.6}\right)\left(\frac{Q_{skin}+Q_{tip}}{1.5}\right)\right)$$

$$3210 = \frac{3662.1+448*\pi*1.2*L_{sock}}{1.3} + \frac{391.1}{1.6}$$

$L_{sock} = 0.36\ m$

$$3210 = \frac{3662.1+448*\pi*1.2*L_{sock}+391.1}{1.5}$$

$L_{sock} = 0.45\ m$

Socket length is usually taken more than pile diameter, our socket length is less than the diameter. Therefore, socket length taken as 2m.

Tension capacity

$$T_{all} = \frac{Q_{skin}\times 0.75}{1.6} + W_{pile} = \frac{7039.9\times 0.75}{1.6} + 213.7 = 3513.8\ kN$$

$$W_{pile} = \pi \times 0.6^2 \times 27 \times (25 - 18) = 213.7\ kN$$

$$\alpha_{avg} = \frac{11\times18+14\times17.5+2\times22}{27} = 18$$

T_{max} on piles due to moment;

$$T_{max} = \frac{Vtotal}{n} - 2 \times \frac{M\times y}{I} = \frac{22000}{16} - 2 \times \frac{103800\sqrt{2}}{2} \times \frac{9}{720} = 459.9\ kN\ (tension)$$

$T_{all} > T_{max} \Longrightarrow$ OK!

Group Action

$$\frac{1}{(Q_{group})^2} = \frac{1}{(n*Q)^2} + \frac{1}{(Q_{block})^2}$$

$Q_{block} = Q_{skin+}\ Q_{tip}$

Dimensions of the block= 19.2*19.2 m

$$Q_{skin} = (8*19{,}2*11*4)_{sand} + (63{,}1*14*19{,}2*4)_{clay} + (19.2*2*4*448)_{rock}$$

$$Q_{skin} = 143416.4\ kN$$

$$Q_{tip} = 19{,}2*19{,}2*345{,}8$$

$$Q_{tip} = 127475.7\ kN$$

$Q_{block} = 180594{,}7\ kN$ (Factors of safety included)

$$Q_{iple} = Q_{skin} + Q_{tip}$$

$$Q_{skin} = (331.8)_{sand} + (3330.3)_{clay} + (3377.8)_{rock}$$

$$Q_{skin} = 7039\ kN$$

$$Q_{tip} = 391.1\ kN$$

$Q_{pile} = 4954\ kN$ (Factors of safety included)

$$\frac{1}{(Q_{group})^2} = \frac{1}{(16*4954)^2} + \frac{1}{(180594{,}7)^2} \qquad Q_{group} = 72580.8\ kN$$

