AIN SHAMS UNIVERSITY

FACULTY OF ENGINEERING

*STRUCTURAL ENGINEERING DEPARTMENT*

***Fourth Year Civil***

*November, 2012*

*Time : 1.50 Hrs*

# MID TERM EXAM

**CES 451** **Foundation Engineering**

The exam consists of *three* questions in *Four* pages.



Reasonably assume any missing data.

## Question 1 (35% of Total Marks)

For the group of columns shown in Figures (1.a, 1.b and 1.c), it is required to:

i) Suggest a suitable type of foundation for each case.

ii) Calculate the footing(s) area required in each case and estimate the footings concrete dimensions.

iii) Without design, draw the estimated concrete dimensions and sketch the reinforcement details in plan for each case (using suitable scale 1: 50 or 1: 100).

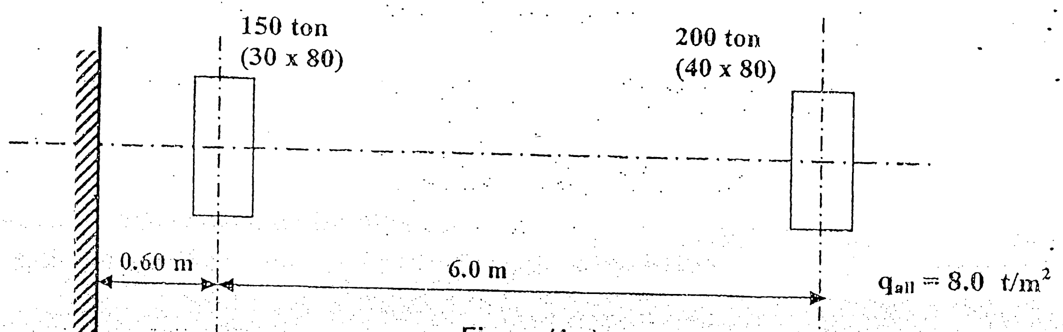



Figure (1.a)

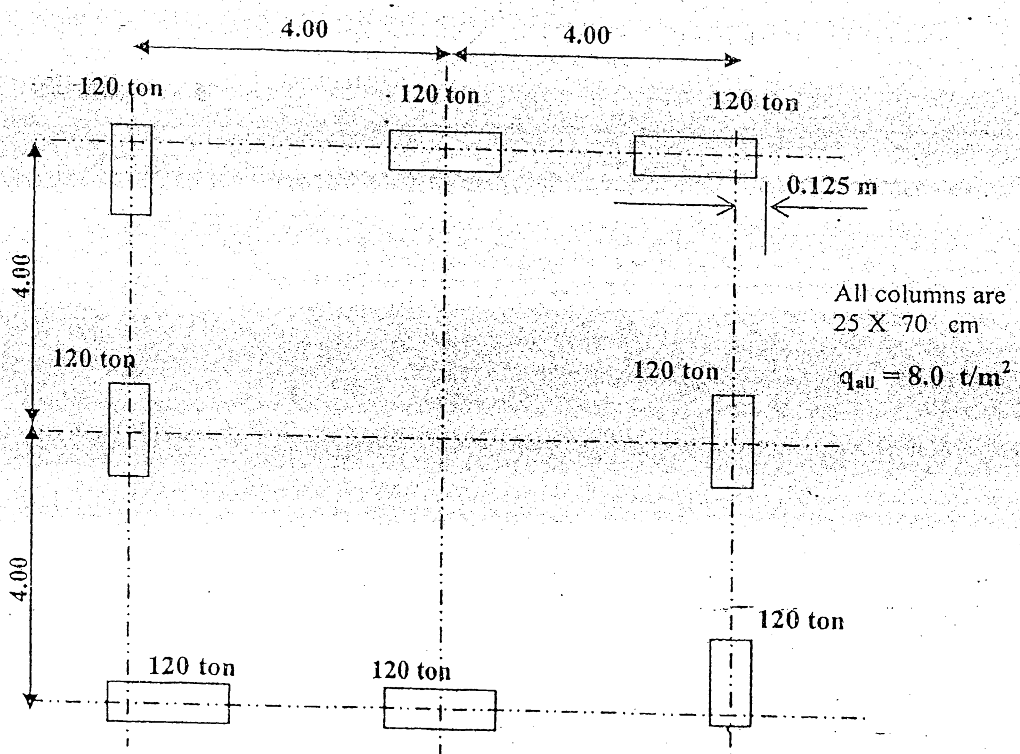



Figure (1.b)

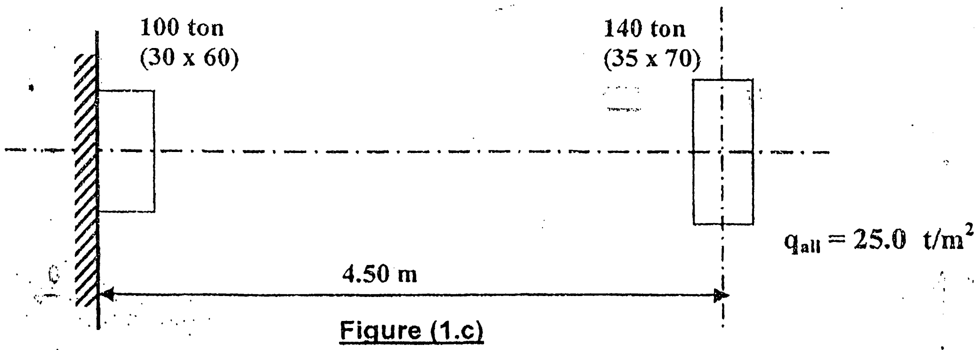



Figure (1.c)

## Question 2 (30% of Total Marks)

Compute and draw the distribution of contact stress and settlement profile below the R.C. isolated footing shown in Figure (2) considering the soil is elastic, homogeneous, isotropic and semi-infinite space. The footing is (2.00 x 6.60 m) and carries one column (120 x 110 cm) which is subjected to a normal compression load of 6000 kN. The footing is divided into 6 elements. The column can be modeled as two loads spaced 1.10 m, 3000 kN for each. It is required also to:

i) Calculate the contact stresses and settlement at point C,
ii) Calculate the maximum bending moment in the footing.

**Data :**
$C_o = 1.5$ $C_1 = 0.65$ $C_2 = 0.35$
$C_3 = 0.25$ $C_4 = 0.18$ $C_5 = 0.13$
$C_6 = 0.09$

$E_c = 2100$ kN/cm²,
$E_s = 70$ MN/m²,
Thickness of footing = 130 cm

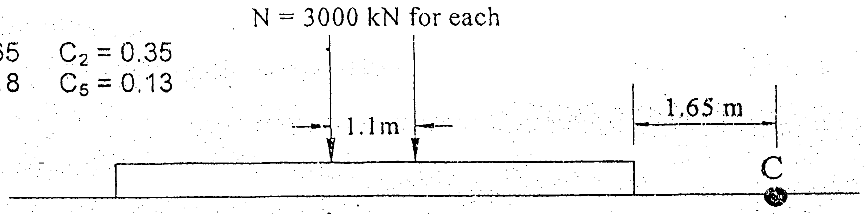



Figure (2)

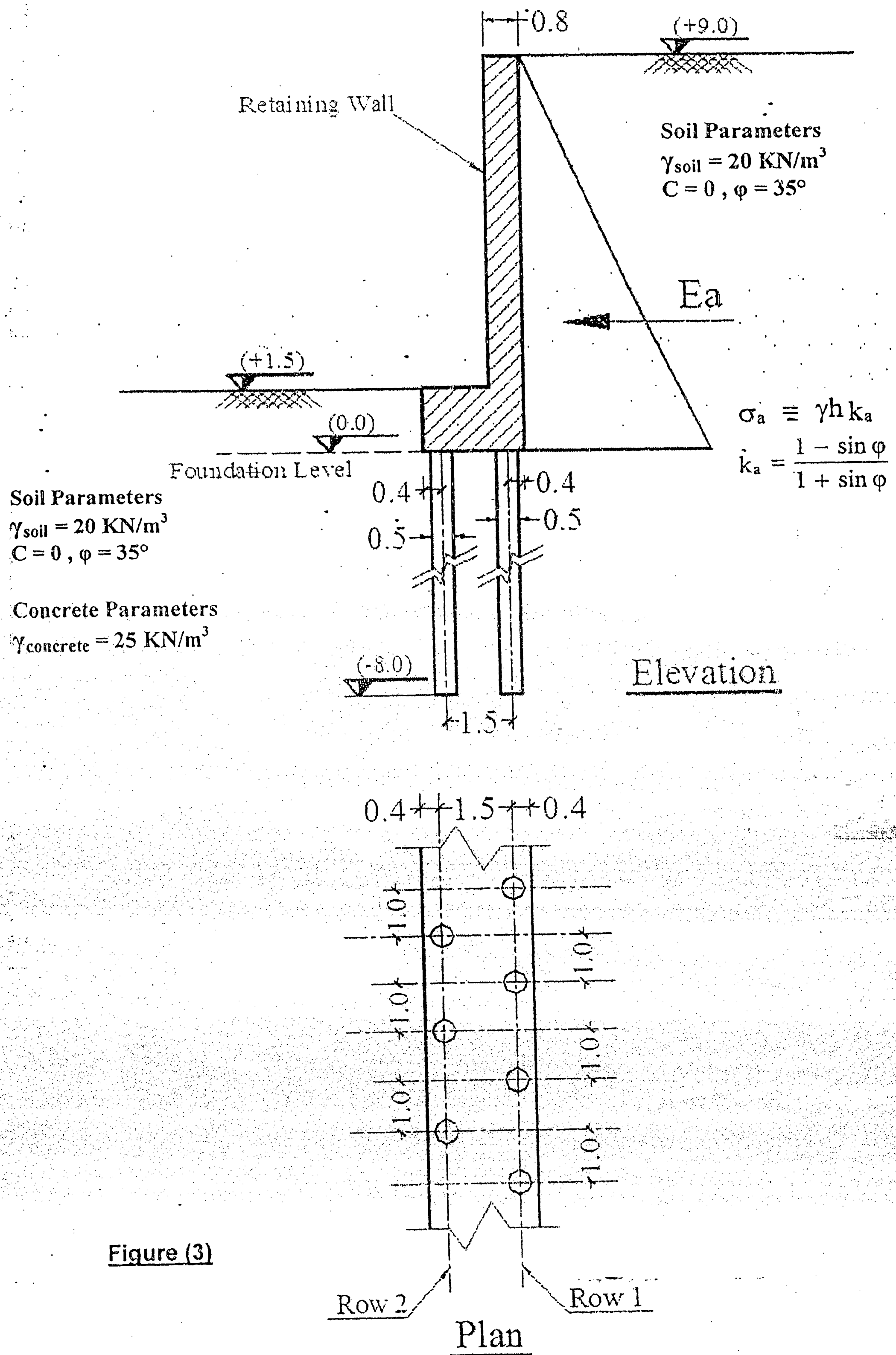
## Question 3 (35% of Total Marks)

Retaining wall should be constructed in a site, as shown in Figure (3). Due to site limitations, the retaining wall have to be founded on a double rows of piles. The soil beneath the foundation level consists of a thick layer of very dense sand down to 20.0 m followed by rock formations. The backfill behind the wall is constructed using very well graded soil. A small diameter bored piles with pile diameter 0.5m were used. The pile length is 8.0 m measured from the foundation level. There is no ground water table. The foundation depth lies 1.50 m below the ground surface. The strip pile cap has a thickness of 1.5m. The following is required:

a) Calculate the active earth pressure acting on the retaining wall.
b) Calculate the pile **vertical** forces considering the weight of the retaining wall and neglecting the passive earth pressure.
c) Check the pile capacity of both rows.

According to the Egyptian Code of Practice, the static formulae (for piles in cohesionless soil, such as sand and gravel) is as follows:

$$Q_{ult} = P_b \, N_q \, \pi \, R^2 + \sum_{H=0}^{H=L} K_{HC} \, P_o \, (\tan\delta) \, 2\pi \, R \, \Delta H$$



0.8
(+9.0)
Retaining Wall
Soil Parameters
$\gamma_{soil} = 20$ KN/m$^3$
C = 0 , φ = 35°
Ea
(+1.5)
(0.0)
Foundation Level
$\sigma_a = \gamma h k_a$
$k_a = \frac{1 - \sin\varphi}{1 + \sin\varphi}$
0.4
0.4
0.5
0.5
Soil Parameters
$\gamma_{soil} = 20$ KN/m$^3$
C = 0 , φ = 35°
Concrete Parameters
$\gamma_{concrete} = 25$ KN/m$^3$
(-8.0)
Elevation
1.5
0.4
1.5
0.4
1.0
1.0
1.0
1.0
1.0
1.0
Row 2
Row 1
Plan


**Figure (3)**

جدول رقم (٤ – ٥) العلاقة بين قيم معامل قدرة التحميل (Nq) وقيم زاوية الإحتكاك الداخلى($\phi$) لتربة غير متماسكة الحبيبات

| $\phi$ بالدرجات | ٢٥ | ٣٠ | ٣٥ | ٤٠ |
|---|---|---|---|---|
| Nq | ١٥ | ٣٠ | ٧٥ | ١٥٠ |

$\phi$ لخوازيق الازاحة = ( $\phi$ ( قبل التنفيذ)+ ٤٠°)/٢

$\phi$ لخوازيق التثقيب العادية = $\phi$ ( قبل التنفيذ)- ٣°

جدول رقم (٤–٦): قيم المعاملات ($K_{HC}$) و($K_{HT}$)

| نوع الخازوق | $K_{HC}$ | $K_{HT}$ |
|---|---|---|
| خازوق ذو قطاع H | ١,٠–٠,٥ | ٠,٥–٠,٣ |
| خازوق ازاحة | ١,٥–١,٠ | ١,٠–٠,٦ |
| خازوق ازاحة متغير القطاع | ٢,٠–١,٥ | ١,٣–١,٠ |
| خازوق ازاحة باستخدام النفاثات | ٠,٩–٠,٤ | ٠,٦–٠,٣ |
| خازوق تثقيب اعتيادى(قطر اقل من ٠,٦٠متر ) | ١,٥–٠,٧ | ١,٠–٠,٤ |

جدول رقم (٤–٧) : قيم زاوية الإحتكاك بين التربة وجزع الخازوق (δ)

| نوع الخازوق | δ (درجة) |
|---|---|
| حديد | ٢٠ |
| خرسانة | ٣/٤ ($\phi$) |
| خشب | ٣/٤ ($\phi$) |

$\phi$ زاوية الاحتكاك الداخلى للتربة

☺ BEST WISHES ☺

جدول رقم (٤ – ٥) العلاقة بين قيم معامل قدرة التحميل (Nq) وقيم زاوية الإحتكاك الداخلى($\phi$) لتربة غير متماسكة الحبيبات

| $\phi$ بالدرجات | ٢٥ | ٣٠ | ٣٥ | ٤٠ |
|---|---|---|---|---|
| Nq | ١٥ | ٣٠ | ٧٥ | ١٥٠ |

$\phi$ لخوازيق الازاحة = ( $\phi$ ( قبل التنفيذ)+ ٤٠°)/٢

$\phi$ لخوازيق التثقيب العادية = $\phi$ ( قبل التنفيذ)- ٣°

جدول رقم (٤–٦): قيم المعاملات ($K_{HC}$) و($K_{HT}$)

| نوع الخازوق | $K_{HC}$ | $K_{HT}$ |
|---|---|---|
| خازوق ذو قطاع H | ١,٠–٠,٥ | ٠,٥–٠,٣ |
| خازوق ازاحة | ١,٥–١,٠ | ١,٠–٠,٦ |
| خازوق ازاحة متغير القطاع | ٢,٠–١,٥ | ١,٣–١,٠ |
| خازوق ازاحة باستخدام النفاثات | ٠,٩–٠,٤ | ٠,٦–٠,٣ |
| خازوق تثقيب اعتيادى (قطر اقل من ٠,٦٠ متر ) | ١,٥–٠,٧ | ١,٠–٠,٤ |

جدول رقم (٤–٧) : قيم زاوية الإحتكاك بين التربة وجزع الخازوق (δ)

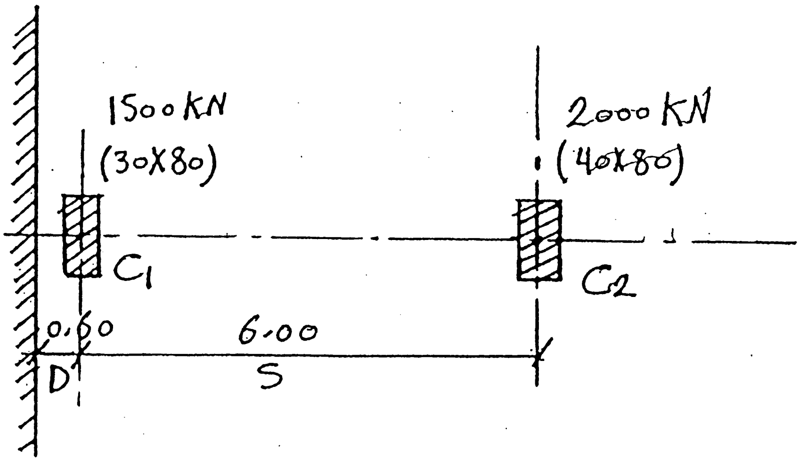
| نوع الخازوق | δ (درجة) |
|---|---|
| حديد | ٢٠ |
| خرسانة | ٣/٤ ($\phi$) |
| خشب | ٣/٤ ($\phi$) |

$\phi$ زاوية الاحتكاك الداخلى للتربة

☺ BEST WISHES ☺

## Q1)

### Figure (1.a):-




- $q_{all\,net} = 80\ kPa$
- Try Isolated footing below ($C_1$)
- Assume $t_{PC} = 30\ cm$

$\therefore A_{PC} = \frac{1500}{80} = 18.75\ m^2 = L_{PC} \times B_{PC}$ ——— ①

$\therefore L_{PC} - B_{PC} = 0.8 - 0.3 = 0.5$ ——— ②

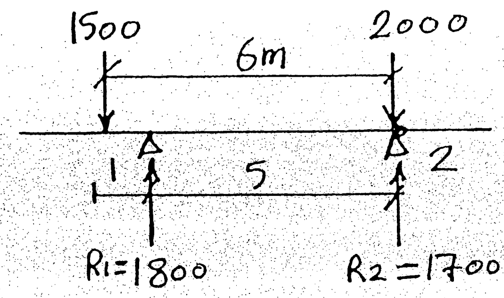
solving eqs ① and ②, we get

$B_{PC} = 4.10\ m$ , $L_{PC} = 4.60\ m$

$\therefore \frac{B_{PC}}{2} > D$

$\therefore$ we can't use isolated footing

### Try strap beam




Assume $e = (0.1 - 0.2)S$

$= 0.6 - 1.20\ m$

$\therefore$ Take $e = 1.00\ m$

$\Sigma M_{@2} = 0 \longrightarrow R_1 = 1800\ kN$

$\Sigma F_y = 0 \longrightarrow R_2 = 1700\ kN$

## F1 -

$L_{PC} = L_{RC} = 2(1+0.6) = 3.20\,m$

$A_{PC} = \dfrac{1800}{80} = 22.5\ m^2$

$\therefore B_{PC} = \dfrac{22.5}{3.20} = 7.03\ m$

$\therefore \dfrac{B_{PC}}{L_{PC}} = 2.2 > 2$ (Not valid)

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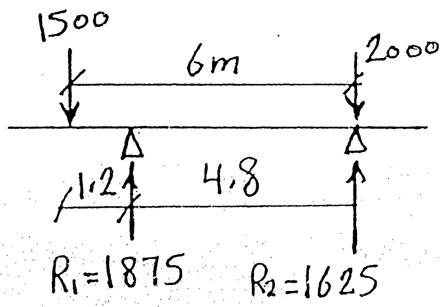
Retry strap beam using $e = 1.20\,m$

## F1 -




$\therefore L_{PC} = L_{RC} = 3.60\,m$

$\therefore A_{PC} = \dfrac{1875}{80} = 23.4375\ m^2$

$\therefore B_{PC} = 6.51 \simeq 6.55\,m$

**Check:** $\dfrac{B_{PC}}{L_{PC}} = \dfrac{6.55}{3.60} = 1.82 < 2$

---

## F2 - (square footing)

$A_{PC} = \dfrac{1625}{80} = B^2_{PC} \longrightarrow B_{PC} = 4.55\,m$

$\therefore B_{RC} = 3.95\,m$

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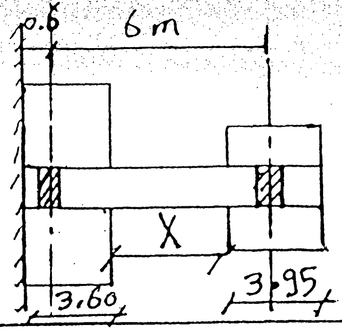
## Check validity of using strap beam -

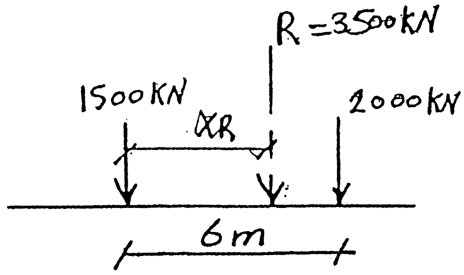



$X = 6 + 0.6 - \dfrac{3.95}{2} - 3.60 = 1.025\,m$

$\therefore X < \dfrac{L_{min}}{2} = \dfrac{3.60}{2} = 1.80$

$\therefore$ We can't use strap beam

## Use Rectangular Combined Footing:-




$\sim R = 1500 + 2000 = 3500\ kN$

$\sim X_R = \frac{2000 \times 6}{3500} = 3.429\ m$

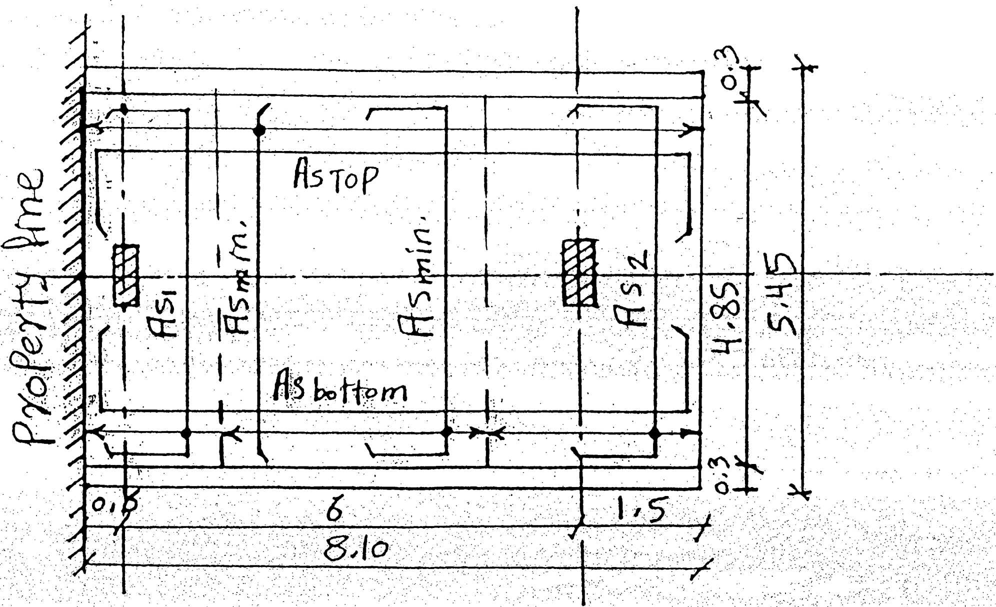
$\sim L_{PC} = L_{RC} = 2\,(3.429 + 0.6) = 8.06\ m$

$\simeq 8.10\ m$

$\sim A_{PC} = \frac{3500}{80} = 43.75\ m^2$

$\sim B_{PC} = 5.45\ m$

$\sim B_{RC} = 4.85\ m$




PLAN

## Figure (1.b)1- (Assume $t_{PC} = 30\,cm$)

- Try isolated footing below each column

$$A_{PC} = \frac{1200}{80} = 15\,m^2 = L_{PC} \times B_{PC} \quad \text{---} \quad (1)$$

$$L_{PC} - B_{PC} = 0.45 \quad \text{---} \quad (2)$$

$\therefore B_{PC} = 3.70\,m$ , $L_{PC} = 4.15\,m$

$\therefore B_{RC} = 3.10\,m$ , $L_{RC} = 3.55\,m$

### Check Validity of Isolated Footing

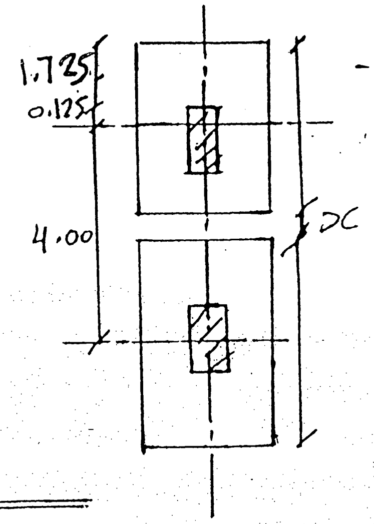
• يتم التأكد من عدم تداخل القواعد

$$x = 4 + 0.125 + 1.725 - 4.15 - \frac{4.15}{2}$$

$$= -0.375$$

∴ يوجد تداخل بين القواعد

$\therefore$ We can't use isolated footing




$\therefore$ Try strip footing

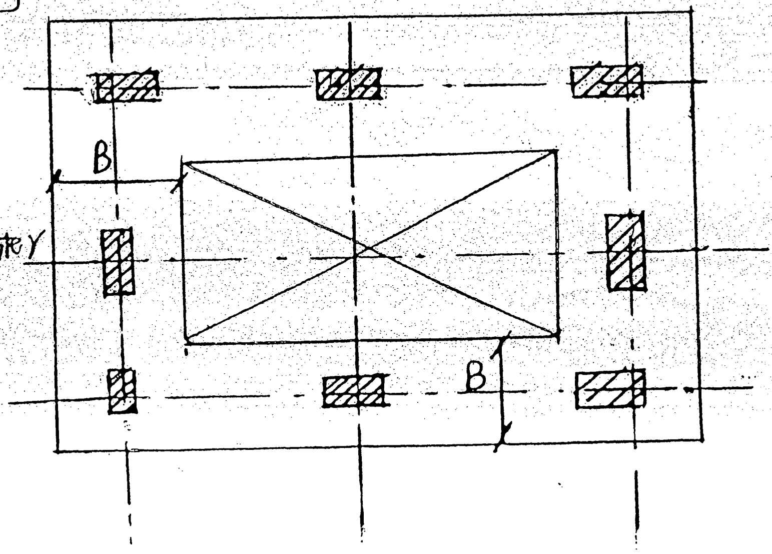
$$A_{PC} = \frac{\Sigma Col.\,loads}{q_{all\,net}} = \frac{120 \times 8}{8} = 120\,m^2$$

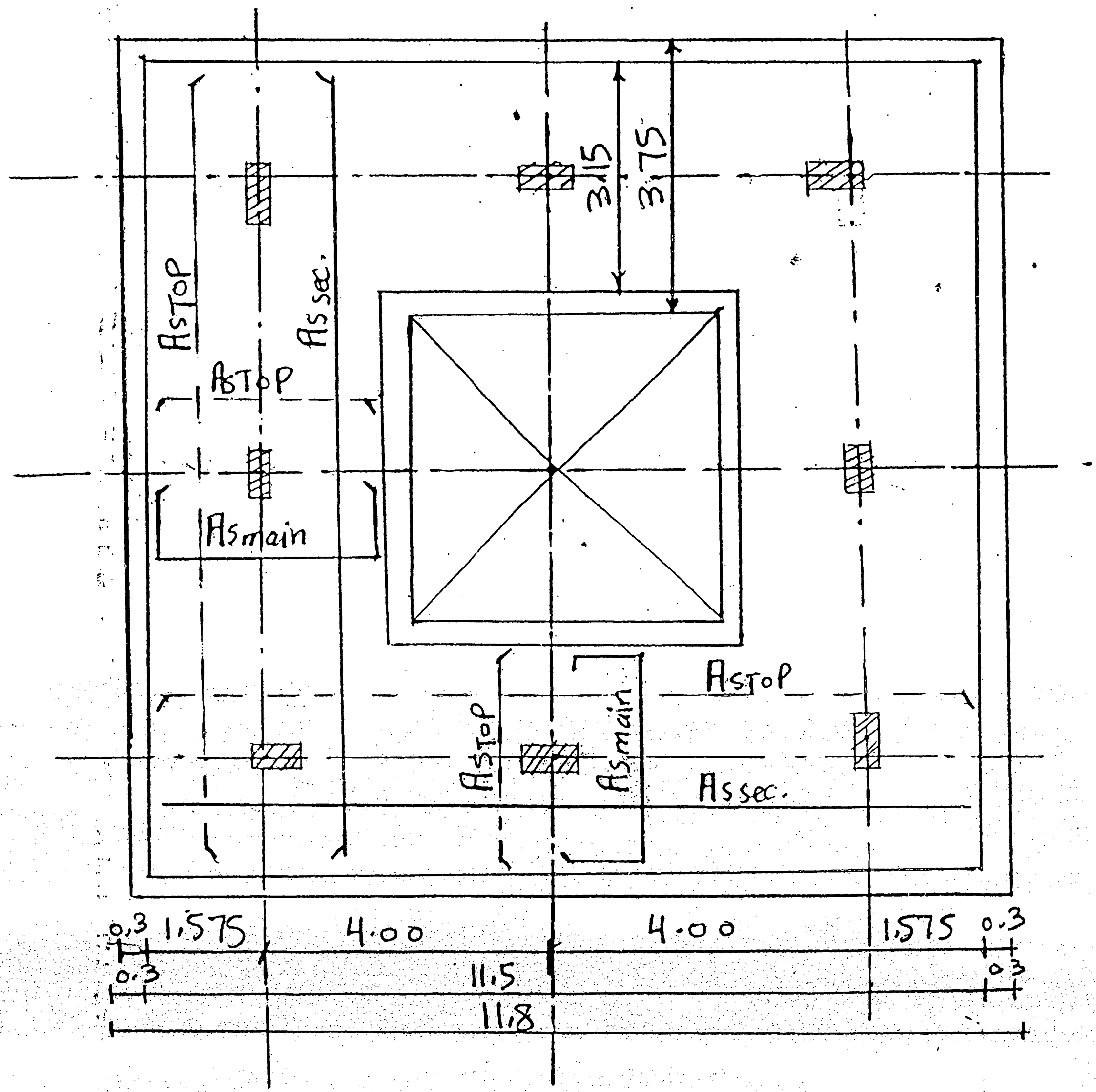
$\therefore A_{PC} = 120\,m^2 = B \times Avg.\,Perimeter$

$Avg.\,Per. = 8 \times 4 = 32\,m$

$$\therefore B = \frac{120}{32} = 3.75\,m$$

$$\therefore B_{RC} = 3.75 - 2(0.3) = 3.15\,m$$

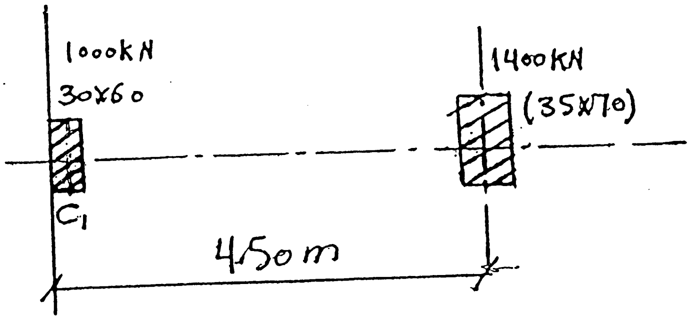




3.15
3.75
AsTOP
As sec.
AsTOP
As main
AsTOP
AsTOP
As main
As sec.
0.3
1.575
4.00
4.00
1.575
0.3
0.3
11.5
0.3
11.8

