CE-366

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MIDTERM

C.E KIRTASİYE İNŞAAT MÜH. K-1 BİNASI ALT KAT

M 1.2-1

Question 1. (20%)

a. (10%) Please fill in the blanks provided below:

1. If structures are to be built on deep soft soil deposits, borings must penetrate to a depth of ..1..to..2.......times the least dimension of the structure where .consolidation.is negligible.

2. The Standard Penetration Test is not very reliable in ..cohesive.. soils and The Standard Penetration Test value of a test result reported as (4-6-7) is ....13...........

3. The linear portion of the Pressuremeter test result curve is used to determine the Pressuremeter (Deformation) Modulus

4. The value of effective overburden pressure itself is deducted when calculating the net...foundation.pressure

5. In an eccentrically loaded shallow foundation, one can make the contact pressure uniform by extending...the...foundation...... or by ..trapezoidal...footing.....................

b. (10%) Please encircle True or False?

1. The Ultimate Load that can be carried by a shallow foundation on a Granular Soil is governed by the shear strength of the soil. T F

The shear strength of such granular soils is dependent on depth. T F

2."Area Ratio" is the name given to the ratio of the Contact Area of a footing to the cross sectional area of the Column it Carries. T F

The contact area of a rigid, eccentrically loaded footing may not be the same as the area determined by its geometric dimensions. T F

3. Soils behave as linearly elastic, isotropic materials. T F

An isotropic soil is also homogeneous. T F

4. The test results obtained in a Vane Shear Test are corrected according to the friction ratio.

T F

A properly obtained friction ratio may be used to classify soils. T F

5. In situ tests are preferred over lab. tests because they are usually less expensive

T F

and

the test results are immediately available. T F m1-4

Question 3. (15%)

a. (8%) Determine the allowable load (kN) for a 3m x 3m square footing in plan at a depth of 2.0m in a moist medium dense sand if the representative uncorrected Standard Penetration Test value at a depth of (0.5) x (footing width) is 25 Blows/ft. The total settlement is to be limited to 25mm. Assume the moist unit weight of the sand as 16 kN/m³.

b. (7%) Determine the allowable load for the same footing if the ground water table is 1m below the footing base. Assume buoyant unit weight of sand as 8.2 kN/m². The uncorrected blow count is 23 this time. The total settlement is limited to 25 mm.

Use equation from amongst those below:

$$S = m_v \times \Delta p \times H \ ; \quad \frac{\Delta H}{H} = \frac{e}{1+e_o}; \quad C_w = 0.5 + 0.5\frac{D_w}{D_f + B}$$

$$C_1 = 1 - 0.5\frac{\sigma_v'}{q - \sigma_v'}; \quad q_a = 11\ N\ C_w \ ; \quad \Delta H = \Delta H_v\left(\frac{B}{B_o}\right)$$

$$C_N = 9.78\sqrt{\frac{1}{\sigma_v'\ (kN/m^2)}} \quad ; \quad q = 0.4\gamma B N_\gamma + \gamma D N_q$$

a) $\sigma_{v_o}' = 16\left(2 + \frac{3}{2}\right) = 56\ kN/m^2$

$C_N = 9.78\sqrt{\frac{1}{56}} = 1.31$

$N_{cor} = 1.31 \times 25 = 33$

$q_{ALL} = 11 \times 33 \times 1.0 = 363\ \frac{kN}{m^2} \Rightarrow Q_{ALL} = 3^2 \times 363 = 3267\ kN$

b) $\sigma_{v_o}' = 16 \times 3 + 8.2 \times 0.5 = 52\ kN/m^2$

$C_N = 9.78\sqrt{\frac{1}{52}} = 1.36$

$N_{corrected} = 1.36 \times 23 = 31$

$C_w = 0.5 + 0.5\ \frac{3}{2+3} = 0.8 \quad (D_w < D_f + B)$

$\Rightarrow q_{ALL} = 11 \times N \times C_w = 11 \times 31 \times 0.8 = 273\ kN/m^2 \ (272.8) \Rightarrow Q_{ALL} = 3^2 \times 273 = 2457\ kN$

Question 4. (10%)

A building has a raft foundation which is 50 m x 50 m square in plan and is to be constructed within a deep bed of soft clay which has an undrained shear strength of 18 kN/m^2 and a saturated unit weight of 18 kN/m^3.

The water table is at the surface of the clay, which is overlain by a 1m thick layer of sand of unit weight 16 kN/m^3.

To keep settlement effects within acceptable limits it is necessary to ensure that the net increase in foundation pressure will not exceed 50 kN/m^2.

Determine the necessary foundation depth if the total applied load will be 300 MN.

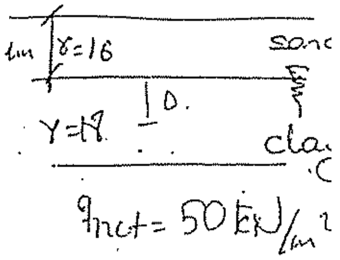
c_u γ_s

Gross Foundation Pressure $= \dfrac{\overbrace{300 \times 1000}^{\text{Applied Load}}}{\underbrace{50 \times 50}_{\text{Area}}} = 120 \text{ kN/m}^2$

300×10^3 kN

d = ?

Required pressure relief by excavation $= \underset{q_{gross}}{120} - \underset{q_N}{50} = 70 \text{ kN/m}^2$



IF D is the required depth of foundation below from the clay bed;

Weight of removed soil

$$1 \times 16 + D \times 18 = 70$$

$D = \underline{3.0 m}$. (from clay surface)

$z = 3.0 + 1.0 m$ (from ground surface)

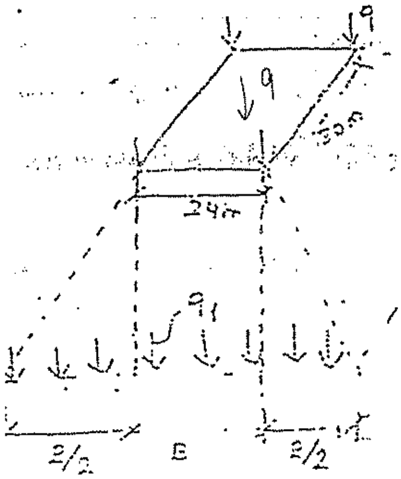
M 1-6

$0.20 qBL = \frac{qBL}{(24+z)(30+z)}$

Question 5. (25%)

a. (7%) For the foundation design of a residential building with preliminary foundation dimensions of 24m x 30m, site investigation studies are to be planned. It was agreed that the depth of borings should reach to a depth where average vertical stress due to the weight of the structure reduce to 20 % of its value at the foundation surface

i) Estimate the depth of borings by using 2V:1H approximation procedure



$q_1 = \frac{qBL}{(B+z)(L+z)}$ (z : Depth of Boring)

(q_1 : Average vertical stress @ depth z)

$q_1 = 0.20 q$

$$0.20q = \frac{q \times 24 \times 30}{(24+z)(30+z)}$$

$$0.20\,(720 + 54z + z^2) = 720$$

$$\Rightarrow z^2 + 54z - 2880 = 0$$

$$z \cong \underline{33.\ m}$$

b. (10%) At 3 m depth a silty sand layer was encountered and a Standard Penetration Test (SPT) was performed. The blowcounts for the first, second and third 15 cm increments were reported as 6,8,9 blows respectively. For the test, safety hammer of an energy ratio of 67% was used. The borehole diameter was measured as 110 mm and the SPT sampler used was a standard sampler with constant inside diameter (no room for liners). The length of the rod from the bottom of the safety hammer to the sampler at 3 m depth was measured as 5.2 m

ii)(7 %) Estimate the overburden and procedure corrected SPT blowcounts ($N_{1,60}$) for 30 cm penetration of the sampler (water table depth is below 3 m and the unit weight of soil in the upper 3 m can be assumed as 18 kN/m^3)

Table 1 : Summary of the correction factors for SPT measurements

(After 1997 NCEER Workshop)

Factor	Term	Equipment Variable	Correction
Overburden Pressure	C_N	-	$(P_a/\sigma'_v)^{0.5}$ $C_N \leq 2$
Energy Ratio	C_E	Safety Hammer Donut Hammer	0.60-1.17 0.45-1.00
Borehole Diameter	C_B	65-115 mm 150 mm 200 mm	1.00 1.05 1.15
Rod Length	C_R	3-4 m 4-6 m 6-10 m 10-30 m > 30 m	0.75 0.85 0.95 1.0 < 1.0
Sampling Method	C_S	Standard Sampler Sampler without liners	1.0 1.15-1.30

$SPT_N = 8+9 = 17$

$E_R = 67\% \Rightarrow C_E = \frac{0.67}{0.60} \times 100 = 1.16$ (Energy Ratio)

$\phi = 110\ mm \Rightarrow C_B = 1.00$ (For BoreHole Diameter $65 < \phi < 115\ mm$)

$L_{Rod} = 5.2\ m \Rightarrow C_R = 0.85$ (Borehole Diameter)

Standard Sampler $\Rightarrow C_S = 1.00$ (Sampling Method)

$\sigma'_{vo} = 3 \times 18 = 54\ kN/m^2 \Rightarrow C_N = \left(\frac{P_a}{\sigma'_{vo}}\right)^{0.5} = \left(\frac{100}{54}\right)^{0.5} = 1.36$ (Overburden Correction)

$N_{1,60} = N \cdot C_N \cdot C_E \cdot C_B \cdot C_R \cdot C_S = 17 \times 1.16 \times 1.00 \times 1.00 \times 0.85 \times 1.00$ 1.36

$N_{1,60} = 23 / 30\ cm$

iii)(3 %) Classify the silty sand layer based on $N_{1,60}$ value you have calculated in part ii by using the information provided in the table given below

$N_{1,60}$	Relative Density
0-4	Very loose
4-10	Loose
10-30	Medium
30-50	Dense
>50	Very Dense

$N_{1,60} = 23 \Rightarrow$ Silty Sand Layer's Relative Density is "Medium."

c. (8%) At 8 m depth a silty clay layer was encountered and a vane shear test was performed. The diameter, the height of the vane apparatus, and the maximum torque applied were reported as D= 51mm, H= 102 mm; and T= 60,000 Nmm respectively.

iv) Estimate the undrained shear strength (c_u) of silty clay layer at 8 m depth by using vane test data. (Hint : $T = c_u \cdot \pi \cdot D^2 \cdot \left(\frac{H}{2} + \frac{D}{6}\right)$. since plasticity index of silty clay layer happened to be 20, there is no need to apply λ correction)

$$T = C_u\, \pi\, D^2 \left(\frac{H}{2} + \frac{D}{6}\right)$$

$$60000 = C_u \cdot \pi \cdot (51)^2 \left(\frac{102}{2} + \frac{51}{6}\right)$$

$$C_u = 0.1234\ N/mm^2 = 0.1234\ MPa \Rightarrow C_u = 123.4\ kPa$$

m1-8

Question 6. (25 %)

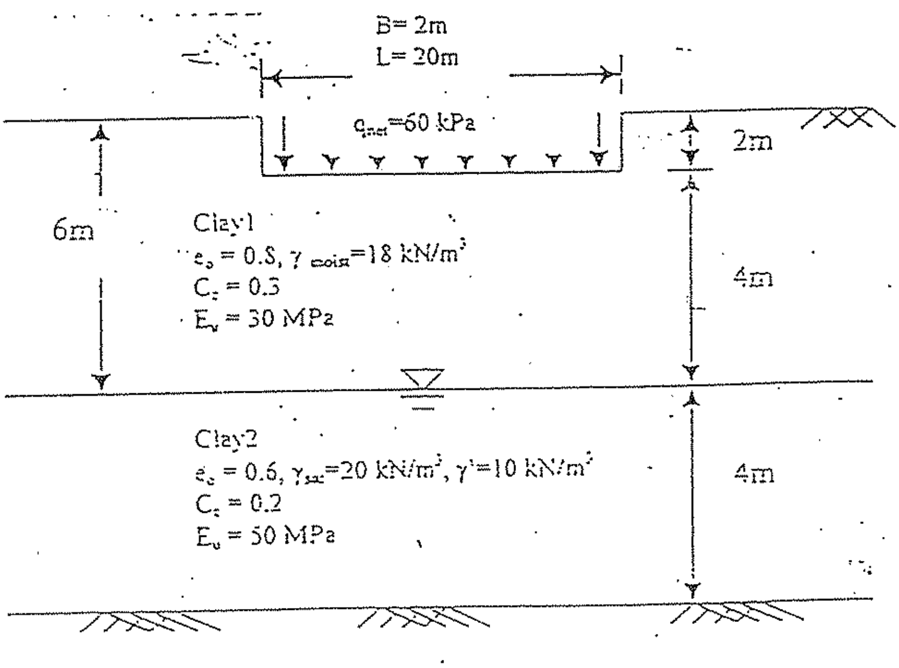
The soil profile in a site consists of two compressible clay layers overlying the bedrock as shown in the figure below. The proposed structure to be built at this site is a L x B = 20 x 2 square meter hydraulic structure, and the magnitude of the net foundation pressure is 60 kN/m^2. The foundation depth is at 2 m below the existing ground level. Both clay layers are normally consolidated, and the ground water table is at 6 m depth from the existing ground level.

a. (10%) Calculate the immediate settlement of the clay layers using undrained modulus (E_u) values are given in the figure below.

b. (15%) Calculate the consolidation settlement of the clay layers using compression index (C_c) values given in the figure below.