Loads on Each Pile

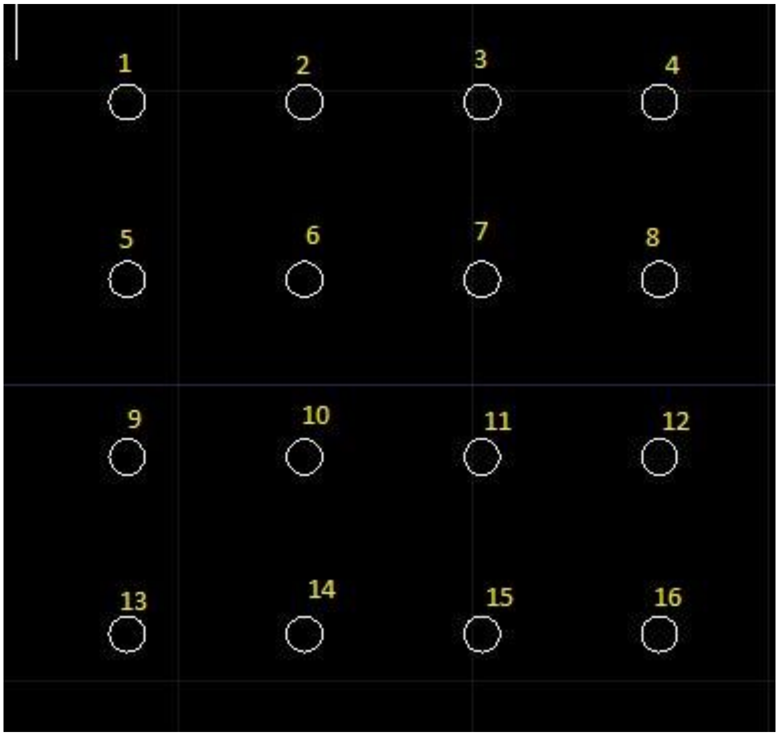


Figure 3:Pile Numbers for Vertical Load Calculation

Pile Label	Q/n (kN)	y (m)	M (kN.m)	I (m^2)	Q (kN)
1	1375	0	103800	720	1375
2	1375	4.242641	103800	720	1986.647
3	1375	8.485281	103800	720	2598.295
4	1375	12.72792	103800	720	3209.942
5	1375	-4.24264	103800	720	763.3526
6	1375	0	103800	720	1375
7	1375	4.242641	103800	720	1986.647
8	1375	8.485281	103800	720	2598.295
9	1375	-8.48528	103800	720	151.7053
10	1375	-4.24264	103800	720	763.3526
11	1375	0	103800	720	1375
12	1375	4.242641	103800	720	1986.647
13	1375	-12.7279	103800	720	-459.942
14	1375	-8.48528	103800	720	151.7053
15	1375	-4.24264	103800	720	763.3526
16	1375	0	103800	720	1375

Settlement Criteria

The settlement of the single for pile in rock formula (from p.153) ;

$\rho = \frac{Q \times I_p}{B \times E_d}$ Where Q: is the total load carried by the pile head

$I_{p:}$ is an influence factor

B: is the diameter of socket

E_d: is the deformation modulus of the rock mass surrounding the shaft

E_c is taken as 30 GPa;

$R = \frac{E_c}{E_d} = 200$; From figure 4.41 on pg. 154; Ip= 0.4

$$\rho = \frac{3210 \times 0.4}{112 \times 150000} = 7.13\ mm$$

$\rho = \frac{P*L}{A*E}$ $\rho = \frac{3210*27}{30000000*\pi*0.6^2}$ =2.55 mm;

$\rho = 7.13 + 2.55 = 9.68\ mm < 25\ mm$ $\Longrightarrow$ OK!

For vertical unit load of 1 MN;

$\rho = \frac{1000 \times 0.4}{112 \times 150000} = 2.22\ mm\ per\ MN$; Vertical stiffness at pile head = k = 450 MN /m

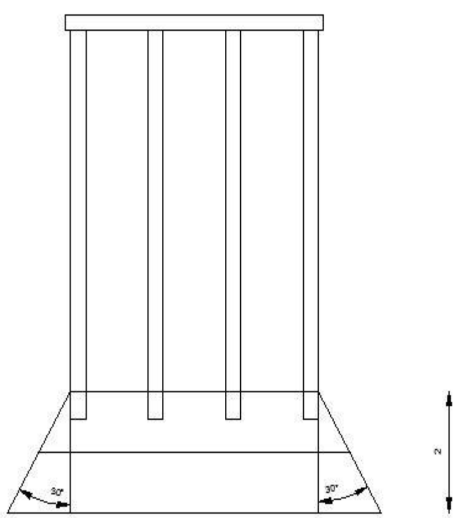


Figure 4: Group Settlement Analysis in Rock with Imaginary Footing Method

Group Settlement Analysis

From CE366 Lecture Notes, P.58

$$S = \frac{q*B}{E} * (1 - v^2) * I_s$$

Where

$I_s = 0.95$ $\longrightarrow$ From page 59, for rocks

$v = 0.2$

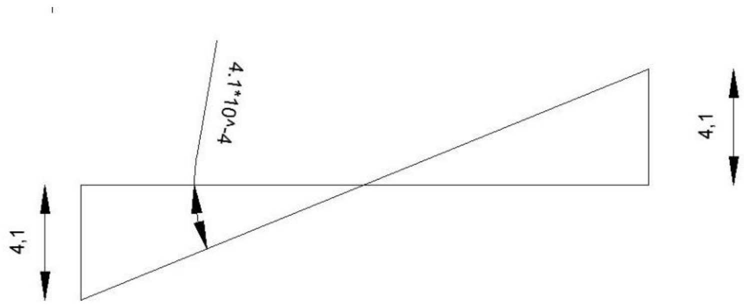


Figure 5: Rotated Shape of Rigid Pilecap

$q = \frac{22000}{21{,}15^2}$

$q = 49.2\ kPa$

$S = 6.32\ mm$

Rotation due to moments;

$\rho = \frac{1834 \times 0.4}{112 \times 150000} = 4.1\ mm;$

Therefore rotation is
$4.53 \times 10^{-4} < 0.001$ OK!