## Figure (1.c):-

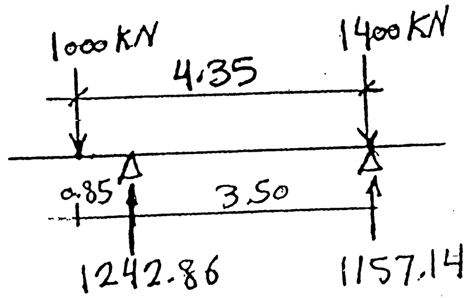
- $q_{all\,net} = 250$ kPa
- Try strap beam




Assume $e = (0.1-0.2)\,S$

$= 0.435 - 0.87$ m

∴ Take $e = 0.85$ m




### F1:-

$L_{PC} = L_{RC} = 2\,(e + c/2)$

$= 2\,(0.85 + 0.15) = 2m$

- Assume $t_{PC} = 30$ cm

∴ $A_{PC} = \frac{1242.86}{250} = 4.97\,m^2$

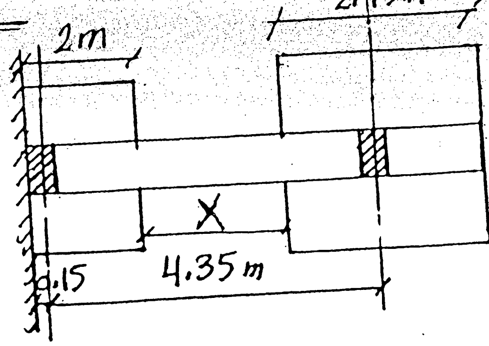
∴ $B_{PC} = 2.50$ m

∴ $B_{RC} = 1.90$ m

### F2:-

$A_{PC} = \frac{1157.14}{250} = 4.63\,m^2 = B_{PC}^2 \Longrightarrow$ $B_{PC} = 2.15m$, $B_{RC} = 1.55m$

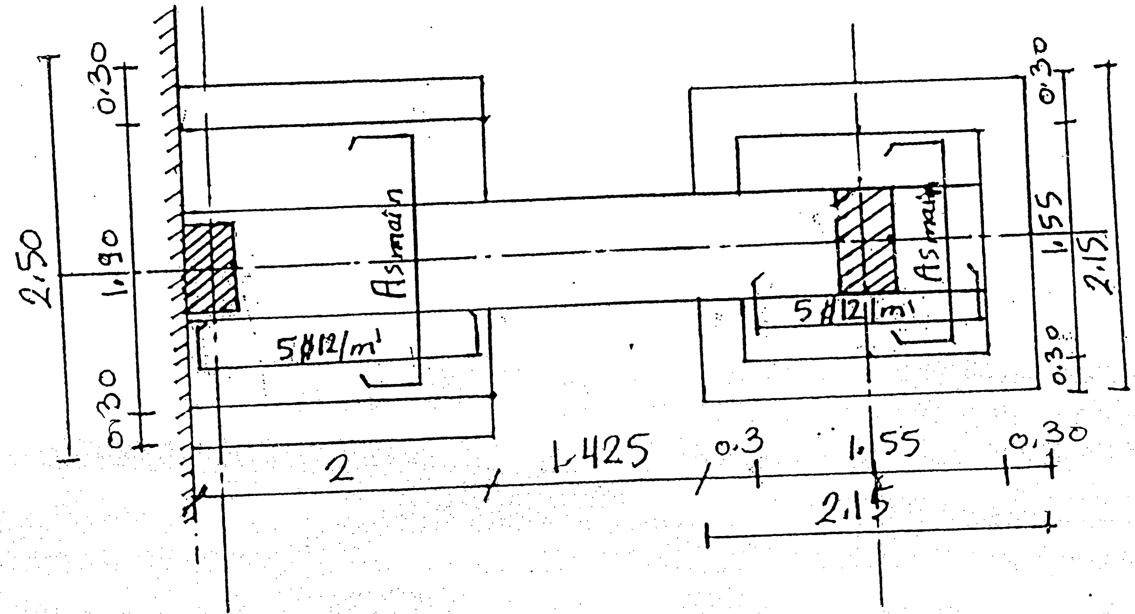
### Check validity of using strap beam:-




$X = 4.35 + 0.15 - \frac{2.15}{2} - 2 = 1.425m$

∴ $X > \frac{L_{min.}}{2} = \frac{2}{2} = 1$

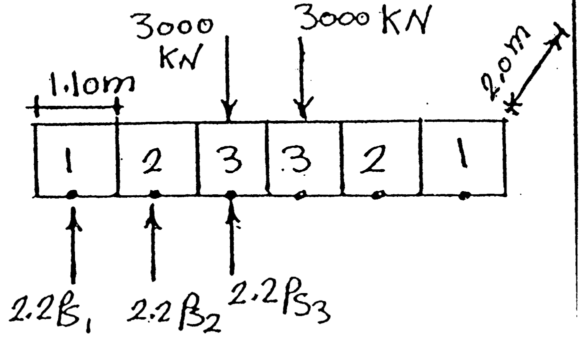
∴ Strap beam is valid


0.30
2,50
1,90
0.30
As main
5Ø12/m'
2
1.425
0.3
1.55
0,30
2.15
0.30
1,55
2,15
0.30
As main
5Ø12/m'

# PLAN

## Q2 :-

- Assume contact stress ($P_{si}$) below each element




- $P_i$ (element reaction) $= 1.1 * 2 * P_{si}$
  $= 2.2\, P_{si}$

### (A) M-Ps eqs :-

$M_1 = 0$ ---- ⓐ

$M_2 = 2.2\, P_{s1} * 1.1 = 2.42\, P_{s1}$ ---- ⓑ

$M_3 = (2.2\, P_{s1} * 2.2) + (2.2\, P_{s1} * 1.1) = 4.84\, P_{s1} + 2.42\, P_{s2}$ -- ⓒ

### (B) M-Δ eqs :-

$$\frac{6 E_c I}{a^2} = \frac{6 \times 2.1 \times 10^7 \times \frac{2 \times (1.3)^3}{12}}{(1.10)^2} = 38.13 \times 10^6 \text{ KN}$$

- Applying 4- Moment equation

**Point (2) :-**

$M_1 + 4 M_2 + M_3 = 38.13 \times 10^6 \, (-\Delta_1 + 2\Delta_2 - \Delta_3)$ ---- ①

**Point (3) :-**

$M_2 + 5 M_3 = 38.13 \times 10^6 \, (-\Delta_2 + \Delta_3)$ ---- ②

c) S-Δ eqs:-

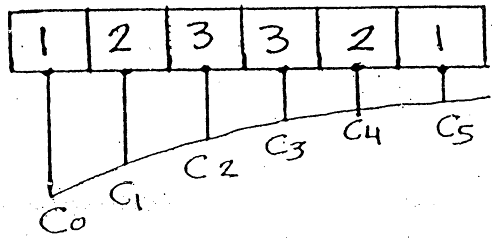
Using assumption of soil is elastic, homogeneous isotropic and semi-infinite

$$\Delta i = \frac{a}{E_s} \sum_{i=1}^{n} C_i * P_{si}$$

$$\frac{a}{E_s} = \frac{1.1}{70 \times 10^3} = 1.57 \times 10^{-5} \; m^3/kN$$

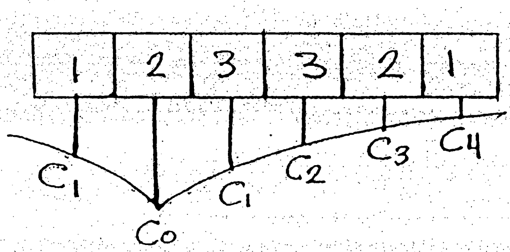
**Point (1):-**

$$\Delta_1 = 1.57 \times 10^{-5} \left[ 1.63 P_{s1} + 0.83 P_{s2} + 0.6 P_{s3} \right] \quad \text{--- (I)}$$



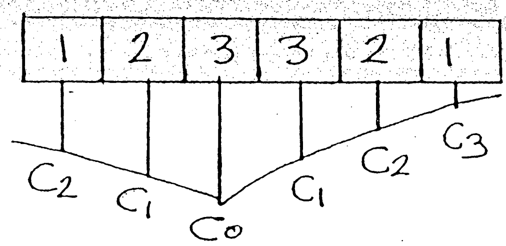

**Point (2):-**

$$\Delta_2 = 1.57 \times 10^{-5} \left[ 0.83 P_{s1} + 1.75 P_{s2} + P_{s3} \right] \quad \text{--- (II)}$$




**Point (3):-**

$$\Delta_3 = 1.57 \times 10^{-5} \left[ 0.6 P_{s1} + P_{s2} + 2.15 P_{s3} \right] \quad \text{--- (III)}$$



## Final eqs:-

Substituting eqs [a, b, c], [I, II, III] into eqs ①, ②

### eq. ①:-

$$0 + 4(2.42\,P_{S1}) + (4.84\,P_{S1} + 2.42\,P_{S2})$$

$$= \frac{38.13*10^6 * 1.57*10^{-5}}{598.641}(-0.57\,P_{S1} + 1.67\,P_{S2} - 0.75\,P_{S3})$$

$$\therefore\ 355.745\,P_{S1} - 997.31\,P_{S2} + 448.98\,P_{S3} = 0 \quad ---\ ①$$

### eq. ②:-

$$2.42\,P_{S1} + 5(4.84\,P_{S1} + 2.42\,P_{S2}) = 598.641(-0.23\,P_{S1} - 0.75\,P_{S2} + 1.15\,P_{S3})$$

$$\therefore\ 164.31\,P_{S1} + 461.1\,P_{S2} - 688.44\,P_{S3} = 0 \quad ---\ ②$$

### eq. ③:-

$$\Sigma F_y = 0$$

$$\therefore\ 2*(2.2\,P_{S1} + 2.2\,P_{S2} + 2.2\,P_{S3}) = 6000$$

$$\therefore\ P_{S1} + P_{S2} + P_{S3} = 1363.7 \quad ------\ ③$$

By solving eqs ①, ②, and ③, we get

$$P_{s1} = 580.724 \ kN/m^2$$

$$P_{s2} = 385.9 \ kN/m^2$$

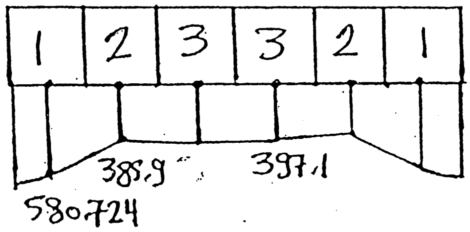
$$P_{s3} = 397.1 \ kN/m^2$$

## Check:-

$$P_{s\,avg.} = 454.5 \ kN/m^2$$

$$= q_{act.} = \frac{6000}{2 \times 6.60} = 454.5 \ kN/m^2$$

(6000 = col load; $q_{act.}$ = Contact stress; $2 \times 6.60$ = Footing Area)

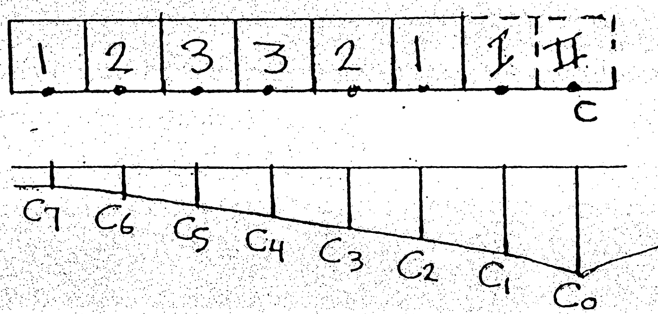



Contact stress
(kN/m²)

## Settlement at Point (c):-

$C_7$ is unknown

$$\sim C_i = \frac{C_o}{1 + k_1 (i)^{k_2}}$$




$$\sim C_1 = \frac{1.5}{1 + k_1 (1)^{k_2}} = 0.65$$

$\hookrightarrow k_1 = 1.3$

$$\sim C_2 = \frac{1.5}{1 + k_1 (2)^{k_2}} = 0.35$$

($k_1$ = 1.3)

$\hookrightarrow k_2 = 1.337$

$\therefore\ C_7 = \dfrac{1.5}{1+1.3(7)^{1.337}} = 0.08$ (check $< C_6$)

$\therefore\ \Delta c = 1.57\times10^{-5}\left[C_0 * P_{S_I} + C_1 P_{S_{II}} + C_2 P_{S1} + C_3 P_{S2} + C_4 P_{S3} + C_5 P_{S3} + C_6 P_{S2} + C_7 P_{S1}\right]$

$\therefore\ \Delta c = 1.57\times10^{-5}\left[1.5 * 0 + 0.65\times0 + 0.43 P_{S1} + 0.34 P_{S2} + 0.31 P_{S3}\right]$

$= 1.57 * 10^{-5}\left[0.43\times580.724 + 0.34\times385.9 + 0.31\times397.1\right]$

$= 7.913\times10^{-3}$ m

$= 7.913$ mm

## Calculation of Max. B.M. in Footing:-

- $M_1 = 0$
- $M_2 = 2.42\, P_{S1} = 2.42\times580.724 = 1405.35$ KN.m.
- $M_3 = 4.84\, P_{S1} + 2.42\, P_{S2}$

$= 4.84\times580.724 + 2.42\times385.9$

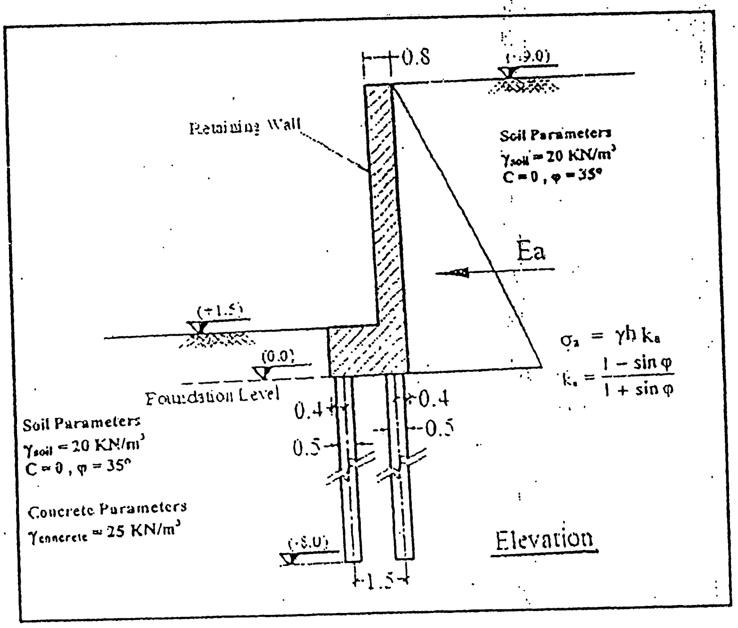
$= 3744.58$ KN.m.

$\therefore$ $M_{max.} = M_3 = 3744.58$ KN.m.

## Q3)

**Given:**

- Bored pile.
- $L_{pile} = 8$ m
- d= 0.50 m




**Solution:**

**a)**

$Ka = (1-\sin\varphi)/(1+\sin\varphi)$

$= (1-\sin35)/(1+\sin35)$

$= 0.27.$

$\sigma a = Ka.\ \sigma_v - 2c\sqrt{ka}$

$= 0.27x20x9- 0$

$= 48.77$ kPa/m.

$Ea = 0.5x9x48.77$

$= 219.465$ kN/m

**b)**

$W_{R.W.} = W_{base} + W_{stem}$

$= (1.5x2.3x25) + (0.8x7.5x25)$

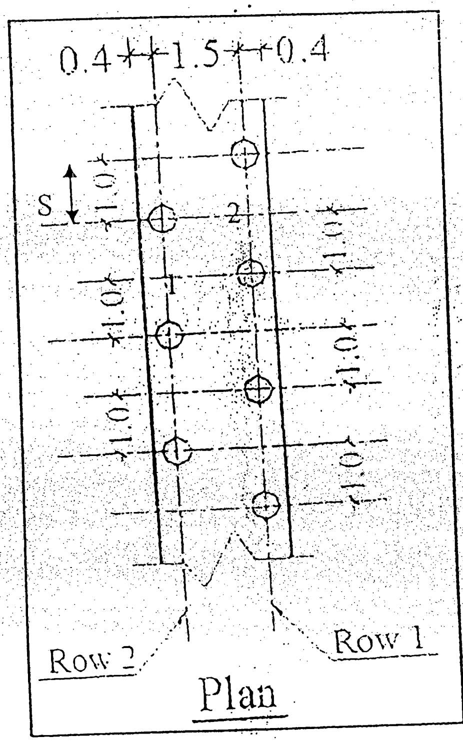
$= 236.25$ kN/m.

$Ma = E_a x 3 = 658.4$ kN.m/m

$Q_1 = W_{R.W.} + Ma/s$

$= 236.25 + (658.4/1.5)$

$= 675.2$ kN (Comp.)



$Q2 = W_{R.W.} - Ma/s$

$= 236.25 - (658.41/1.5)$

$= -202.7$ kN (Tension)

## c) Pile capacity:

**Compression capacity:**

$Q_{ult.} = Q_b + Q_s$

- **Bearing resistance:**

$Q_b = (q \cdot Nq) \cdot A_b$ ($\phi^* = 35-3 = 32$)

$= (20x9.5) \cdot (48) \cdot (0.196)$

$= 1787.52$ kN

- **Skin friction**

$Qs = K_{Hc} \cdot P_o \cdot \tan\delta \cdot A_{side}$

Assume $K_{Hc} = 1$

$Po = 20x5.5 = 110$ kPa

$Qs = 1x110x\tan(0.75x35)x\ 12.56$

$= 681.33$ kN.

$Q_{ult.} = 1787.52 + 681.33$

$= 2468.85$ kN.

$Q_{all.} = Q_{ult.} / F.O.S$

$= 2468.85/3$

$= 822.95$ kN > Q1 (Safe)

**Tension capacity:**

$T_{all.} = (Qs/ F.O.S) + O.W.$

$= (681.33/3) + [\pi \times (0.5^2/4) \times 8 \times 25]$

$= 227.11 + 39.27$

$= 266.38$ kN > Q2 (Safe)

Ain Shams University
Faculty of Engineering
November 2011
CES 451: Foundation Engineering

4th Year, Civil Engineering
Students of Structure Eng. Div.
Time: 1.5 hours
Mid-term Exam

| Question | 1 | 2 | 3 | Total | **Name:** | |
|---|---|---|---|---|---|---|
| Mark | | | | | **Section:** | |
| Initials | | | | | **BN:** | |

The examination consists of 3 questions in 4 pages.
Make a reasonable assumption of any missing data.
All sketches should be neatly drawn and properly dimensioned.

---

## Question 1

Design a reinforced concrete isolated footing for an R.C. column 100 cm in diameter. The proposed configuration of the footing is shown in figure. The column is carrying net working load of 850 ton. The net bearing capacity of supporting soil is 12 t/m2. Thickness of plain concrete is 10 cm.

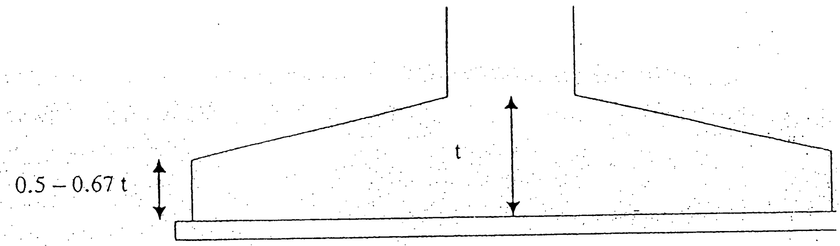
**Data :**

Materials: Concrete $f_{cu} = 300$ kg/cm$^2$, Steel 36/52, $f_y = 3600$ kg/cm$^2$

Working stress: $f_c$=100 kg/cm$^2$, $f_s$ =2200 kg/cm$^2$, $q_c$=7 kg/cm$^2$, $q_{cp}$=10 kg/cm$^2$, local bond stress = 12 kg/cm$^2$

Ultimate stress: $q_{cu}$ = 9 kg/cm$^2$, $q_{cpu}$=14.5 kg/cm$^2$, $q_{bu}$= 18 kg/cm$^2$

$C_{1\,min.}$ = 3.0 , j = 0.74
$C_1$ = 3.5 , j = 0.78
$C_1$ = 4.0 , j = 0.80
$C_1$ = 4.85 , j = 0.826




Area of different reinforcement steel bars:
Φ12: area = 1.13 cm$^2$ ; Φ16: area = 2.00 cm$^2$ ; Φ18: area = 2.84 cm$^2$ ;
Φ20: area = 3.14 cm$^2$ ; Φ22: area = 3.80 cm$^2$ ; Φ25: area = 4.91 cm$^2$

$d = C_1 \left( M_u / ( f_{cu} . b) \right)^{½}$ $As = M_u / f_y . d . j$

## Question 2

1) Evaluate the following statements (right or wrong) and comment on your evaluation (Any answer without comments is not accepted):

   a) Driven piles are suitable piling technique in city centers beside historical buildings.

   b) Pile load tests (static and/or dynamic) should be performed at least on 50% of working piles.

   c) Pile skin friction is fully mobilized at small settlement.

   d) The structural loads are carried completely by the piles of a piled raft foundation.

   e) Hammering and lifting process should be considered in the design of precast piles.

2) Attached figure shows the subsurface soil profile at a site and the geotechnical parameters of the soil layers in that site. A tower crane is to be constructed in this site. The foundation is a square footing 5m x 5m x 1.5m. The vertical load of the crane is 400 kN. The footing weight is 1100 kN The crane can rotate in any direction around its vertical axe. The maximum weight to be transported by the crane is 150 kN. The maximum lifting distance is 25.0 m.
The crane footing is founded on 4.0 driven concrete piles with pile diameter 0.6 m.

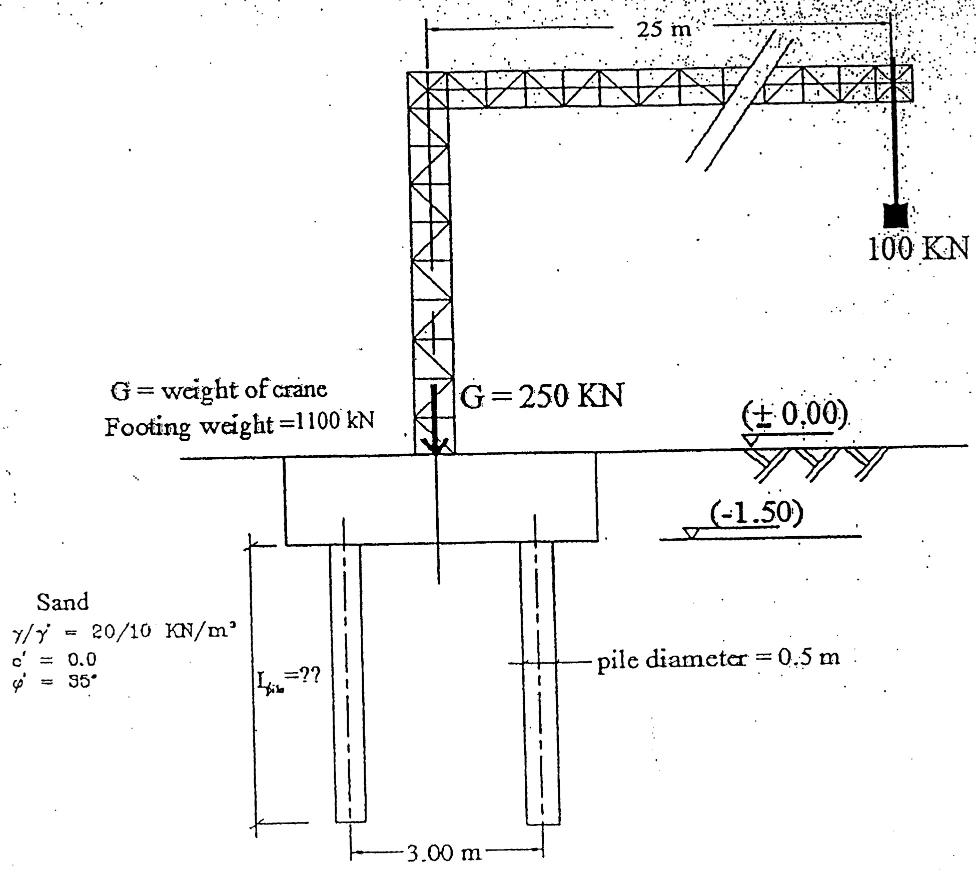
   a) Calculate the pile loads in case the crane is on x axis.

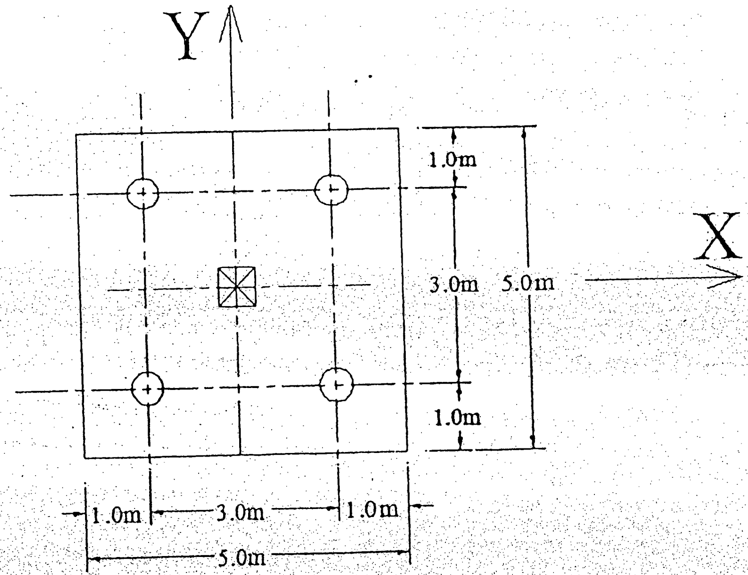
   b) Determine the pile length applying a factor of safety of 3.0.

For driven piles:
$K_{HC} = 1.0$
$\delta = \frac{3}{4}\varphi$
$N_q = 75$

$$Q_{ult} = P_b\, N_q\, \pi R^2 + \sum_{H=0}^{H=L} K_{HC}\, P_o\, (\tan\delta)\, 2\pi R\, \Delta H$$

Please assume any missing data reasonably


25 m
100 KN
G = weight of crane
Footing weight = 1100 kN
G = 250 KN
(± 0.00)
(-1.50)
Sand
γ/γ' = 20/10 KN/m³
c' = 0.0
φ' = 35°
L_pile = ??
pile diameter = 0.5 m
3.00 m


Y
X
1.0m
3.0m
5.0m
1.0m
1.0m
3.0m
1.0m
5.0m

## Question 3

A reinforced concrete footing (9.0 m long X 1.20 m wide X 1.00 m thick) is carrying a masonary wall. Use ohde equation, to determine the required masonary wall load (kN/m') equivalent to uniform settlement 8 cm beneath the footing.

<u>Input Data:</u>

Modulus of elasticity of clay = 20 MN/m$^2$.

Modulus of elasticity of R. C. = 21000 kN/cm$^2$.

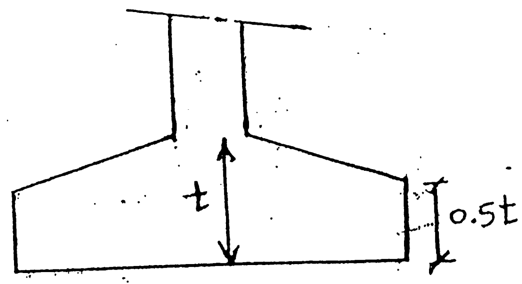
The footing should be divided into six elements.

$C_0$ = 1.60

$$C_i = \frac{C_0}{\left[1 + 1.2(i)^{1.4}\right]}$$

## Q1)

### Given:-




- $P_{col.} = 8500$ kN
- circular column, $\phi = 100$ cm
- $q_{all\ net} = 120$ KPa
- $t_{p.c.} = 10$ cm
- $F_{cu} = 300$ kg/cm$^2$ $= 30$ N/mm$^2$
- $q_{bu} = 18$ kg/cm$^2$ $= 1.8$ N/mm$^2$ (bond strength)
- $q_{cu} = 9$ kg/cm$^2$ $= 0.9$ N/mm$^2$ (shear strength)
- $q_{p_{cu}} = 14.5$ kg/cm$^2$ $= 1.45$ N/mm$^2$ (Punching strength)

### Required:-

- Design footing and Draw R.F.T. details.

### Solution:-

#### 1) Footing Plan dimensions:-

∵ col. is circular

∴ Footing will be square

∴ $t_{P.C.} = 10\ cm < 20\ cm$

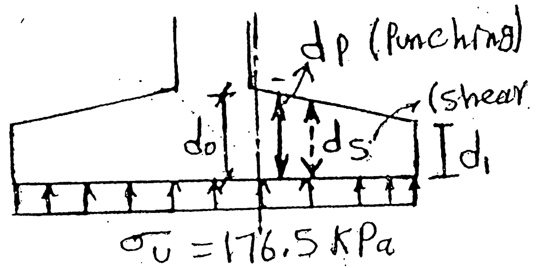
∴ $A_{RC} = \frac{8500}{120} = 70.83\ m^2 = B_{RC}^2$

∴ $B_{RC} = 8.416\ m \simeq \underline{8.50\ m}$

∴ $B_{PC} = 8.50 + 2(0.1) = \underline{8.70\ m}$

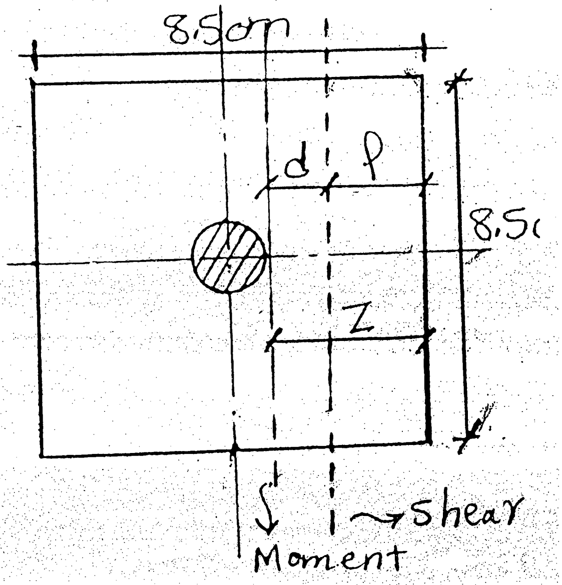
2) ultimate stress on Footing

$\sigma_U = \frac{1.5 \times 8500}{(8.5)^2} = 176.5\ kPa$




3) Moment at critical section

• على وش العمود الدائري عند أكبر depth للقاعدة $(d_o)$.