Note: You may use 2V: 1H approximate stress distribution. Do not consider Skempton and Bjerrum correction for settlements.

$$s_{oed} = \Sigma\left[H\frac{C_c}{1+e_o}\log\left(\frac{\sigma_o' + \Delta\sigma'}{\sigma_o'}\right)\right] \qquad s_i = \mu_0\mu_1\left[\frac{qB}{E_u}\right]$$



ROCK (Incompressible)

a) $\frac{D}{B} = \frac{2}{2} = 1.0 \Rightarrow \mu_0 \cong 0.93$

First layer: $H = 4m$ $\quad H/B = 2.0 \quad L/B = 10 \quad \mu_1 = 0.65$

$$S_{i_1} = 0.93 \times 0.65 \times \frac{60 \times 2}{30\,000} = 2.4 \times 10^{-3}\,m = 2.4\,mm$$

Two layer combined with $E_u = 50$ MPa

$H = 8m \quad H/B = 4 \quad L/B = 10 \quad \mu_1 \cong 1.0$

$$S_{i_2} = 0.93 \times 1.0 \times \frac{60 \times 2}{50\,000} = 2.2 \times 10^{-3}\,m = 2.2\,mm$$

First layer with $E_u = 50$ MPa

$\mu_0 = 0.93 \quad \mu_1 = 0.65$

$$S_{i3} = 0.93 \times 0.65 \times \frac{60 \times 2}{50\,000} \cong 1.5 \times 10^{-3}\,m = 1.5\,mm$$

$S_i = 2.4 + 2.2 - 1.5 \cong \underline{\underline{3.1}}$ mm (not significant)

b) First Layer

$$\Delta\sigma = \frac{qBL}{(B+z)(L+z)} = \frac{2 \times 20 \times 60}{(2+2)(20+2)} = 27.3\,kPa$$

$\sigma_0' = 4 \times 18 = 72$ kPa, $H = 4m$, $e_0 = 0.8$, $C_c = 0.3$

$$S_{c_1} = 4.0 \times \frac{0.3}{1+0.8} \log \frac{72 + 27.3}{72} = 0.093\,m$$

Second Layer

$$\Delta\sigma = \frac{2 \times 20 \times 60}{(2+6)(20+6)} = 11.5\,kPa$$

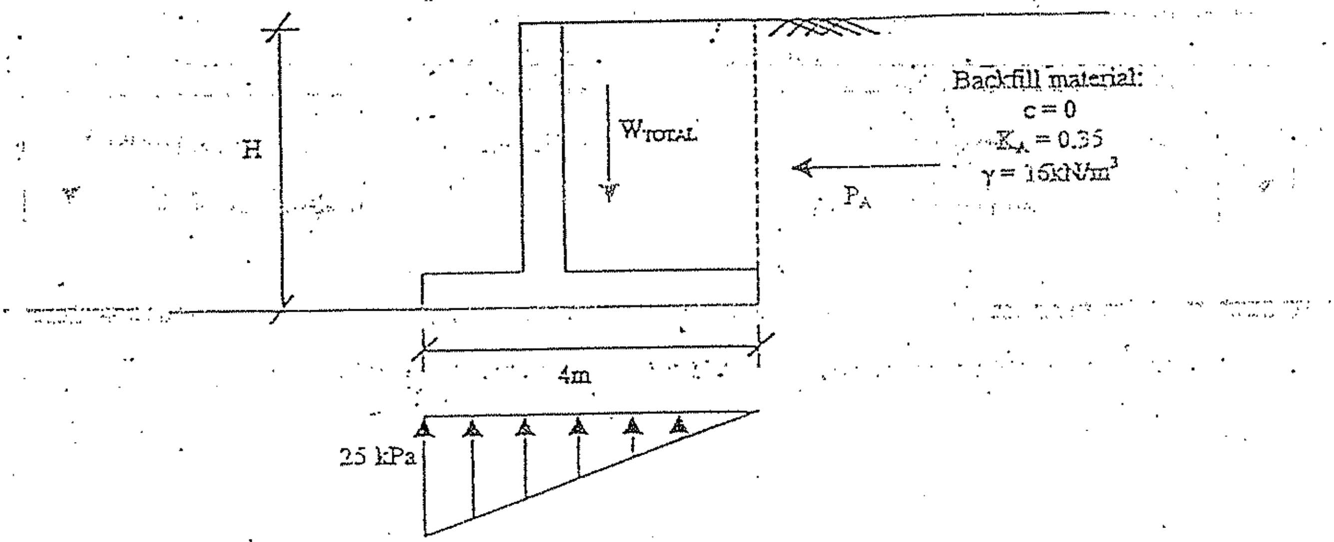
$\sigma_0' = 6 \times 18 + 2 \times 10 = 128$ kPa $\quad H = 4m$, $e_0 = 0.6$, $C_c = 0.2$

$$S_{c_2} = 4.0 \times \frac{0.2}{1+0.6} \log \frac{128 + 11.5}{128} = 0.019\,m$$

$S_{c_1} + S_{c_2} = 0.093 + 0.019 = 0.112\,m = 112\,mm$

Question 2. (20%)

A cantilever retaining wall and the pressure distribution beneath its foundation are given in the figure below. (Hint: Assume that the soil pressure doesn't change. for all the parts of the question)

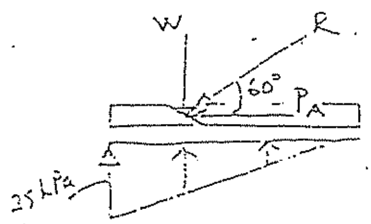


a. (7%) Determine the height of the wall if the resultant of the total weight (weight of the wall + weight of the soil on the wall) and the active thrust acting on the wall is inclined at the 60° to the horizontal.

b. (7%) Assuming the height of the wall is 3 m, determine the factor of safety (F.S.) against overturning, also state whether the factor of safety is sufficient or not.

c. (6%) If the angle of friction between the base of the wall and soil is 30°, determine the F.S. against base sliding. Assume the wall height is 3m. Also state whether the F.S. is sufficient from the point of design against sliding or not.

(a)



$\Sigma F_{ver} = 0 \Rightarrow \quad W = 25 \times 4 \times \frac{1}{2} = 50 \text{ kN/m}$

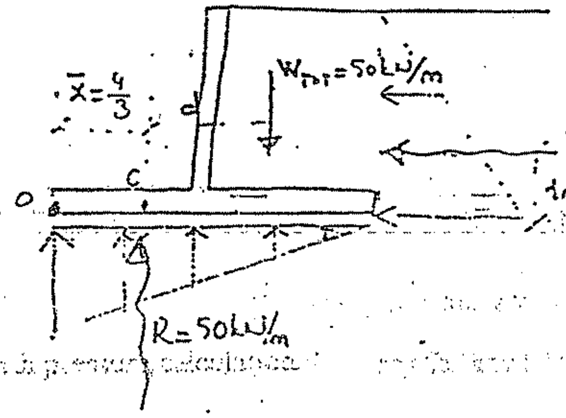
$P_A = \frac{W_{TOT}}{\tan 60} = \frac{50}{1.732} = 28.87 \text{ kN/m}$

$P_A = \frac{1}{2} \gamma H^2 . K_A$

$28.87 = \frac{1}{2}(16)(H)^2 (0.35) \quad H = 3.21 \text{ m}$

For $H = 3m$

$$P_A = \frac{1}{2}\gamma H^2 K_A = \frac{1}{2}(16)(3)^2(0.35) = 25.2\ kN/m$$



$\curvearrowleft + \sum M_c = 0$

$50 \times d - 25.2 \times 1.0 = 0$

$d = 0.504m$

Force (kN/m)	Arm (about pt O)	Moment (kN·m/m)
$W_{TOT} = 50$	$0.504 + \frac{4}{3} = 1.84$	92 (Resisting
$P_A = 25.2$	1.0	25.2 (Overturn

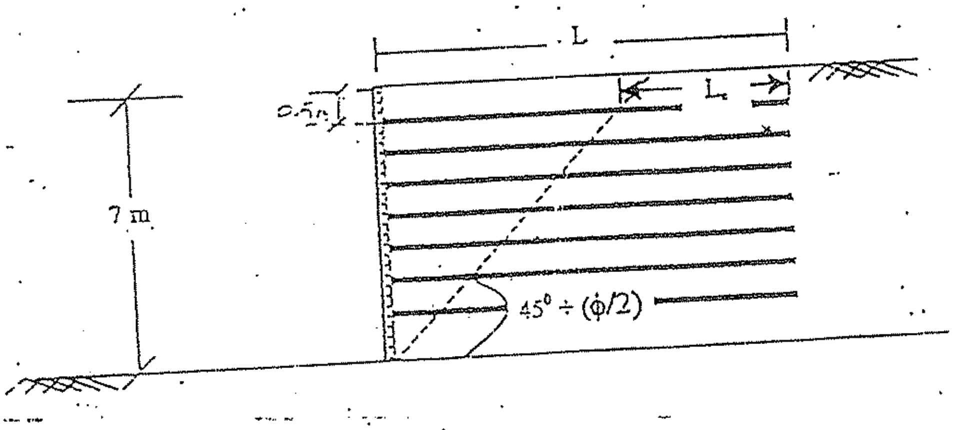
$$F.S._{OVERTURN} = \frac{\sum M_{RESIST}}{\sum M_{OVERTURN}} = \frac{92}{25.2} = 3.65 > 2.0$$

$\therefore$ O.K ✓

(c) $F.S._{SLIDING} = \frac{\sum F_R}{\sum F_D} = \frac{\sum V \cdot \tan\delta}{\sum H} = \frac{50 \times \tan 30}{25.2} = 1.15 < 1.5$ $\therefore$ Not O.K ✗

Question 3. (10%)

A 7m high reinforced earth retaining wall is to be constructed. Dimensions of the precast concrete panels are $S_v = 1.0$ m and $S_h = 1.25$ m. The unit weight and internal friction angle of backfill material are $\gamma = 17\ kN/m^3$ and $\phi = 30^0$, respectively.



Extensible galvanized steel strips, which will be used as reinforcement have the following properties:

Width (w) = 90 mm

Yield Stress (f_y) = 2.4 x 10^5 kN/m^2

Soil – Reinforcement Friction Angle (δ) = 28^0 Hint: $K_A = \dfrac{1-\sin\phi}{1+\sin\phi}$

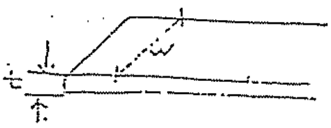
Corrosion Rate: 0.03 mm/year

Use Rankine's Theory for any lateral earth pressure calculation.

a. (5%) Determine the design thickness of reinforcements (t_d) for a service life of 50 years and a factor of safety of 3 against breaking (FS_b).

b. (5%) Determine the strip length ($L = L_r + L_e$) for a factor of safety of 3 against pull-out (FS_p).

(a) $F.S._b = \dfrac{w \cdot t \cdot f_y}{\gamma z\, K_A\, S_V\, S_H}$ $K_A = \dfrac{1-\sin 30}{1+\sin 30} = \dfrac{1}{3}$



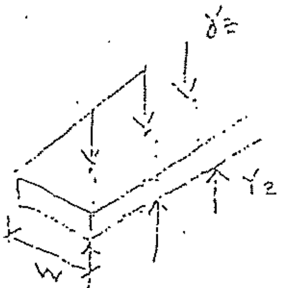
$T_{max} = \gamma \cdot (z_{max} = 6.5)\, K_A\, S_V\, S_H$

$3.0 = \dfrac{0.09 \times t \times 2.4 \times 10^5}{17 \times (6.5) \times 1/3 \times 1 \times 1.25} \Rightarrow t = 6.4 \times 10^{-3}$ m

Assume $t = 6.5$ mm

$t_{design} = 6.5 + 0.03 \times 50 = 8.0$ mm

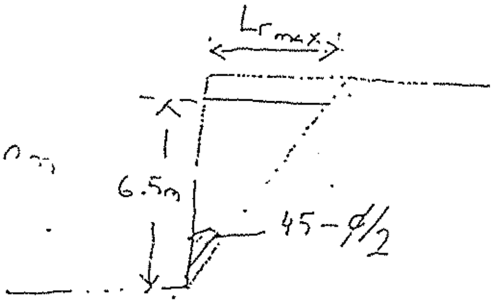
b) $F.S._p = \dfrac{2 \cdot w \cdot L_e\, \cancel{\gamma z}\, \tan\delta}{K_A\, \cancel{\gamma z}\, S_V \cdot S_H}$



$3.0 = \dfrac{2 \times 0.09 \times L_e \times \tan 28^{\circ}}{\frac{1}{3} \times 1.0 \times 1.25}$

$L_e = 13$ m

$L_{r\,max} = (6.5)\tan\left(45 - \frac{30}{2}\right) \cong 4.0$ m

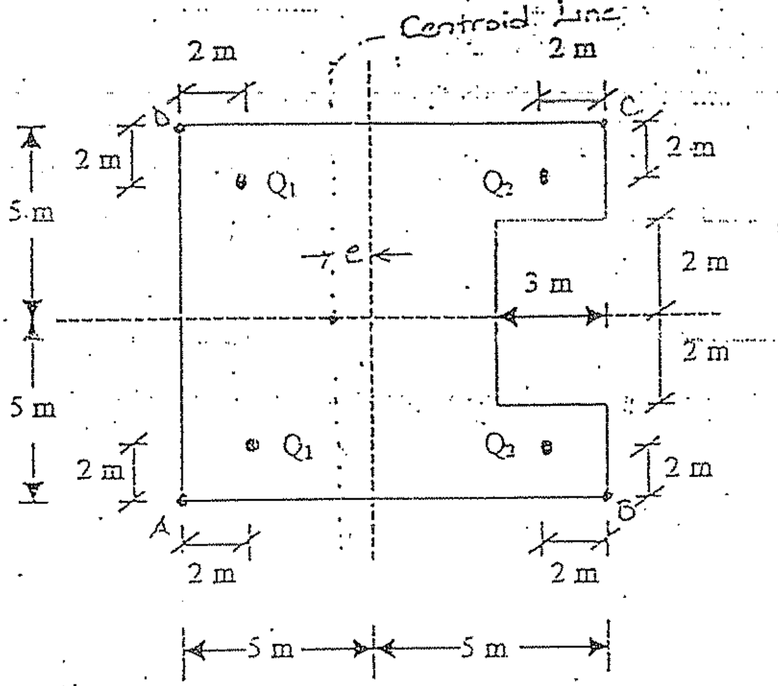


$L = L_e + L_r = 13 + 4 = 17$ m

$m\ 1:25$

Question 4. (15%)

A 10 x 10 m^2 with a cut area of 3 x 4 m^2 supports four columns as shown in the figure below. Determine the magnitude of the column loads Q_1 and Q_2 so that the soil pressure beneath the foundation would be uniform and equal to 120 kPa.



Taking area moments about B to find the centroid of the final shape,

$$(10\times10)\times5 - (3\times4)\times\frac{3}{2} = (10\times10-3\times4)\times[5+e] \Rightarrow (5+e) = 5.477$$

$$e = 0.477\ m$$

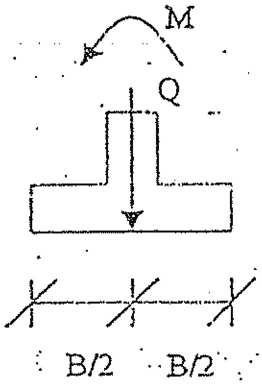
$$\Sigma F_{vertical} = 0 \Rightarrow 2Q_1+2Q_2 = 120\times[10\times10-3\times4] \Rightarrow Q_1+Q_2 = 5280\ kN \quad \text{--- (1)}$$

$$\Sigma M_{centroid} = 0 \Rightarrow 2Q_1[3-0.477] = 2Q_2[3+0.477] \Rightarrow Q_1 = 1.378\ Q_2 \quad \text{--- (2)}$$

Solving (1) & (2) together $\rightarrow$ $Q_2 = 2220\ kN$ $\quad Q_1 = 3060\ kN.$

Question5. (10%)

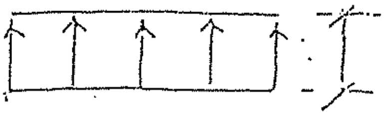
Derive an expression relating the soil pressures beneath a footing subjected to a moment M and a concentric column load of Q in terms of footing dimensions B, L and eccentricity e.



Hint: Derive $q = \dfrac{Q}{BL}\left(1 \mp \dfrac{6e}{B}\right)$

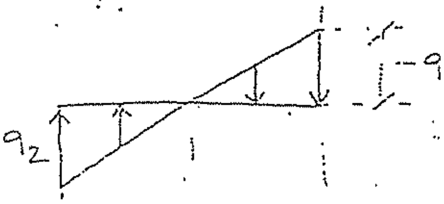
Note : Moment of inertia of rectangle

$$I = \frac{B^3 L}{12}$$



$q_1 = \dfrac{Q}{B \times L}$ due to Q.