Lateral Load Analysis

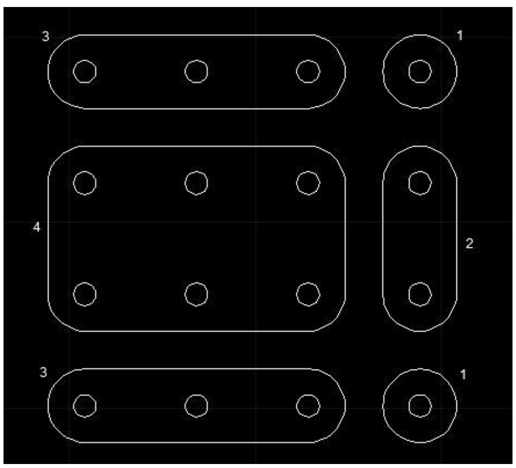


Figure 6: Pile Groups According to DIN 4014 Method for Lateral Load & Deflection Calculations

Since our S/B ratio is 5, pile interaction in diagonal direction is relatively smaller. Most critical case is when the load is in sideways (in X or Y axis, orientation is the same).

Pile groups are determined according to DIN 4014 procedure and calculations were done accordingly.

$K_h = \frac{0.65}{B} \sqrt[12]{\frac{Es*B^4}{Ep*Ip}} * \frac{Es}{(1-v^2)}$ = 12.8 MN/ m^3.

R = $\sqrt[4]{\frac{Ep*Ip}{kh*B}}$ = 3.75 m.

L/R = 7.2 > 4, therefore $\alpha_i^{4/3}$ values should be calculated.

From figures on p.396, for S/B = 5, $\alpha_{QA} = 1$, $\alpha_{QZ} = 1$, $\alpha_L = 0.875$.

$\alpha_1 = (\alpha_{QA})^{4/3} = 1$,

$\alpha_2 = (\alpha_{QZ})^{4/3} = 1$,

$\alpha_3 = (\alpha_L * \alpha_{QA})^{4/3} = 0.837$,

$\alpha_4 = (\alpha_L * \alpha_{QZ})^{4/3} = 0.837$,

Poulos' elastoplastic solutions are going to be used.

$\rho = (I_{\rho f} * H) / (E_S * L * F_{\rho F})$ where $F_{\rho F} = 1$ for unit (small) H.

Pile 1: $E_S = 25*1 = 25$ Mpa. $K_{R1} = \frac{30000*\pi*0.6^4}{4*25*27^4} = 2.3*10^{-4}$. Corresponding IρF value where

L/B = 22.5 is read as 6.8 from p.326 fig.8.19.

$\rho = \frac{6.8*H1}{25*27}$ (*)

Pile 2:

$\rho = \frac{6.8*H2}{25*27}$ (*)

Pile 3:

$\rho = \frac{6.6*H3}{20.9*27}$ (*)

Pile 4:

$\rho = \frac{6.6*H4}{20.9*27}$ (*)

Horizontal force equilibrium gives $3400*2 = 4H_1 + 12H_3$. Solving (*) with force equilibrium

$H_1 = H_2 = 474.5$ kN, $H_3 = H_4 = 408.7$ kN are calculated.

Then, ρ_{cap} is calculated as 4.78 mm. <25 mm OK!

$\rho_{unit} = 6.6*1/20.9*27 = 0.0117$ mm/kN

Lateral $k = (\rho_{unit})^{-1} = (0.0117)^{-1} = 85.47$ kN / mm

Note that the maximum lateral load carried by a single pile is 474.5 kN.

To find moment at pile head,

$\theta = I_{\theta H} * \frac{H}{E_s*L^2} + I_{\theta m} * \frac{M}{E_s*L^3}$ where;

$K_R = 7.22 * 10^{-4}$

$I_{\theta H} = 38$ (from page 324, figure 8.14 – 8.15)

$I_{\theta M} = 300$

H = 474.5 kN

L = 27 m

E_s=25000, Note that in E_s calculations, we took weighted average of all layers.