$Z = \frac{8.5 - 1.0}{2} = 3.75\ m$

∴ $M_U = 176.5 \times \frac{(3.75)^2}{2} \times 8.50$

$= 10549\ kN.m / B$

∴ $d_o = 5\sqrt{\frac{10549 \times 10^6}{30 \times \frac{8500}{2}}}$ → $\frac{B_{R.C.}}{2}$

$= 1438\ mm$

∴ Take $d_o = 1430\ mm$

$d_1 = 0.5 \times 1430 = 715 \simeq \underline{730\ mm}$

تم تقريبه لأقرب 30mm

## 4) Check Shear:-

• على بعد $d_o$ من وش العمود الدائرى

$$\ell = z - d_o = 3.75 - 1.43 = 2.32\ m$$

∴ footing depth at shear sec. $= d_s$

• يتم حساب $d_s$ بالنسبة والتناسب بين $d_o, d_1$

$$\therefore d_s = d_1 + \frac{\ell}{z}(d_o - d_1)$$

$$= 0.73 + \frac{2.32}{3.75}(1.43 - 0.73) = 1.163\ m$$

لا نحتاج لتقريبه لأنه لـ check shear فقط.

$$\therefore Q_{su} = 176.5 \times 2.32 \times 1$$

$$= 409.48\ kN/m'$$

$$\therefore q_{su} = \frac{409.48 \times 10^3}{1000 \times 1163} = 0.352\ N/mm^2$$

(1000 = عرض الشريحة ، 1163 = $d_s$)

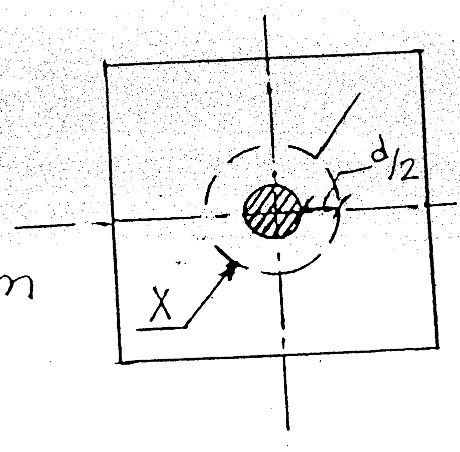
$$< q_{scu} = 0.9\ N/mm^2$$

(0.9 = Given في المسألة)

∴ Footing is safe shear

## 5) Check Punching:-

• الـ critical sec. دائرة على بعد $d_o/2$ من وش العمود الدائرى.




$$\therefore X = \phi + d_o = 1 + 1.43 = 2.43\ m$$

$\because Q_p = P_{col,u} - \sigma_U \left[ \frac{\pi X^2}{4} \right]$

$= (1.5 \times 8500) - 176.5 * \left[ \frac{\pi * (2.43)^2}{4} \right]$

$= 11931 \text{ KN}$

$\sim$ footing depth $= \underline{d_p}$ at $\underline{\text{Punching critical sec}}$

$\because d_p = d_1 + \frac{Z - d_o/2}{Z} (d_o - d_1)$

$= 0.73 + \frac{3.75 - 1.43/2}{3.75} (1.43 - 0.73)$

$= 1.296 \text{ m} \rightarrow$ للرتبي تقريبها ل 80 او 30

$\because q_{pu} = \frac{Q_{pu}}{d_p * \text{Perimeter}}$

$= \frac{11931 \times 10^3}{1296 \times \pi \times 2430} = 1.205 \text{ N/mm}^2$

($1296 \rightarrow d_p$, $2430 \rightarrow X$)

$< q_{pcu} = 1.45 \text{ N/mm}^2$ (Given)

(Safe)

---

## 6) R.F.T. :-

$As_{min.} = 1.5 * \underline{d_o} = 1.5 \times 1430 = 2145 \text{ mm}^2/\text{m}'$

$As_{req.} = \frac{10549 * 10^6}{360 * 0.826 * 1430} / 8.5 = 2919 \text{ mm}^2/\text{m}'$

**$8 \phi 22/\text{m}'$**

2

## 7) Check bond:-

(shear ← عند وش العمود)

$$q_{bond} = \frac{Q_{s\,max}}{d_o \times \pi \times n \times \phi} = \frac{[176.5 \times 3.75 \times 1m] \times 10^3}{1430 \times \pi \times 8 \times 22}$$

($d_o$: depth أكبر ; $n$: عدد الأسياخ / م' ; $\phi$: قطر السيخ بالـ mm)

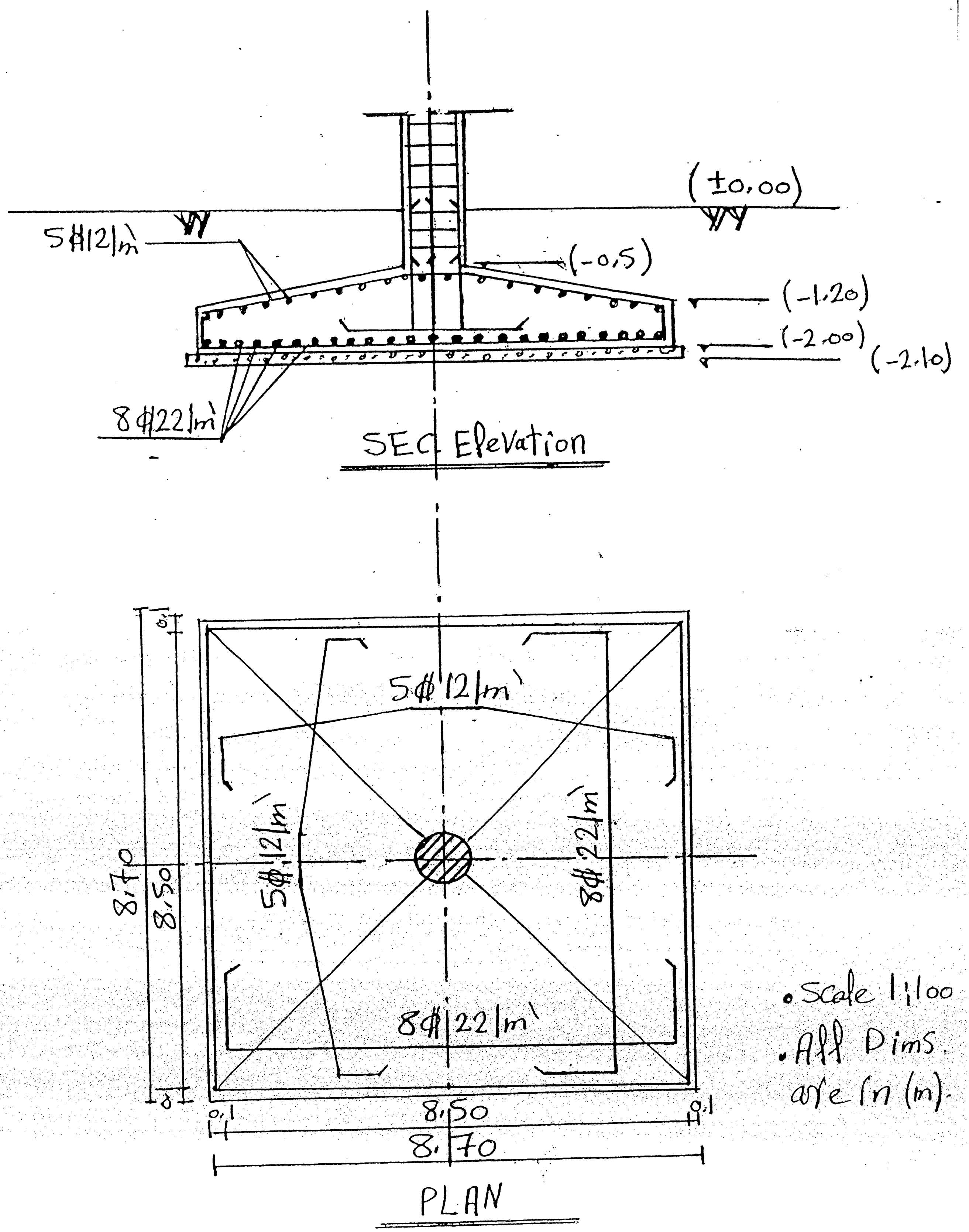
$$= 0.837 \ N/mm^2$$

$$< q_{bu} = 1.8 \ N/mm^2 \quad \text{(Given)}$$

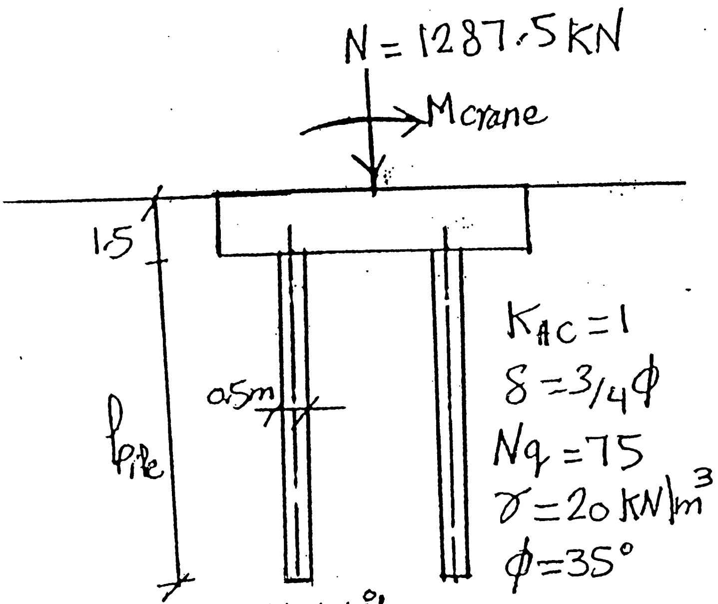
## 8) Final R.C. thickness:-

$$t_o = d_o + \text{cover} = 1430 + 70 = 1500 \ mm$$

$$t_1 = d_1 + \text{cover} = 730 + 70 = 800 \ mm$$

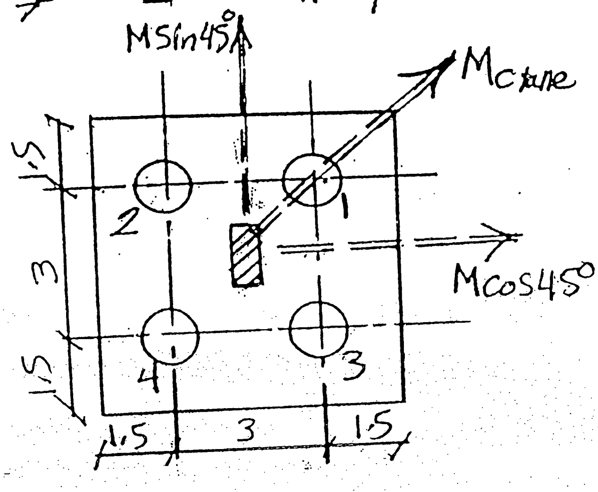

(±0,00)
5Φ12/m'
(-0,5)
(-1.20)
(-2.00)
(-2.10)
8Φ22/m'
SEC Elevation
5Φ12/m'
5Φ12/m'
8Φ22/m'
8Φ22/m'
8,70
8,50
0,1
8,50
8,70
PLAN
• Scale 1:100
• All Dims. are in (m).

## Question (2):-




### Given:-

- Footing weight = 937.5 KN
- Crane vl. load = 250 KN
- Max. weight carried by crane = 100 KN
- Crane arm = 30 m
- Driven Piles, $\phi = 0.5$ m




### Solution:-

#### i) Calculation of crane loads (X-Axis)

- $N = 937.5 + 250 + 100$
  $= 1287.5$ KN
- $M = \text{load} * \text{arm} = 30 * 100$
  $= 3000$ KN.m.

#### ii) Required pile length:-

The critical loading case is when crane plays at 45° from X-Axis. This case yields maximum effect of moment ($\cos 45° = \sin 45°$)

Pile loads:-

$M_x = M_y = 3000 \cos(45°) = 2121.32 \ KN.m.$

$$Q_{1Pile} = \frac{N}{n} \pm \frac{M_x . Y_i}{\sum Y_i^2} \pm \frac{M_y X_i}{\sum X_i^2}$$

$$\sum (X_i^2) = \sum (Y_i^2) = 4(1.5)^2 = 9 m^2$$

$Q_{max}$. (Pile No. ①)

$$Q_{max} = Q_1 = \frac{1287.5}{4} + \frac{2121.32 * 1.5}{9} + \frac{2121.32 * 1.5}{9}$$

$= 321.875 + 353.55 + 353.55$

$= 1028.98$ KN (critical Pile in Comp.)

$Q_{min}$:- (Pile No. ④)

$Q_{min} = Q_4 = 321.875 - 353.55 - 353.55$

$= -385.225$ KN (critical Pile in Tension)

Compression Pile capacity:-

$Q_{ult.} = 1028.98 * 3 = 3086.94$ KN ($Q_{ult.}$ = Acting load * F.o.)

$Q_{ult.} = Q_b + Q_s$

$Q_{b1}$ -

$$A_b = \pi \frac{(0.5)^2}{4} = 0.196\ m^2$$

$$Q_b = (\underbrace{20*1.5 + 20\ l_{pile}}_{q}) * \underbrace{75}_{N_q} * \underbrace{0.196}_{A_b}$$

$$= (441 + 294\ l_{pile})\ KN$$

$Q_{s1}$ -

$$Q_s = K_{HC}\ P_o \tan\delta * A_s$$

$$= 1\left(20*1.5 + \tfrac{20}{2} * l_{pile}\right) * \tan(3/4 * 35^\circ) * \pi * 0.5 * l_{pile}$$

$$= (23.24\ l_{pile} + 7.746\ l^2_{pile})\ KN$$

$Q_{ult.1}$ -

$$Q_{ult.} = 3086.94 = 441 + 294\ l_{pile} + 23.24\ l_{pile} + 7.75\ l^2_{pile}$$

$$\therefore 7.75\ l^2_{pile} + 317.24\ l_{pile} - 2645.94 = 0$$

$$\therefore l_{pile} = 7.10\ m$$

الطول اللازم حتى يكون Pile (1) Safe

## Tension Capacity:-

Assume $K_{HC} = K_{HT} = 1$

$T_{ult.} = Q_s + O.W. = (385.225 * 3) = 1155.675\ KN$

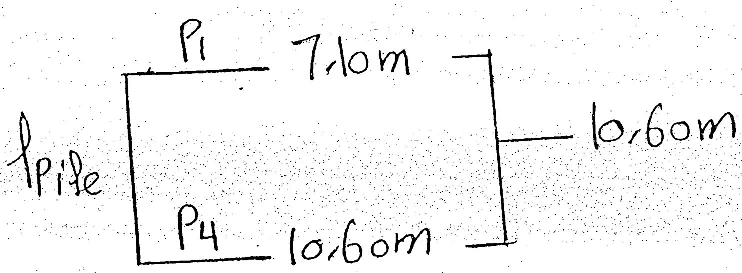
$Q_s = 23.24\ l_{pile} + 7.75\ l^2_{pile}$

$O.W. = A_b * l_{pile} * \gamma_{R.C.} = 0.196 * l_{pile} * 25 = 4.9\ l_{pile}$

$\therefore 23.24\ l_{pile} + 7.75\ l^2_{pile} + 4.9\ l_{pile} = 1155.675$

$\therefore 7.75\ l^2_{pile} + 28.14\ l_{pile} - 1155.675 = 0$

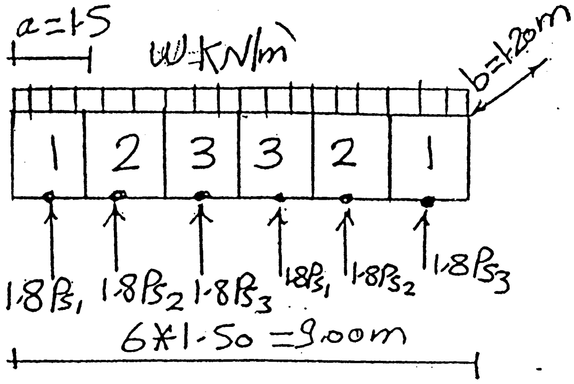
$\therefore l_{pile} = 10.53\ m \simeq 10.60\ m$



## Q3)

Given:-

* $E_s = 20\ MN/m^2 = 20*10^3\ kPa$
* $E_{RC.} = 2.1*10^7\ kPa$
* $C_o = 1.6$ , $C_i = \frac{C_o}{1+1.2(i)^{1.4}}$
* uniform settlement = 8 cm




Solution:-

∴ Wall uniform load
∴ footing is symmetric

* Assume contact stress below each element ($P_{si}$)

∴ Element reaction = $a*b*P_{si} = 1.5*1.2*P_{si} = 1.8\,P_{si}$

∴ uniform settlement

∴ $S_1 = S_2 = S_3 = 8\ cm = 0.08\ m$

Using ohde's formula:-

$$S_i = \frac{a}{E_s}\sum_{i=1}^{n} (P_{si}*C_i$$

$$\therefore C_i = \frac{C_o}{1+1.2(i)^{1.4}} = \frac{1.6}{1+1.2(i)^{1.4}}$$

$$\therefore C_1 = \frac{1.6}{1+1.2(1)^{1.4}} = 0.73$$

$$C_2 = \frac{1.6}{1+1.2(2)^{1.4}} = 0.38$$

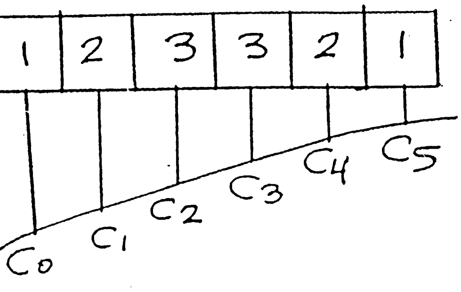
$$C_3 = \frac{1.6}{1+1.2(3)^{1.4}} = 0.24$$

$$C_4 = \frac{1.6}{1+1.2(4)^{1.4}} = 0.17$$

$$C_5 = \frac{1.6}{1+1.2(5)^{1.4}} = 0.13$$

---

## Element (1):-




$$S_1 = \frac{1.5}{20*10^3}\left[(C_0+C_5)P_{S_1} + (C_1+C_4)P_{S_2} + (C_2+C_3)P_{S_3}\right]$$

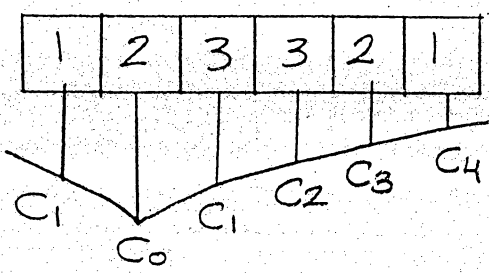
$$= \frac{1.5}{20*10^3}\left[1.73\,P_{S_1} + 0.9\,P_{S_2} + 0.62\,P_{S_3}\right]$$

$$= 0.08\ m$$

$$\therefore 1.73\,P_{S_1} + 0.9\,P_{S_2} + 0.62\,P_{S_3} = 1066.67 \quad \text{---} \quad (1)$$

---

## Element (2):-




$$S_2 = \frac{1.5}{20*10^3}\left[(C_1+C_4)P_{S_1} + (C_0+C_3)P_{S_2} + (C_1+C_2)P_{S_3}\right]$$

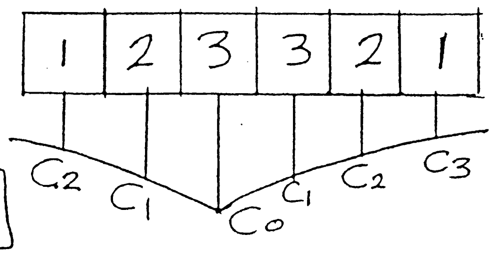
$$= \frac{1.5}{20*10^3}\left[0.9\,P_{S_1} + 1.84\,P_{S_2} + 1.11\,P_{S_3}\right]$$

$$= 0.08\ m$$

$$\therefore 0.9\,P_{S_1} + 1.84\,P_{S_2} + 1.11\,P_{S_3} = 1066.67 \quad \text{---} \quad (2)$$

---

## Element (3):-




$$S_3 = \frac{1.5}{20*10^3}\left[(C_2+C_3)P_{S_1} + (C_1+C_2)P_{S_2} + (C_0+C_1)P_{S_3}\right]$$

$$\therefore S_3 = \frac{1.5}{20*10^3}\left[0.62\,P_{S_1} + 1.11\,P_{S_2} + 2.33\,P_{S_3}\right]$$

36

$\therefore\ 0.62\, P_{s_1} + 1.11\, P_{s_2} + 2.33\, P_{s_3} = 106667$ ——— ③

---

Solving eqs ①, ②, ③, we get

$P_{s_1} = 408.4$ KPa

$P_{s_2} = 237.63$ KPa

$P_{s_3} = 235.92$ KPa

---

$\Sigma F_y = 0$

$2\,(1.8\, P_{s_1} + 1.8\, P_{s_2} + 1.8\, P_{s_3}) = W * 9.00\,m$

$\therefore\ 2 * 1.8\,(408.4 + 237.63 + 235.92) = W * 9$

$\therefore\ W = 352.78$ KN/m

Ain Shams University
Faculty of Engineering
November 2010
CES 451: **Foundation Engineering**

$4^{th}$ Year, Civil Engineering
Students of Structure Eng. Div.
Time: **1.5** hours
Mid-term Exam

| Question | 1 | 2 | 3 | 4 | Total | **Name:** | |
|---|---|---|---|---|---|---|---|
| Mark | | | | | | **Section:** | |
| Initials | | | | | | **BN:** | |

The examination consists of **4** questions in **12** pages.
Make a reasonable assumption of any missing data.
All sketches should be neatly drawn and properly dimensioned.

## Question 1

A square R.C. isolated footing is proposed to support a square column of a composite section (as shown in figure 1). The column carries an axial compression load of 2000 KN. The concrete dimensions of the column cross section are 0.4 x 0.4 m. The proposed dimensions of the R.C. footing are 2.5 x 2.5 m, and its thickness is 0.5 m as estimated from a preliminary design. It is required to check the safety of the proposed footing thickness **only for punching shear**. If the given footing thickness is unsafe, estimate the footing thickness that will be safe in punching shear (no more calculations or checks are required).

- Column dimensions: 0.4 x 0.4 m
- R.C. footing thickness: 0.5 m
- $f_{cu}$ = 30 N/mm$^2$

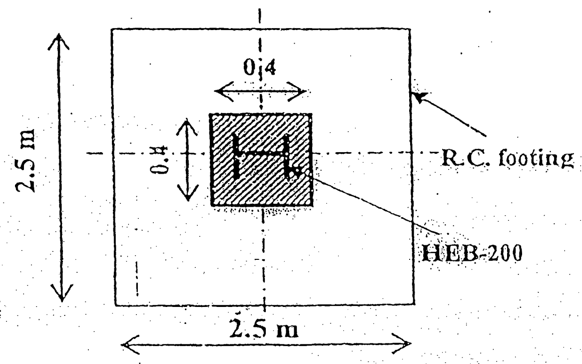



Figure (1)

## Question 2

Two isolated square footings ($F_1$ and $F_2$) are founded on a thick layer of quite homogenous silty sand followed by a rock deposit (as shown in figure 2). The thickness of the silty sand layer is 13.0 m from the ground surface. The foundation depth of the two footings is 2.0 m. The average value of the compression modulus of the silty sand is 20 MPa. It is required to:

i) Calculate the average contact pressure below each footing.
ii) Calculate the average settlement below each footing (the compressible silty sand can be divided into two layers).
iii) Estimate a reasonable value for the coefficient of subgrade reaction below each footing.
iv) Estimate the differential settlement between the two footings.
v) If the allowable foundation settlement is 30 mm and the allowable differential settlement is 1:500, check on the satisfaction of both the settlement and the differential settlement of ($F_1$ and $F_2$) with respect to the given permissible limits.

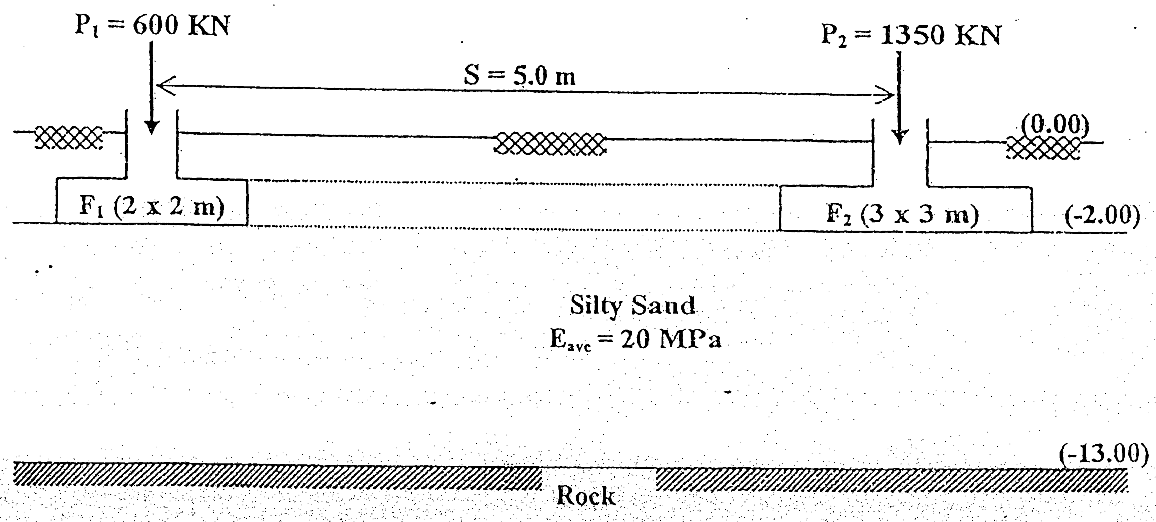



Figure (2)

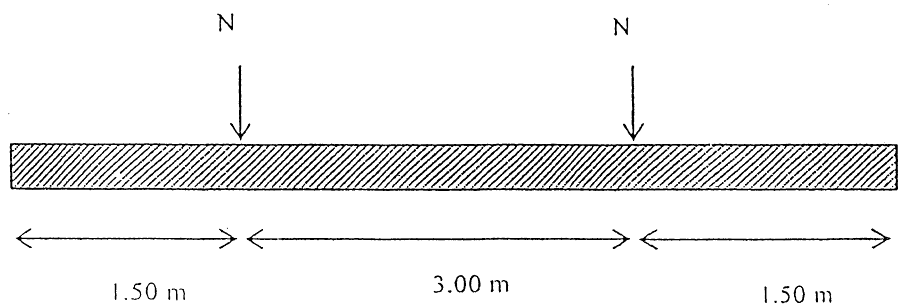
## Question 3

The R.C. combined footing shown in Figure (4.a) is 1.50 x 6.00 m, and carries two columns 3.00 m center to center. Each column is subjected to N vertical load. The footing is divided into 6 elements. The footing thickness is 100 cm. The expected settlement profile below the footing due to the columns loads is shown in Figure (4.b). It is required to determine:

i. The contact stress under the footing assuming the soil is elastic, homogenous, isotropic and semi-infinite. ($C_0$=1.33, $C_1$=0.52, $C_2$=0.30, $C_3$=0.20, $C_4$=0.16, $C_5$=0.12), $E_c$= 2000 kN/cm$^2$, $E_s$ = 20000 kN/m$^2$.

ii. The Value of N.

Figure (4.a)



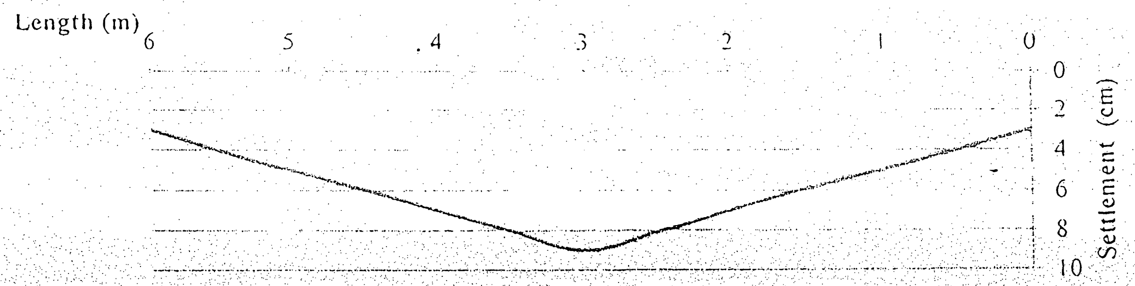




Figure (4.b) "settlement profile"

## Question 4

a- Advice the most suitable pile for following conditions:
   1- Offshore platform with water depth of 20 m
   2- Foundation of an extension building of a hospital where the soil consists of 15 m sandy silt followed by very dense sand and the ground water table lies 2.0 below ground surface
   3- Foundation of a road bridge where the subsoil consists of dense sand and the ground water table lies just at ground surface

b- Comment on static pile load test using anchors or Osterberg cell as a reaction system.

c- Results of geotechnical investigation at a site showed that the soil profile (see figure 4) consists of a thick layer of medium to dense sand down to 15.0 m followed by rock formations. A column with working load (dead and live loads) of 5000 kN is to be founded on driven piles with pile diameter 0.5 m in the sand layer. The pile length is 10 m below foundation level. The foundation depth is just at the ground water table that lies 1.0 m below ground surface. Calculate the bearing capacity of the single pile under compression and under tension loads.

Soil Parameters

| Soil parameter | Sand |
|---|---|
| E [MN/m²] | 50 |
| ν [-] | 0.3 |
| γ / γ′ [kN/m³] | 20/10 |
| c′ [kN/m²] | 0.0 |
| φ′ [°] | 33 |

$E$ Primary loading stiffness

$\nu$ Unloading/reloading Poisson's ratio

$\gamma / \gamma'$ Total / Effective unit weight of soil

$c'$ Cohesion

$\varphi'$ Angle of internal friction

Pile diameter D = 0.5 m

Pile length L = 10 m

$E_{concrete}$ = 30000 MPa

For driven piles:

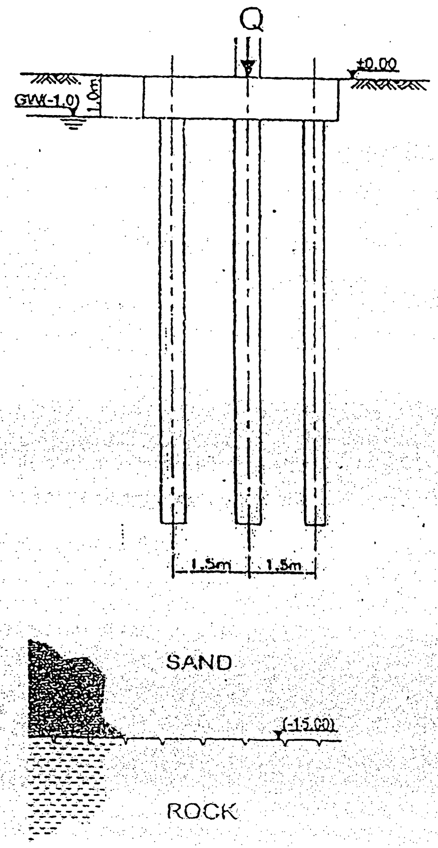
$K_{HC} = 1.0$

$K_{HT} = 0.7$

$\delta = \frac{3}{4}\varphi$

$N_q = 60$

$$Q_{ult} = P_b\, N_q\, \pi R^2 + \sum_{H=0}^{H=L} K_{HC}\, P_o\, (\tan\delta)\, 2\pi R\, \Delta H$$




section

Figure (4)

# Answer of Midterm Exam

## 4. str (2010-2011)

## Question (1):-

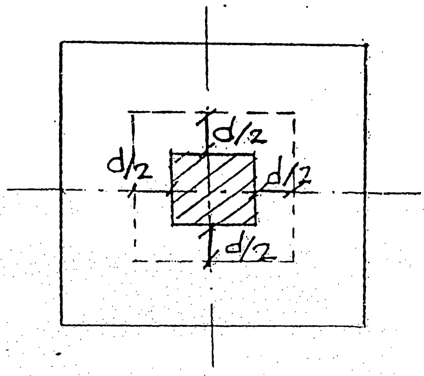
### Given:-

$P_{col.} = 2000\ KN$ , $(a*b) = 0.4*0.4$ , $L_{R.C.}*B_{R.C.} = 2.5m*2.5m$

$t_{R.C.} = 0.5m$ , $F_{cu} = 30\ N/mm^2$

### Solution:-

* $\sigma_u = \frac{1.5*2000}{(2.5)^2} = 480\ KPa$




* $Q_p = 1.5*2000 - 480*(0.83)^2$
$= 2669.33\ KN$

* $q_{pu} = \frac{2669.33*10^3}{430*4*830} = 1.87\ N/mm^2$

* $q_{pcu} = 0.316\sqrt{\frac{30}{1.5}} = 1.413\ N/mm^2$

∴ $q_{pu} > q_{pcu}$ (unsafe) ⟹ increase dimensions

∴ Try d = 480 m.m.

∴ $Q_{pu} = 2628.28\ KN$ ⟹ $q_{pu} = 1.55\ N/mm^2 > q_{pcu}$ (unsafe)

∴ increase d ⟹ Try d = 530 m.m.