$+$



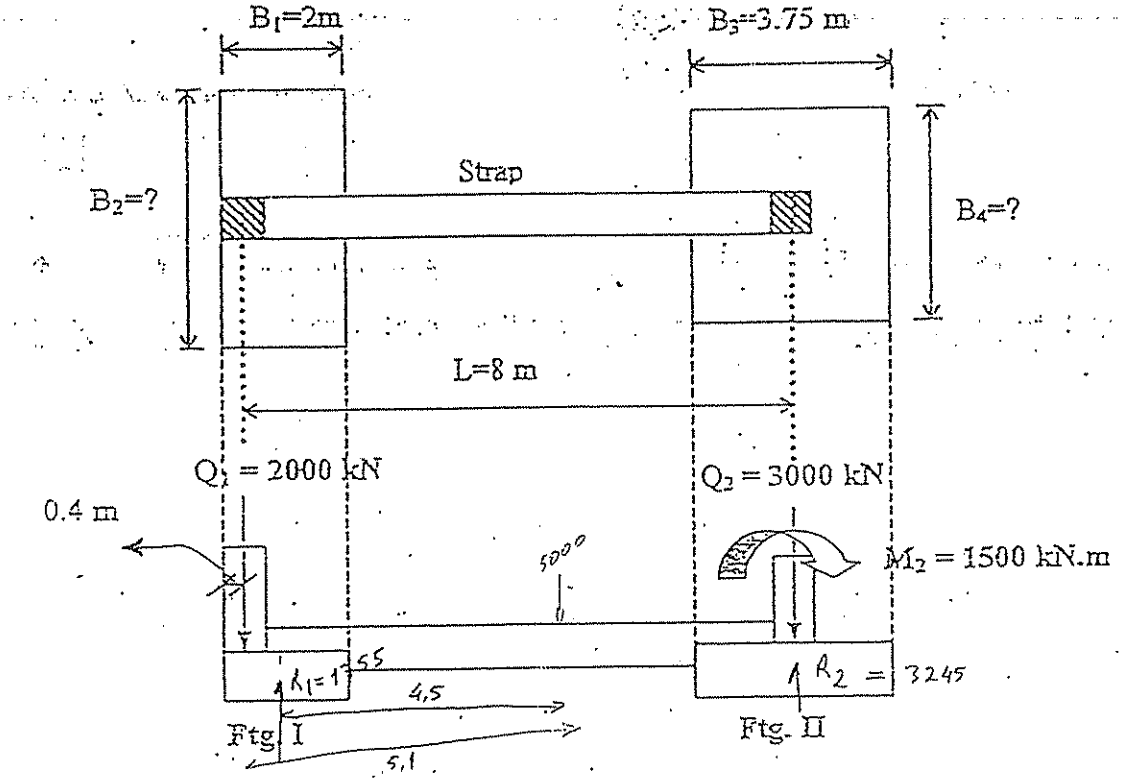
$$q_2 = \frac{M\,y}{I} = \frac{(Q.e)\left(B/2\right)}{\left(\frac{B^3 L}{12}\right)} = \frac{6.e\,Q}{B^2 L}$$

$$q_1 + q_2 = q = \frac{Q}{BL} \mp \frac{6eQ}{B^2 L} = \frac{Q}{BL}\left(1 \mp \frac{6e}{B}\right)$$

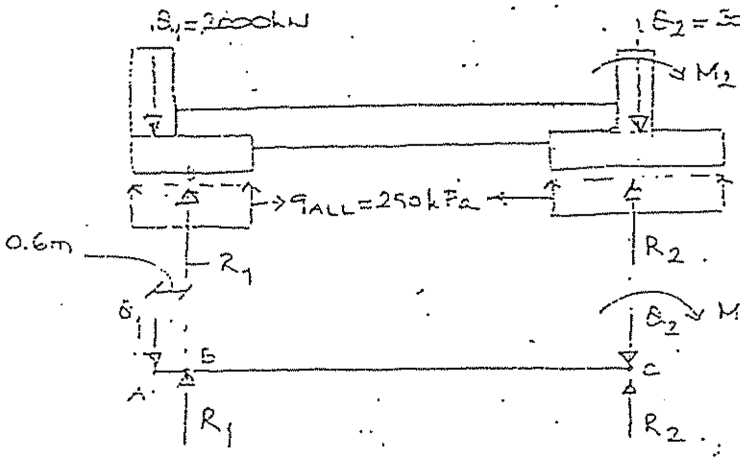
m1-2-7

Question 6. (25%)

The figure given below shows the foundation plan view and cross-section of a residential building. To achieve uniform pressure distribution beneath the footings two footings were combined by a strap. The net allowable bearing capacity for the foundation soils is estimated as 250 kN/m^2.



a. (13%) Estimate the minimum footing dimensions B_2 and B_4.



$\curvearrowleft + \sum M_B = 0$

$(Q_2 - R_2)\times 8 + M_2 - Q_1\times 0.6 = 0$

$(3000 - R_2)\times 8 + 1500 - 2000\times 0.6 = 0$

$\Rightarrow R_2 = 3037.5$ kN

$\sum F_{vertical} = 0$

$Q_1 + Q_2 = R_1 + R_2$

$2000 + 3000 = 3037.5 + R_1$

$\Rightarrow R_1 = 1962.5$ kN

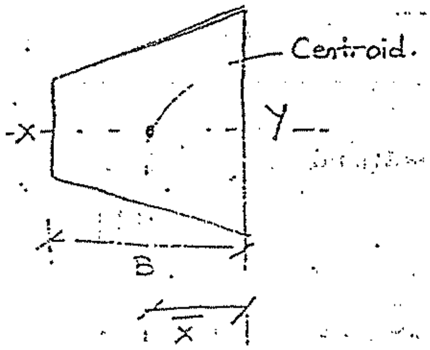
$R_1 = 250\times(B_2\times 2)$

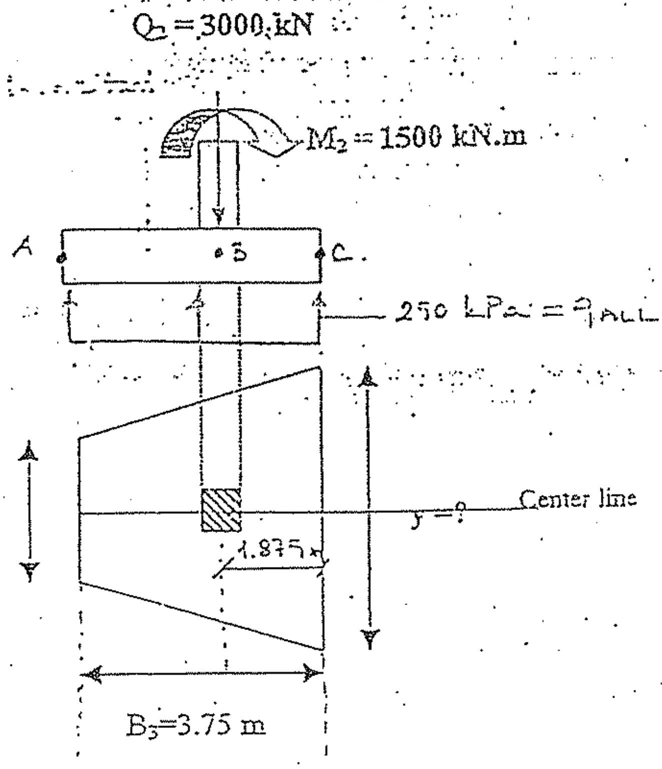
$R_2 = 250\times(B_4\times 3.75)$

$1962.5 = 250\times(B_2\times 2)$ $\quad B_2 = 3.925$ m

$3037.5 = 250\times(B_4\times 3.75)$ $\quad B_4 = 3.24$ m

b. (12%) If footing II is planned to be designed as an individual footing without a strap, to achieve the same uniform stress distribution beneath the footing, it needs to be designed as a trapezoidal footing as shown in the figure given below. Estimate the dimensions x and y of this trapezoidal footing.





Ftg. II

$$\left(\frac{x+y}{2}\right)B = \frac{x\cdot B}{2}\cdot\frac{2B}{3} + \frac{y\cdot B}{2}\cdot\frac{B}{3}$$

$$\left(\frac{x+y}{2}\right)B = \frac{B^2}{6}\left(2x+y\right)$$

$$\bar{x} = \frac{B}{3}\cdot\left(\frac{2x+y}{x+y}\right)$$

$$\sum M_C = 0 \Rightarrow \quad 3000\times 1.875 - 1500 - 250\times\left[3.75\times\left(\frac{x+y}{2}\right)\right]\left[\frac{3.75}{3}\left(\frac{2x+y}{x+y}\right)\right] = 0$$

$$\sum F_{vertical} = 0 \quad 3000 = 250\times 3.75\left[\frac{x+y}{2}\right]$$

eqn ① $4125 = 585.9375\,(2x+y)$

eqn ② $6.4 = (x+y)$

$x = 0.64$, $y = 5.76$

m 1-2

Question 2. (15%)

A footing for an industrial complex having a width of 4 m is to be constructed at a site consisting of deep deposit of fine sand. Foundation depth is 3 m and the gross foundation pressure is 180 kN/m^2. Results of a Cone Penetration Test performed at this site are given below. Calculate the settlement of the footing 10 years after the construction, using Schmertman's method. (No ground water is observed at the site.)

$S = C_1 C_2 q_{net} \sum \frac{I_z}{E_s} \Delta z$

$q_{net} = q_{gross} - \sigma'$

$\sigma' =$

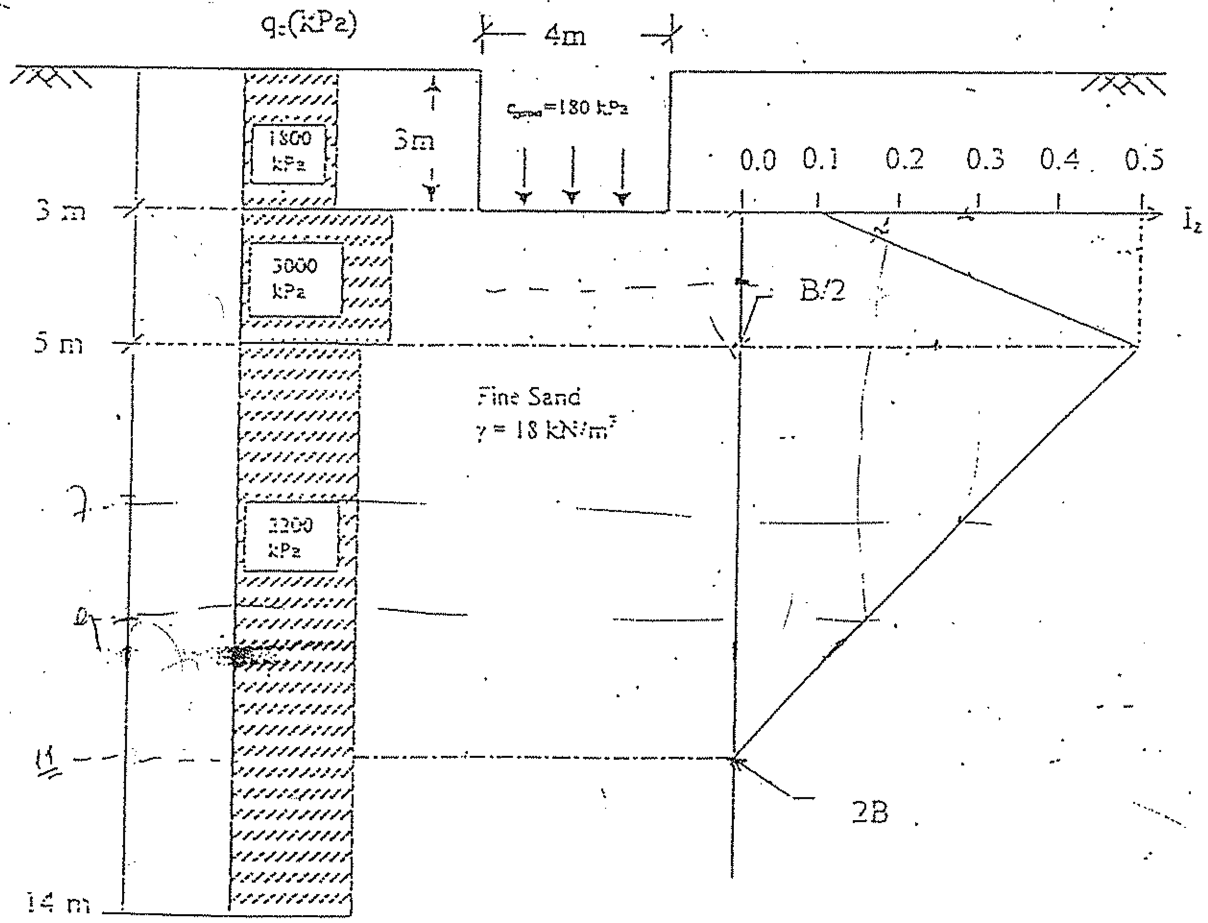
Note: Variation of strain influence factor is as follows:

$I_z = 0.1$ at $z = 0$
$I_z = 0.5$ at $z = B/2$
$I_z = 0.$ at $z = 2B$

$$S = C_1 C_2 q_{net} \sum \frac{I_z}{E_s} \Delta z$$

where,

$$E_s = 2q_c\,,\; C_1 = 1 - 0.5\frac{\sigma_0'}{q_{net}},\quad C_2 = 1 + 0.2\log\left(\frac{t}{0.1}\right)$$



Layer #	Depth	Δz(m)	$E_s = 2q_c$(kPa)	I_z	$\frac{I_z}{E_s} \cdot \Delta z$
1	0–2.0	2.0	6000	0.3	1.10^{-4}
2	2.0–8.0	6.0	4400	0.25	3.41×10^{-4}
	?			?	$\Sigma = 4.41 \times 10^{-4}$

$$\sigma_z' = 18 \times 3 = 54 \text{ kPa}$$

$$q_{NET} = q_{gross} - \underbrace{\gamma D}_{\sigma_z'} = 180 - 54 = 126 \text{ kN/m}^2$$

$$C_1 = 1 - 0.5 \frac{\sigma_z'}{q_{NET}} = 1 - 0.5 \times \frac{54}{126} = 0.786$$

$$C_2 = 1 + 0.2 \log\left(\frac{t}{0.1}\right) = 1 + 0.2 \log\left(\frac{10}{0.1}\right) = 1.4$$

$$S = C_1 \, C_2 \, q_{NET} \sum \frac{I_z}{E_s} \Delta z$$

$$= (0.786)(1.4)(126)(4.41 \times 10^{-4})$$

$$= 0.0611 \text{ m} = 61 \text{ mm}$$

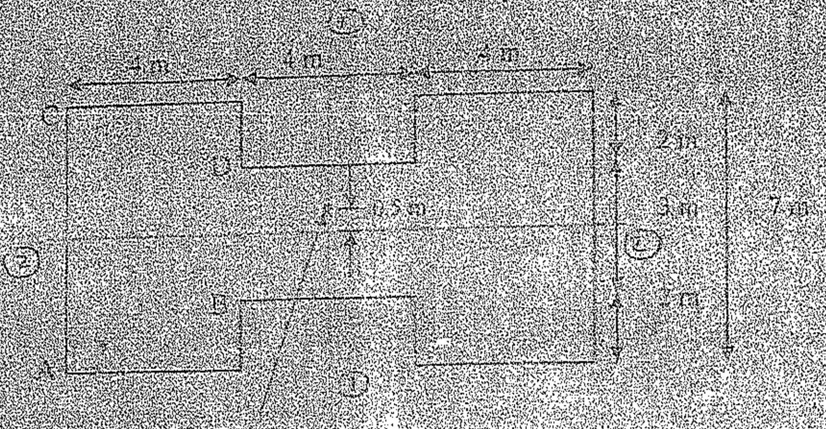
2003-2004 Spring

CE 366 MT-2

– Solutions –

Question 1 (15%)

Neglecting the weight of the mat, calculate the base pressures at corners A, B, C and D. Total vertical load applied over the foundation is 14000 kN.