Then; $M = -1622.79\ kNm$

Moment along the pile is calculated of z= 0, 5, 10, 15, 20 and 27 m.

$$M = \frac{H}{\beta} K_{MH} + M_0 K_{MM}$$

$\beta = \frac{1}{R*\sqrt{2}} = 0.19\ (\frac{1}{m})$ Where, R=3.75 $\beta * L = 5.1 > 5$ therefore; values corresponding to β*L=5 are used.

- @ z = 0 m
  M = - 1622.79 kNm
  V= - 474.5 kN
- @ z = 5 m
  M = - 100.1 kNm
  V= 234.69 kN
- @ z = 10 m
  M = 202.34 kNm
  V= 181.14 kN
- @ z = 15 m
  M = 108.17 kNm
  V= 46.2 kN
- @ z = 20 m
  M= 22.81 kNm
  V= -9.43 kN
- @ z = 27 m
  M= 0 kNm
  V= 0 kN

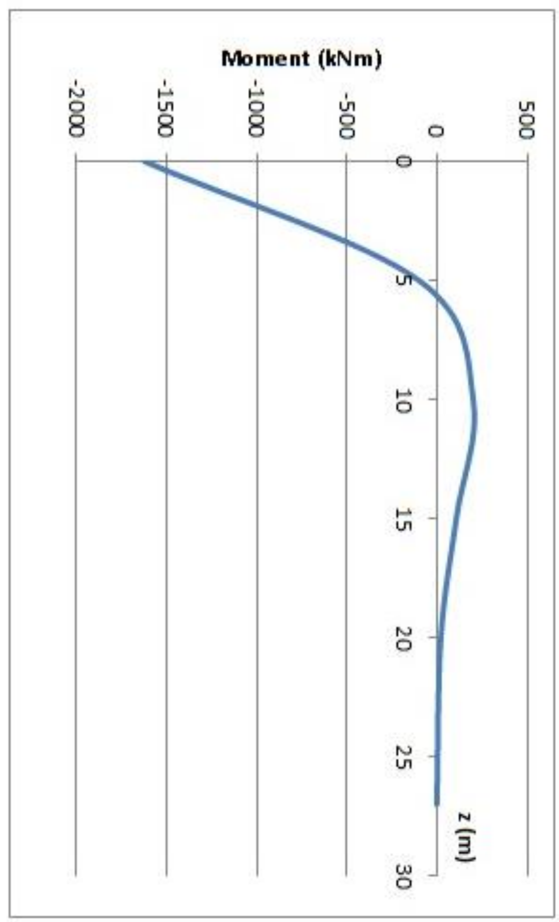


Figure 7: Distirubition of moment along the pile length

Lateral Load Capacity

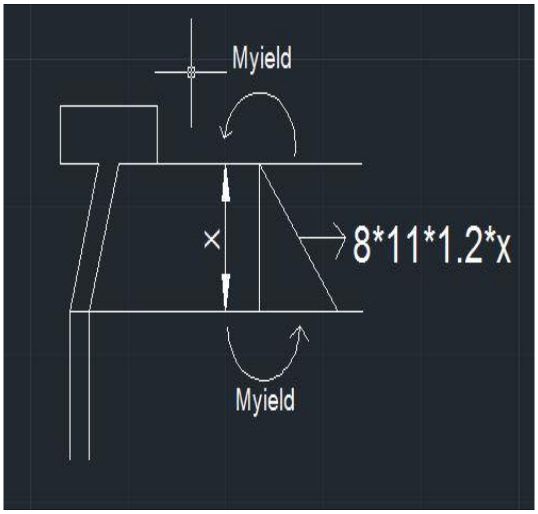


Figure 8: Deflected Shape of a Pile

We chose our hinge in sand layer, and for σ_v' as an average of below and above the GWT.