∴ $Q_{pu} = 1.5*2000 - 480*(0.93)^2 = 2584.85\ KN$

∴ $q_{pu} = \frac{2584.85*10^3}{530*4*930} = 1.311\ N/mm^2 < q_{pcu} = 1.413\ N/mm^2$ (safe)

∴ $t_{R.C.} = 530 + 70 = 600\ mm.$

Question (2):-

i)

(F1)

$$q_1 = \frac{600}{(2)^2} = 150\ KN/m^2$$

(F2)

$$q_2 = \frac{1350}{(3)^2} = 150\ KN/m^2$$

ii)

**(F1)**

• calculation of effective depth

$$\frac{600}{(2+H_1)^2} = 0.1\ q_{act}$$

$$q_{act} = \frac{600}{2*2} = 150\ KPa$$

$$\therefore \frac{600}{(2+H_1)^2} = 0.1*150$$

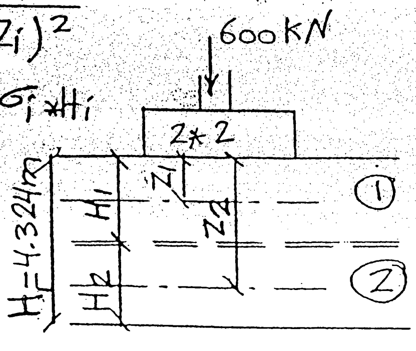
$$\therefore H_1 = 4.324\ m$$

| layer | $H_i$ (m) | $Z_i$ (m) | $\Delta\sigma_i$ (KN/m²) | $E_i$ (KN/m²) | $\Delta h_i$ (m) |
|---|---|---|---|---|---|
| 1 | 2.162 | 1.081 | 63.2 | $20*10^3$ | $6.83*10^{-3}$ |
| 2 | 2.162 | 3.243 | 21.83 | $20*10^3$ | $2.36*10^{-3}$ |
| | | | | Σ | $9.19*10^{-3}$ |

$$*\ \Delta\sigma_i = \frac{600}{(2+Z_i)^2}$$

$$*\ \Delta h_i = 1/E_i * \Delta\sigma_i * H_i$$

$$*\ \Delta h_{F_1} = 0.919\ Cm$$




**(F2)**

• calculation of effective depth

$$\frac{1350}{(3+H_2)^2} = 0.1\ q_{act}$$

$$q_{act} = \frac{150}{3*3} = 150\ KPa$$

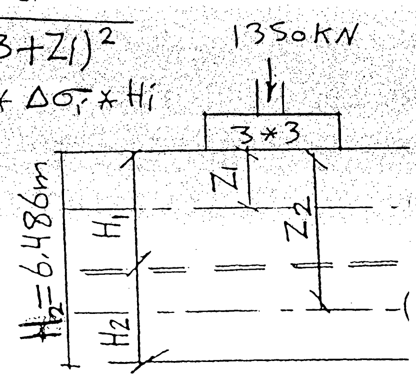
$$\therefore \frac{1350}{(3+h_2)^2} = 0.1*150$$

$$\therefore H_2 = 6.186\ m$$

| layer | $H_i$ (m) | $Z_i$ (m) | $\Delta\sigma_i$ (KN/m²) | $E_i$ (KN/m²) | $\Delta h_i$ (m) |
|---|---|---|---|---|---|
| 1 | 3.243 | 1.6215 | 63.2 | $20*10^3$ | 0.010 |
| 2 | 3.243 | 4.8645 | 21.83 | $20*10^3$ | 3.54 |
| | | | | Σ | 0.013 |

$$*\ \Delta\sigma_i = \frac{1350}{(3+Z_i)^2}$$

$$*\ \Delta h_i = 1/E_i * \Delta\sigma_i * H_i$$




$$\therefore \Delta h_{F_2} = 1.3785\ Cm$$

iii)

(F1)

$$K_1 = \frac{\text{Contact stress}}{\text{Settlement}}$$

$$= \frac{150}{9.19*10^{-3}}$$

$$= 16322.09 \ KN/m^3$$

(F2)

$$K_2 = \frac{\text{Contact stress}}{\text{Settlement}}$$

$$= \frac{150}{0.013785}$$

$$= 10881.4 \ KN/m^3$$

iv)

$$\text{Differential settlement} = \frac{\Delta h_{F2} - \Delta h_{F1}}{S} = \frac{1.3785 - 0.919}{500}$$

$$= \frac{0.4595}{500}$$

$$= 9.19*10^{-4}$$

v)

- $\Delta h_{all} = 30$ m.m.
- $\Delta h_{F_1} = 9.19$ mm. $\angle$ 30 mm. (Satisfy)
- $\Delta h_{F_2} = 13.785$ m.m. $\angle$ 30 m.m. (Satisfy)

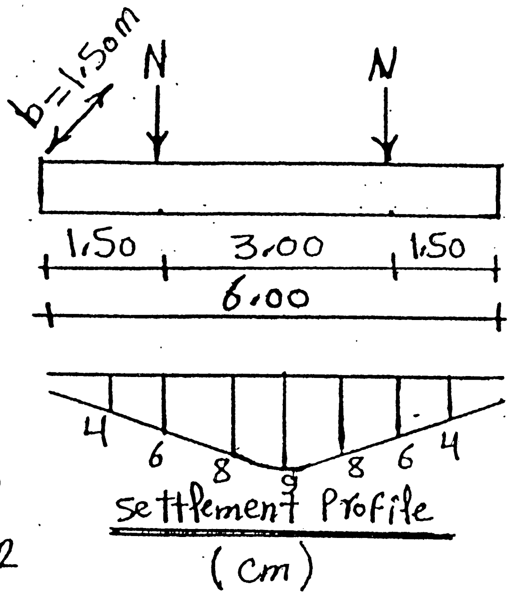
---

- Allowable differential settlement $= 1/500$
- Differential settlement $= \frac{0.4595}{500} \angle 1/500$ (Satisfy)

## Question (3):-

### Given:-

- thickness of Footing $= 100\,cm$
- settlement Profile
- $E_c = 2000\ KN/cm^2 = 2 \times 10^7\ KPa$
- $E_s = 20000\ KPa$
- $C_0 = 1.33,\ C_1 = 0.52,\ C_2 = 0.30,\ C_3 = 0.20,\ C_4 = 0.16,\ C_5 = 0.12$

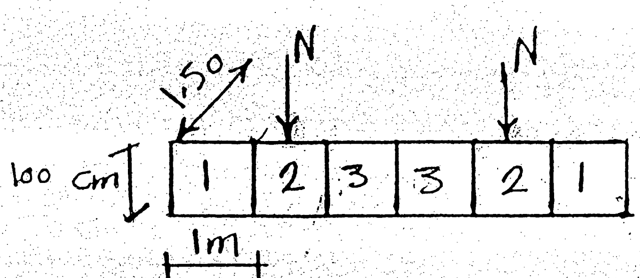



### Required:-

1) Contact stress under the footing, assuming the soil is elastic, homogenous, isotropic and semi infinite.
2) The Value of (N)

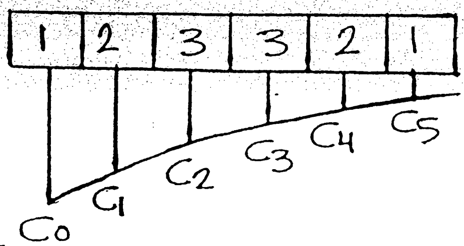
### Solution:-

- Assume Contact stress ($P_{si}$) below each element.




### Δ-Ps eqs:-

$$\Delta_i = \frac{a}{E_s} \sum_{i=1}^{n} C_i P_{si}$$



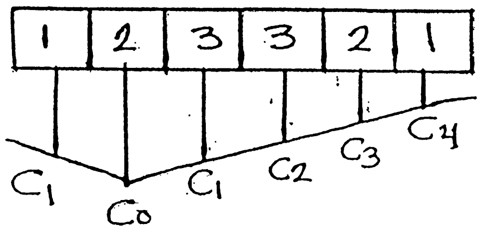

$$\therefore \Delta_1 = \frac{1}{20000}\left[C_0 P_{s1} + C_1 P_{s2} + C_2 P_{s3} + C_3 P_{s3} + C_4 P_{s2} + C_5 P_{s1}\right]$$

$\therefore \Delta_1 = 0.04 = \frac{1}{20000}\left[1.45\, P_{S_1} + 0.68\, P_{S_2} + 0.5\, P_{S_3}\right]$

↓ Sett. Profile ال من الخطي

$$\therefore 1.45\, P_{S_1} + 0.68\, P_{S_2} + 0.5\, P_{S_3} = 800 \quad \text{------} \quad (1)$$

- $\Delta_2 = \frac{1}{20000}\left[C_1 P_{S_1} + C_0 P_{S_2} + C_1 P_{S_3} + C_2 P_{S_3} + C_3 P_{S_2} + C_4 P_{S_1}\right]$

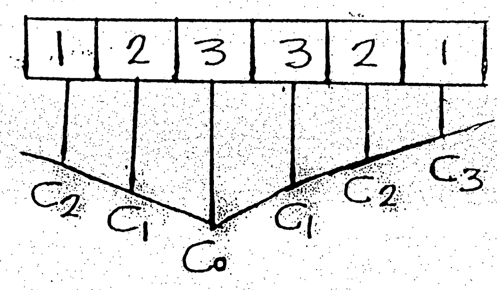



$\therefore \Delta_2 = 0.06 = \frac{1}{20000}\left[0.68\, P_{S_1} + 1.53\, P_{S_2} + 0.82\, P_{S_3}\right]$

Sett. Profile الوسط

$$\therefore 0.68\, P_{S_1} + 1.53\, P_{S_2} + 0.82\, P_{S_3} = 1200 \quad \text{------} \quad (2)$$

- $\Delta_3 = \frac{1}{20000}\left[C_2 P_{S_1} + C_1 P_{S_2} + C_0 P_{S_3} + C_1 P_{S_3} + C_2 P_{S_2} + C_3 P_{S_1}\right]$




$\therefore \Delta_3 = 0.08 = \frac{1}{20000}\left[0.50\, P_{S_1} + 0.82\, P_{S_2} + 1.85\, P_{S_3}\right]$

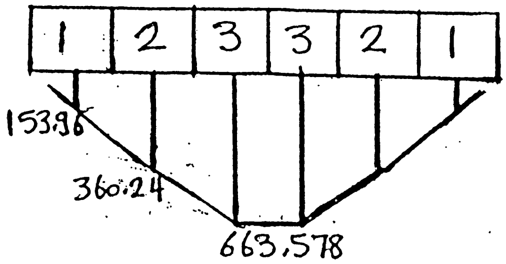
$$\therefore 0.50\, P_{S_1} + 0.82\, P_{S_2} + 1.85\, P_{S_3} = 1600 \quad \text{------} \quad (3)$$

Solving eqs ①, ②, ③, we get

$P_{s_1} = 153.96 \ KN/m^2$

$P_{s_2} = 360.24 \ KN/m^2$

$P_{s_3} = 663.578 \ KN/m^2$




Contact stress
(kPa)

• $\sum F_y = 0$

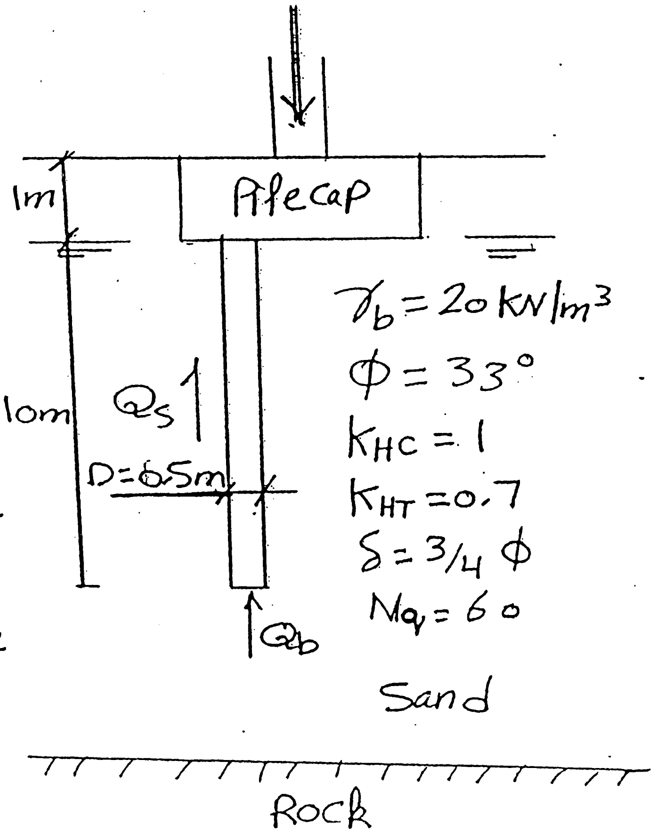
$\therefore \ \phi N = \phi \left[1.5 P_{s_1} + 1.5 P_{s_2} + 1.5 P_{s_3}\right]$

$\therefore \ N = \left[1.5 * 153.96 + 1.5 * 360.24 + 1.5 * 663.578\right]$

$= 1771.17 \ KN$

## Question (4) :-

### 1) Compression Capacity :-




- $Q_{ult.} = Q_b + Q_s$

**A) $Q_b$ :- ($\phi$-Soil)**

$$Q_b = P_b\ N_q\ \pi R^2$$

- $P_b = 20*1 + 10*10 = 120\ kN/m^2$
- $N_q = 60$ (Given)
- $A_b = \frac{\pi (D)^2}{4} = \frac{\pi (0.5)^2}{4} = 0.196\ m^2$

$\therefore Q_b = 120 * 60 * 0.196$
$= 1411.2\ kN$

---

**B) $Q_s$ :- ($\phi$-Soil)**

$$Q_s = (K_{HC}\ P_o\ \tan\delta)\ A_{side}$$

- $A_s = \pi * 0.5 * 10 = 15.7\ m^2$
- $K_{HC} = 1$ (Given)
- $P_o = 20*1 + 10*5 = 70\ kN/m^2$
- $\delta = 3/4\ \phi = 3/4 * 33° = 24.75°$

$\therefore Q_s = (1 * 70 * \tan 24.75) * 15.7 = 506.646\ kN$

---

**$Q_{ult.}$ :-**

- $Q_{ult.} = 1411.2 + 506.646 = 1917.846\ kN$

## 2) Tension Capacity :-

$$T_{ult.} = Q_s + O.W.$$

- $O.W. = \frac{\pi (0.5)^2}{4} * 10 * 25 = 49.087 \text{ kN}$

  ($10 \rightarrow L_{pile}$, $25 \rightarrow \gamma_{R.C.}$)

- $Q_s = (K_{HT} \; P_o \; \tan\delta) \; A_{side}$

  $= (0.7 * 70 * \tan 24.75) * 15.7$

  $= \underline{354.652 \text{ KN}}$

$\therefore T_{ult.} = 354.652 + 49.087 = \underline{403.739 \text{ KN}}$

Ain Shams University
Faculty of Engineering
November 2009
**CES 451: Foundation Engineering**

4th Year, Civil Engineering
Students of Structure Eng. Div.
Time: **1.5** hours
Mid-term Exam

| Question | 1 | 2 | 3 | Total | **Name:** | |
|---|---|---|---|---|---|---|
| Mark | | | | | **Section:** | |
| Initials | | | | | **BN:** | |

The examination consists of **3** questions in 4 pages.
Make a reasonable assumption of any missing data.
All sketches should be neatly drawn and properly dimensioned.

---

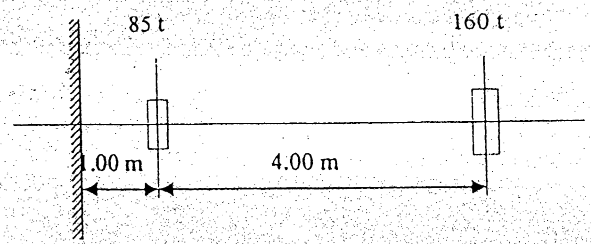
## Question 1

A) A footing is square (10.5x10.5 m) and carries total load of 1800 ton. The soil profile at the site is 20 meters of medium to very dense silty sand followed by a layer of moderately strong sandstone. The modulus of compressibility for the silty sand layer varies linearly with depth from 200 kg/cm2 at foundation level (-3.00 m) to 1000 kg/cm2 at the base of stratum. In analyzing the footing using computer software, it was divided into 230 elements (each element 0.5x0.5 m), and the soil was represented by springs. It is required to:

i) Determine the average settlement of the footing due to the given load. (The silty sand layer should be divided into 4 layers).
ii) Estimate the value of the coeficient of subgrade reaction below the footing.
iii) Calculate the stiffness of interior spring, edge spring, and corner spring.

B) R.C. column (40x100 cm) carries a working net vertical load equals 30 ton, and permenant working bending moment equals 45 m.t. . The net bearing capacity of the ssoil is 6 t/m2. It is required to determine the concrete dimensions of the footing so that the contact stress beneath thefooting is uniform, and sketch the reinforcement details.

C) For the two columns shown in figure, suggest a suitable type of foundation in each case. Draw neat sketches showing the concrete dimensions.




Case 1 : $q_{all} = 16.5$ t/m$^2$

Case2 : $q_{all} = 8.0$ t/m$^2$

# Question 2

Evaluate the following statements (right or wrong) and comment on your evaluation *(Any answer without comments will not be accepted)*:

i. Bored piles are the most suitable piling technique in off-shore structure

ii. Bentonite slurry is suitable to be used to stabilize the drilling hole during construction of large diameter bored piles in sandy soil.

iii. Pile integrity test is a quick and economic way to determine ultimate pile capacity.

iv. Pile skin friction is mobilized under relatively small displacements where pile base resistance needs relatively large displacements to be mobilized.

v. In a pile group under tension loads, the own weight of soil between the piles can govern the pullout resistance of the pile group.

vi. In a pile group that is connected with a rigid pile cap and subjected to horizontal load, the horizontal loads are distributed equally among all piles.

vii. Settlement of pile group under vertical compression load is smaller that the settlement of the single pile under average load.

**2.2** Results of geotechnical investigation at a site showed that the soil profile consists of a thick layer of very dense sand down to 25.0 m followed by rock formations, as shown in Figure (3). A bridge pier has working load (dead and live loads including the weight of the superstructure, pier and pile cap) of 36000 KN is to be founded on large diameter bored piles with pile diameter of 1.0 m in the sand layer. Horizontal forces due to braking and wind loads in both X and Y directions are as follows: $H_x$ = 1000 KN and $H_y$= 400 KN. The pile length is 15 m. The foundation depth is just at the ground water table that lies 1.5 m below ground surface. Nine piles were arranged below the pile cap, as shown in Figure (3). The following are required:

i. Calculate the settlement of the pile group, neglecting the applied moments.

ii. If the settlement of the single pile under a working load of 4000 KN is 6 mm, calculate the pile group action.

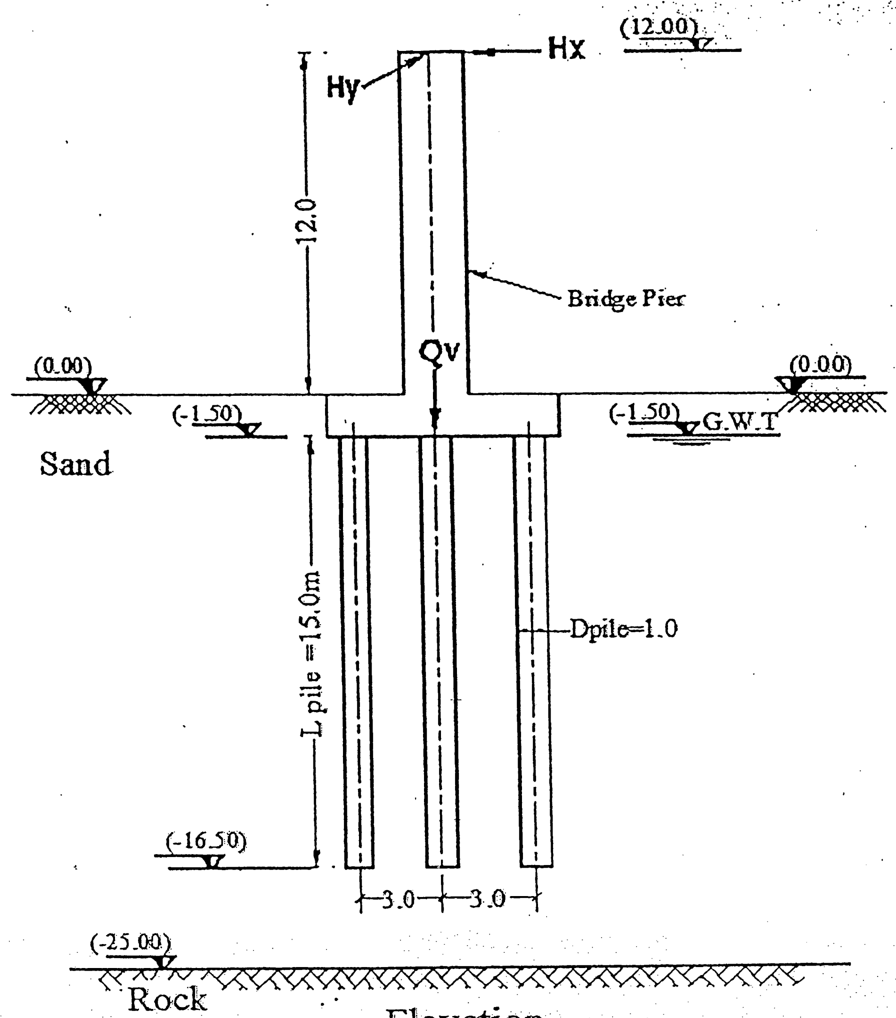
iii. Determine the vertical loads taken by piles no. P1, P2 and P3.

iv. Determine the factor of safety of piles no. P1, P2 and P3 using the following data taken from a pile load test on a nonworking pile:

a) The average ultimate pile skin friction along the pile shaft = 180 kPa

b) The ultimate pile base resistance = 3000 kPa.

v. Comment on the results of point (iv).

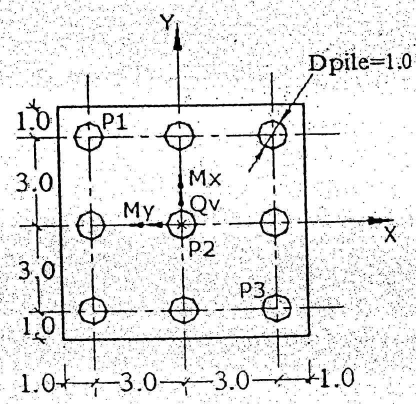



Elevation

Soil Parameters

| Soil parameter | Sand |
|---|---|
| E [MN/m²] | 100 |
| ν [-] | 0.3 |
| γ/γ′ [kN/m³] | 20/10 |
| c′ [kN/m²] | 0.0 |
| φ′ [°] | 38 |

E = Primary loading stiffness
ν = Poisson's ratio
γ/γ′ = Total / Effective unit weight of soil
c′ = Cohesion
φ′ = Angle of internal friction

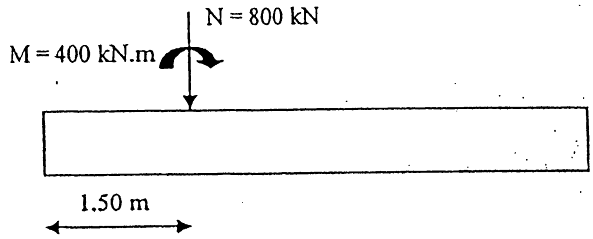



Plan

## Question 3

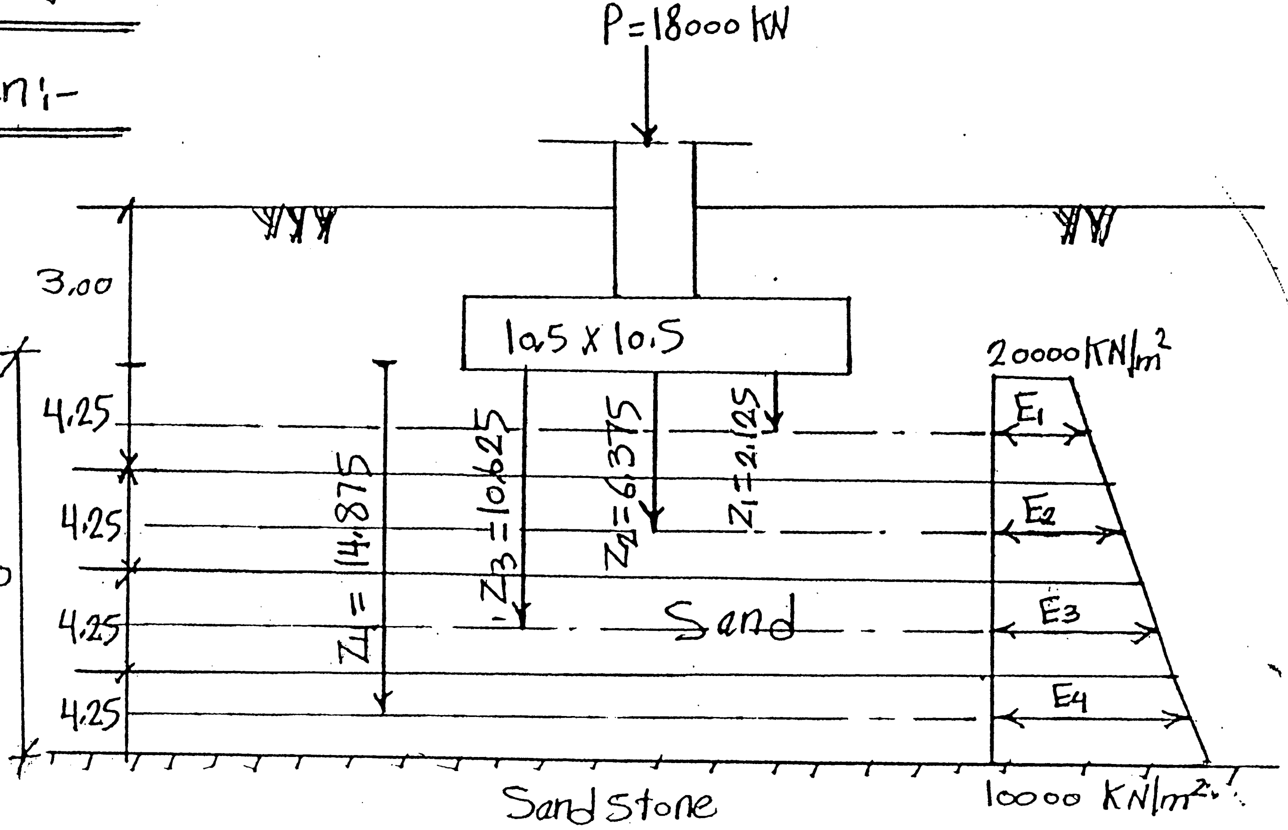
The R.C. footing shown in figure is 1.5 x 4 m, with thickness = 110 cm. The footing can be divided into 4 elements. <u>It is required to:</u>

i) Determine the contact stress under the footing assuming the soil is elastic, homogeneous, isotropic, and semi infinite. ($C_o$ = 1.33, $C_1$= 0.52, $C_2$= 0.3, $C_3$= 0.2, $C_4$= 0.15, $C_5$ = 0.12) Ec = 2000 kN/cm$^2$ and Es = 1000 kN/m$^2$.



## Q1) A)

**Given:-**




• **Determine the effective depth:-**

$$q_{act} = \frac{18000}{(10.5)^2} = 163.265 \text{ KPa} \quad \text{(contact stress)}$$

$$\Delta\sigma = 0.1\, q_{act} \quad \text{(at effective depth)}$$

$$\therefore \frac{18000}{(10.5+H)^2} = 0.1 \times 163.265$$

$$\therefore H = 22.7 \text{ m}$$

• لا يوجد معلومات عن طبقة ال Sandstone وبالتالى يتم حساب الهبوط لطبقة ال Sand فقط.

$$\therefore \underline{H = 17 \text{ m}}$$

## i) Calculation of average settlement:-

$$\Delta h = \frac{1}{E_s} \times \Delta\sigma_Z \times H$$

- Average settlement
- use 2:1 method
- $\Delta\sigma_Z = \frac{18000}{(10.5+Z)^2}$

| layer | Hi (m) | Zi (m) | Δσ (kN/m²) | E (KPa) | Δh (m) |
|---|---|---|---|---|---|
| 1 | 4.25 | 2.125 | 113 | 30000 | 0.016 |
| 2 | 4.25 | 6.375 | 63.2 | 50000 | 0.0054 |
| 3 | 4.25 | 10.625 | 40.3 | 70000 | 0.0024 |
| 4 | 4.25 | 14.875 | 28 | 90000 | 0.0013 |
| | | | | | 0.025 |

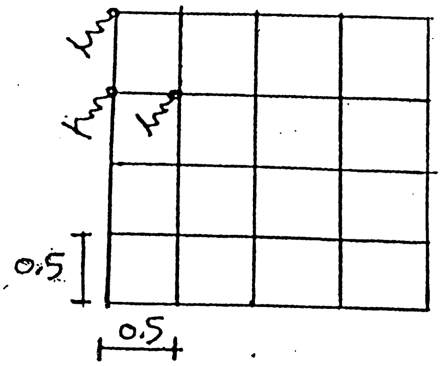
- $\Delta h = 0.025\ m = 2.50\ cm$

## ii) Modulus of subgrade reaction:-

$$K_{S_o} = \frac{\text{Contact stress}}{\text{Average Settlement}} = \frac{163.265}{0.025} = 6530.6\ KN/m^3$$

## iii) Stiffness of springs:-

### Interior spring:-




$$K_{si} = K_{so} * [0.5 \times 0.5]$$

$$= 6530.6 * [0.5 * 0.5]$$

$$= 1632.65 \ KN/m$$

### Edge spring:-

$$K_{se} = \frac{K_{si}}{2} = \frac{1632.65}{2} = 816.325 \ KN/m$$

### Corner spring:-

$$K_{sc} = \frac{K_{si}}{4} = 408.16 \ KN/m$$

## Q1) B)

### Given:-

- Col. (40x100) cm , $N = 300\ kN$ , $M = 450\ kN.m$ ↳ Permenant moment
- $q_{all_{net}} = 60\ kPa$

### Required:-

1) concrete dimensions of the footing so that the contact stress is uniform.
2) sketch reinforcement details.

### Solution:-

- Assume t.p.c. = 10 cm

(لاحظ: في مسائل ال N و M لو اذا لم تكن t.p.c. معطاة، يتم فرضها ١٠ سم).