Point of application of vertical load

SOLUTION:

– Center of gravity is the same as figure (CG)

$\rightarrow e_x = 0$ and $e_y = 0.5$

$\rightarrow \Sigma M = \Sigma P \cdot e = 14000 \times 0.5 = 7000$ kNm

$\rightarrow A_{mat} = 7 \times 12 - 2 \times 4 \times 2 = 68\ m^2$

$\rightarrow I_{x-x} = \frac{1}{12}(12 \cdot 7^3) - 2 \cdot \left(\frac{1}{12}(4 \cdot 2^3) + 4 \times 2 \times \ldots\right)$

$= 237.67\ m^4$

$q = \frac{\Sigma Q}{A} \mp \frac{M \cdot y}{I_{x-x}} = \frac{14000}{68} \mp \frac{7000}{237.67} y$

$q = 205.88 \mp 29.45 y$

$\Rightarrow q_A = 205.88 - 29.45(3.5) = 102.8$ kPa

$q_B = 205.88 - 29.45(1.5) = 161.7$ kPa

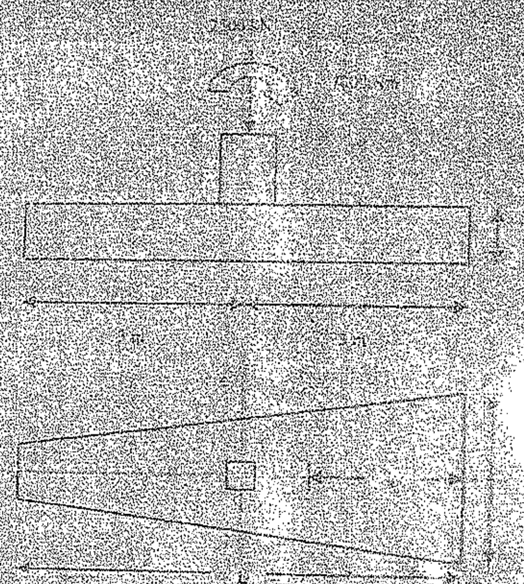
$q_C = 205.88 + 29.45(3.5) = 308.96$ kPa

$q_D = 205.88 + 29.45(1.5) = 250.1$ kPa

Question 2: (12%)

A uniform pressure distribution of 200 kN/m² is desired beneath the foundation shown in figure below. Due to the [illegible] the foundation is to be designed as a trapezoidal [illegible]. Consider [illegible] 2500 kN, [illegible] determine B_1 and B_2. [illegible] 24 kN/m³.

Geometric center: $x = \frac{L}{3}\left(\frac{2B_1 + B_2}{B_1 + B_2}\right)$



Area, $A = L\left(\frac{B_1 + B_2}{2}\right)$ weight of conc, $W = A \times 1 \times \gamma_{conc}$

$\Sigma Q = 2500 + A \times 1 \times \gamma_{conc}$

$A = \frac{\Sigma Q}{q} = \frac{2500 + A \times \gamma_{conc}}{200} \Rightarrow 200A = 2500 + A \times 24$

$\Rightarrow \boxed{A = 14.2\ m^2}$

$\Rightarrow L\left(\frac{B_1 + B_2}{2}\right) = 14.2 \Rightarrow L\left(\frac{B_1 + B_2}{2}\right) = 14.2 \Rightarrow B_1 + [illegible]$

From moment equilibrium, $\Sigma M = 0$ (+)

$2500(3 - x) - 700 + (W_F) \times 0 - (q_{soil}) \times 0 = 0 \Rightarrow$ [illegible]

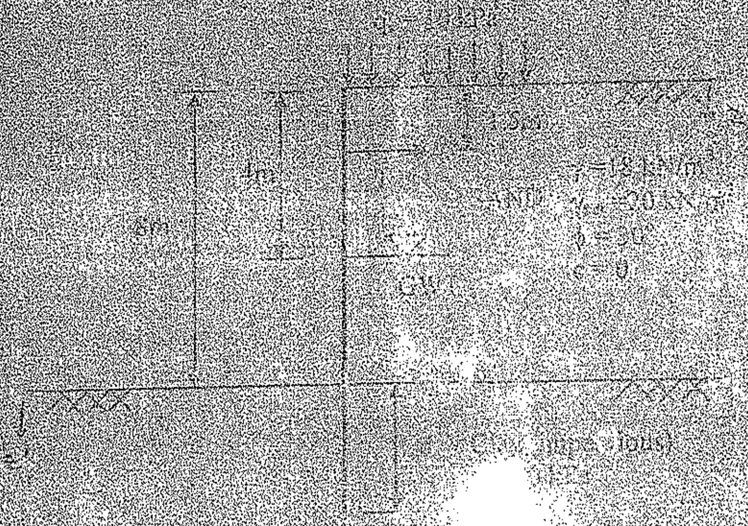
From given geometric center formula

$\Rightarrow x = \frac{L}{3}\left(\frac{B_1 + 2B_2}{B_1 + B_2}\right) = 2.72\ m \Rightarrow \frac{L}{3}\left(\frac{B_1 + 2B_2}{14.2}\right)$ [illegible]

$\boxed{\begin{matrix} B_2 = 3.0\ m \\ B_1 = 1.70\ m \end{matrix}}$

Question 2 (15%)

[illegible]

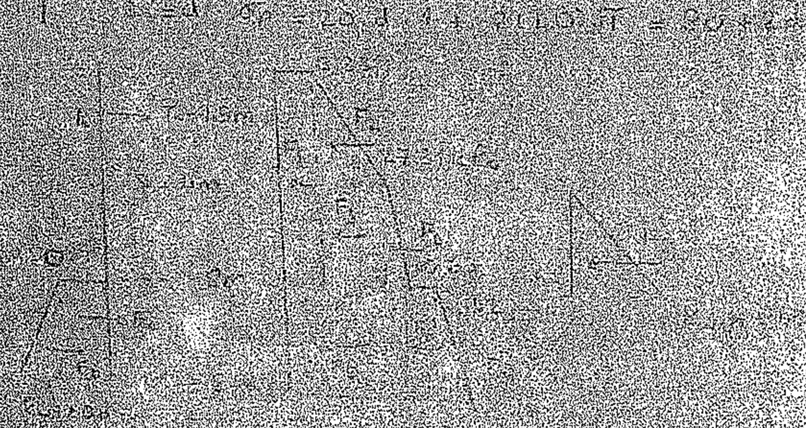


Active pressure: $P_a = \gamma Z K_a - 2c\sqrt{K_a}$ $K_a = \frac{1-\sin\phi}{1+\sin\phi}$ $K_p = \frac{1}{K_a}$

$K_a = \tan^2(45 - \phi/2) = 0.333$ $K_p = \frac{1}{K_a} = 3$

Active pressure
@ $z = 0$ $\sigma_a = q_s K_a = 10 \times (0.333) = 3.33$ kPa
@ $z = 4$m $\sigma_a = (18(4) + 10)(0.333) = 27.306$ kPa
@ $z = 8$m $\sigma_a = (18(4) + (20-10)4 + 10)\ 0.333 = 40.626$ kPa
$z = 8$ $\sigma_a = (18(4) + (20-10)4 + 10) \ldots$ [illegible]
@ $z = 8+d$ m $\sigma_a = (10 + 18(4) + (20-10)4 + 20(8)) \ldots$ [illegible]

Passive pressure
@ $z = 0$ $\sigma_p = 2(40)\sqrt{3} = 138$ kPa
$z = d$ $\sigma_p = 20d \ldots 2(40)\sqrt{3} = 20 \ldots$ [illegible]



$F_1 = [illegible] = 13.32$

$F_2 = (27.3 - [illegible]) \frac{[illegible]}{2} = 17.95$

$F_3 = [illegible] = 10.422$

$F_4 = (19.23 - [illegible]) \frac{1}{2} = 24.4$

$F_5 = [illegible] \times \frac{1}{2} = 80$

$F_6 = 88.2$

$\sum F = 277.136 - [illegible] - T = 0$

$T = 277.136 - 88(13.52)$

$\boxed{T = 143.4 \text{ kN/m}}$

Question 4 (10%)

A reinforced earth wall is 6.0 m high. The [illegible], which are spaced at [illegible] vertically and 1.2 m horizontally, measure 85 [illegible] section and are [illegible]. Their ultimate tensile strength is 240 N/mm². The unit weight of the selected fill is 20 kN/m³. Determine the factors of safety against bond failure (pull-out) and tensile failure for an element 3.0 m below the top of the wall if the lateral pressure at that level and the effective length L_e of the reinforcement at that particular level are 0.25 and 3.8 m, respectively. The friction angle between the fill and the reinforcements is 25°. Is the design adequate?

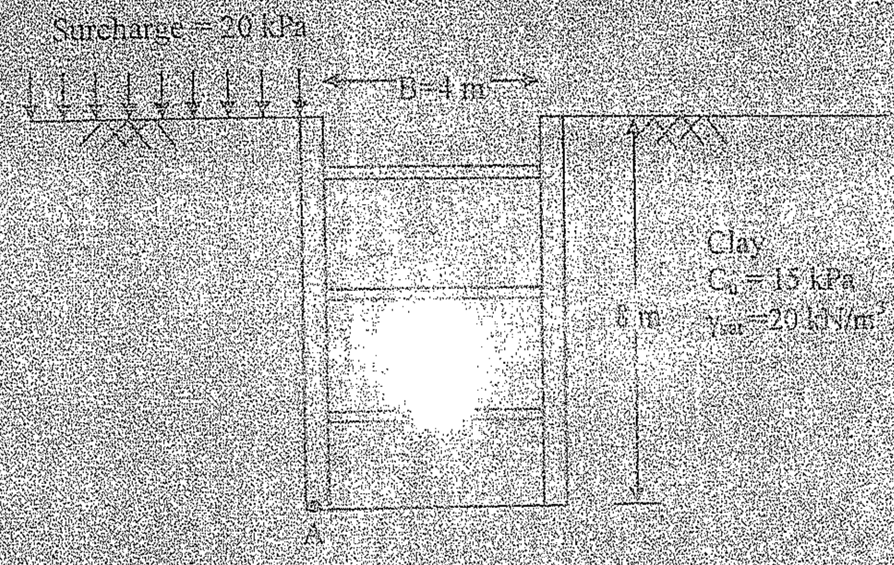
(1) $(FS)_{pull\text{-}out} = \dfrac{2\, b\, L_e \tan\delta}{[illegible]} = \dfrac{2 \times 0.085 \times [illegible]}{0.25 \times 0.85 \times 1.2}$

(2) $(FS)_{tension} = \dfrac{T_{[illegible]}}{T_{max}} = \dfrac{[illegible]}{[illegible]}$

$= 8.5$

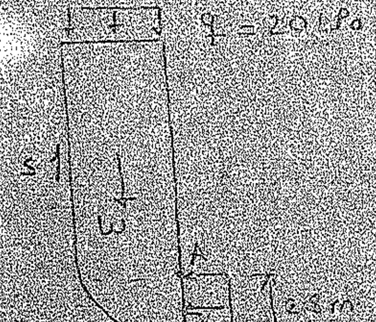
Question 5. (10%)

Calculate the factor of safety against base heave for the braced cut shown in the figure below. Assume $B_1 = 0.7B$ and do not consider tension crack.



Note: $$FS = \frac{\Sigma \text{ Moment of resisting forces with respect to point A}}{\Sigma \text{ Moments of driving forces with respect to point A}}$$

Present your results in a table format.



Assume $B_1 = 0.7B$

$$FS = \frac{8\cdot(15)(2.8) + \frac{2.8(2.8)\ldots(15)\ldots}{4}}{\underbrace{(20)(2.8)\left(\frac{2.8}{2}\right)}_{\text{due to surcharge}} + (20)(8)\ldots}$$

$\Rightarrow$ $\underline{FS = 0.905 < 1}$

Question6. (15%)

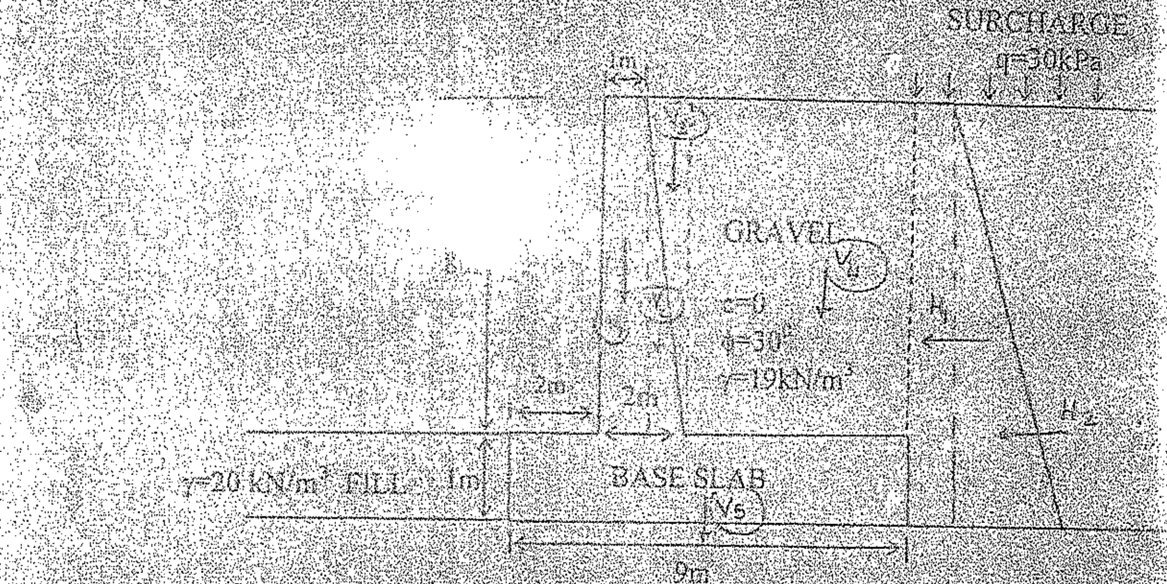
The figure given below shows the cross-section of a retaining wall. Estimate:

a. (5%) The safety factor for overturning.

b. (5%) The safety factor for sliding.

c. (5%) Determine the base pressure distribution.

Hint: Do not consider the passive resistance of the fill in front of the wall.