M_{yield}=0.1875*A_c*f_{cd}*B

M_{yield}=5090 kNm

$\sigma_v' = 11\ kPa$

$K_p = \frac{1+\sin(27)}{1-\sin(27)}$

$K_p = 2.66$

$2 * M_{yield} = 1.2 * 3 * 2.66 * 11 * x * \frac{x}{2} * \frac{2*x}{3}$

$$x = 6.61 < 11\ m$$

Assumption is valid, hinge occurs in sand layer.

$H_u = 1.2 * 3 * 2.66 * 11 * x * \frac{x}{2}$

For x=6.61;

H_u=2303.3 kN

H_{max}=474.5; H_u>F.s*H_{max} $\Longrightarrow$ 2303.3>2*474.5 OK!

Cost Analysis

$Total\ cost\ of\ a\ single\ pile = 130 * D^2 * L + 35 * D^{0,5} * L^{1.2}$

$Total\ cost\ of\ a\ single\ pile = 130 * 1.2^2 * 27 + 35 * 1.2^{0,5} * 27^{1.2}$

$Total\ cost\ of\ a\ single\ pile = 7055.6\ TL$

16 piles=112889,9 TL

$Cost\ of\ the\ pile\ cap = 11 * B^3$

$Cost\ of\ the\ pile\ cap = 11 * 20^3$

$Cost\ of\ the\ pile\ cap = 88000\ TL$

$Total\ cost = 200900\ TL$

Buckling Capacity

Piles are socketed into rock layer. Therefore, the following formula is used.

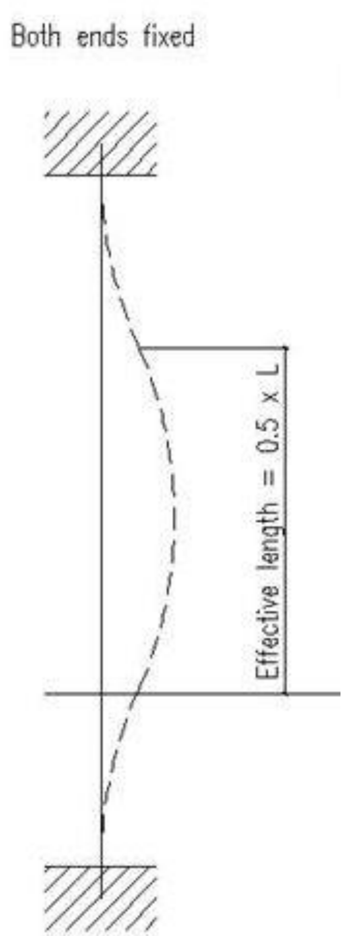


Figure 9: In case of Buckling, the Deflected Shape of a Pile

$$Q_{euler} = \frac{\pi^2 * E * I}{L_e^2}$$

where;