∵ Permenant moment , and uniform stress is required

∴ The effect of <u>M</u> should be mitigated

∴ يتم ترحيل .c.g القاعدة عن .c.g العمود مسافة (e) في اتجاه رأس سهم ال moment.

$$e = \frac{M}{N} = \frac{450}{300} = 1.50\ m$$

- $A_{R.C.} = \dfrac{N}{q_{all_{net}}} = \dfrac{300}{60} = 5\,m^2 = L_{R.C.} \times B_{R.C.}$ ——— (1)

- $L_{RC} - B_{RC} = b - a = 0.6\,m$ ——— (2)

solving eqs (1) , (2) we get

$L_{R.C.} = 2.60m \quad , \quad B_{R.C.} = 2.00m$

Check Footing dimensions:-

- $\dfrac{L_{R.C.}}{2} = 2.60/2 = 1.30\,m$

- $e + b/2 = 1.50 + 1/2 = 2.00m$

$\therefore \ (e + b/2) > \dfrac{L_{R.C.}}{2}$

∴ العمود يقع خارج حدود القاعدة

∴ يتم تعديل أبعاد القاعدة كالآتي :-

Take $\dfrac{L_{R.C.}}{2} = e + b/2 = 1.5 + 0.5 = 2.00m$

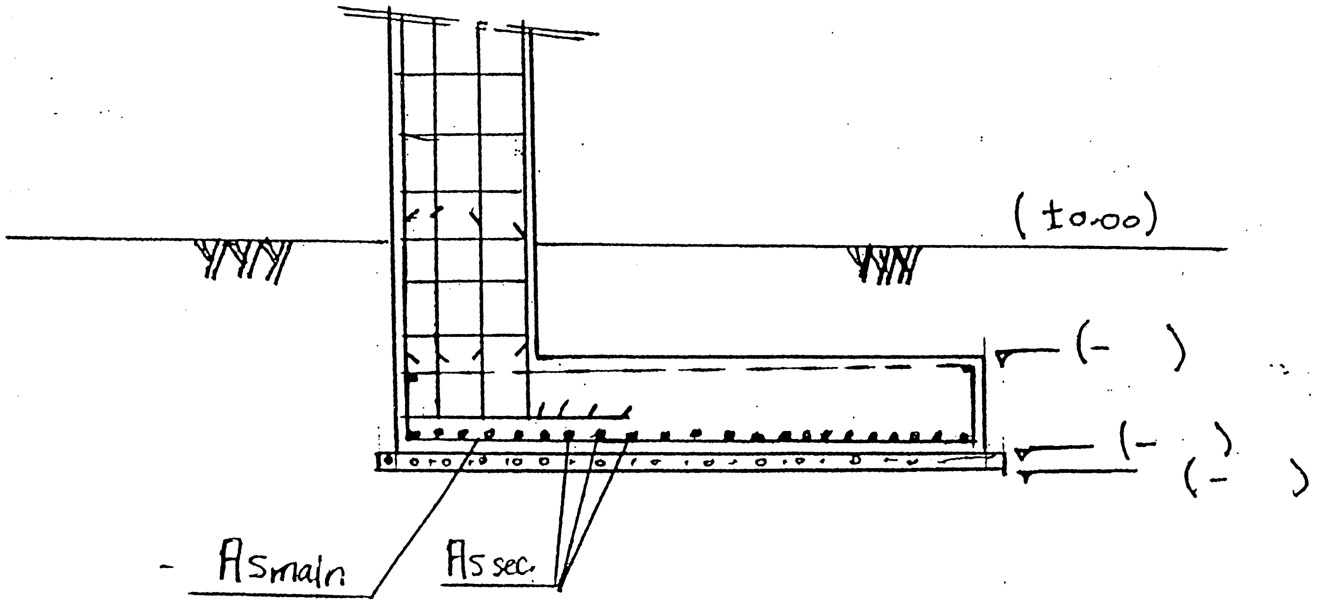
$\therefore \ L_{R.C.} = 4.00m$ (Minimum $L_{R.C.}$)

$\therefore \ B_{R.C.} = \dfrac{A}{L_{R.C.}} = \dfrac{5}{4} = 1.25m$

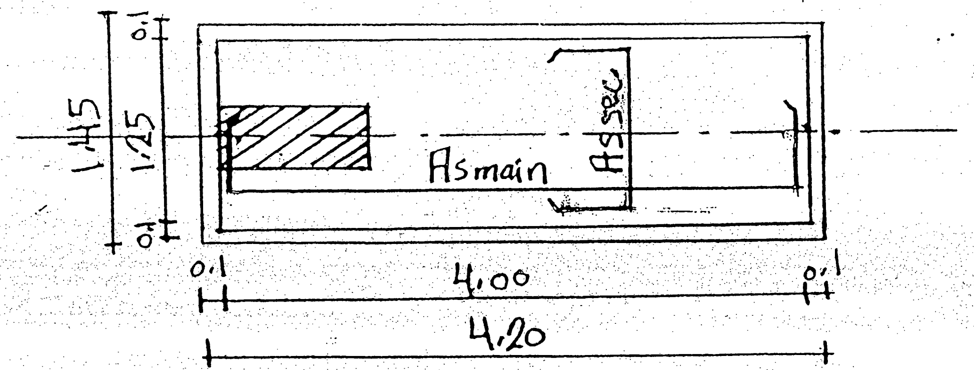
Check:-

$B_{R.C.} = 1.25\,m > a = 0.40m$ (يتم التأكد أن عرض القاعدة المسلحة أكبر من عرض العمود)

$\therefore \ L_{PC} = 4 + 2(0.10) = 4.20m \quad , \quad B_{PC} = 1.25 + 2(0.1) = 1.45$



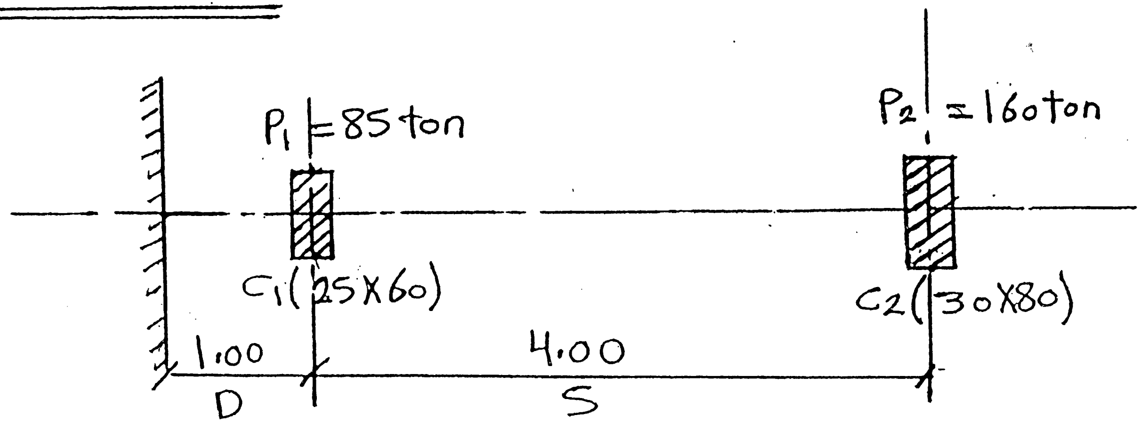

Elevation




PLAN

## Q1) c)

### Case ① :-




• $q_{all\,net} = 16.5\ t/m^2$ .

### $C_1$ :-

ـ العمود $(C_1)$ يبعد مسافة (D) عن حد الجار

∴ Try Rectangular Isolated Footing

Assume $t_{P.C.} = 40$ cm

∴ $A_{PC} = \dfrac{85}{16.5} = 5.20\ m^2$ ——— ①

$L_{PC} - B_{PC} = 0.35 m$ ——— ②

Solving eqs ① , ② we get

$B_{PC} = 2.15 m > D/2$

∴ يتم تعديل أبعاد القاعدة كالآتي :-

Take $B_{PC} = 2.00$ m

$\therefore L_{PC} = \frac{5.20}{2.00} = 2.60\,m$

$\therefore L_{RC} = 2.60 - 2(0.4) = 1.80\,m$

$\therefore B_{R.C.} = 2.00 - 2(0.4) = 1.20\,m$

---

## C2 :-

$A_{PC} = \frac{160}{16.5} = 9.70\,m^2 = L_{PC} \times B_{PC}$ ------ (1)

$L_{PC} - B_{PC} = 0.8 - 0.3 = 0.50\,m$ ------ (2)

$\therefore L_{PC} = 3.40\,m$

$B_{PC} = 2.90\,m$

$\therefore L_{RC} = 3.40 - 2(0.4) = 2.60\,m$

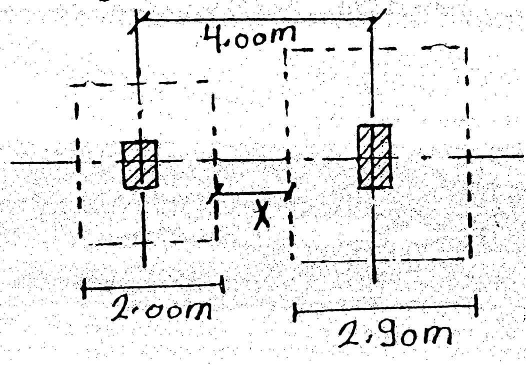
$B_{RC} = 2.90 - 2(0.4) = 2.10\,m$

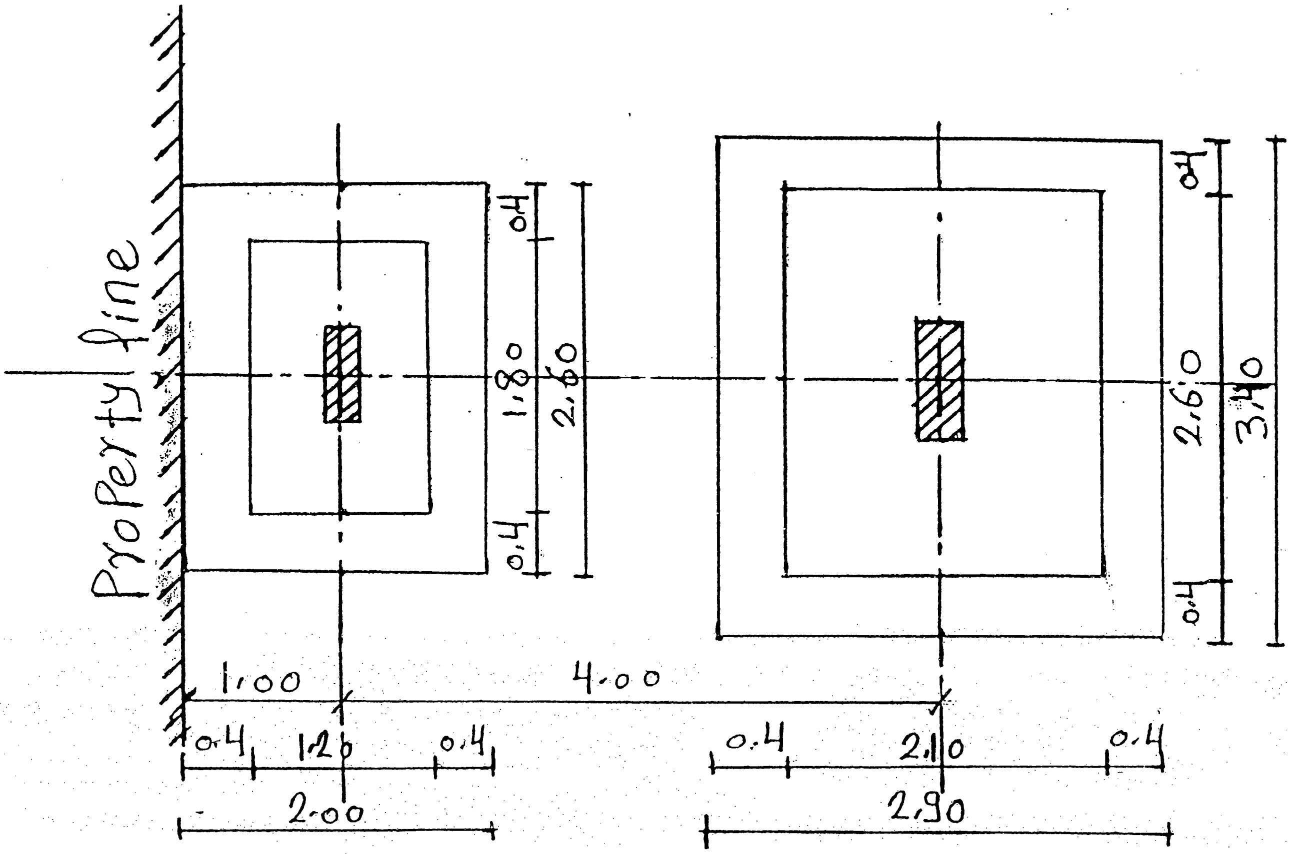
---

## Check intersection of footings :-

يتم التأكد من تداخل القواعد الشغالة (على حسب قيمة tpc)

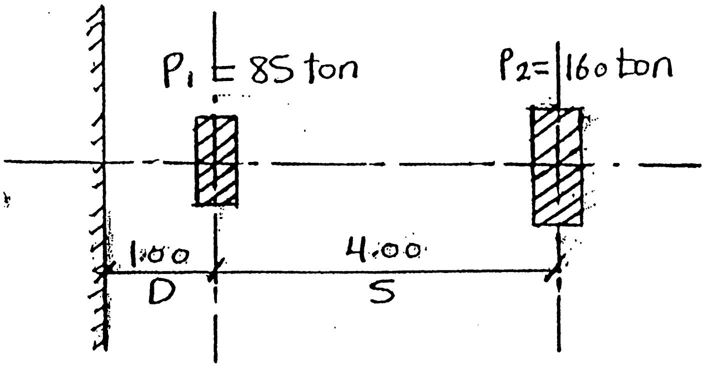
- clearance $(X) = 4.00 - \frac{2}{2} - \frac{2.9}{2}$
  $= 1.55\,m$



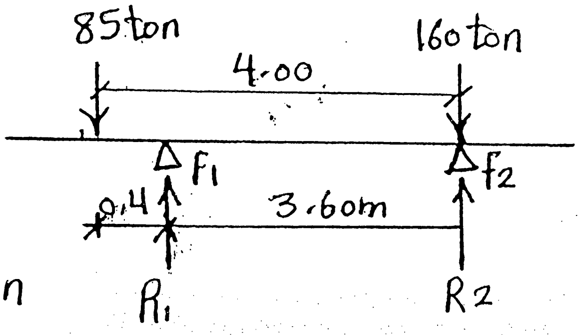

Property line
1.00
4.00
0.4
1.20
0.4
2.00
0.4
2.10
0.4
2.90
0.4
1.80
2.60
0.4
0.4
2.60
3.40
0.4

## Case (2):-

- $q_{all\,net} = 8.0\ t/m^2$
- Based on case (1), we can't use Isolated footing




## Try strap beam:-




Assume $e = 0.1\,S$

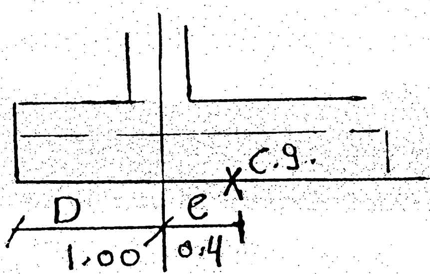
$= 0.1 * 4$

$= 0.40\ m$

$\Sigma M_{@2} = 0 \longrightarrow R_1 = 94.4$ ton

$\Sigma F_y = 0 \longrightarrow R_2 = 150.6$ ton

## $(F_1)$:-




Assume $t_{PC} = 40\ cm$

$L_{PC} = L_{RC} = 2(e + D)$

$= 2(0.4 + 1) = 2.80m$

$A_{PC} = \frac{94.4}{8} = 11.8\ m^2 = L_{PC_1} \times B_{PC_1}$

$\therefore B_{PC} = \frac{11.8}{2.8} = 4.21 \simeq 4.25m$ , $B_{RC} = 4.25 - 2(0.4) = 3.45$

### check:-

$\frac{B_{PC}}{L_{PC}} = \frac{4.21}{2.8} = 1.5 < 2$

## (F2)- Square footing

$$A_{PC} = \frac{150.6}{8} = 18.83\ m^2 = B_{PC}^2$$

$\therefore B_{PC} = 4.35\,m$

$\therefore B_{RC} = 3.55\,m$

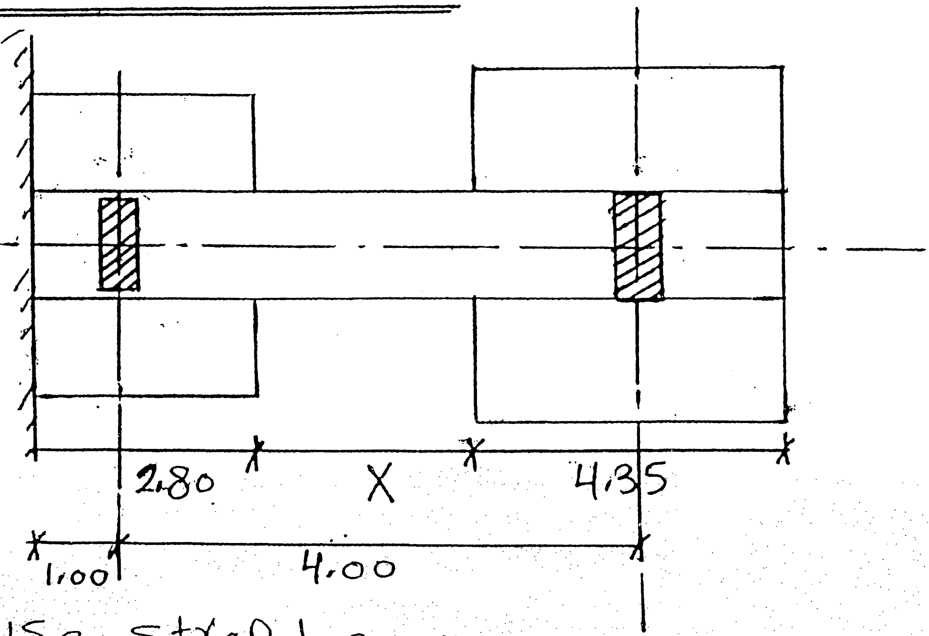
## Check Validity of using strap beam

$$X = D + S - L_1 - \frac{B_2}{2}$$

$$= 1 + 4 - 2.80 - \frac{4.35}{2}$$

$$= 0.025\,m$$

$$< \frac{L_{min}}{2}$$

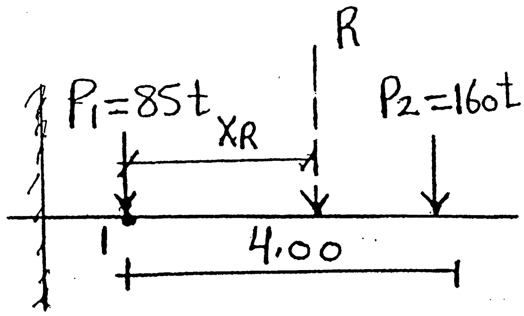



$\therefore$ We can't use strap beam

$\because P_1 < P_2$

$\therefore$ Try Rectangular Combined footing.

## Combined footing dimensions:-

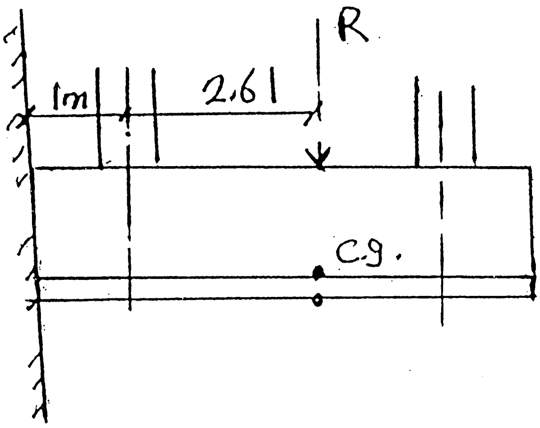



$R = 85+160 = 245\ t$

$M_{a_1} = 0$

$\therefore X_R = \dfrac{160 \times 4}{245} = 2.61m$

- Assume $t_{P.C.} = 40\ cm$




$\dfrac{L_{PC}}{2} = X_R + 1m$

$= 2.61 + 1 = 3.61m$

$\therefore L_{PC} = 2 \times 3.61 = 7.22m$

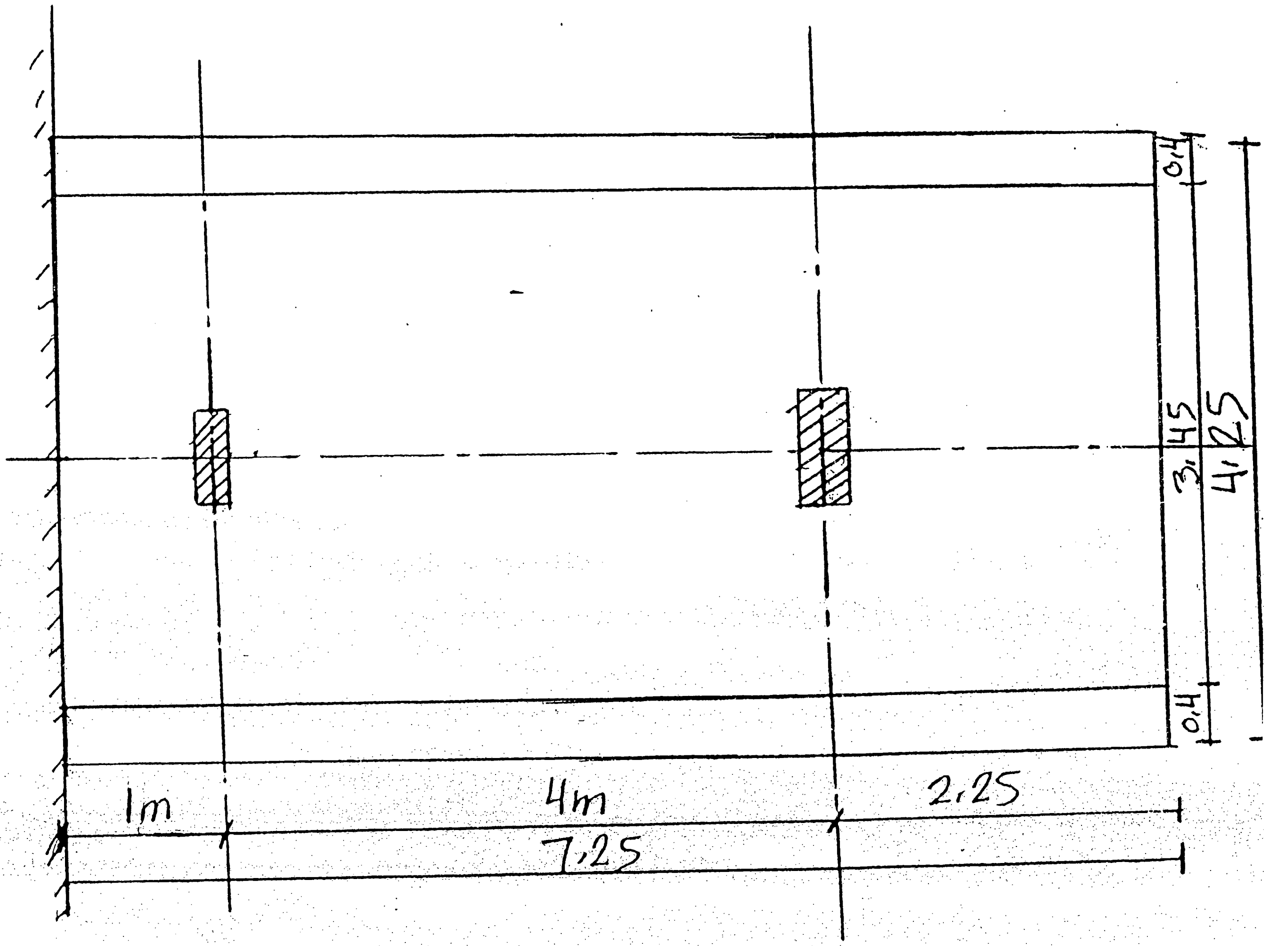
$\simeq 7.25m$

$\therefore L_{RC} = L_{PC} = 7.25m$

- $A_{PC} = \dfrac{245}{8} = 30.625\ m^2$

$\therefore B_{PC} = \dfrac{30.625}{7.25} = 4.23m \simeq 4.25m$

$\therefore B_{RC} = 4.25 - 2(0.4) = 3.45m$

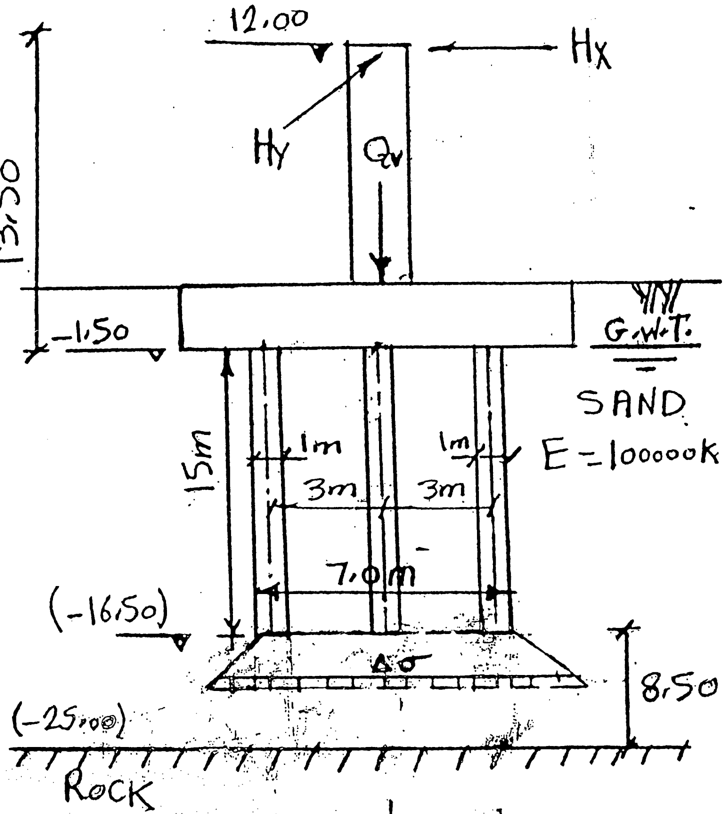



PLAN
Scale 1:50

# Question (2):-

## Given:-

- $Q_v = 36000$ KN
- $H_x = 1000$ KN
- $H_y = 400$ KN




## solution:-

i) $S_g = \frac{1}{E} * \Delta\sigma * H$

$= \frac{1}{100000} * \frac{36000}{\left[7+\frac{8.5}{2}\right]^2} \times 8.50$

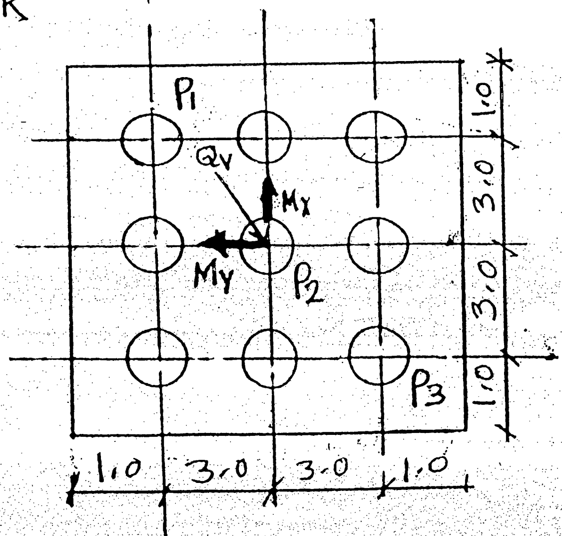
(pier length)

$= 24.18 \times 10^{-3}$ m

$= 24.18$ mm

ii) $G_A = \frac{S_g}{S_o} = \frac{24.18}{6}$

$= 4.03$




iii) $Q_{/Pile} = \frac{Q_v}{n} \pm \frac{M_x \cdot y_i}{\Sigma(y_i^2)} \pm \frac{M_y \cdot X_i}{\Sigma(X_i^2)}$

$\Sigma(X_i^2) = \Sigma(Y_i^2) = 3*(3)^2 * 2$

$= 54 \; m^2$

$M_x = H_y * 13.50 = 400 \times 13.5 = 5400$ KN.m.

$M_y = H_x * 13.50 = 1000 \times 13.5 = 13500$ KN.m.

$$\therefore Q/_{pile} = \frac{36000}{9} \pm \frac{5400(y_i)}{54} \pm \frac{13500(x_i)}{54}$$

$$= 4000 \pm 100(y_i) \pm 250(x_i)$$

$$\therefore Q_{P_1} = 4000 + 100(3) + 250(3) = 5050 \text{ KN}$$

$$Q_{P_2} = 4000 + 100(0) + 250(0) = 4000 \text{ KN}$$

$$Q_{P_3} = 4000 - 100(3) - 250(3) = 2950 \text{ KN}$$

IV)

- $F_s = 180$ KPa (Skin friction along pile shaft)
- $q_b = 3000$ KPa (Pile base resistance)

$$\therefore Q_{ult.} = q_b A_b + F_s A_s$$

$$= 3000 * \pi * \frac{(1)^2}{4} + 180 * \pi * 1 * 15$$

$$= 10838.49 \text{ KN}$$

- $F.o.S. = \dfrac{Q_{ult.}}{Q/_{pile}}$

$$F.o.S._{P_1} = \frac{10838.49}{5050} = 2.146$$

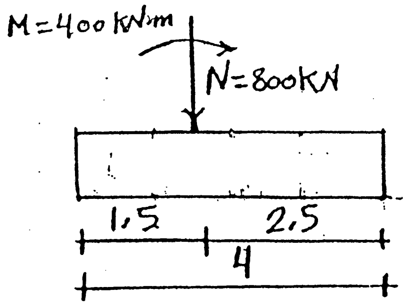
$$F.o.S._{P_2} = \frac{10838.49}{4000} = 2.71$$

$$F.o.S._{P_3} = \frac{10838.49}{2950} = 3.674$$

iv) $P_1$ has the minimum factor of safety, because it is subjected to the maximum compression force.

## Question (3):-

### Given:-

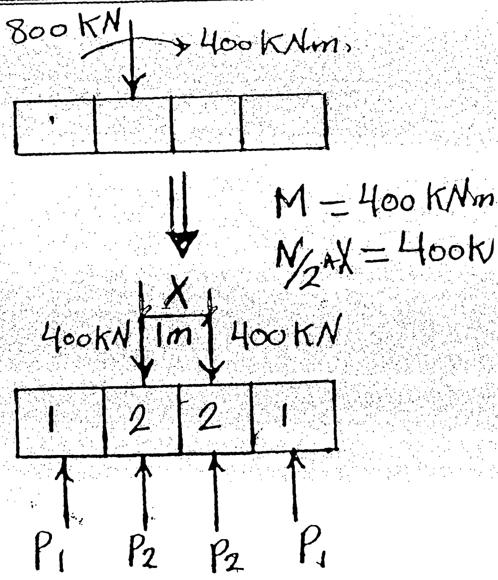



- 4x1.5m footing
- Divide into 4 elements
- $E_s = 10000\ kN/m^2$
- $E_c = 2000\ kN/cm^2 = 2\times10^7\ kPa$
- $t = 110\ cm$
- $(C_0 = 1.33,\ C_1 = 0.52,\ C_2 = 0.3,\ C_3 = 0.2,\ C_4 = 0.16,\ C_5 = 0.12)$

### Required:-

1) Contact stress under the footing, assuming the soil is elastic, homogeneous, isotropic and semi-infinite.

### Solution:-




- يتم تحويل القاعدة إلى قاعدة متماثلة وذلك يتحقق فقط عندما يكون :-
- $M = \frac{N}{2} * X$

ويجب أن يكون اتجاه دوران (M) حول مركز القاعدة عكس اتجاه دوران (N).