MEDIUM DENSE SILTY SAND

c=0, φ=30°, γ=20kN/m³, f=0.5(base friction), γ_{conc}=24kN/m³

$$P_A=(\gamma z+q)K_A-2c\sqrt{K_A}\ ;\ P_P=(\gamma z+q)K_P-2c\sqrt{K_P}\ ;\ K_A=\frac{1}{K_P}=\frac{1-\sin\phi}{1+\sin\phi}$$

Force (kN/m)	Arm about toe (m)	Moment (kNm/m)
$V_1 = 1\times8\times24=192$	2.5	480
$V_2=\frac{1}{2}\times1\times8\times24=96$	3.33	320
$V_3=\frac{1}{2}\times1\times8\times19=76$	3.67	278.9
$V_4=5\times8\times19=760$	6.5	4940
$V_5=9\times1\times24=216$	4.5	972
$H_1=10\times9=90$	4.5	405
$H_2=57\times9\times\frac{1}{2}=256.5$	3	769.5

$\Sigma V = 1$[illegible]

$\Sigma M_V =$ [illegible]

$\Sigma H =$ [illegible]

$\Sigma M_H =$ [illegible]

7

a) $(FS)_{overturning} = \dfrac{6990.9}{1174.5} = 5.95$

b) $(FS)_{sliding} = \dfrac{1340 \times 0.5}{346.5} = 1.93$

c) Base pressure

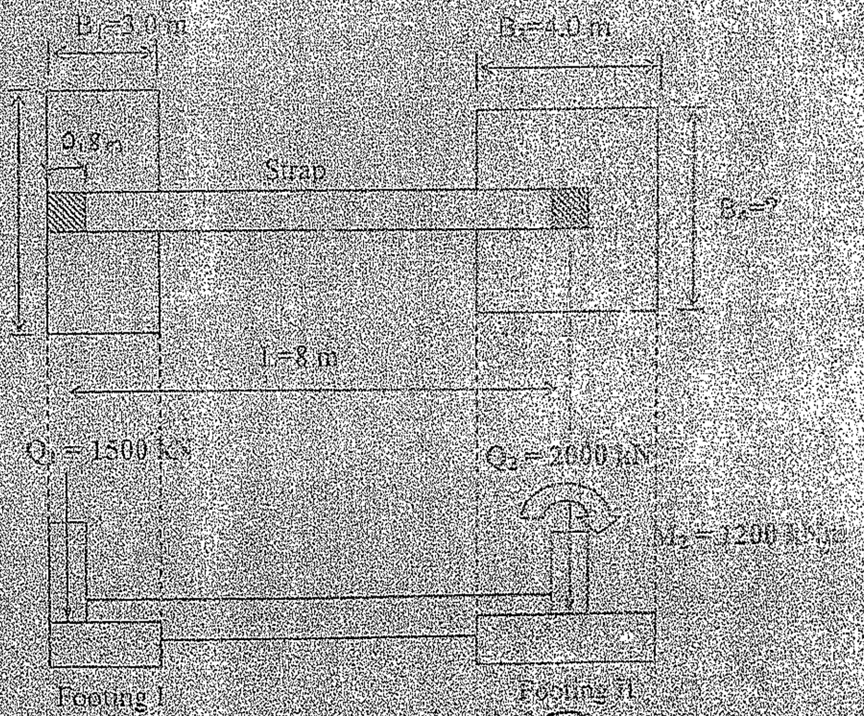
$\Sigma M_{net} = 6990.9 - 1174.5 = 5817$ kN m/m

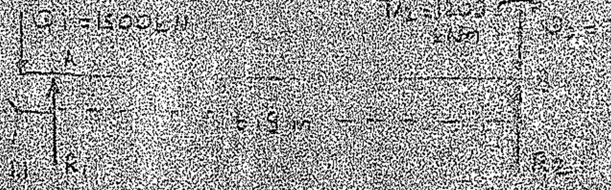
$\dfrac{5817}{1340} = 4.34 \implies e = 4.50 - 4.34 = 0.16$ m

$q_{\substack{max \\ min}} = \dfrac{\Sigma V}{B}\left(1 \pm \dfrac{6e}{B}\right) \rightarrow$ $q_{max} = 164.77$ kPa, $q_{min} = 133$ kPa

<u>Question7. (13%)</u>

The figure given below shows the f[illegible]dation plan view and cross-section of a residential building. To achieve uniform [illegible] distribution beneath the footings two footings were combined by a strap. The net allow[illegible] bearing capacity for the foundation soils is estimat[illegible] as 250 kN/m². Estimate the [illegible] footing dimensions B_2 and B_1 (Take width of the columns as 0.8 m).





$\Sigma M_A = 0 \quad (+\curvearrowright)$

$$1500 \times 1.1 + R_2 \times 6.9 - 2000 \times 6.9 - 1200 = 0$$

$$R_2 = 1934.78 \text{ kN}$$

$$R_2 = q_{all} \times B_3 \times B_4 \Rightarrow B_4 = \frac{1934.78}{250 \times 4.0} = 1.94 \text{ m}$$

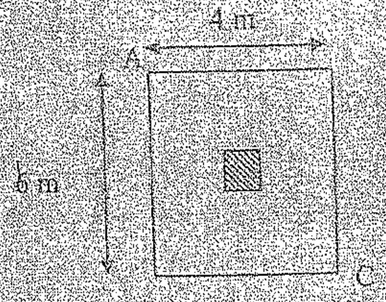
$\Sigma M_B = 0 \quad (+\curvearrowright)$

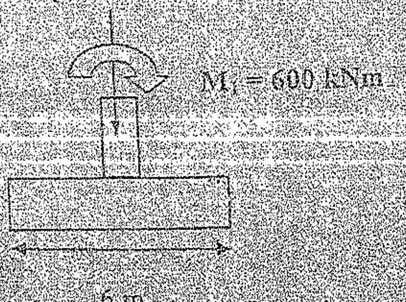
$$1500 \times 8 - R_1 \times 6.9 - 1200 = 0 \Rightarrow R_1 = 1565.22 \text{ kN}$$

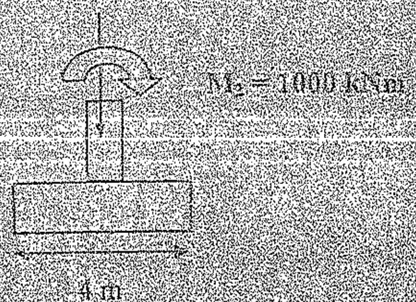
$$R_1 = q_{all} \times B_1 \times B_2 \Rightarrow B_2 = \frac{1565.22}{250 \times 3} = 2.09 \text{ m}$$

Question8. (12%)

Estimate vertical stresses beneath corners A and C of the below given individual footing under given loading conditions.







$$e_1 = \frac{600}{3000} = 0.2$$

$$e_2 = \frac{1000}{3000} = 0.33$$

$$q_C = \frac{3000}{6 \times 4}\left(1 - \frac{6 \times 0.2}{6} + \frac{6 \times 0.33}{4}\right) = 1$$

$$q_A = \frac{3000}{6 \times 4}\left(1 + \frac{6 \times 0.2}{6} - \frac{6 \times 0.33}{4}\right) =$$

366
m2-1

Middle East Technical University
Civil Engineering Department
Geotechnical Engineering Division

CE 366-FE 1
(www.ce.metu.edu.tr/~ce366)
Spring' 2002
Midterm 2

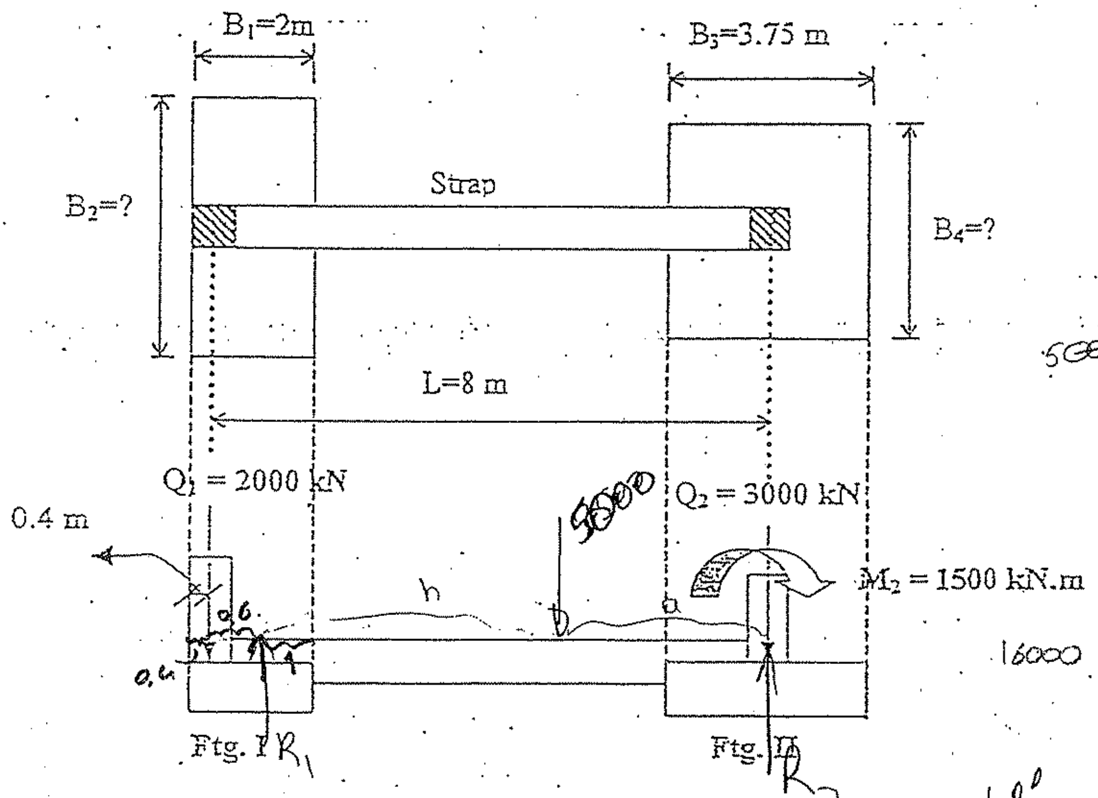
Examination Date: 26.04.2002, Friday
Examination Duration: 100 minutes

GOOD LUCK IN THE EXAM!!!

Question	Grade
1 (20%)	
2 (20%)	
3 (10%)	
4 (15%)	
5 (10%)	
6 (25%)	
Total (100%)	

Question 6. (25%)

The figure given below shows the foundation plan view and cross-section of a residential building. To achieve uniform pressure distribution beneath the footings two footings were combined by a strap. The net allowable bearing capacity for the foundation soils is estimated as 250 kN/m².



$5000 \cdot a = 2000 \times 8$

$a = 3,2$

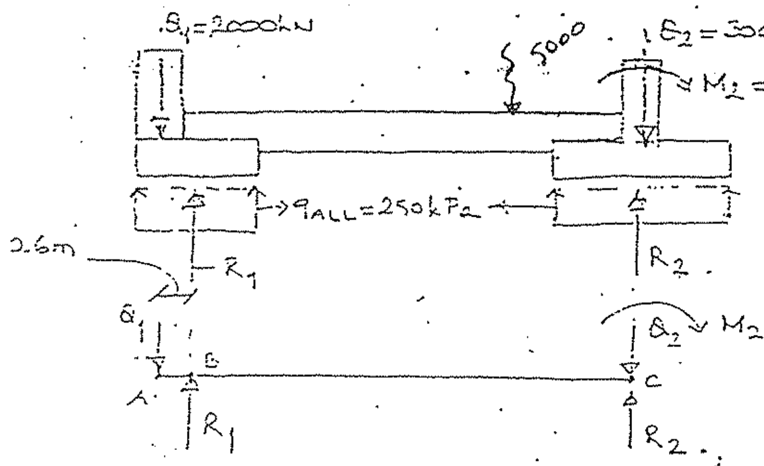
$b = 4,2$

$16000 = R_1 \times 7.4$

1600

1.9

a. (13%) Estimate the minimum footing dimensions B_2 and B_4.



$\overset{+}{\curvearrowleft} \sum M_B = 0$

$(Q_2 - R_2) \times 7.4 + M_2 - Q_1 \times 0.6 = 0$

$(3000 - R_2) \times 7.4 + 1500 - 2000 \times 0.6 = 0$

$\Rightarrow R_2 = 3037.5$ kN

$\sum F_{vertical} = 0$

$Q_1 + Q_2 = R_1 + R_2$

$2000 + 3000 = 3037.5 + R_1$

$\Rightarrow R_1 = 1962.5$ kN

$R_1 = 250 \times (B_2 \times 2)$

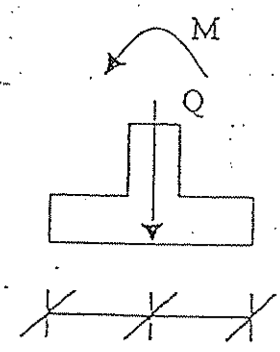
$R_2 = 250 \times (B_4 \times 3.75)$

$1962.5 = 250 \times (B_2 \times 2)$ $B_2 = 3.925$ m

$3037.5 = 250 \times (B_4 \times 3.75)$ $B_4 = 3.24$ m

Question5. (10%)

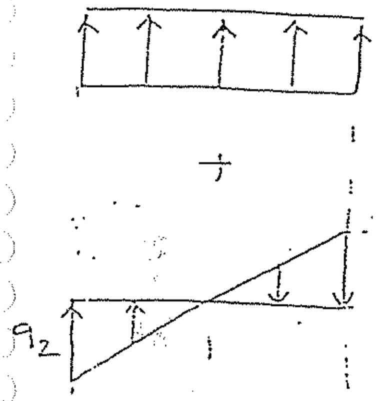
Derive an expression relating the soil pressures beneath a footing subjected to a moment M and a concentric column load of Q in terms of footing dimensions B, L and eccentricity e.



Hint: Derive $q = \frac{Q}{BL}\left(1 \mp \frac{6e}{B}\right)$

Note : Moment of inertia of rectangle

$$I = \frac{B^3 L}{12}$$



$q_1 = \frac{Q}{B \times L}$ due to Q.

$+$

$-q_2$ $\quad q_2 = \frac{M\,y}{I} = \frac{(Q.e)(B/2)}{\left(\frac{B^3 L}{12}\right)} = \frac{6.e\,Q}{B^2 L}$

$$q_1 + q_2 = q = \frac{Q}{BL} \mp \frac{6eQ}{B^2 L} = \frac{Q}{BL}\left(1 \mp \frac{6e}{B}\right)$$

CE-366 Final

CE-366 F-1
FINAL

Q.1. (1%) Open ended pipe pile is a

a. small displacement pile

b. large displacement pile

c. replacement pile

d. non-displacement pile

e. none

Q.2. (1%) It is observed in practice that group capacity of piles may sometimes be less than the number of piles times the capacity of a single pile. Which of the following statements is not true?

a. The reduction in single pile or group capacity due to the behavior explained above is expressed and quantified as a "efficiency factor".

b. The behavior explained above is not observed in granular soils.

c. Converse-Labarre formula may be used to calculate the reduction in pile capacity to be used in group design.

d. The soil type has no effect in Converse-Labarre formula

e. None.