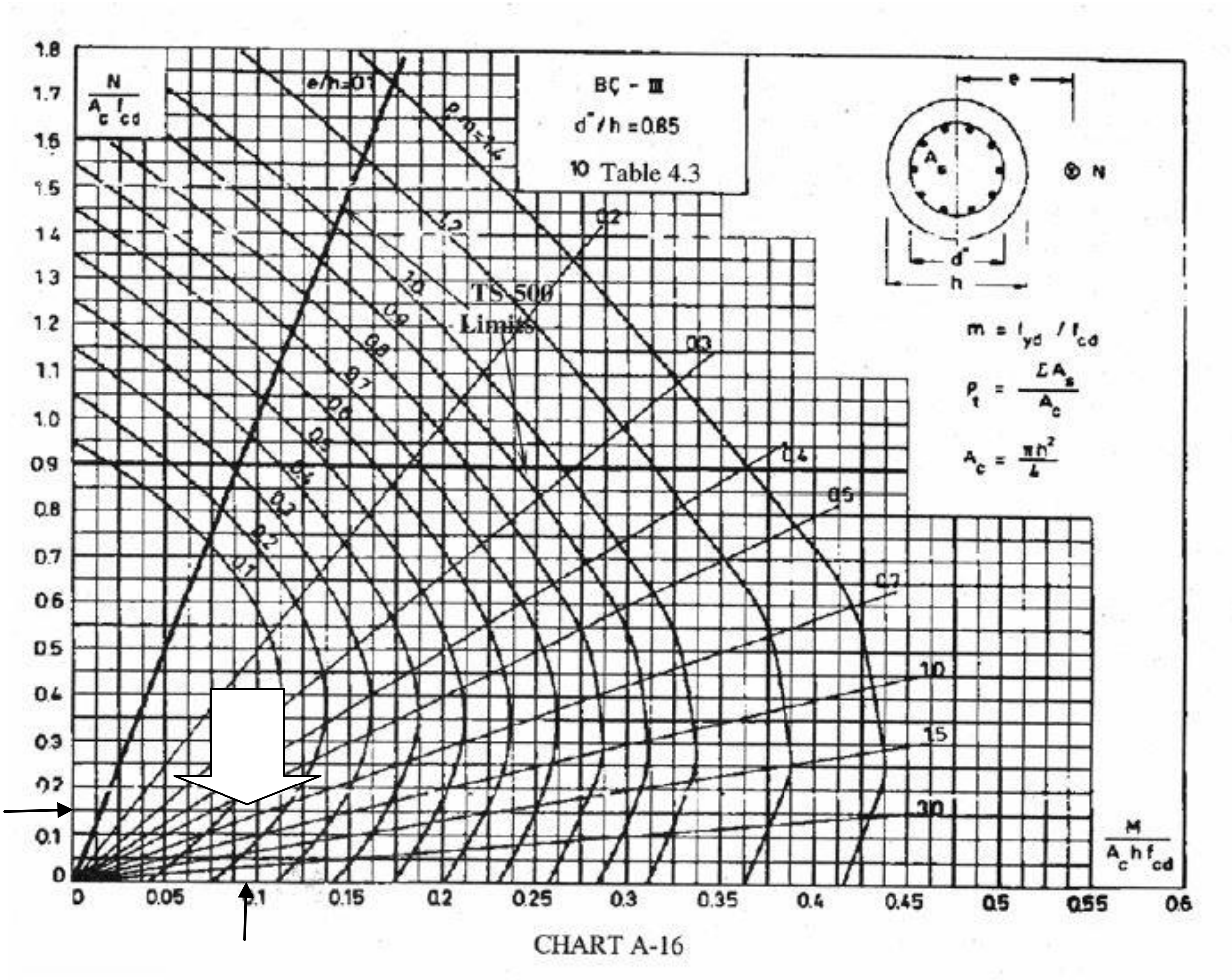
E=30 GPa

Le=L/2=13.5m

$I = 0.6^4 * \pi/4$

$Q_{euler} = 165367\ kN > 3 * Q_{max}$

Pile Section & Longitudinal Reinforcement



Reinforced concrete interaction diagram for circular cross-sections with St420.

Figure 10: Reinforced Concrete Interaction Diagram for Circular Cross- Sections with ST420, Ersoy, 1986.

Note that FS=1.5 for moment,

$N_d = 3210\ kN$ & $M_d = 1.5 * -1622.8\ kN.m$ $M_d = -2434.2 kN.m$

Using design chart, converting N_d and M_d into dimensionless values, we obtained a point inside the ρ_tm=0.2. But we ρ_{min}=0.03, therefore ρ_t is taken as 0.03.

$\rho = \frac{A_{st}}{A_c}$ $A_c=0.6^2*\pi=1{,}13\ m^2$

$A_{st} = 33929.2\ mm^2$, 27 ϕ40 is sufficient.