- Assume contact stress ($P_{si}$) below each element
- $P_i$ (Reaction) $= a * b * P_{si} = 1.5 P_{si}$

## A) M – $P_s$ eqs:-

- $M_1 = 0$ ——— (a)
- $M_2 = 1.5 P_{s_1}$ ——— (b)

## B) M – Δ eqs:-

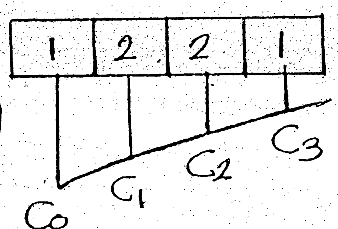
$$\frac{6EI}{a^2} = \frac{6 \times 2 \times 10^7 * 1.5 * (1.1)^3}{(1)^2 * 12} = 19.965 \times 10^6 \text{ KN}$$

### Point (2):-

$$M_1 + 5M_2 = 19.965 \times 10^6 (-\Delta_1 + \Delta_2) \quad \text{--- (1)}$$

## Δ – $P_s$ eqs:-

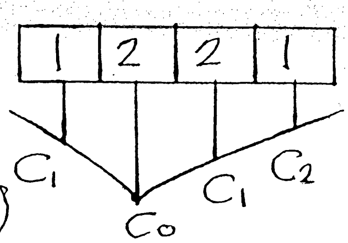
### Point (1):-




$$\Delta_1 = \frac{a}{E_s} [C_0 P_{s_1} + C_1 P_{s_2} + C_2 P_{s_2} + C_3 P_{s_1}]$$

$$\therefore \Delta_1 = \frac{1}{10000} [1.53 P_{s_1} + 0.82 P_{s_2}] \quad \text{--- (I)}$$

### Point (2):-




$$\Delta_2 = \frac{1}{10000} [C_1 P_{s_1} + C_0 P_{s_2} + C_1 P_{s_2} + C_2 P_{s_1}]$$

$$\therefore \Delta_2 = \frac{1}{10000} [0.82 P_{s_1} + 1.85 P_{s_2}] \quad \text{--- (II)}$$

## Final eqs:-

Substituting eqs ⓐ, ⓑ, and (I), (II) into eq. (1''), we get

$$0 + 7.5\,P_{S_1} = \frac{19.965\times10^{6}}{10000}\left[-0.71\,P_{S_1} + 1.03\,P_{S_2}\right]$$

$$\Rightarrow \quad 1425.015\,P_{S_1} - 2056.395\,P_{S_2} = 0 \quad \text{———— (1)}$$

$$\Sigma F_y = 0$$

$$\Rightarrow \quad P_{S_1} + P_{S_2} = 266.667 \quad \text{———— (2)}$$

Solving eqs (1) and (2), we get

$$P_{S_1} = 157.51 \text{ KPa}$$

$$P_{S_2} = 109.152 \text{ KPa}$$

**Check:-**

$$P_{S_{avg.}} = \frac{157.51 + 109.152}{2} = 133.33 \text{ KPa}$$

$$= \frac{\Sigma \text{loads}}{\text{Area}}$$

$$= \frac{2\times400}{4\times1.5}$$

$$= 133.33 \text{ KPa}$$

Ain Shams University
Faculty of Engineering

CES 451: **Foundation Engineering**

4th Year, Civil Engineering
Students of Structure Eng. Div.
Time: **1.5** hours
Mid-term Exam

| Question | 1 | 2 | 3 | Total | **Name:** | |
|---|---|---|---|---|---|---|
| Mark | | | | | **Section:** | |
| Initials | | | | | **BN:** | |

The examination consists of **3** questions in 4 pages.
Make a reasonable assumption of any missing data.
All sketches should be neatly drawn and properly dimensioned.

---

## Question 1

**1.1** Explain using neat sketches the purposes of tie beams.

**1.2** For the two columns, shown in Figure (1), it is required to:

i. Suggest a suitable foundation type.

ii. Give a complete design of the foundation (dimensions and reinforcement). Thickness of plain concrete is only 10 cm. The allowable net bearing capacity is 20 t/m$^2$.

iii. Draw neat sketches showing the concrete dimensions and reinforcement details in both plan and elevation (using scale 1:50).

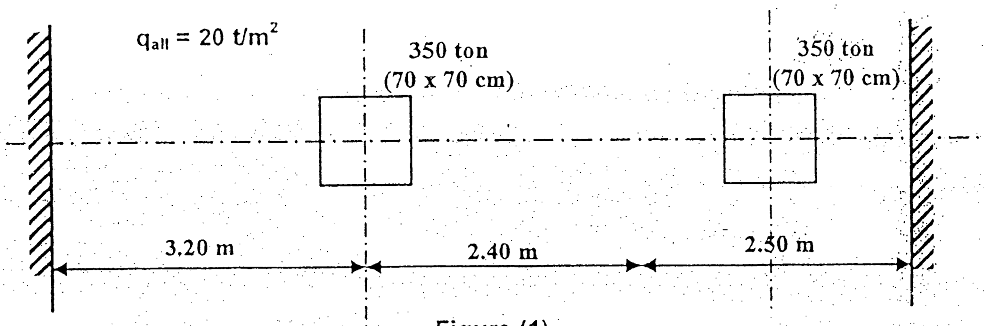



Figure (1)

**Data :**

Materials: Concrete $f_{cu}$ = 300 kg/cm$^2$, Steel 40/60, $f_y$ = 4000 kg/cm$^2$

Working stress: $f_c$=100 kg/cm$^2$, $f_s$ =2200 kg/cm$^2$, $q_c$=7 kg/cm$^2$, $q_{cp}$=10 kg/cm$^2$, local bond stress = 12 kg/cm$^2$

Ultimate stress: $q_{cu}$ = 9 kg/cm$^2$, $q_{cpu}$=14.5 kg/cm$^2$, qbu= 18 kg/cm$^2$

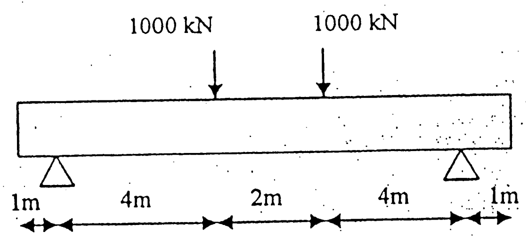
$C_{1\,min}$ = 3.0 , j = 0.74
$C_1$ = 3.5 , j = 0.78
$C_1$ = 4.0 , j = 0.80
$C_1$ = 4.85 , j = 0.826

Area of different reinforcement steel bars:
Φ12: area = 1.13 cm$^2$ ; Φ16: area = 2.00 cm$^2$ ; Φ18: area = 2.84 cm$^2$ ; Φ20: area = 3.14 cm$^2$ ;
Φ22: area = 3.80 cm$^2$ ; Φ25: area = 4.91 cm$^2$

$d = C_1 \left( M_u / (f_{cu} \cdot b) \right)^{1/2}$ $\quad\quad$ $As = M_u / f_y \cdot d \cdot j$

## Question 2

A strip footing of dimensions of 12.0 m x 2.0m is to be constructed on two piles. The footing is loaded by two columns of equal loads of 1000 kN. The piles located at 1.0 m from the footing edge can be **assumed as hinged support**. The footing is founded on well graded gravel with a modulus of subgrade reaction of 30,000 kN/m$^3$. The footing thickness is 90 cm and the concrete elastic modulus is 21 GPa. The footing can be divided into 6 elements. **Find the settlement value below the two columns and forces in piles, taking into cinsideration that both the piles and foundation soil contribute in resisting the acting loads.**

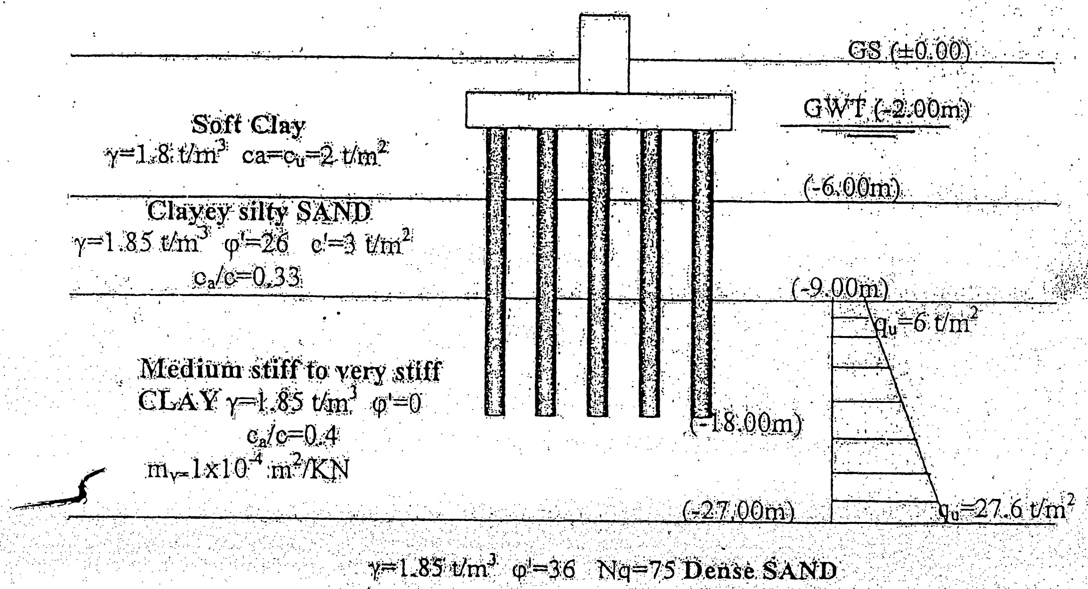


# Question 3

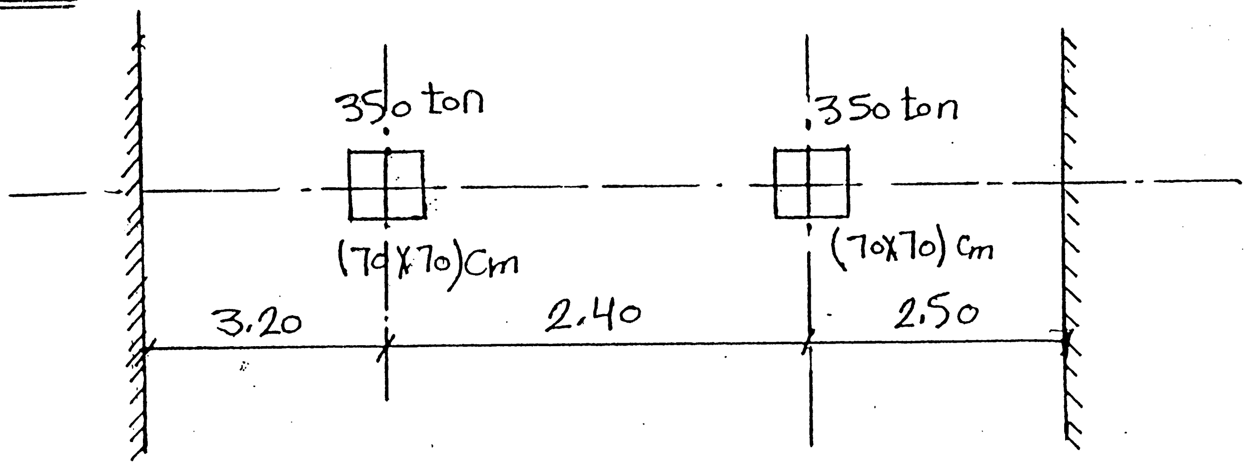
i) Comment on the pile load test using Osterberg cell. Mention its advantages compared to other loading techniques.

ii) A group of piles (5x6 piles) having a diameter of 0.45 m and spaced at 1.35 m are driven to a depth of 18 m below the ground surface. The soil formation at the site is illustrated in Figure. If the maximum allowable settlement of the group is 2.5 cm. Determine the maximum allowable compression capacity of the group ($K_{HC}$ = 1.0, $K_{HT}$=0.6).

| Level | -2 | -4 | -6 | -8 | -10 | -12 | -14 | -16 | -18 | -20 | -22 | -24 | -26 | -28 | -30 | -32 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| N30 | 3 | 3 | 8 | 18 | 8 | 8 | 9 | 11 | 11 | 12 | 12 | 13 | 13 | 40 | 48 | 50 |



## Q1)




**Given:-**

- Col. loads, Col dims.
- $q_{all\,net} = 20\,t/m^2$
- $t_{PC} = 10$ cm
- $F_{cu} = 300\ Kg/cm^2 = 30\ N/mm^2$
- $F_y = 4000\ Kg/cm^2 = 400\ N/mm^2$
- $q_{cu} = 9\ Kg/cm^2$ (allowable shear strength)
- $q_{cpu} = 14.5\ Kg/cm^2 = 1.45\ N/mm^2$ (allowable punching strength)

**Required:-**

Design suitable foundation type and Draw R.F.T.

## Solution:-

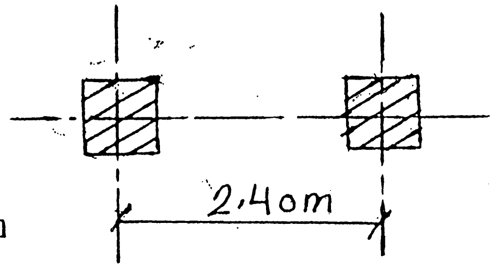
### Try isolated footing

$\therefore t_{pc} = 10\ cm$

$\therefore A_{RC} = \frac{350}{20} = 17.5\ m^2 = B_{RC}^2$

$\therefore B_{RC} = 4.18\ m \simeq 4.20\ m$

### Check intersection of footings



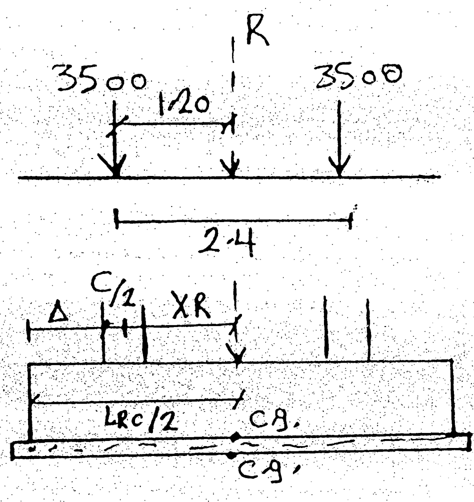

$X = 2.40 - \frac{4.20}{2} - \frac{4.20}{2} = \ominus 1.8\ m$

X ↓ المسافة بين القواعد

∴ يحدث تداخل بين القواعد المنفصلة

$\therefore$ We can't use isolated footing

### Try combined footing:-




$R = 7000\ KN$

$\therefore \frac{L_{RC}}{2} = 1.20 + \frac{0.7}{2} + 1.5$

$= 3.05\ m$

$\therefore L_{RC} = 6.10\ m$

$\therefore L_{PC} = 6.10 + 2(0.1) = 6.30\ m$

$\therefore A_{RC} = \dfrac{7000}{200} = 35\ m^2$

$\therefore B_{R.C.} = 5.75$

$\therefore B_{PC} = 5.75 + 2(0.1) = 5.95 m$

## Ultimate limit loads:-

$P_u = 1.5 \times 3500 = 5250\ KN$

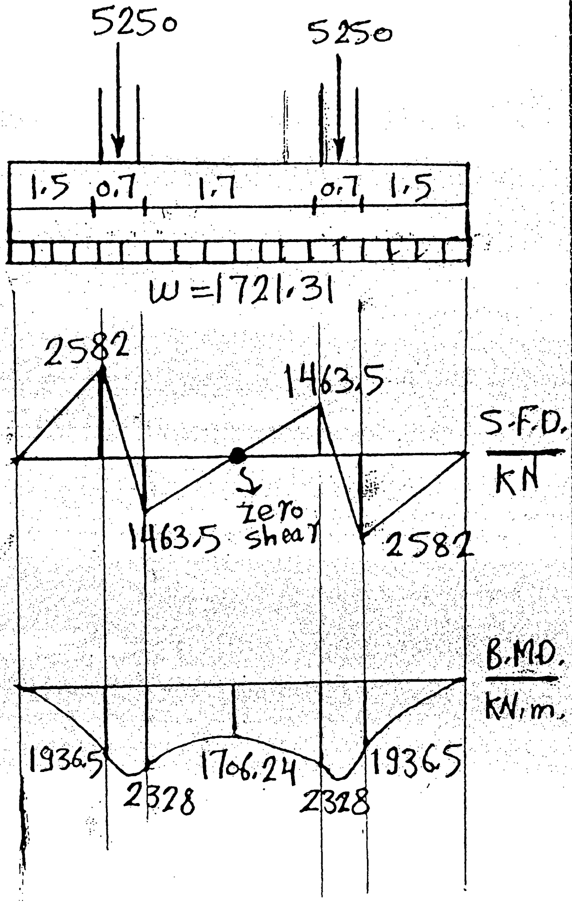
$W_u = \dfrac{10500}{6.1} = 1721.31\ KN/m'$

$\sigma_u = \dfrac{10500}{6.1 \times 5.75} = 299.36\ KN/m^2$

## Design in longitudinal direction:-

### B.M.D., S.F.D.

- Point of zero shear lays at the center of footing (القاعدة متماثلة)




### Footing depth:-

$d = 5\sqrt{\dfrac{2328 \times 10^6}{30 \times 5750}} = 580.8\ mm$

Take $d = 630\ mm$

### Check shear:-

$Q_{su} = 2582 - (1721.31) \times 0.63 = 1497.6\ KN$

(2582 ↓ $Q_{max}$, 1721.31 ↓ $w$, 0.63 ↓ $d$)

$\therefore q_{su} = \dfrac{1497.6 \times 10^3}{5750 \times 630} = 0.413\ N/mm^2 < q_{scu} = 0.9\ N/mm^2$ (safe)

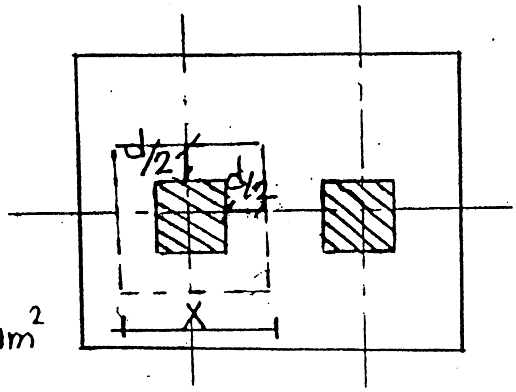
## Check Punching:-

$X = 0.7 + 0.63 = 1.33\ m$

$Q_{Pu} = 5250 - 299.36\,(1.33)^2$

$= 4720.46\ KN$

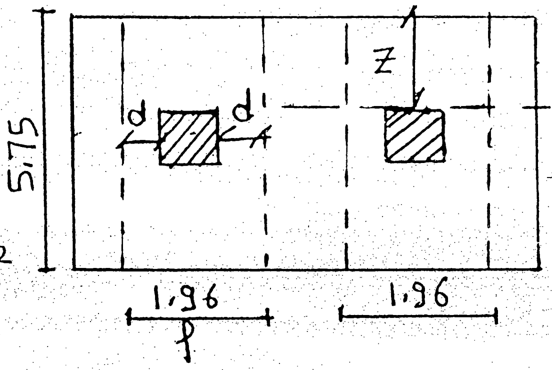
$q_{Pu} = \dfrac{4720.46 \times 10^3}{630 \times 4 \times 1330} = 1.41\ N/mm^2$




$q_{Pcu} = 1.45\ N/mm^2 > q_{Pu}$
↓
Given

## Design in Transverse direction:-

- $l = b + 2d$
  $= 0.7 + (2 \times 0.63) = 1.96\ m$
- $Z = \dfrac{5.75 - 0.7}{2} = 2.525\ m$
- $\sigma_u = \dfrac{5250}{5.75 \times 1.96} = 465.84\ KN/m^2$
- $M = 465.84 \times \dfrac{(2.525)^2}{2} \times 1m$
  $= 1485\ KN.m/m$




## Check depth:-

$630 = C_1 \sqrt{\dfrac{1485 \times 10^6}{30 \times 1000}} \Longrightarrow C_1 = 2.83 > 2.78$ (safe)

## Final thickness of R.C.:-

$t = 630 + 70 = 700$ mm

## R.F.T.:-

$As_{min.} = 1.5 \times 630 = 945$ mm²/m'

**longtudinal direction:-**

- $As_{bot.} = \frac{2328 \times 10^6 / 5.75}{400 \times 0.826 \times 630} = 1945$ mm²/m' → **8∅18/m'**

- $As_{Top} = As_{min.} = 945$ mm²/m' → **5∅16/m'**

  ↓ (لا يوجد Moment علوي)

**Transverse direction:-**

- $As_{bot.} = \frac{1485 \times 10^6}{400 \times 0.73 \times 630} = 8072.4$ mm²/m' → **10∅32/m'**

لاحظ :-

أ) غير مسموح في الـ Combined بأن تزيد B عن L .

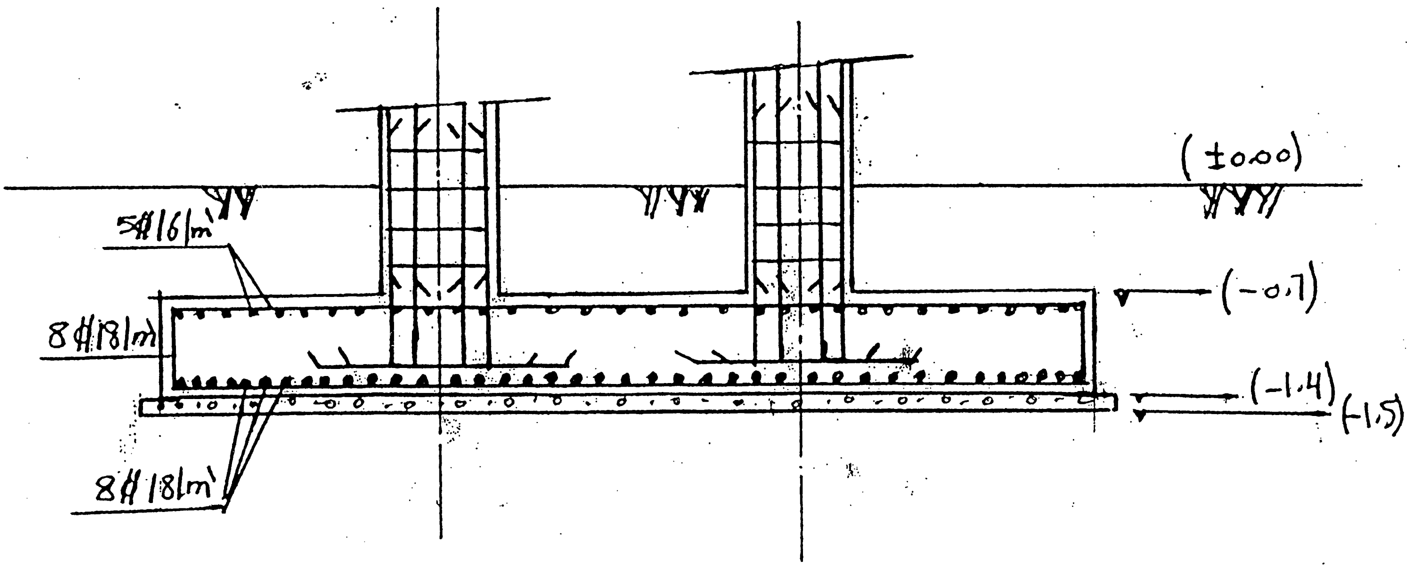
ب) في هذه المسألة ، يمكن زيادة d لتقليل حديد التسليح :-

inc. d ⟶ 730 mm

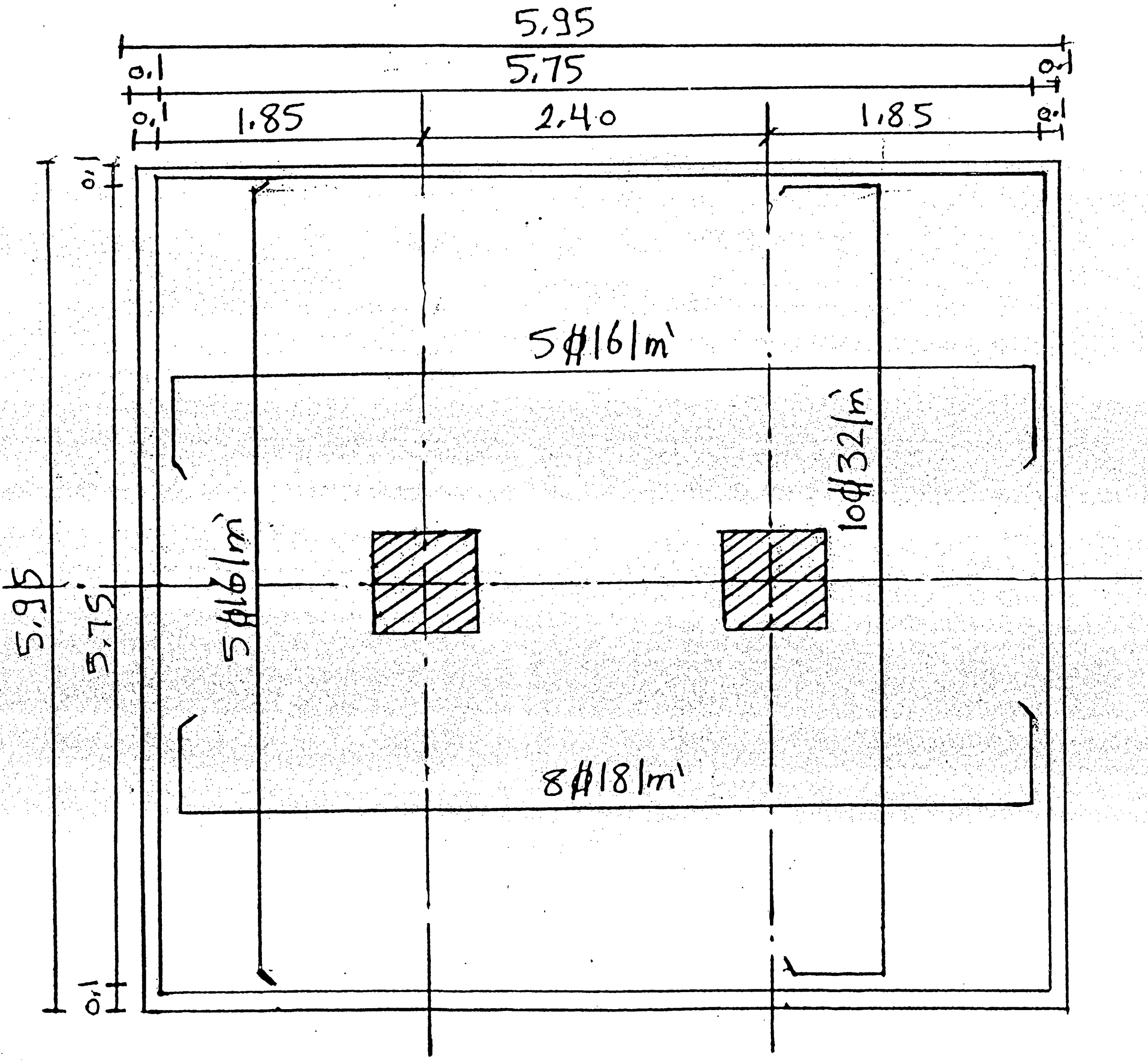
long. direction:- $As_{bot.} = 7∅18/m'$ ; $As_{Top} = 6∅16/m'$

Transverse direction:- $As = 9∅32/m'$

## R.F.T. details:-



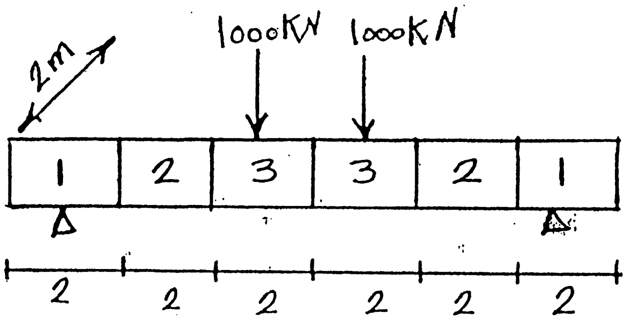
(±0.00)
5∅16/m'
8∅18/m'
(-0.7)
(-1.4)
(-1.5)
8∅18/m'

5.95
5.75
0.1
0.1
1.85
2.40
1.85
0.1
0.1
5∅16/m'
10∅32/m'
5∅16/m'
5.95
5.75
8∅18/m'

## Question (2):-

### Given :-



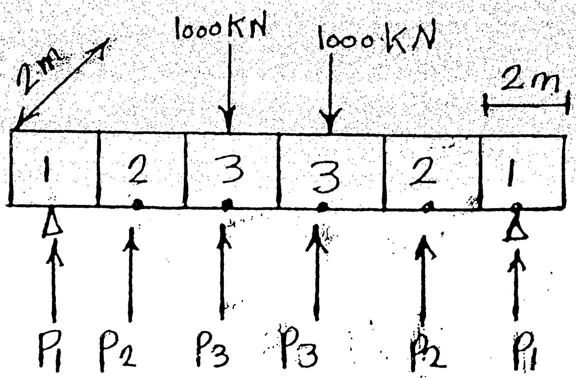

- Footing (12x2) m
- Divide in 6 elements
- Footing is supported on piles.
- The pile is modeled as hinged support (For approximation)
- $K_{so} = 30000 \ KN/m^3$
- $t = 90$ cm
- $E = 21 \times 10^7$ KPa
- The footing is piled Raft (both pile and soil will carry load)

### Required :-

i) Force in each pile.

ii) Settlement at the piled Raft center.