Q.3. (1%) Which of the following statements is not correct?

a. A hydrostatic level behind the retaining walls is not considered in design, because a drainage system is planned as an integral part of the design

b. A base key is designed under a retaining wall to increase the sliding stability of the wall.

c. A buttressed wall is basically cantilever retaining wall and is higher than the cantilever wall. Buttresses are needed for the structural stability of the wall

d. Berlin(er) wall is a type of sheet-pile wall and preferred in watertight applications.

e. None.

Q.4. (1%) Which of the following statements is not correct

The Skempton-Bjerrum correction factor

a. is not applied in extensive load applications

b. is not applied if one-dimensional compression occurs in the soil layers

c. depends on the stress history of a cohesive soil

d. depends on the foundation shape (square, strip etc.)

e. none.

F2

Q.5. (1%) Values of relative rotation (angular distortion) observed in structures due to differential settlement which lead to structural damage are in the range of

a. 1/500 b. 1/150 c. 1/1000 d. 1/700 e. none

Q.6. (1%) If the structural plan of a building is not symmetrical and irregular the most suitable type of raft foundation is

a. plain slab b. slab and beam c. cellular d. pedestal type e. none

Q.7 (1%) Which of the following is not related to dewatering?

a. sump b. shallow well c. well-point d. trial pit e. none

Q.8. (1%) In the following field tests, ... is the only one during the conduct of which soil samples are obtained.

a. dynamic penetration test
b. standard penetration test
c. static cone penetration test
d. push-in vane test (without opening a borehole)
e. none

Q.9. (1%) In most soils especially below ground water level .... is required during boring operation because of the collapse of the borehole wall.

Q.10. (1%) Identification of soils using cone penetration test (CPT) can be accomplished by evaluating the end resis~~tance (q_c) and~~ ......f_s.............................. values

Q.11. (1%) The information obtained during drilling, sampling and field testing is described on a page showing the soil profile, water level, penetration test results etc. This is called a ....

F3

Q.12. (2%) The critical depth for the base heaving (failure) in an excavation in clay is expressed as $H_{cr} = N_c \times C_u / \gamma$ where N_c is the bearing capacity factor, C_u is the undrained shear strength, γ is the bulk unit weight. If a base failure occurs in an excavation in a clay at a depth of 8m, what will be the factor of safety at the depth of [illegible]m? (N_c=8, γ=18 kN/m^3, C_u= [illegible] kN/m^2)

a. 2.25 b. 2.0 c. 3.0 d. 1.50 e. none

Q.13. (2%) Find the correct answer.

Consider a deep silty sand soil profile (γ_{sat}=20 kN/m^3, ϕ'=32°), where the water table is at the ground surface. The end bearing capacity of a 35cmx35cm, 20 m long precast concrete pile is .... kN. Bearing capacity factor for end bearing of a driven pile in the silty sand is 35. Assume that the critical depth concept of D_{cr}=20B is valid.

a. 1715 b. 875 c. 305 d. 600 e. none

Q14 .(10%) A reinforced earth wall, 8m high and 6m wide is given in the figure below. All the strips are 6m long and spaced at 0.5m centers both in vertical and horizontal directions. Calculate the factor of safety of a strip at 3m depth against both pullout and tie break. Note that the lateral earth pressure coefficient at 3m depth can be taken as 0.4.

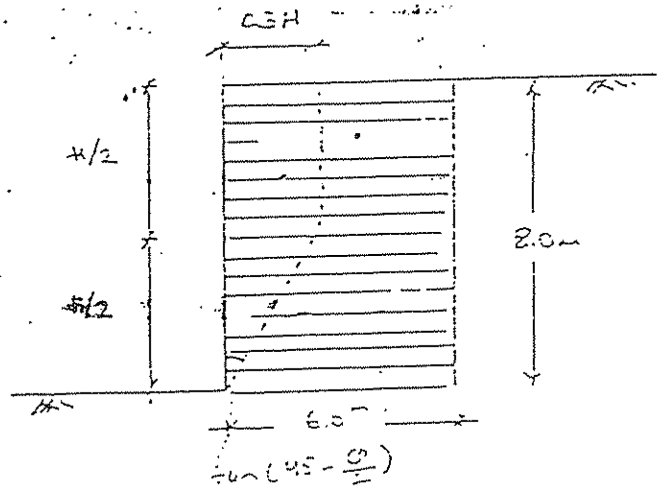
γ_{fill}=20 kN/m^3

Width of strips=50mm

Thickness of strips=5mm

Yield strength of steel=200 MPa

Steel-fill friction angle=20^0



F4

Q15 .(10%) A 3.0m wide strip foundation on a layered sand profile is given in the figure below. The cone penetration tip resistance (q_c) of the layers are as given in the figure. Calculate the settlement of the footing under a net foundation pressure of 160 kPa using Schmertmann's strain distribution method. Consider no creep effects (c_2=1.0).

Note: $E_s = 2.5q_c$

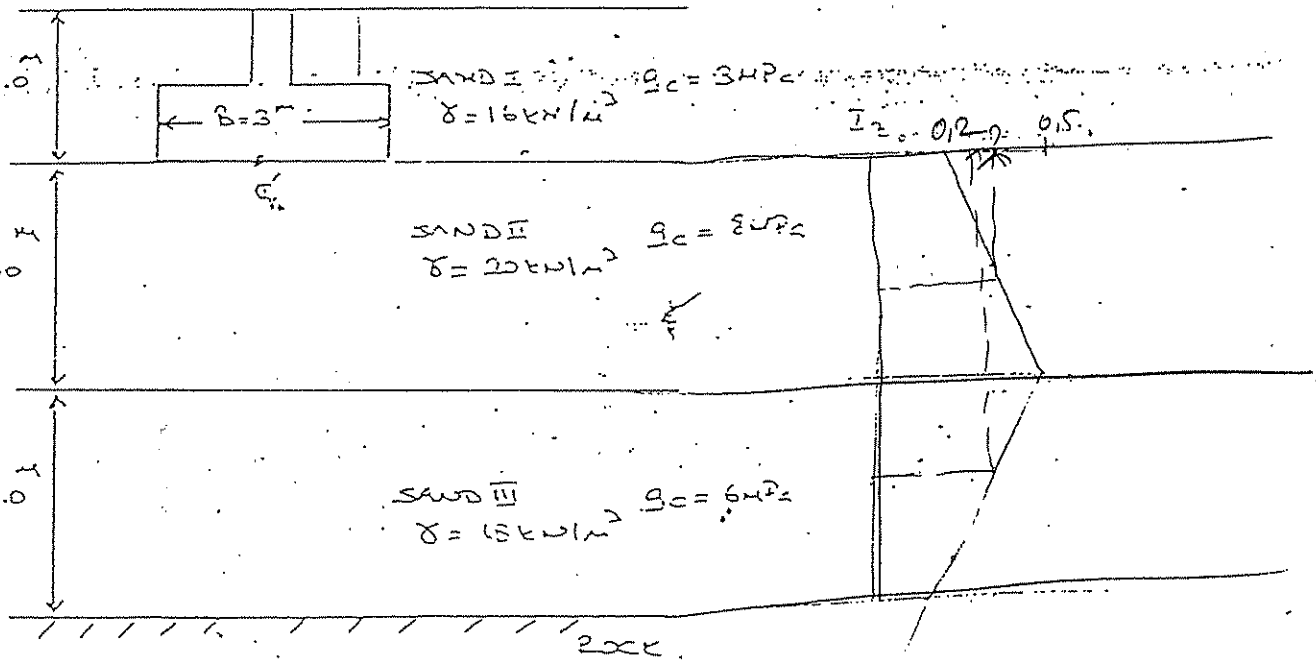
I_z=0.2 at z=0

Iz=0.5 at z=B

Iz=0.0 at z=4B

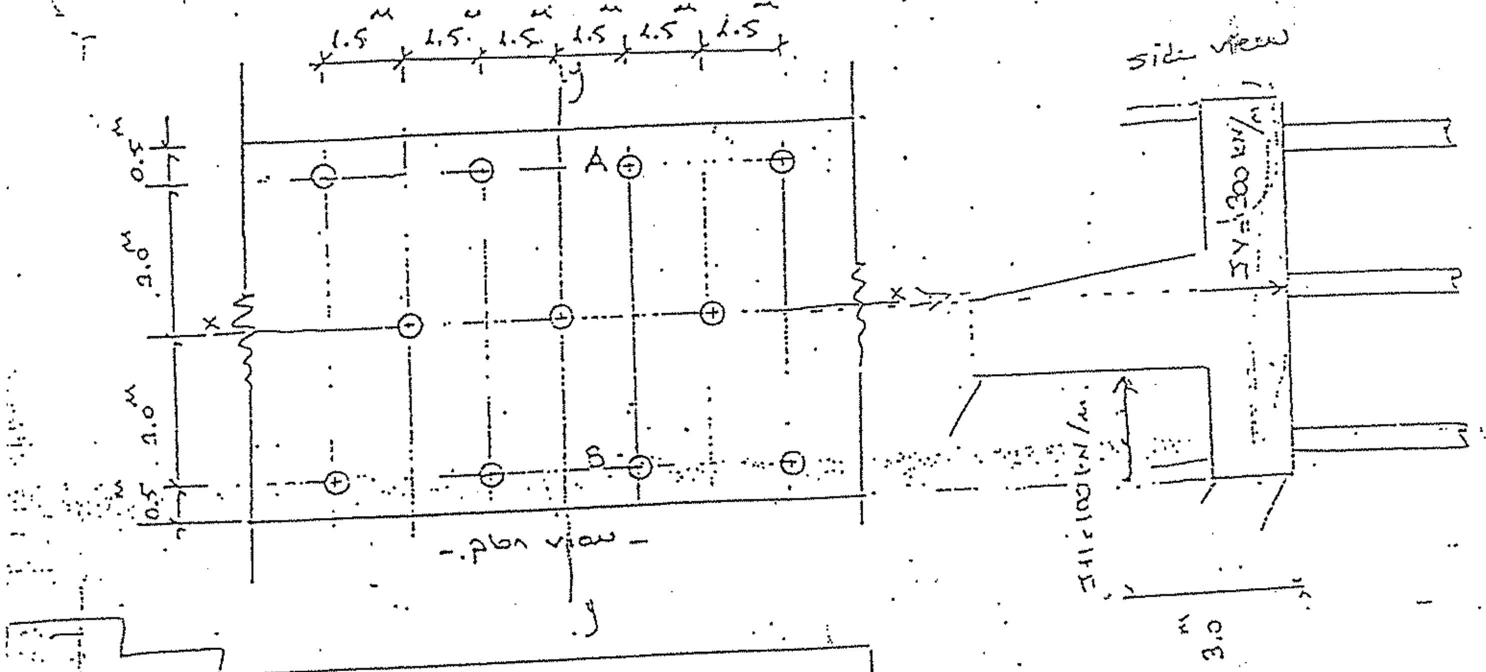
$S=c_1c_2q_n\Sigma(I_z/E_s)\Delta z$

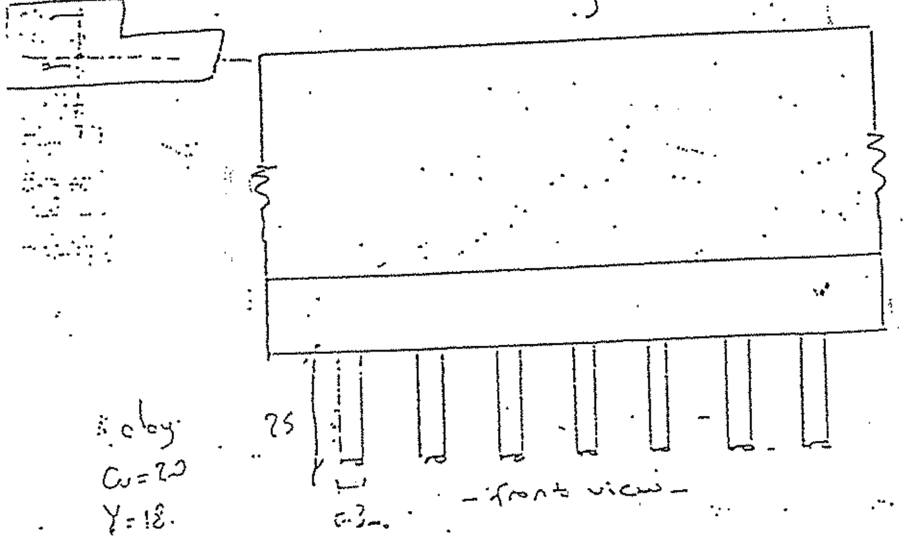
$C1=1-0.5(\sigma'_{vo}/q_n)$



F5

Q.16. (29%) The figure below shows a retaining wall of infinite length. The pile pattern of the foundation is as shown. The soil is soft clay (C_u=20 kPa, γ=18 kN/m^3) to infinite depth. The piles are circular, 25 m long with a diameter of 0.3m.





$\Sigma M = 300$ kN × 9

$q = \frac{\Sigma Q}{n} \pm \frac{M}{I} d_1 \pm \frac{M}{J} d_2$

$\Sigma Q = 300$ kN/m × 7

$q_A = \frac{2100}{11} \pm \frac{2700}{72}(3) = \cancel{2700}\ (3)$

$q_A =$

a) (7%) Calculate the loads on piles A and B.

b) (7%) What is the ultimate capacity of a single pile? What is the factor of safety of piles A and B against failure?