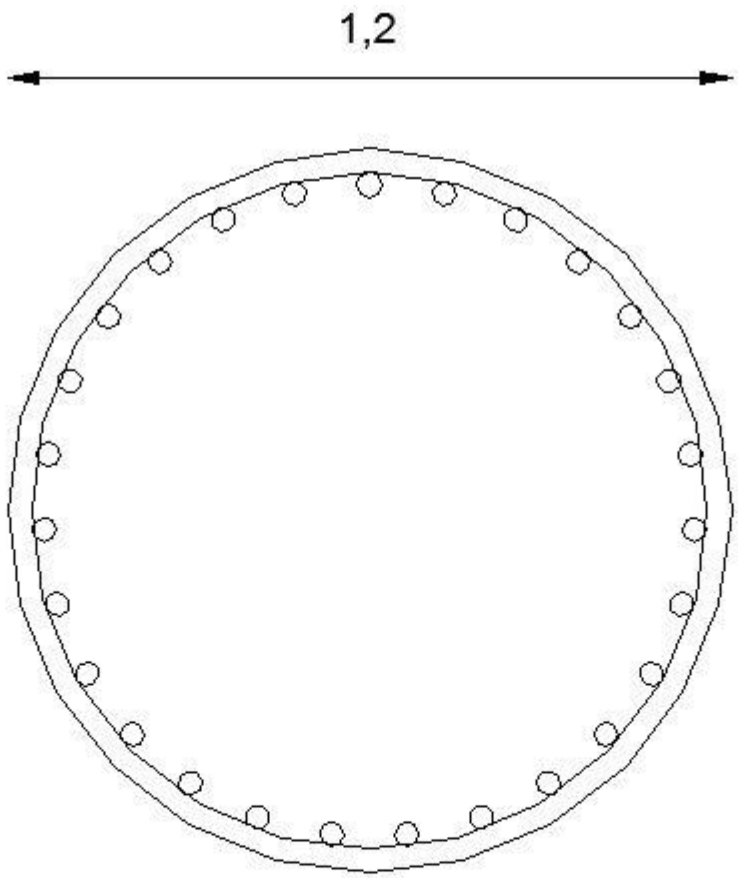


Figure 11: Pile Reinforcement Layout

Shear Reinforcement

B = 1200 mm.

Assume 800 mm. rectangular section due to lack of formulas for circular cross section

d=800 mm.

$f_{ctd} = \frac{\sqrt{fck}}{4.3} = 1.27\ Mpa$

f_{yd} = 365 MPa

f_{cd}=20 Mpa

$V_{cr} = 0.65 f_{ctd} b_w d(\psi)$ = 528 kN, where $\psi = 1 + 0.07*3210/800^2$ (Ersoy, Reinforced concrete)

$V_c = 0.8*V_{cr}$ = 422 kN.

V_d = 474.5*2 = 949 kN.

$$\frac{Asw}{s} = \frac{Vd - Vc}{fywd * d} = 0.18$$

A_{sw} of ϕ12 stirrups is 113 mm^2.

S = 113/0.18 = 627.8 mm.

Minumum shear reinforcement check:

$$\frac{Asw}{s} > \frac{0.3\, fctd}{fywd} * bw = 0.835$$

S = 136 mm.

ϕ12 steel with 120 mm. intervals will be used as shear reinforcement.

Load on Each Pile	3210 kN
Group Action	72581 kN
Average Settlement	9.68 mm.
Rotation	$4.53*10^{-4}$ radians
Horizontal Loads	474.5 kN
Horizontal Movement	4.78 mm.
Buckling	165367 kN
Pile sect. & Longit. Reinfor.	ok
Shear Reinfor.	ok

It is seen that the most critical criterion is vertical settlement. This can be reduced by several methods including:

- ✓ Extending the socket length in rock layer,
- ✓ Increasing pile cap dimensions,
- ✓ Using a higher- strength concrete, etc...

However, all the methods are going to increase total cost of the project, obviosuly. In our opinion, the most reasonable option is to increase the pile cap dimensions in order to have a reduced settlement value.

Conclusion

To sum up, a brief overview of designing a pile foundation is provided in this term project. This is done so by means of checking nine different design criteria including settlements, load distributions, reinforcement calculations, etc… With the help of this project, we had a chance to comment on numerical results calculated from graphs, charts, formulas and see whether they are compatible with our engineering judgment or not.

Soil mechanics is a complex discipline due to the fact that there are lots of different variables in soil properties and the real case cannot always be modeled by present equations. Hence, in some calculations, we made some simplifications to have a relation between our case and given equations. For example, we take weighted average in modulus of elasticity and specific weight values.

In our opinion, one of the most challenging issues in design is to use empirical charts. Calculations based on these graphs have a high standard deviation with respect to different readings. Therefore, in order to overcome potential capacity and settlement problems, high factor of safeties and reasonable amount of overdesign is inevitable.

References

Birand, A. (2011) *“ Ce 366 Lecture Notes ”* Ankara.

Ersoy, U. (1984) *“ Reinforced Concrete “* Ankara: Metu- Press.

Toker, K. (2013) *“ Ce 462 Course Reference Material “* Ankara.