### Solution:-




- Assume contact stress ($P_{si}$) below each element
- Reaction ($P_i$) below each element $= a \times b \times P_{si}$

- $P_1 = 4P_{s1}$ (لايوجد contact stress أسفل element (1)، ولكن يتم تحويل رد فعل ال Pile إلى contact stress لسهولة الحل)
- $P_2 = 4P_{s2}$
- $P_3 = 4P_{s3}$

---

**M – Ps eqs:-**

$$M_1 = 0 \quad \text{------ (a)}$$

$$M_2 = 8P_{s1} \quad \text{------ (b)}$$

$$M_3 = 16P_{s1} + 8P_{s2} \quad \text{------ (c)}$$

---

**M – Δ eqs:-**

$$\frac{6E_cI}{a^2} = \frac{6 \times 2.1 \times 10^7 \times 2 \times \frac{(0.9)^3}{12}}{(2)^2} = 3.82725 \times 10^6 \text{ KN}$$

**Point (2):-**

$$M_1 + 4M_2 + M_3 = 3.82725 \times 10^6 \left(-\Delta_1 + 2\Delta_2 - \Delta_3\right) \quad \text{------ (1)}$$

**Point (3):-**

$$M_2 + 5M_3 = 3.82725 \times 10^6 \left(-\Delta_2 + \Delta_3\right) \quad \text{------ (2)}$$

---

**Ps – Δ eqs:-**

$$\Delta_1 = \text{zero} \quad \text{(hinged support)} \quad \text{------ (I)}$$

$$\Delta_2 = \frac{P_{s2}}{30000} \quad \text{------ (II)}$$

$$\Delta_3 = \frac{P_{s3}}{30000} \quad \text{------ (III)}$$

**Final eqs:-**

Substituting eqs ⓐ, ⓑ, ⓒ, and (I), (II), (III) into eqs ① & ②

**equation ①:-**

$$\cancel{M_1} + 4M_2 + M_3 = 3.82725 * 10^6 \left(-\cancel{\Delta_1} + 2\Delta_2 - \Delta_3\right)$$

($M_1 = 0$, $\Delta_1 = 0$)

$$\therefore\ 32 P_{S1} + 16 P_{S1} + 8 P_{S2} = \frac{3.82725 \times 10^6}{30000} \left(2P_{S2} - P_{S3}\right)$$

$$\therefore\ 48 P_{S1} - 247.15 P_{S2} + 127.575 P_{S3} = 0 \quad \text{--- ①}$$

**equation ②:-**

$$M_2 + 5M_3 = 3.82725 \times 10^6 \left(-\Delta_2 + \Delta_3\right)$$

$$\therefore\ 8 P_{S1} + 80 P_{S1} + 40 P_{S2} = \frac{3.82725 \times 10^6}{30000} \left(-P_{S2} + P_{S3}\right)$$

$$\therefore\ 88 P_{S1} + 167.575 P_{S2} - 127.575 P_{S3} = 0 \quad \text{--- ②}$$

**equation ③:-**

$$\Sigma F_y = 0$$

$$\therefore\ P_{S1} + P_{S2} + P_{S3} = 250 \quad \text{--- ③}$$

Solving eqs ①, ②, ③, we get

$$P_{S1} = 44.296 \text{ KPa}$$

$$P_{S2} = 75.7 \text{ KPa}$$

$$P_{S3} = 129.997 \text{ KPa}$$

- Force in each Pile $= 4\,P_{S1} = 4 \times 44.296$

$= 177.184$ KN

- Settlement at the center of the Piled Raft $= \Delta_3$

$$\Delta_3 = \frac{P_{S3}}{K_{S0}} = \frac{129997}{30000} = 4.33 \times 10^{-3}\,m$$

$$= 4.33\ mm$$

## Q3) Solution :-

### 1) Allowable Comp. Capacity of Pile group

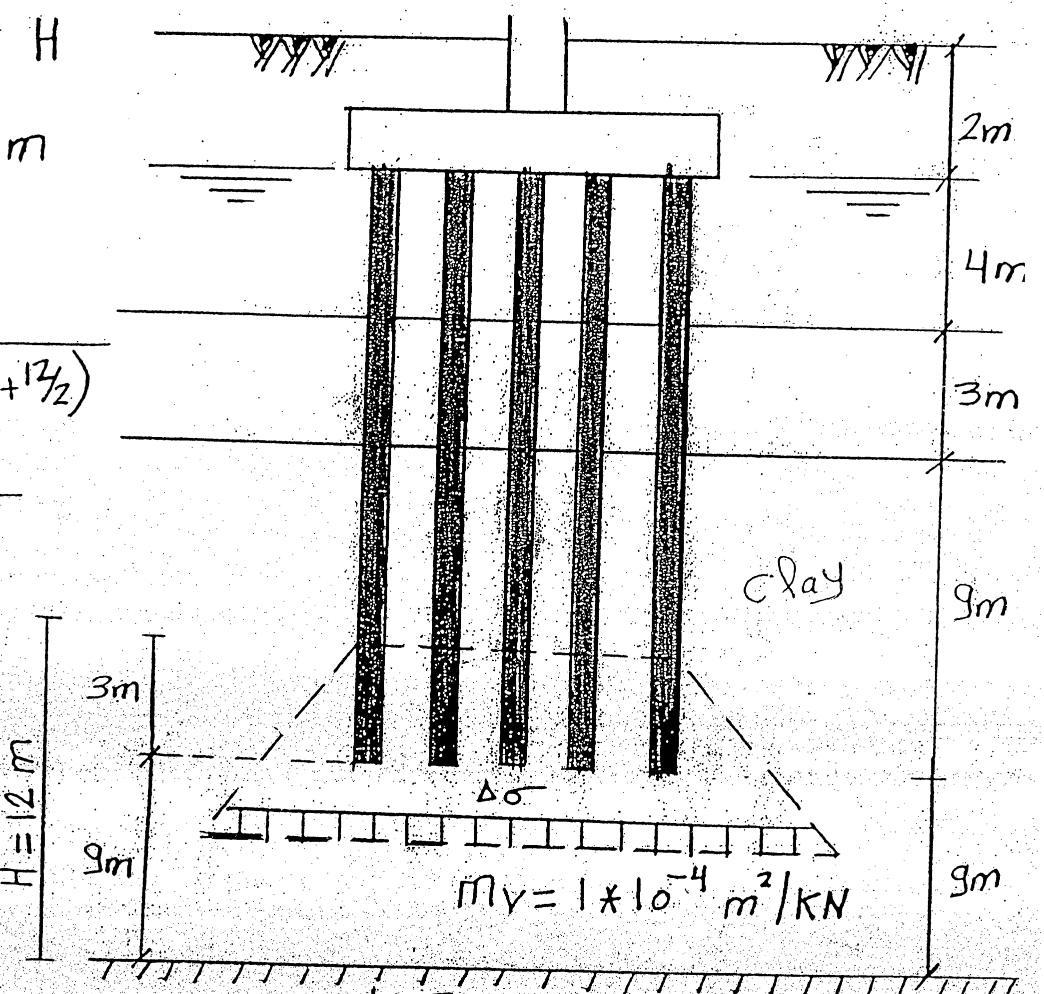
### From allowable group settlement

1.5B = 8.8 m

$$S_{g_{all}} = m_v \, \Delta\sigma \, H$$

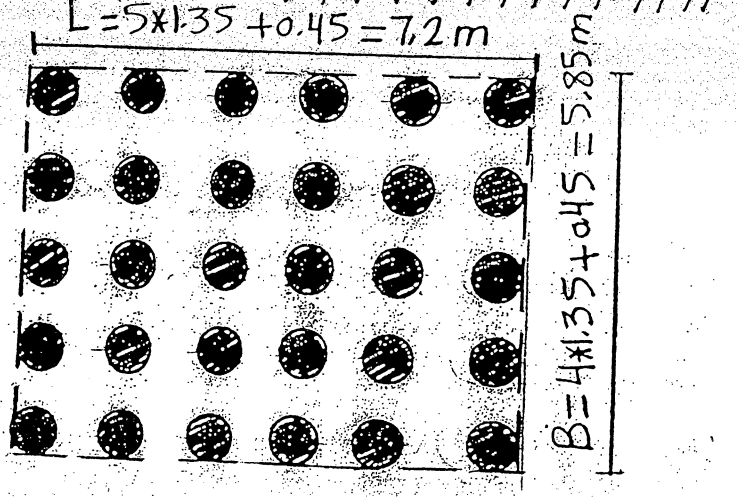
- $H = 9/3 + 9 = 12 \, m$

- $$\Delta\sigma = \frac{Q_{allg}}{(7.2 + 12/2)(5.85 + 12/2)} = \frac{Q_{allg}}{13.2 * 11.85}$$

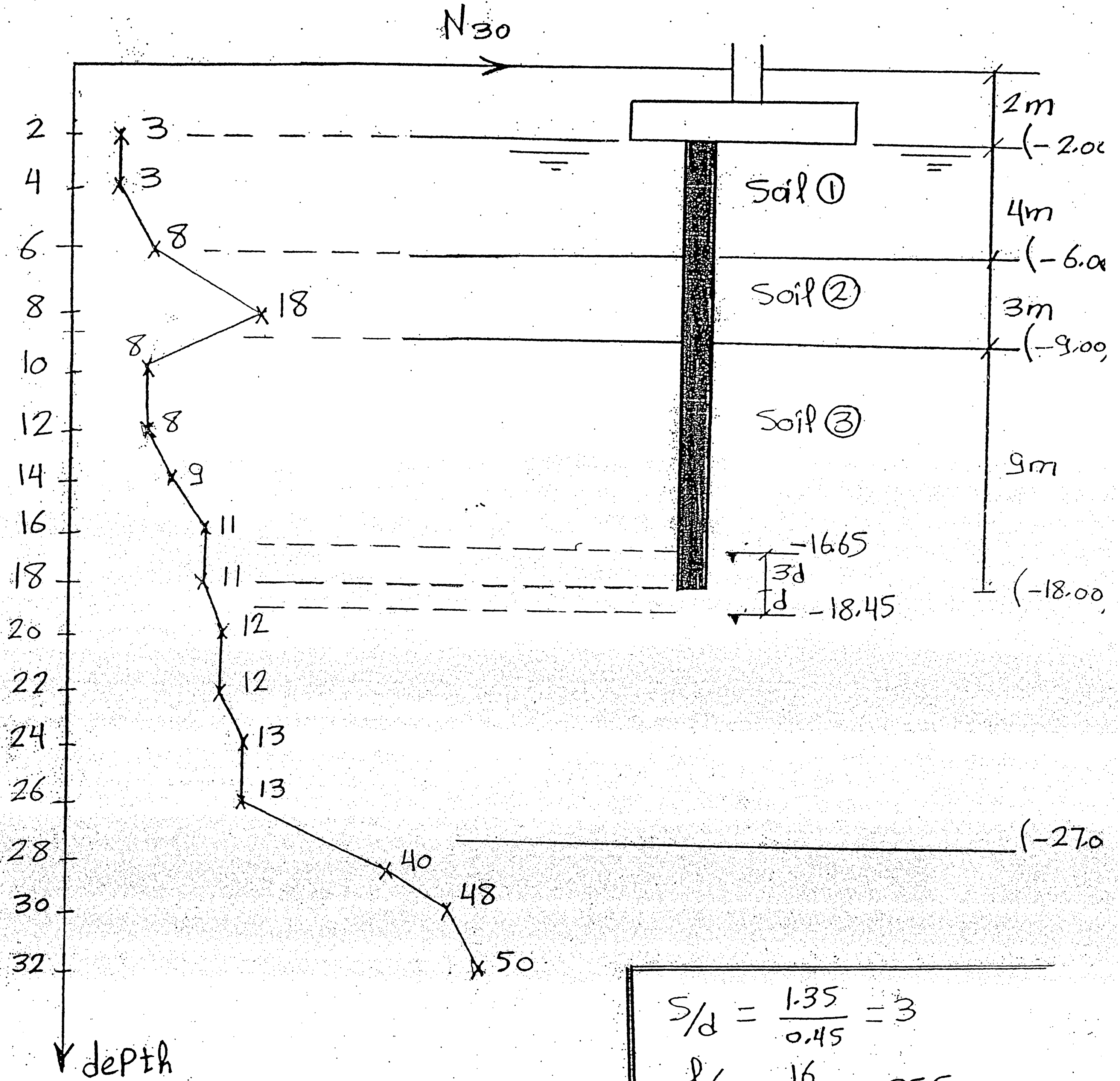



$$\therefore \frac{2.5}{100} = 1*10^{-4} \frac{Q_{allg}}{13.2 * 11.85} * 12$$

$$\therefore Q_{allg} = 3258.75 \, KN$$



## 2) All. Comp. Capacity of Pile group from S.P.T. data :-




$S/d = \frac{1.35}{0.45} = 3$

$l/d = \frac{16}{0.45} = 35.5$

5*6 Pile group

جدول ٤-٢٢ → $G_e = 0.77$

- $Q_{all\ driven} = 90\, N_b\, A_b + \bar{N}_1 A_{s1} + \bar{N}_2 A_{s2} + \bar{N}_3 A_{s3}$

  - $A_b = \pi \dfrac{(0.45)^2}{4} = 0.159\ m^2$
  - $A_{s1} = \pi * 0.45 * 4 = 5.65\ m^2$
  - $A_{s2} = \pi * 0.45 * 3 = 4.24\ m^2$
  - $A_{s3} = \pi * 0.45 * 9 = 12.72\ m^2$

- $N_b$ = Average $N_{30}$ between $-16.65 \rightarrow -18.45$

  $= 11$

- $\bar{N}_1$ = Average $N_{30}$ between $-2 \rightarrow -6$

  $= \dfrac{3+3+8}{3} = 4.67$

- $\bar{N}_2$ = Average $N_{30}$ between $-6 \rightarrow -9$

  $= \dfrac{8+18}{2} = 13$

- $\bar{N}_3$ = Average $N_{30}$ between $-9 \rightarrow -18$

  $= \dfrac{8+8+9+11+11}{5} = 9.4$

$\therefore Q_{all\ driven} = 90*11*0.159 + 4.67*5.65 + 13*4.24 + 9.4*12.72$

$= 358.5\ KN$ 356 → اذا كل الطبقات $\bar{N}$

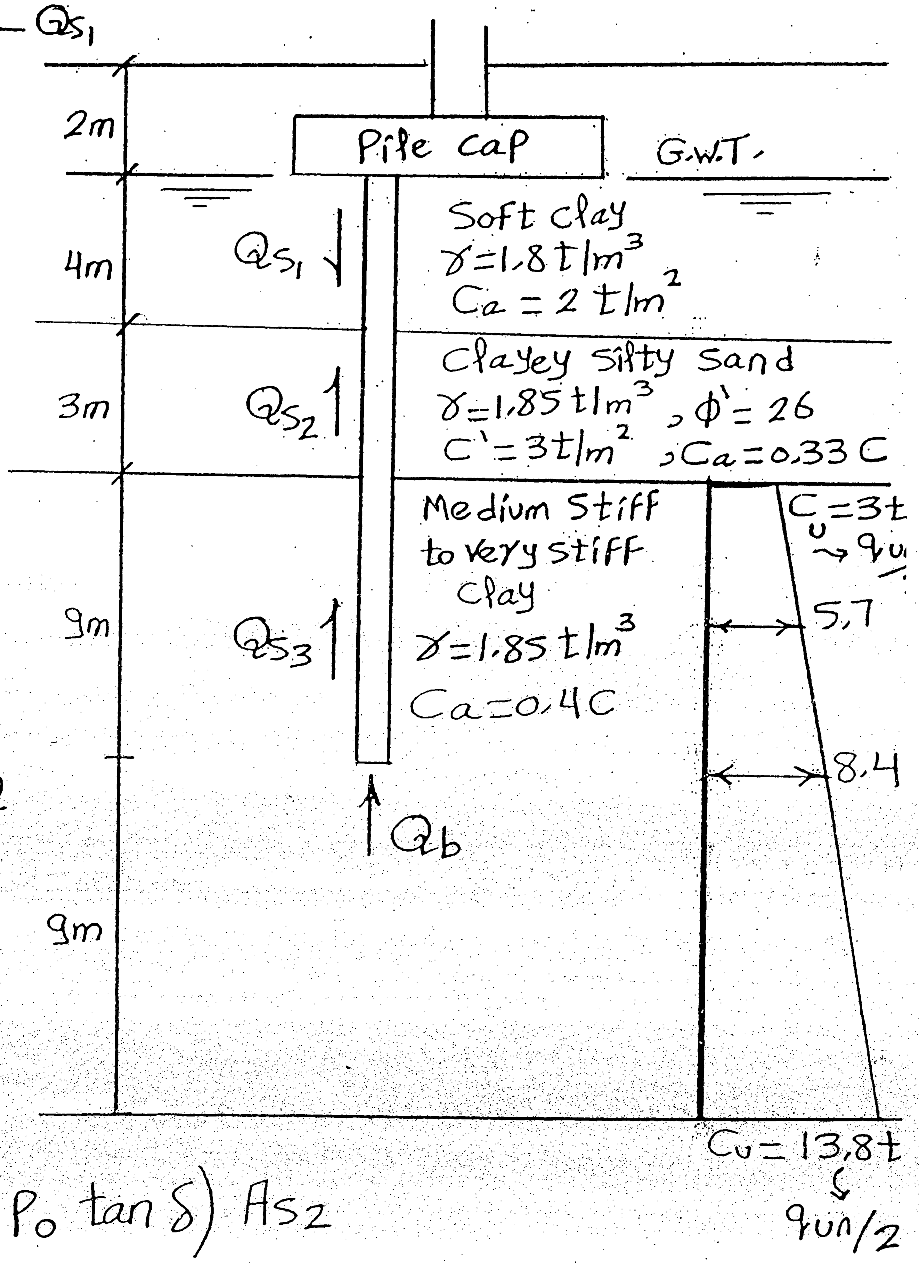
$\therefore Q_{all_g} = G_e * n * Q_{all_s} = 0.77 * 30 * 358.5$

$= 8281.35\ KN$

## 3) All. Comp. Capacity of Pile group

### From Statical Formula :-

$\because Q_{ult} = Q_b + Q_{s2} + Q_{s3} - Q_{s1}$




**Qb :- (C-Soil)**

$Q_b = (C\ N_c)\ A_b$

$= (8.4 * 9) * 0.159$

$= 12.024$ ton

**Qs3 :- (C-Soil)**

$Q_{s3} = C_a\ A_{s3}$

$= (0.4 * 5.7) * 12.72$

$= 29$ ton

**Qs2 :- (C-φ) Soil**

$Q_{s2} = (C_a + K_{HC}\ P_o \tan\delta)\ A_{s2}$

- $C_a = 0.33 * C = 0.33 * 3 = 0.99\ t/m^2$
- $K_{HC} = 1.0$ (given)
- $\delta = 3/4\ \phi = 3/4 * 26 = 19.5°$

• $P_o = 1.8*2 + 0.8*4 + 0.85*1.5 = 8.075 \ t/m^2$

$\therefore Q_{s2} = (0.99 + 1*8.075*\tan 19.5)*4.24$

$= 16.3$ ton

**$Q_{s1}$ :- (C-Soil)**

→ Soft clay , $C_u = 20 \ KN/m^2 < 25 \ KN/m^2$

∴ Consider negative skin friction

$Q_{s1} = C_a \ A_{s1}$

$= 2*5.65 = 11.3$ ton

**$Q_{alls}$ :-**

$$Q_{alls} = \frac{Q_b + Q_{s2} + Q_{s3}}{F.O.S.} - Q_{s1}$$

F.O.S. → not given

• Assume F.O.S. = 3.00

$$\therefore Q_{alls} = \frac{12.024 + 29 + 16.3}{3} - 11.3$$

$= 7.808$ ton $= 78.08$ KN

**$Q_{allg}$ :-**

$Q_{allg} = G_e * n * Q_{alls}$

$= 0.77*30*78.08$

$= 1803.65$ KN

## • Maximum allowable compression capacity of pile group :-

$$Q_{allg} = \min \begin{cases} 3258.75 \\ 8281.35 \\ 1803.65 \end{cases} \rightarrow 1803.65 \text{ KN}$$

Ain Shams University
Faculty of Engineering
November 2007
CES 451: Foundation Engineering

4th Year, Civil Engineering
Students of Structure Eng. Div.
Time: 1.5 hours
Mid-term Exam

| Question | 1 | 2 | 3 | Total | Name: | |
|---|---|---|---|---|---|---|
| Mark | | | | | Section: | |
| Initials | | | | | BN: | |

The examination consists of **3** questions in 4 pages.
Make a reasonable assumption of any missing data.
All sketches should be neatly drawn and properly dimensioned.

---

## Question 1

i) Discuss briefly the different methods for analyzing a raft foundation.

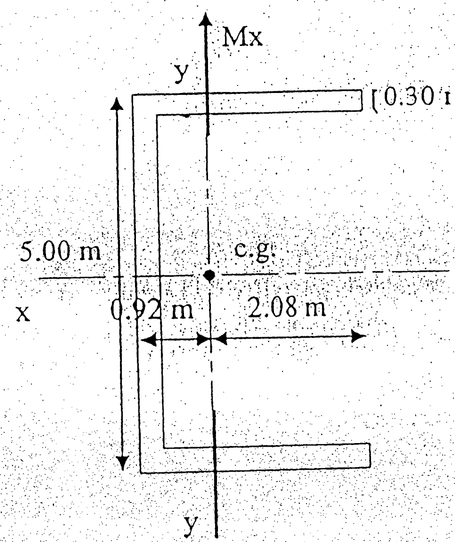
ii) For the shown reinforced concrete core shown in figure, design a suitable foundation assuming rigid footing. The core is carrying net working load 1000 ton, and working bending moment (case of loading: earthquake) $M_x$ = 800 m.t. The net allowable bearing capacity of supporting soil is 35 t/m$^2$.

**Data :**
Materials: Concrete $f_{cu} = 300$ kg/cm$^2$, Steel 40/60, $f_y = 4000$ kg/cm$^2$
Working stress: $f_c$=100 kg/cm$^2$, $f_s$ =2200 kg/cm$^2$, $q_c$=7 kg/cm$^2$, $q_{cp}$=10 kg/cm$^2$, local bond stress = 12 kg/cm$^2$
Ultimate stress: $q_{cu}$ = 9 kg/cm$^2$, $q_{cpu}$=14.5 kg/cm$^2$, $q_{bu}$= 18 kg/cm$^2$

$C_{1\,min.}$ = 3.0 , j = 0.74
$C_1$ = 3.5 , j = 0.78
$C_1$ = 4.0 , j = 0.80
$C_1$ = 4.85 , j = 0.826

Area of different reinforcement steel bars:
Φ12: area = 1.13 cm$^2$ ; Φ16: area = 2.00 cm$^2$ ;
Φ18: area = 2.84 cm$^2$ ; Φ20: area = 3.14 cm$^2$ ;
Φ22: area = 3.80 cm$^2$ ; Φ25: area = 4.91 cm$^2$

$d = C_1 \left( M_u / (f_{cu} \cdot b) \right)^{1/2}$     $As = M_u / f_y \cdot d \cdot j$



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## Question 2

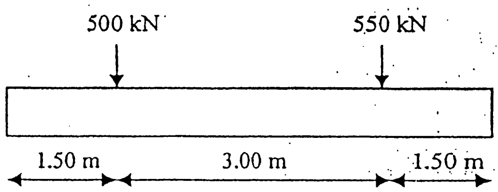
i) Explain the field test used for (direct & indirect) detemination of coefficient of subgrade reaction.

ii) Explain berifly the difference between modulus of soil elasticity ($E_s$) and the coefficient of subgrade reaction ($K_{so}$).

iii) The shown slab on grade is loaded with two **unequal** loads (500 kN, 550 kN) spaced by 2.0 m. The slab is 5.0 m long by 1.5 m width. The footing can be divided into 5 elements. The slab thickness is 45 cm and made of concrete with elastic modulus of 20 GPa. The modulus of subgrade reaction is 10000 kPa/m. It is required to:

Calculate the stress value at the slab center using:

a. Winkler assumption
b. Pseudeo-coupled method



## Question 3

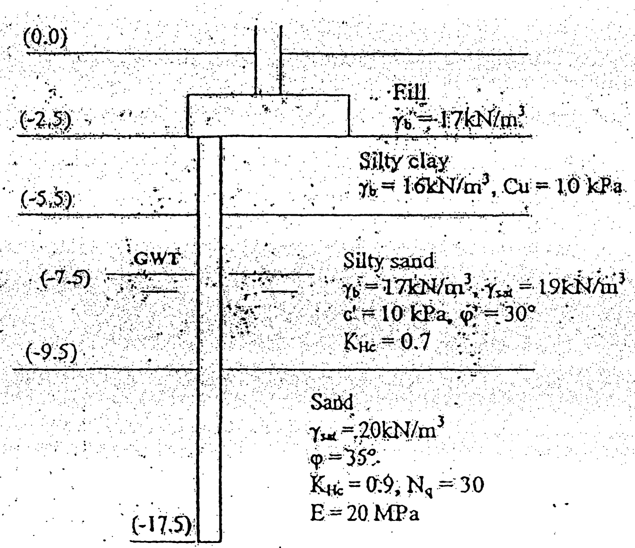
i) Explain the factors which affect the lateral capacity of a single pile subjected to horizaontal load. Discuss briefly how to distribute horizaontal loads on each pile in a pile group.

ii) Discuss the quality control of piles using pile (sonic) integrity test through the following points:

1. Advantages and limitations of the test.
2. Phenomena which can be detected by sonic integrity test, and those which can not be detected by sonic integrity test.

iii) A group of bored piles (60 cm in diameter and 15 m long) were proposed for the shown soil profile. The foundation level of the pile caps is 2.5 m below the ground surface. It is required to:

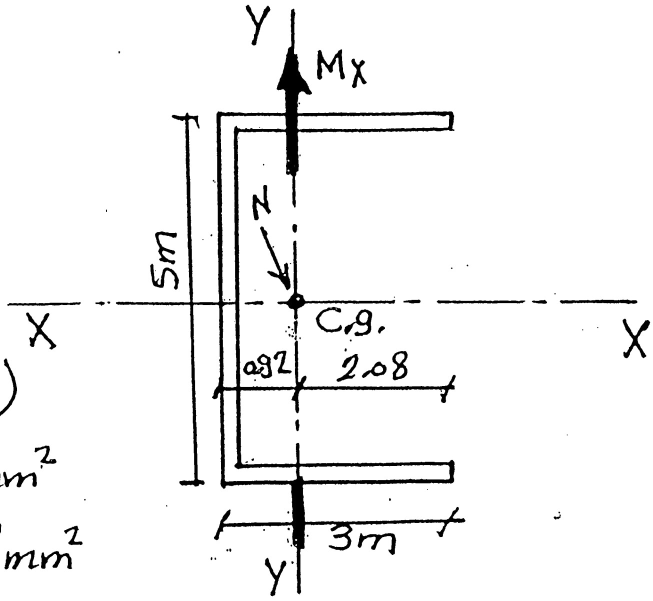
1. Estimate the allowable single pile capacity from the given soil properties using a factor of safety equals 3.0.
2. Determine the required number of piles if the column loads are:
   Vertical load Qv = 7000 kN
   Moments Mx = 1500 kN.m
   My = 1000 kN.m
3. Draw in plan (scale 1:50) the pile cap with arrangement of the estimated number of piles beneath it.
4. Calculate the settlement of the pile group considering the effective depth of the lower sandy layer is 10.0 m.
5. Use the above calculated pile group settlement to estimate the corresponding allowable settlement of the single pile.



Q1) '

## Given:-

- N = 10000 KN
- $M_X$ = 8000 KN.m (Due to earthquake → variable)
- $F_{cu}$ = 300 Kg/Cm² = 30 N/mm²
- $F_y$ = 4000 Kg/Cm² = 400 N/mm²
- $q_{all\,net}$ = 35 t/m² = 350 KPa




## Required:-

Design footing and Draw R.F.T. details

## Solution:-

~ Moment is due to Earthquake

~ Design the isolated footing on M and N

~ C.g. Core ≡ C.g. Footing

Assume t.p.c. = 10 Cm (Footing subjected to variable moment)

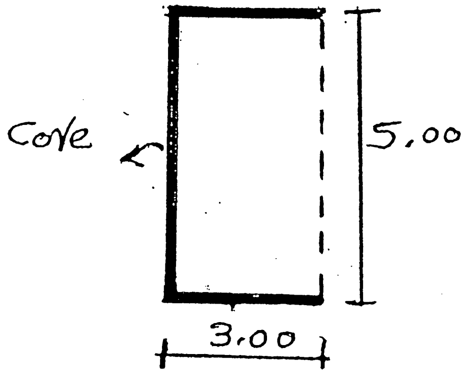
$$A_{R.C.} = \frac{N}{q_{all\,net}} + \frac{6M}{\sqrt{N \cdot q_{all\,net}}}$$

$$= \frac{10000}{350} + \frac{6 \times 8000}{\sqrt{10000 \times 350}} = 54.29\ m^2 \quad \text{--- (1)}$$

$\therefore L - B = b - a$

$= 5 - 3$

$\therefore L - B = 2$ —— (2)




Solving eqs (1) and (2)

$B_{R.C.} = 6.45 m$ , $L_{R.C.} = 8.45 m$

## Check Contact stress :-

- $F_{max} = \frac{N}{A} + \frac{6M}{BL^2}$

$= \frac{10000}{6.45 \times 8.45} + \frac{6 \times 8000}{6.45 \times (8.45)^2} = 287.7 \ kN/m^2$

$< q_{all_{net}} = 350 \ kPa$

- $F_{min.} = \frac{N}{A} - \frac{6M}{BL^2}$

$= \frac{10000}{6.45 \times 8.45} - \frac{6 \times 8000}{6.45 \times (8.45)^2} = 79.256 \ kPa > Zero$

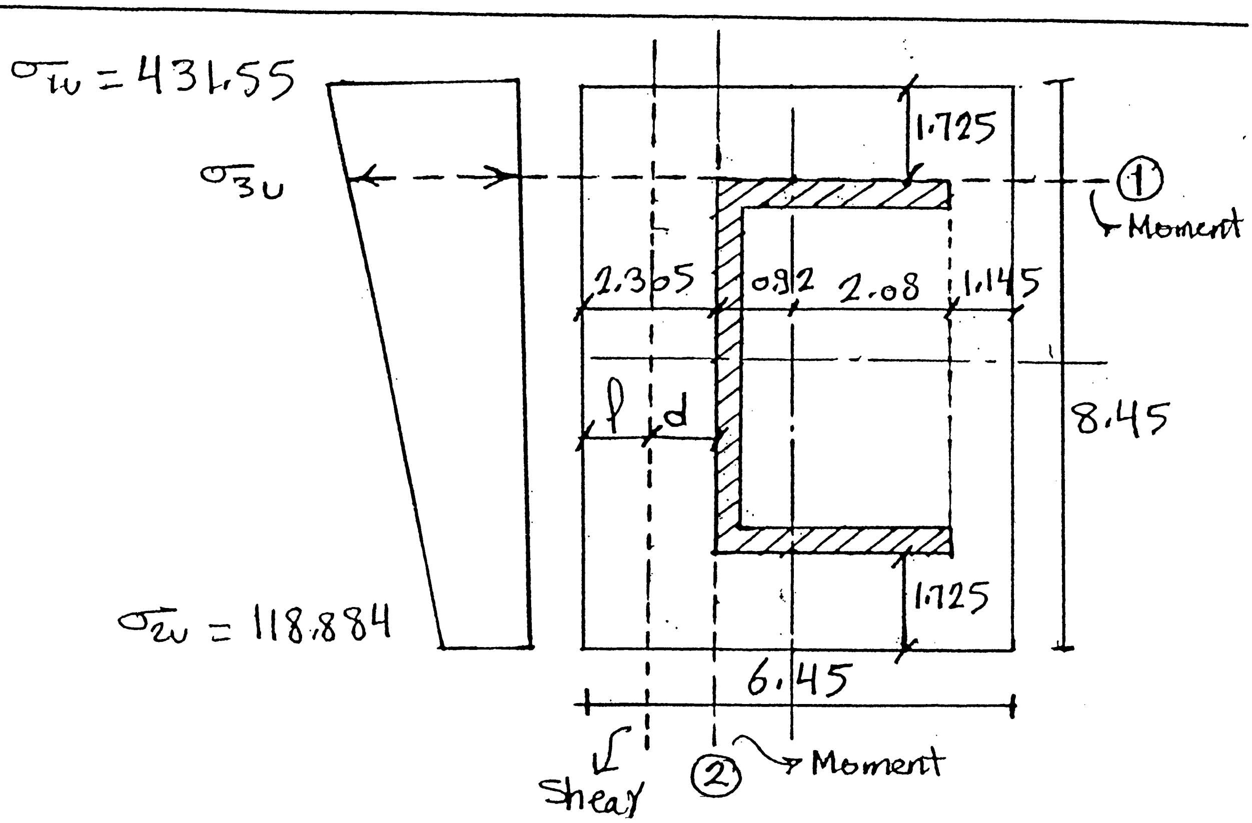
(No Tension on Soil)

- $L_{PC} = 8.65 m$ , $B_{PC} = 6.65 m$

## Calculate ultimate limit stress on footing :-

- $\sigma_{1u} = \frac{1.5 \times 10000}{6.45 \times 8.45} + \frac{1.5 \times 6 \times 8000}{6.45 \times (8.45)^2} = 431.55 \ kPa$

- $\sigma_{2u} = \frac{1.5 \times 10000}{6.45 \times 8.45} - \frac{1.5 \times 6 \times 8000}{6.45 \times (8.45)^2} = 118.884 \ kPa$




## Moment at critical sections:-

### Sec ①:-

- $\sigma_{3u} = 118.884 + \frac{8.45 - 1.725}{8.45}(431.55 - 118.884)$

  $= 367.722 \text{ KPa}$

- $\sigma_{avg.\,sec①} = \frac{\sigma_{1u} + \sigma_{3u}}{2} = \frac{431.55 + 367.722}{2} = 399.636 \text{ KPa}$

- $M_1 = 399.636 \times \frac{(1.725)^2}{2} \times 6.45 = 3835.063 \text{ kN.m/F}$

- $d_1 = 5\sqrt{\frac{3835.063 \times 10^6}{30 \times \frac{6450}{2}}} = 995.48 \text{ mm}$

Sec ② :-

- $\sigma_{avg.\ sec②} = \dfrac{\sigma_1 + \sigma_2}{2} = \dfrac{431.55 + 118.884}{2} = 275.217$ kPa

- $M_2 = 275.217 \times \dfrac{(2.305)^2}{2} \times 8.45 = 6177.94$ kN.m/L

- $d_2 = 5\sqrt{\dfrac{6177.94 \times 10^6}{30 \times \frac{8450}{2}}} = 1103.87$ mm

∴ Choose $d = d_{bigger} = d_2 = 1103.87$ mm

∴ Take $d = 1130$ mm

Check shear:-

- ال critical sec. في ال shear من ناحية sec. ② وهو القطاع الذى يعمل d أكبر.