$J_{yy} = I = \left[2(4.5)^2 + 1(3)^2 + 2(1.5)^2\right] 2$ pile m^2

$I_{xx} = I = \left[4(3)^2\right] \times 2 = 72$ pile m^2

clay; $f_s = \alpha \cdot C_u$, $q_b = 9 \cdot C_u$

$\alpha = 1 - 0.0085(C_u - 25) \Rightarrow \alpha = 1$

$Q_s = 20(25)(\pi\, 0.3) =$

$Q_b = 9(20)\left(\frac{\pi\, 0.3^2}{4}\right) =$

Q_{ult} =

$F.S. = \frac{Q_{ult}}{q_{all}}$

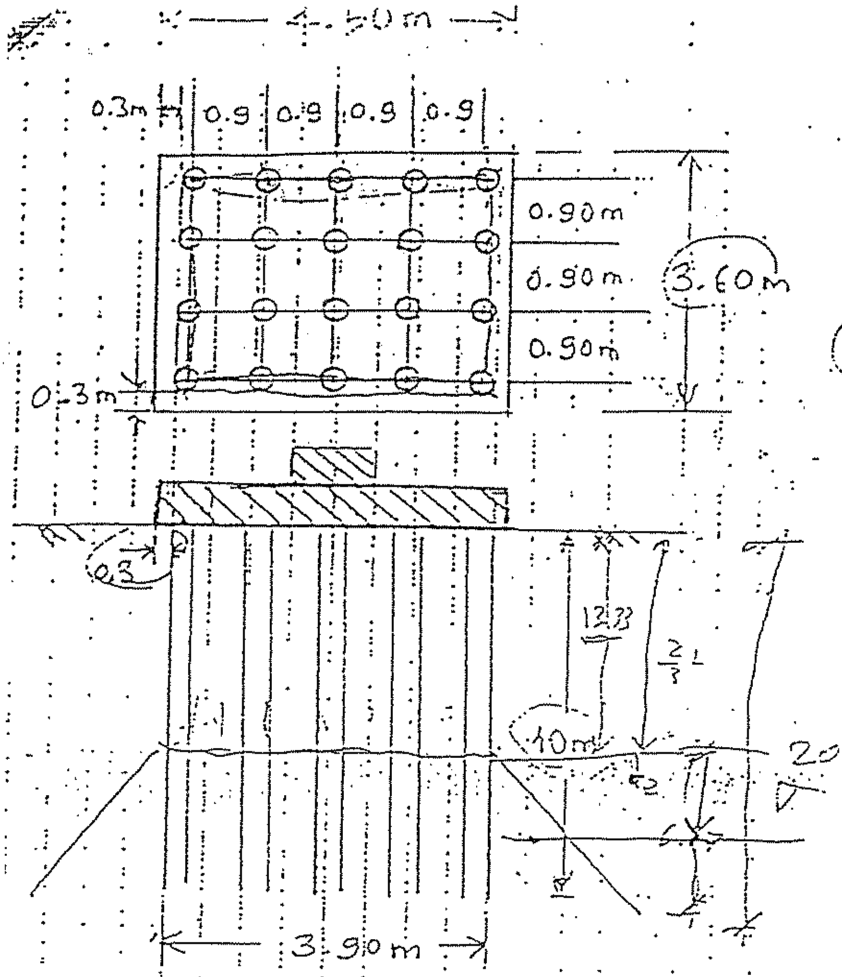
F 6

c) A 10m long 20 – pile group shown below (0.3m diameter) supports a heavy column in this soft clay.

i) (7%) Calculate the group capacity using the Terzaghi block analysis. Is there any need for group capacity reduction? If yes, what is the efficiency?

ii) (8%) Estimate the group settlement assuming that the compressible soil depth is 20m from the ground surface. Take the average compressibility of the soil 0.2×10^{-3} m^2/kN and make any assumption necessary.

F 7



$\frac{2L}{3}$

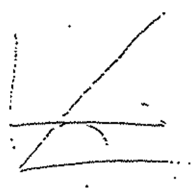
$Q_T = (3.6)(\cancel{4.5})(20)\cancel{N_c} + (3.6+4.5)\,2 \times 10 \times 20$

$Q_T = (4.5-0.6)(3)(20)\,N_c + (3+3.5)\times 2 \times 20 \times 10 = Q_T$

$Q_{single} = (\alpha . C_u)(\pi\, 0.3)(10) + 9(20)\left(\pi \frac{0.3^2}{4}\right) =$

$Q_{Ent} = \mu = 1 - \theta \frac{[(3)5 + (6)4]}{90 \cdot 4 \times 5} = 1$

$Q_{up} = \mu .$



$S = m_v \cdot \Delta\sigma' H \Rightarrow S = 0.2 \times 10^{-5} (6.67)(\Delta\sigma') = S$

$\Delta\sigma' = (13.33)(18) + (3.33)(18) =$

F 8.

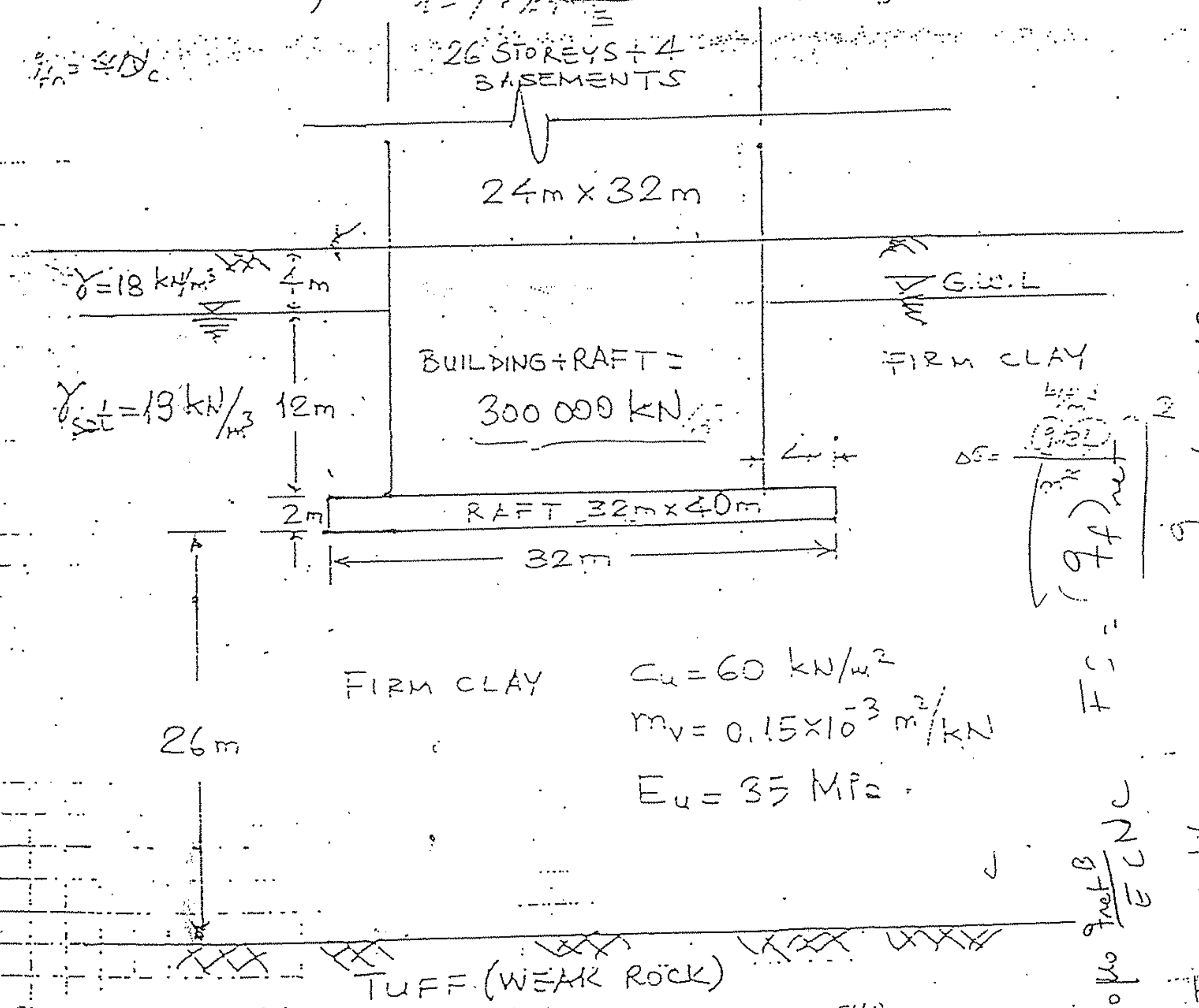
Q.17. (20%) A deep firm clay layer (44 m thick) overlies a tuff formation, which may be considered incompressible compared to clay. A tall building with four basements is to be constructed. The dimensions, building load and soil properties are given in the figure below.

a) (10%) Calculate the net foundation pressure to be used in the settlement analysis.

b) (5%) Assume that net foundation pressure is 100 kPa. What is the factor of safety of the foundation against bearing capacity failure? $q_{nf} = c_u \cdot N_c$

c) (5%) If the net foundation pressure is 100 kPa, calculate the total settlement of the building. (i.e. immediate + consolidation)

You may use 2 to 1 stress distribution. Assume that Skempton - Bjerrum correction coefficient is 0.6.



$32 \times 40 \times 2 \times 79 = 48640$

$\ldots \times 12 \times 19 = 275200$

$\ldots \times 4 \times 18 = 4608$

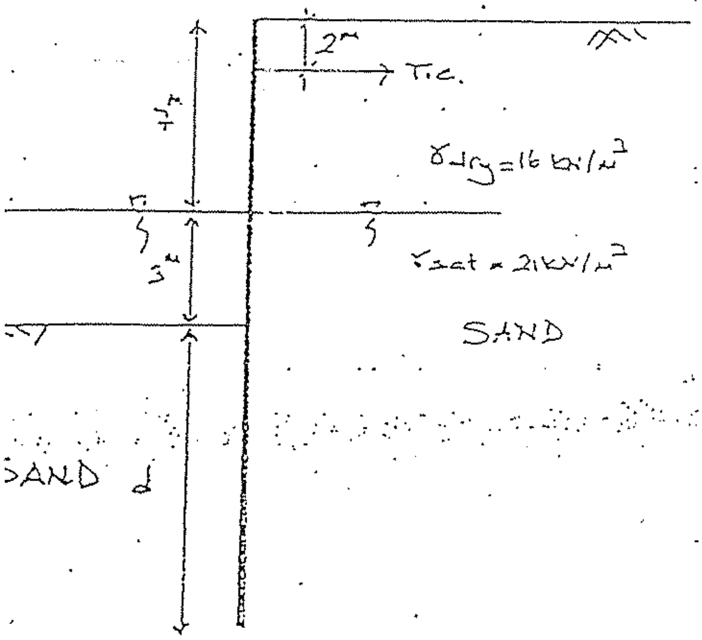
228352

$S_c = m_v \Delta\sigma H$

$= 55.575$ kPa

F9

Q.18 (10%) A quay wall is to be constructed by use of sheet piling, as depicted below. The shear strength parameters are $c'=0$, and $\phi'=38^0$. Calculate the required depth of penetration and the force in each tie if these are spaced at 2m centers. A factor of safety of 2 with respect to the gross passive resistance is needed.



F 10

Q.19. (6%) A building is to be constructed at a site whose soil profile is shown in the figure. The weight of the structure is about 250 kN/m² on a construction area of 20x40m.

What would you recommend at the preliminary stage of the soil and foundation study as far as the type of foundation is concerned:

a. Individual or combined footings above water level

b. A mat foundation above water level

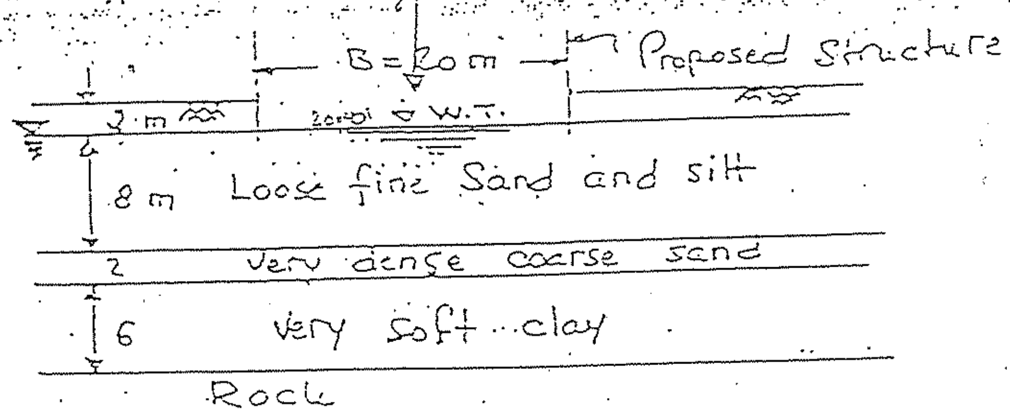
c. A mat foundation below water level (D_f=?)

d. Friction piles in fine sand and silt

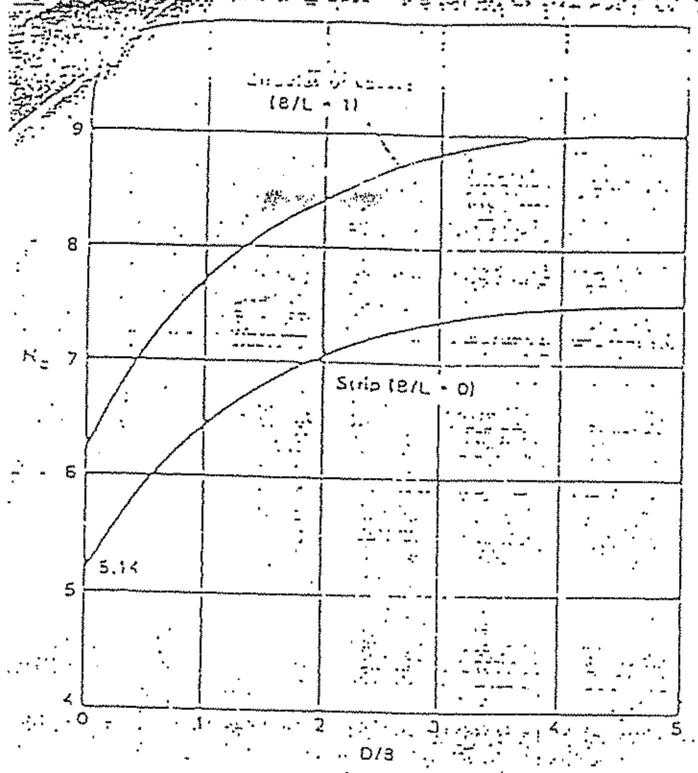
e. End bearing piles on dense sand

f. End bearing piles on rock

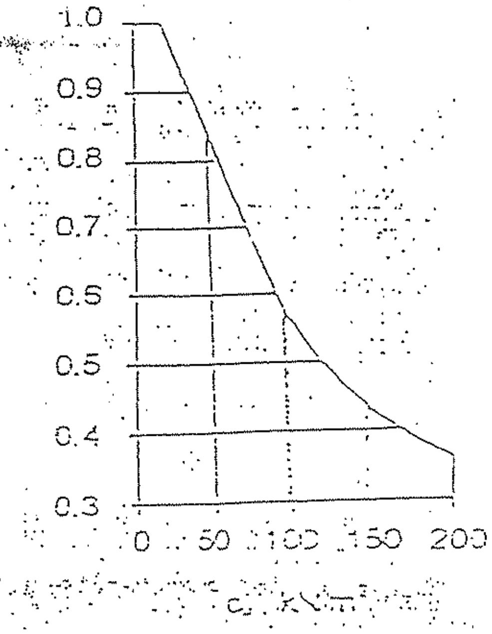
State your reason if you recommend one of the foundations listed above or propose a new alternative type of foundation: $q = 250\ kN/m^2$