∴ $l = Z - d = 2.305 - 1.13 = 1.175$ m

∴ $Q_{su} = \sigma_{avg.\ sec 2} \times l \times 1 = 275.217 \times 1.175 \times 1 = 323.38$ kN/m'

∴ $q_{su} = \dfrac{323.38 \times 10^3}{1000 \times 1130} = 0.286\ N/mm^2 < q_{scu} = 0.715\ N/mm^2$

(Safe)

## Check Punching:-

- The core load is distributed over large area (like walls)

∴ No Need to check Punching

## Final Thickness:-

$t = 1130 + 70 = 1200\ mm$

## R.F.T.:-

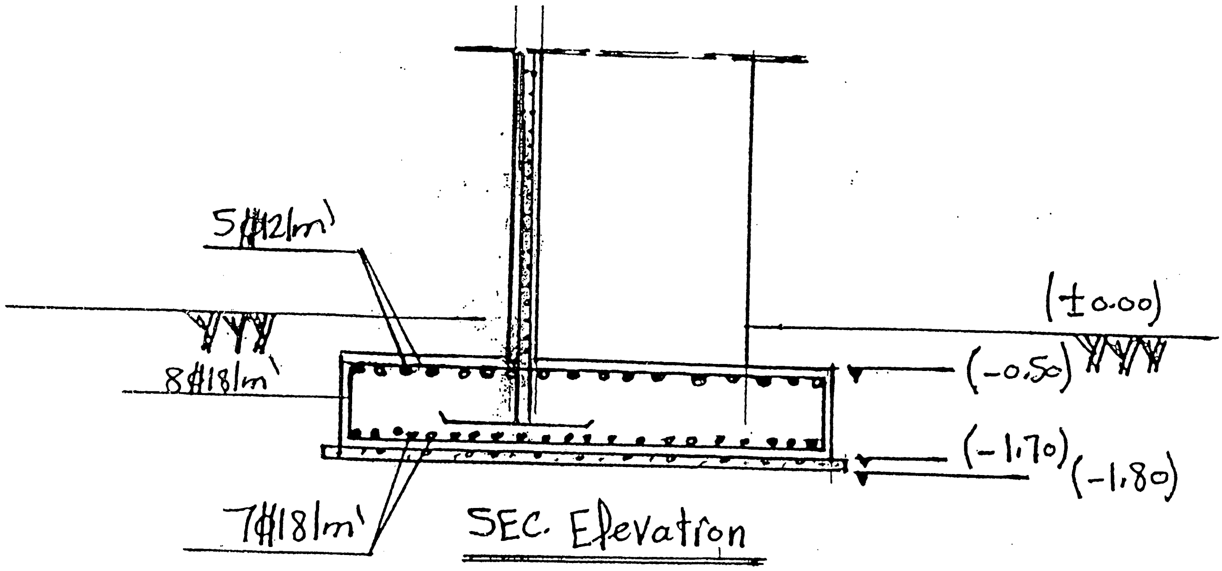
- $A_{s_{min.}} = 1.5 \times 1130 = 1695\ mm^2/m'$

- $A_{s_1} = \frac{3835.063 \times 10^6}{400 \times 0.826 \times 1130} / 6.45 = 1592\ mm^2/m' < A_{s_{min.}}$

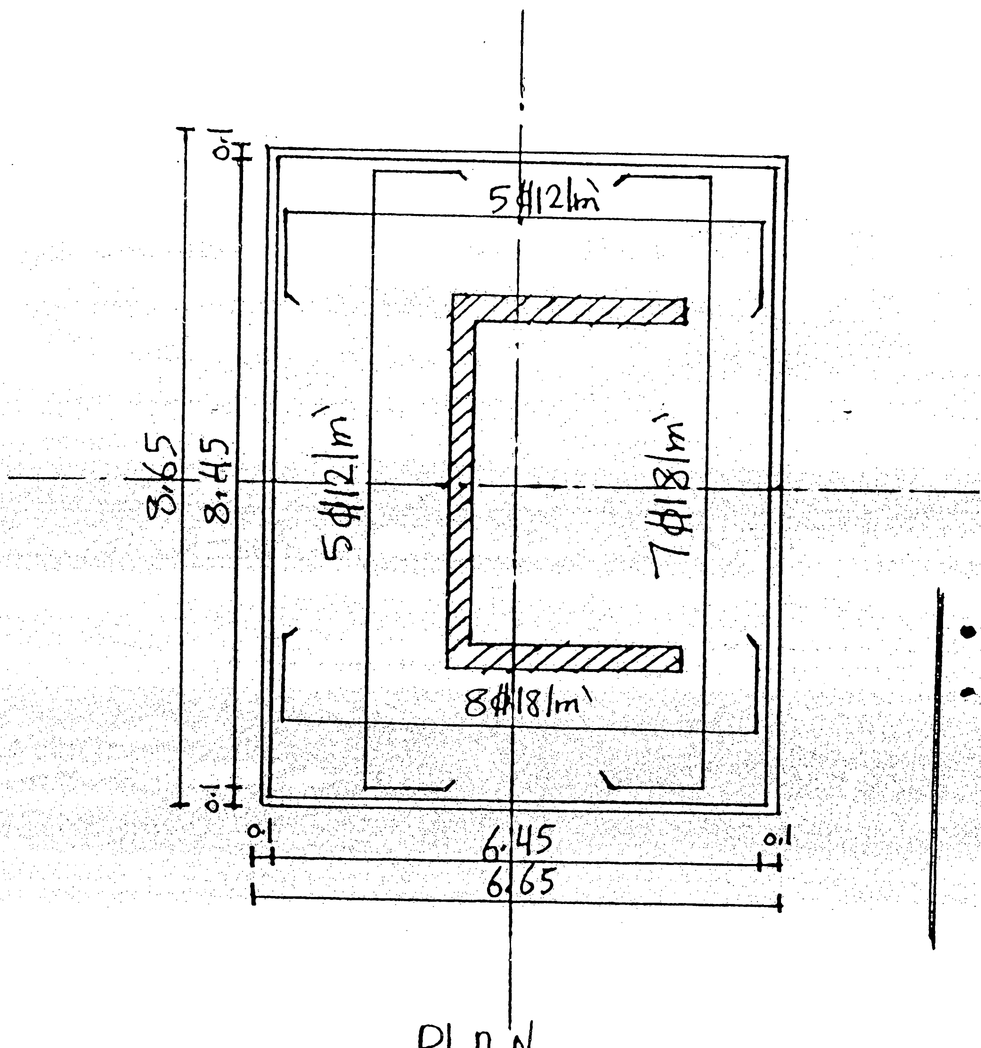
∴ Take $A_{s_1} = A_{s_{min.}} = 1695\ mm^2/m'$ **7Φ18/m'**

- $A_{s_2} = \frac{6177.9 \times 10^6}{400 \times 0.826 \times 1130} / 8.45$

$= 1958.25\ mm^2/m'$ **8Φ18/m'**




SEC. Elevation



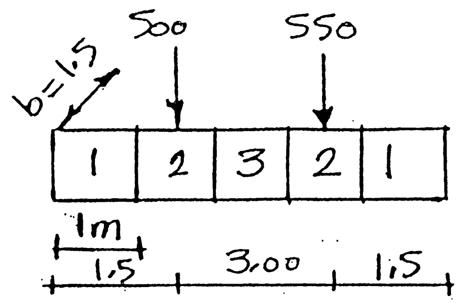

PLAN

- Scale 1/100
- All Dims are in (m).

# Question (2)

## Given:-

- $t = 45\ cm$
- $E_{RC} = 2\times10^{7}\ KPa$
- $K_{so} = 10000\ KPa/m'$ ($\rightarrow KN/m^3$)

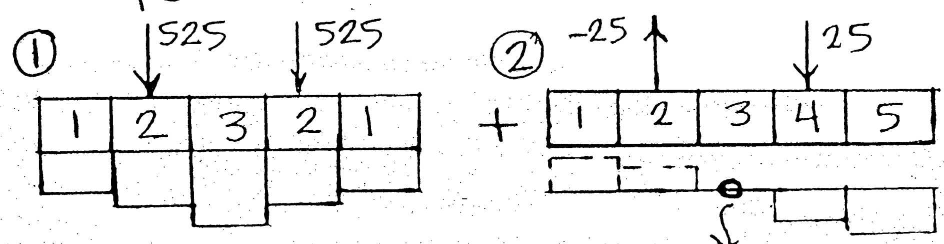



## Required:-

- Calculate the stress on the slab center based on winkler assumption, and Pseudo Coupled method.

## Solution:-

- يتم حل المسألة عن طريق تحويلها لمسألة symmetric كالآتي:-

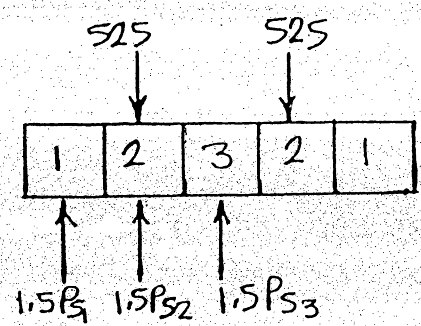



[stress at center equal zero → R=0]

ال stress عند ال slab center هو الناتج من case ① وبالتالي يتم حل case ① فقط.

### 1) Winkler assumption

#### M-Ps eqs:-




$M_1 = 0$ ——— ⓐ

$M_2 = 1.5\,P_{s_1}$ ——— ⓑ

$M_3 = 3P_{s_1} + 1.5P_{s_2} - 525$ ——— ⓒ

## M – Δ eqs:-

$$\frac{6E_cI}{a^2} = \frac{6*2*10^7*1.5*\frac{(0.45)^3}{12}}{(1)^2} = 1.367*10^6 \text{ KN}$$

### Point ②:-

$$M_1 + 4M_2 + M_3 = 1.367*10^6\left(-\Delta_1 + 2\Delta_2 - \Delta_3\right) \quad \text{--- ①}$$

### Point ③:-

$$M_2 + 4M_3 + M_2 = 1.367*10^6\left(-\Delta_2 + 2\Delta_3 - \Delta_2\right)$$

$$\therefore 2M_2 + 4M_3 = 1.367*10^6\left(-2\Delta_2 + 2\Delta_3\right) \quad \text{--- ②}$$

## Δ – $P_s$ eqs:-

$$\Delta_1 = \frac{P_{S1}}{10000} \quad \text{--- I}$$

$$\Delta_2 = \frac{P_{S2}}{10000} \quad \text{--- II}$$

$$\Delta_3 = \frac{P_{S3}}{10000} \quad \text{--- III}$$

## Final eqs:-

Substituting eqs ⓐ, ⓑ, ⓒ, and I, II, III into eqs ①, ②

We get:-

### equation ①:-

$$0 + 6P_{S1} + 3P_{S1} + 1.5P_{S2} - 525 = \frac{1.367*10^6}{10000}\left(-P_{S1} + 2P_{S2} - P_{S3}\right)$$

$\therefore\ 145.7\, P_{S1} - 271.9\, P_{S2} + 136.7\, P_{S3} = 525$ ——— (1)

equation (2):-

$3 P_{S1} + 12 P_{S1} + 6 P_{S2} - 2100 = -273.4\, P_{S2} + 273.4\, P_{S3}$

$\therefore\ 15 P_{S1} + 279.4\, P_{S2} - 273.4\, P_{S3} = 2100$ ——— (2)

equation (3):-

$\Sigma F_y = 0$

$2 P_{S1} + 2 P_{S2} + P_{S3} = 700$ ——— (3)

solving eqs (1), (2), (3)

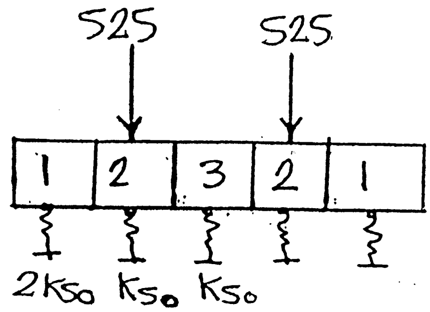
$P_{S1} = 133.83$ KPa

$P_{S2} = 143.17$ KPa

$P_{S3} = 145.98$ KPa

$\therefore$ stress at the slab center $= P_{S3} = 145.98$ KPa

## Pseudo-Coupled method :-




### Δ-Ps eqs :-

$$\Delta_1 = \frac{P_{s1}}{20000} \quad \text{--- (I')}$$

$$\Delta_2 = \frac{P_{s2}}{10000} \quad \text{--- (II')}$$

$$\Delta_3 = \frac{P_{s3}}{10000} \quad \text{--- (III')}$$

### Final eqs :-

**equation (1) :-**

$$9 P_{s1} + 1.5 P_{s2} - 525 = -68.35 P_{s1} + 273.4 P_{s2} - 136.7 P_{s3}$$

$$\therefore\ 77.35 P_{s1} - 271.9 P_{s2} + 136.7 P_{s3} = 525 \quad \text{--- (1)}$$

**equation (2) :-**

$$15 P_{s1} + 279.4 P_{s2} - 273.4 P_{s3} = 2100 \quad \text{--- (2)}$$

**equation (3) :-**

$$2 P_{s1} + 2 P_{s2} + P_{s3} = 700 \quad \text{--- (3)}$$

Solving eqs (1), (2), (3), we get :-

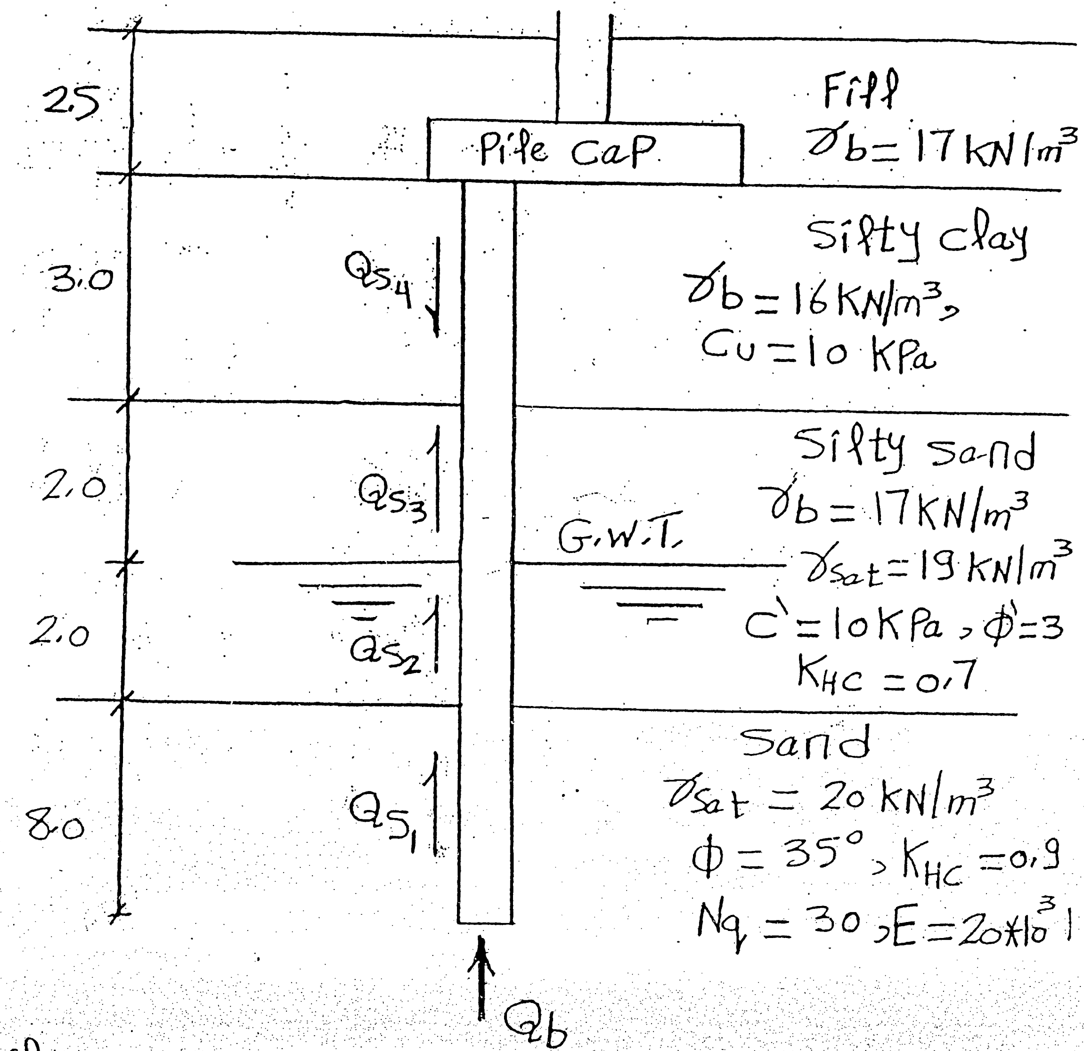
$P_{s1} = 186.1$ KPa

$P_{s2} = 107.58$ KPa

$P_{s3} = 112.47$ KPa ⟶ stress at slab center

Q3)

Given :-




- bored piles
- $d = 60$ cm
- $D_F = -2.5$ m

## 1) Calculation of the allowable single pile capacity:-

∴ For silty clay layer

$C_u = 10 \ kPa < 25 \ kPa$ (i.e. soft clay)

∴ Negative skin friction occures

∴ $Q_{ult.} = Q_b + Q_{s1} + Q_{s2} + Q_{s3} - Q_{s4}$

- $Q_b = q \, N_q \, A_b$ ($\phi$-soil)

$* A_b = \pi \frac{(0.6)^2}{4} = 0.283 \ m^2$

$* N_q = 30$ (Given)

$* q = 17*2.5 + 16*3 + 17*2 + 9*2 + 10*8$
$= 222.5 \ kN/m^2$

∴ $Q_b = 222.5 * 30 * 0.283 = 1889 \ kN$

- $Q_{s1} = [K_{HC} \, P_o \, \tan\delta] \, A_{s1}$ ($\phi$-soil)

$* A_{s1} = \pi * 0.6 * 8 = 15.08 \ m^2$

$* P_o = 222.5 - 10 * 8/2 = 182.5 \ kN/m^2$

$* \delta = 3/4 \, \phi = 3/4 * 35^\circ = 26.25^\circ$

$* K_{HC} = 0.9$ (Given)

∴ $Q_{s1} = [0.9 * 182.5 * \tan 26.25] * 15.08 = 1221.47$

• $Q_{S2} = [C_a + K_{HC} P_o \tan\delta] * A_{S2}$ (C–φ soil)

$* A_{S2} = \pi * 0.6 * 2 = 3.77 \ m^2$

$* C_a = \underbrace{0.35 \ C_{soil}}_{\text{bored Pile}} = 0.35 * 10 = 3.5 \ KN/m^2$

$* P_o = 17 * 2.5 + 16 * 3 + 17 * 2 + 9 * 1$

$= 133.5 \ KN/m^2$

$* \delta = 3/4 \ \phi = 3/4 * 30° = 22.5°$

$* K_{HC} = 0.7$ (Given)

$\therefore Q_{S2} = [3.5 + 0.7 * 133.5 * \tan 22.5] * 3.77$

$= 159.125 \ KN$

• $Q_{S3} = [C_a + K_{HC} P_o \tan\delta] * A_{S3}$ (C–φ soil)

$* A_{S3} = \pi * 0.6 * 2 = 3.77 \ m^2$

$* P_o = 17 * 2.5 + 16 * 3 + 17 * 1 = 107.5 \ KN/m^2$

$* K_{HC} = 0.7$ , $\delta = 22.5°$ , $C_a = 3.5 \ KN/m^2$

$\therefore Q_{S3} = [3.5 + 0.7 * 107.5 * \tan 22.5] * 3.77$

$= 130.7 \ KN$

• $Q_{s4} = C_a * A_{s4}$ (c-soil)

$* A_{s4} = \pi * 0.6 * 3 = 5.65 \ m^2$

$* C_a = \underbrace{0.35 \ C_{soil}}_{\text{bored Pile}} = 0.35 * 10 = 3.5 \ KN/m^2$

$\therefore \underline{\underline{Q_{s4}}} = 3.5 * 5.65 = \underline{\underline{19.775 \ KN/m^2}}$

$\therefore \underline{\underline{Q_{all_s}}} = \dfrac{Q_b + Q_{s1} + Q_{s2} + Q_{s3}}{F.O.S.} - Q_{s4}$

$= \dfrac{1889 + 1221.47 + 159.125 + 130.7}{3 \rightsquigarrow \text{Given}} - 19.775$

$= \underline{\underline{1113.66 \ KN}}$

---

## 2) Calculation of number of Piles;-

• $N = 7000 \ KN$ , $M_x = 1500 \ KN.m$ , $M_y = 1000 \ KN.$

$e_x = \dfrac{M_y}{N} = \dfrac{1000}{7000} = 0.143 \ m$

$e_y = \dfrac{M_x}{N} = \dfrac{1500}{7000} = 0.214 \ m$

$$n = \frac{1.15 N}{Q_{all}} (1 + e_x)(1 + e_y)$$

$$= \frac{1.15 * 7000}{1113.66} (1 + 0.143)(1 + 0.214)$$

$$= 10.03 \text{ Piles}$$

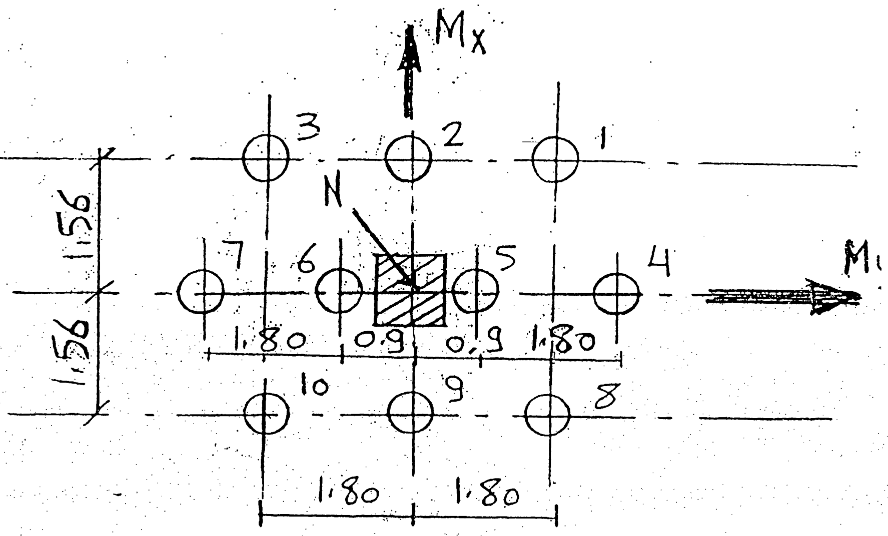
$\approx$ **Use 10 Piles**

**Assume** • $S = 3d$

$= 1.80 m$

• $e = d$

$= 0.60 m$




**Check $Q_{max}$ > $Q_{min}$ :-**

• $\Sigma(X_i^2) = 4(1.8)^2 + 2(0.9)^2 + 2*(2.7)^2 = 29.16 \, m$

• $\Sigma(Y_i^2) = 6*(1.56)^2 = 14.6 \, m^2$

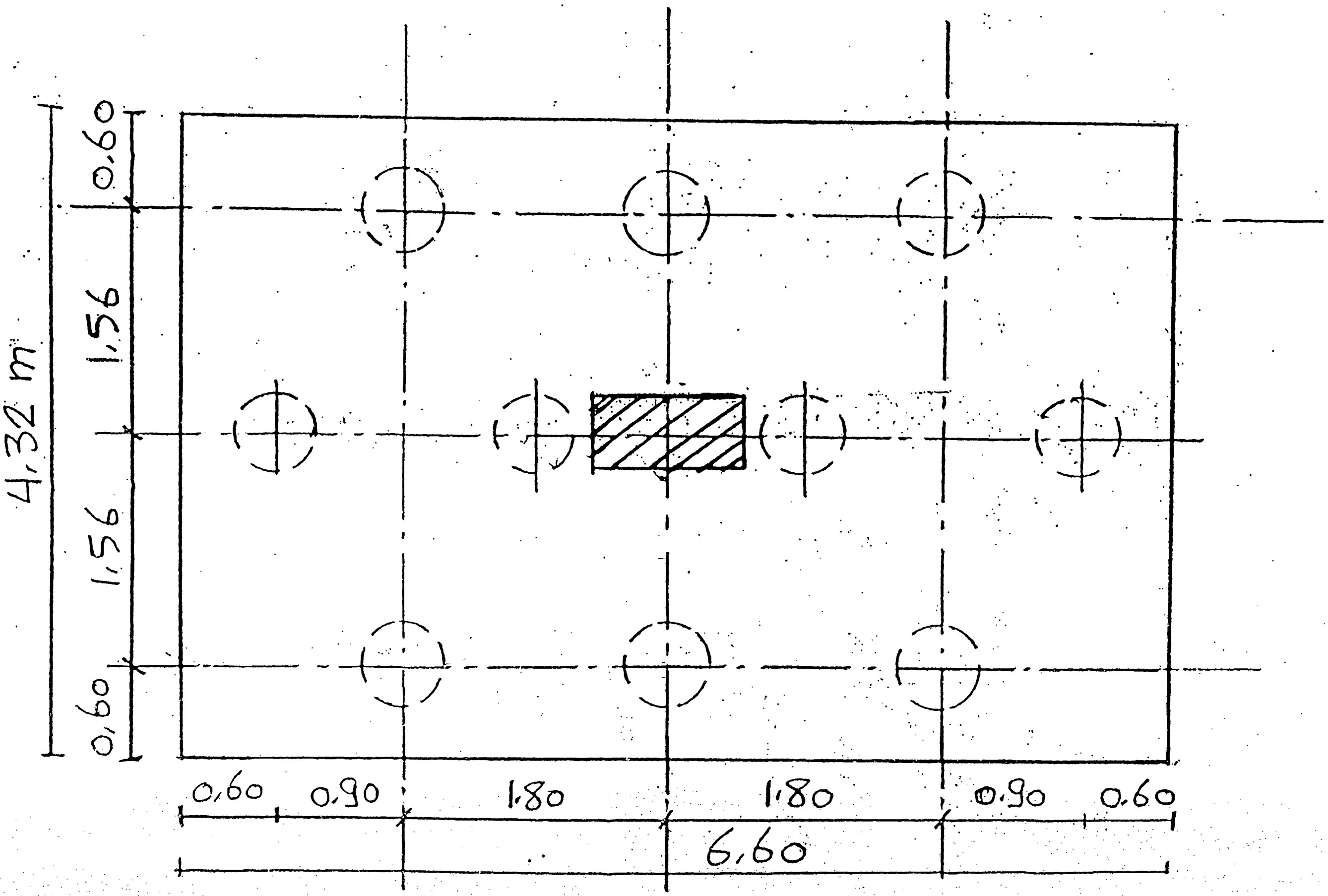
• $Q_{max} = Q_{pile ①} = \frac{1.15*7000}{10} + \frac{1000*1.80}{29.16} + \frac{1500*1.5}{14.6}$

$= 1027 \text{ KN} < Q_{all} = 1113.66$ K

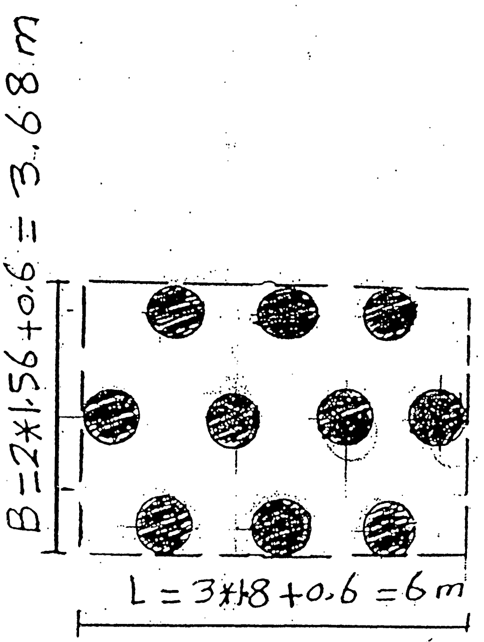
• $Q_{min} = Q_{pile_{10}} = \frac{1.15*7000}{10} - \frac{1000*1.80}{29.16} - \frac{1500*1.56}{14.6}$

$= 583 \text{ KN} > \text{Zero}$

3) Pile Cap and Piles arrangement :-

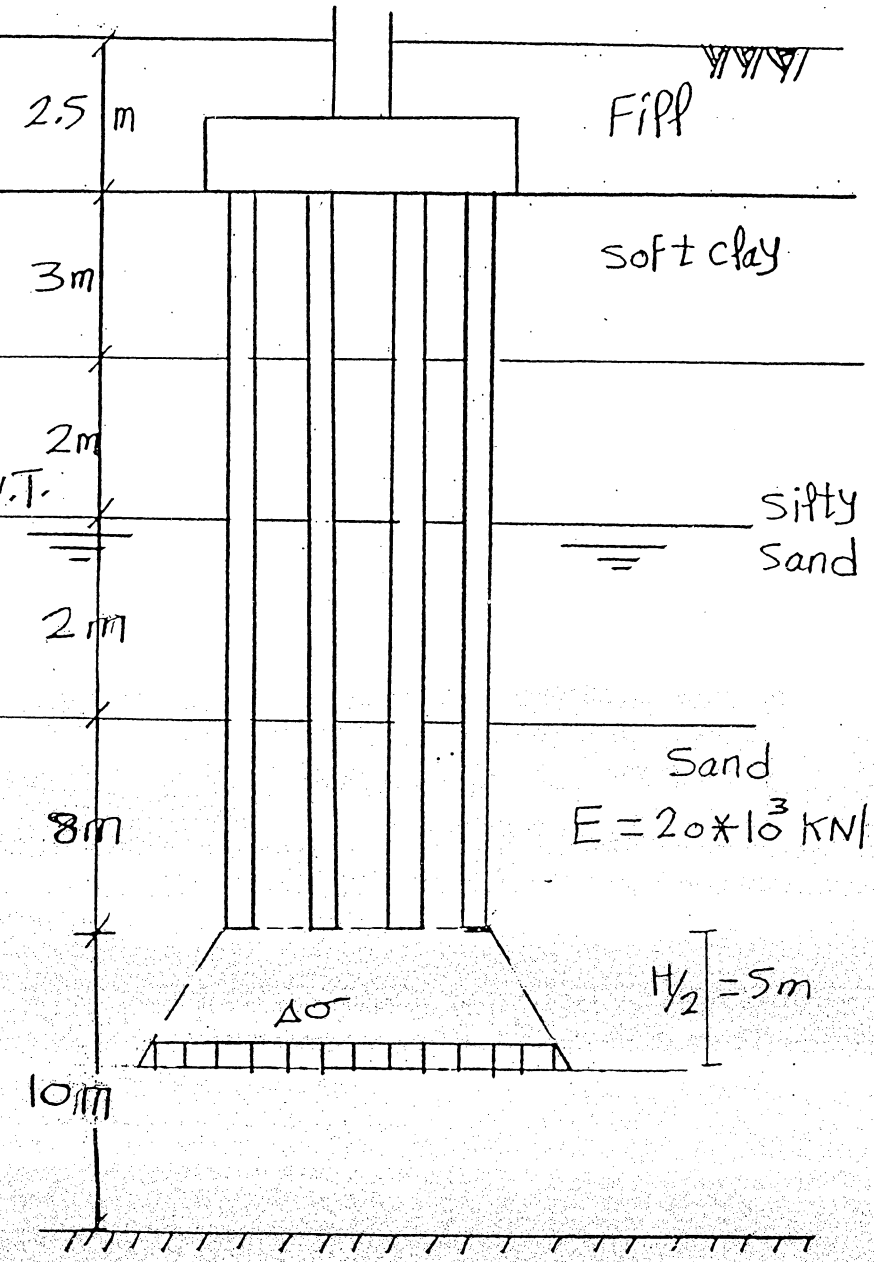


# 4) Settlement of Pile group:-




Pier dims.

حل تقريبي




* Settlement due to compressibility of sand

$$\bullet\ S_g = \frac{1}{E}\ \Delta\sigma\ H$$

$$\bullet\ \Delta\sigma = \frac{P_{col} = N}{(L+H/2)(B+H/2)} = \frac{7000}{(6+5)(3.68+5)}$$

$$= 73.31\ KN/m^2$$

$$\therefore S_g = \frac{1}{20 * 10^3} * 73.31 * 10$$

$$= 0.0366\ m$$

$$= 3.66\ cm$$

## 5) Allowable Settlement of Single Pile :-

$$\therefore S_{g_{sand}} = S_o \sqrt{\frac{B}{d}}$$

$$\therefore 0.0366 = S_o \sqrt{\frac{3.68}{0.6}}$$

$$\therefore S_o = 0.0147\ m$$

$= 1.47$ cm → allowable settlement of single pile correspondin to the allowable group settlement.