F 11



Skempton's values of N_c for $\phi_u = 0$



$$N_{c_{rect}} = N_{c_{square}} * \left[0.84 + 0.16 \frac{B}{L} \right]$$

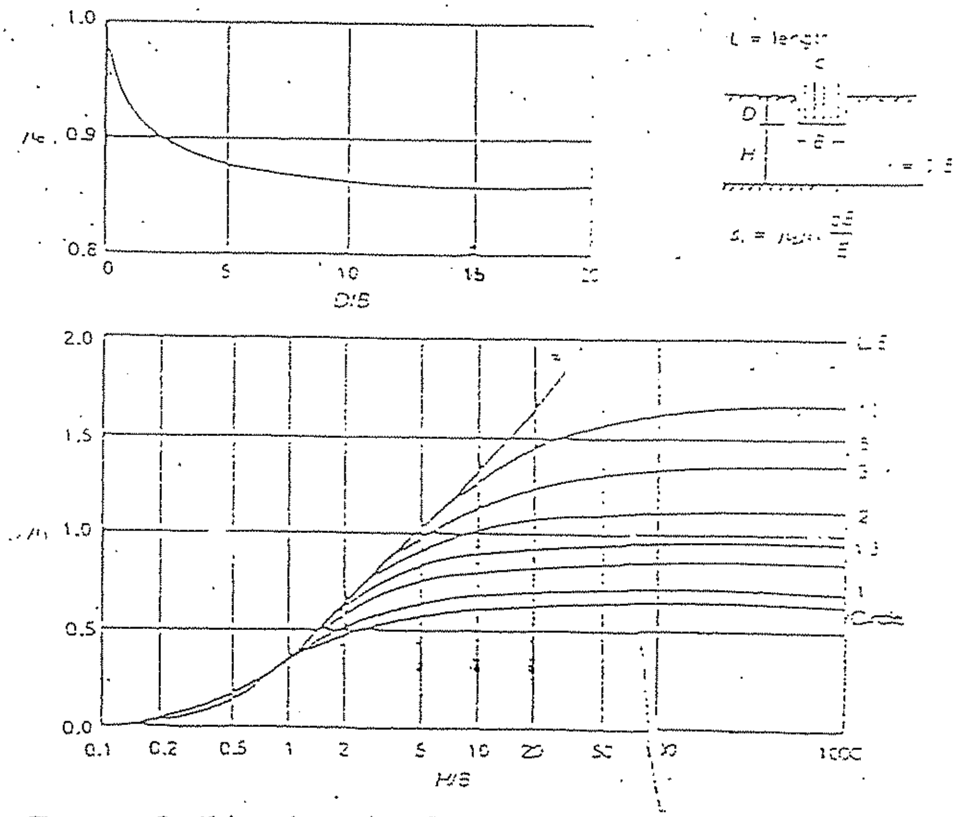
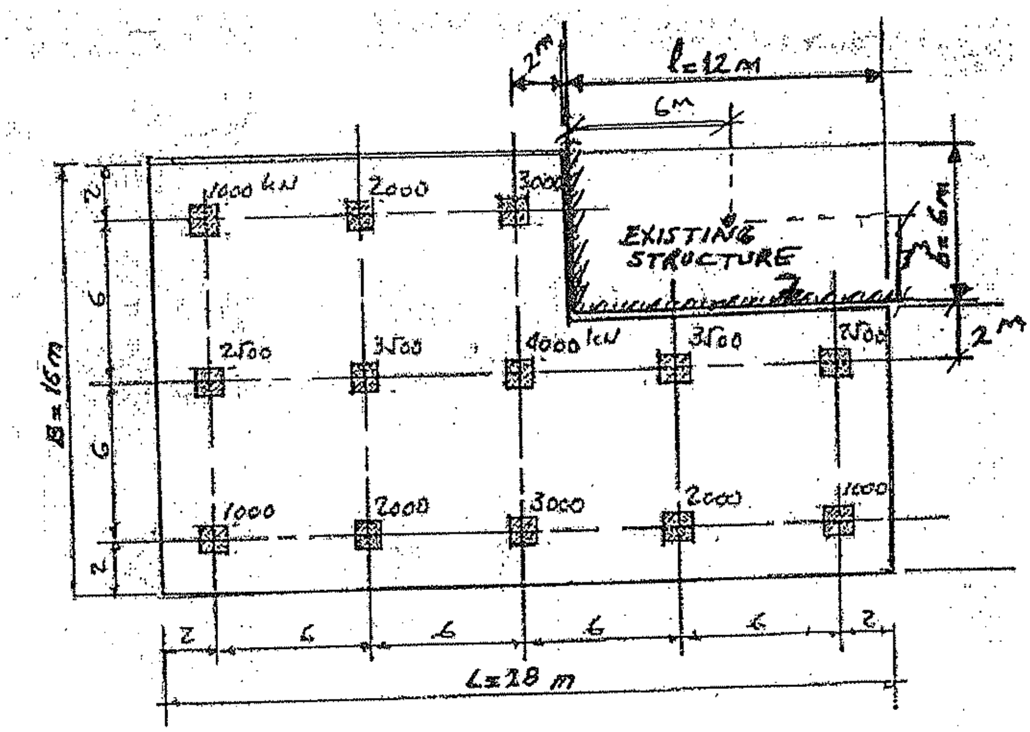


Fig. Coefficients for vertical displacement

MAT FOUNDATION

A mat foundation rests on a sand deposit whose allowable bearing value is 150 kN/m^2. Column loads are given in the figure. The thickness of the mat is 2.0 m ($\gamma_{concrete}$ = 24 kN/m^3). Calculate base pressures assuming that the lines passing through the centroid of the base and parallel to the sides are principal axes. Is the mat foundation given safe?



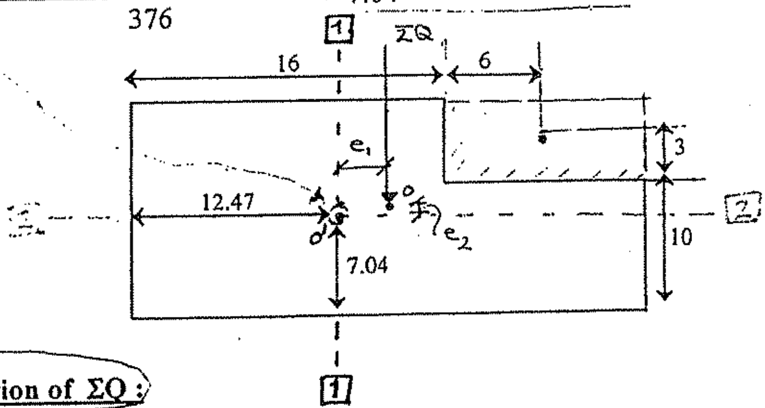
Total vertical load = ΣV = [Column loads + (376) × 24 × 2] = 49048 kN

Area of foundation = 28 × 16 − 12 × 6 = 376 m²

*** Center of gravity of base :** (loaded area)

$$\frac{28\times16\times14-6\times12\times(16+6)}{376}=12.47 \text{ m from left}$$

$$\frac{28\times16\times8-6\times12\times(3+10)}{376}=7.04 \text{ m from bottom}$$



enter of gravity of base Area

*** Location of ΣQ :**

Take moment of Loads about the left side :

= (1 / 49048) . [2 × (1000+2500+1000) + 8 × (2000+3500+2000) + 14 × (3000 + + 4000 + 3000) + 20 × (3500+2000) + 26 × (2500+1000) + 28 × 16 × 2 × 24 × 14 − 12 × 6 × 2 × 24 × 22]

(28/2) (16+6)

= 12.95 m from left

Take moment of Loads about bottom side :

= (1 / 49048) . [2 × (1000+2000+3000+2000+1000) + 8 × (2500 + 3500 + 4000 + + 3500 + 2500) + 14 × (1000+2000+3000) + 28 × 16 × 2 × 24 × 8 − 12 × 6 × 2 × 24 × 13]

(6+2) (3+2+6+2)

= 7.28 m from bottom

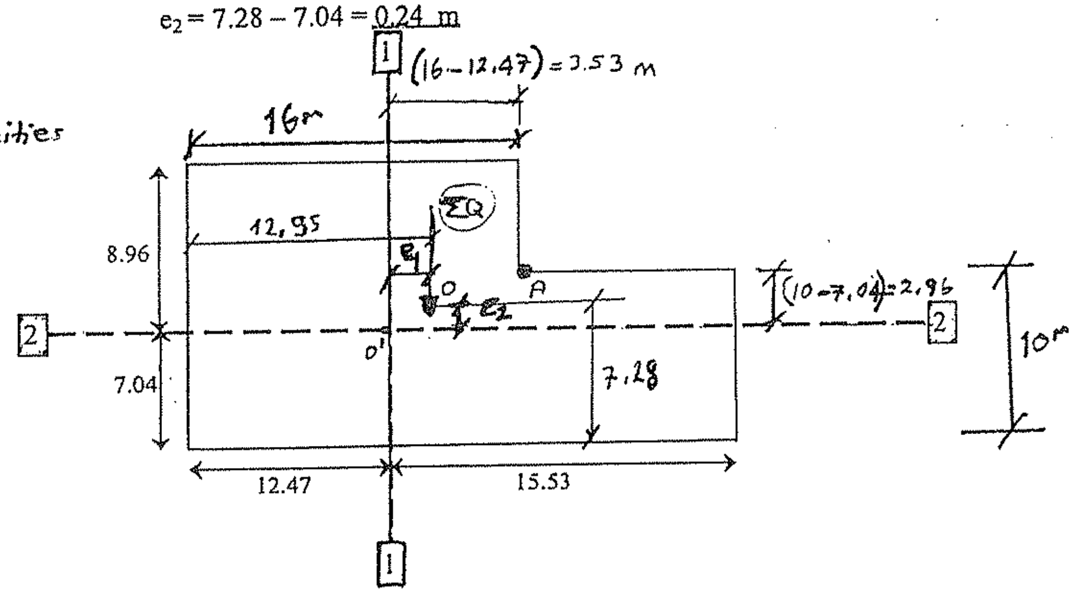
ocation of ΣQ

Eccentricities e_1, e_2

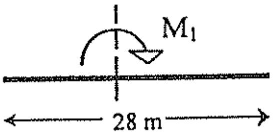
Eccentricity :

$e_1 = 12.95 - 12.47 = \underline{0.48\ m}$

$e_2 = 7.28 - 7.04 = \underline{0.24\ m}$

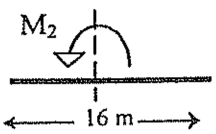


Moments



$\underline{M_1 \text{ around 1-1 axis :}}$

$M_1 = \Sigma Q \cdot e_1 = 49048 \cdot (0.48) = \underline{23543\ kN.m}$



$\underline{M_2 \text{ around 2-2 axis :}}$

$M_2 = \Sigma Q \cdot e_2 = 49048 \cdot (0.24) = \underline{11772\ kN.m}$

Area Moment of Inertia

$$I_{1-1} = \left[\frac{B \cdot L^3}{12} + B \cdot L \cdot (D_1)^2\right] - \left[\frac{b \cdot l^3}{12} + b \cdot l \cdot (d_1)^2\right] =$$

before cut / After cut

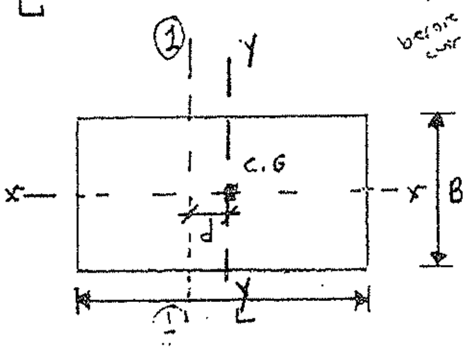
$$I_{1-1} = \left[\frac{16\times 28^3}{12} + 16\times 28\times(14 - 12.47)^2\right] - \left[\frac{6\times 12^3}{12} + 6\times 12\times(22 - 12.47)^2\right] = 22915 \quad m^4$$

before cut / After cut

$$I_{2-2} = \left[\frac{L \cdot B^3}{12} + B \cdot L \cdot (D_2)^2\right] - \left[\frac{l \cdot b^3}{12} + b \cdot l \cdot (d_2)^2\right] =$$

$$I_{2-2} = \left[\frac{28\times 16^3}{12} + 16\times 8\times(8 - 7.04)^2\right] - \left[\frac{12\times 6^3}{12} + 12\times 6\times(13 - 7.04)^2\right] = 7197 \quad m^4$$

before cut / After cut



$$I_{Y-Y} = \frac{B.L^3}{12}, \quad I_{①①} = \frac{B.L^3}{12} + A \cdot d^2$$

ase
ressures

Note: In soil mechanics compression is taken as positive (+)

$\overline{BN}$ = Distance to 1-1 axis
(x-distance to o')

$\overline{FK}$ = Distance to 2-2 axis
(y-distance to o')

$$q = \frac{\Sigma Q}{Area} \pm \frac{M_1 \cdot y_1}{I_{1-1}} \pm \frac{M_2 \cdot y_2}{I_{2-2}}$$

$$q = \frac{49048}{376} \pm \frac{23543 \cdot y_1}{22915} \pm \frac{11772 \cdot y_2}{7197} = 130.4 \pm 1.03 y_1 \pm 1.64 y_2$$

Distance to 1-1 axis
" " 2-2 "

$$q_A = 130.4 \pm 1.03 \underbrace{y_1}_{\overline{BN}} \pm 1.64 \underbrace{y_2}_{\overline{FK}} = 130.4 + 1.03 \cdot \underbrace{(3.53)}_{(16-12.47)} + 1.64 \cdot \underbrace{(2.96)}_{(10-7.04)} = 138.9 \quad \text{kN/m}^2$$

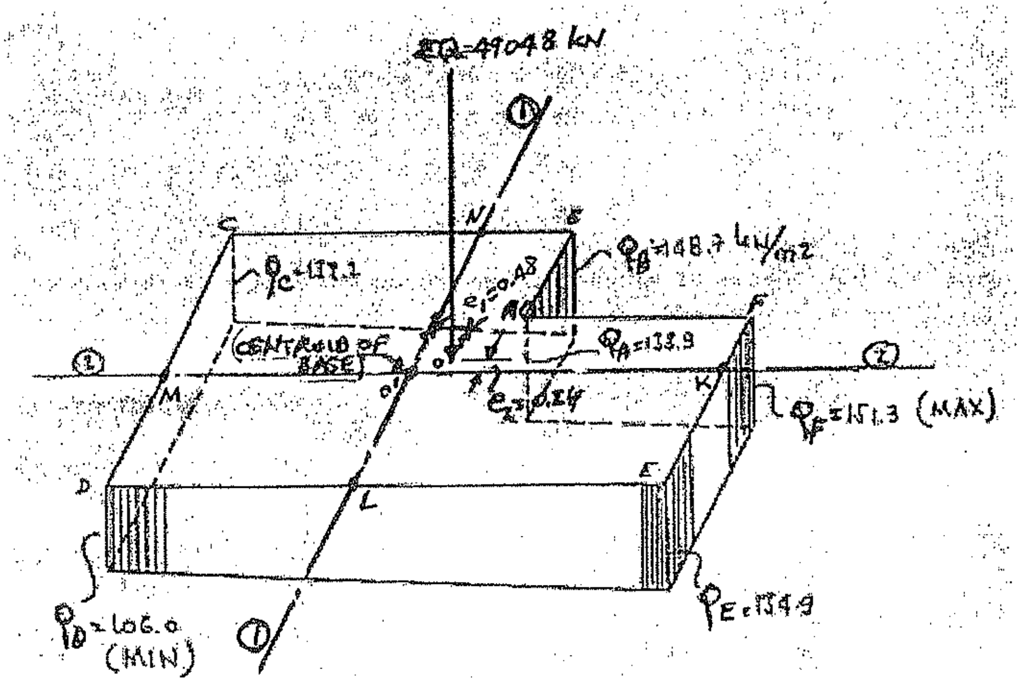
$q_B = 130.4 + 1.03 \cdot (3.53) + 1.64 \cdot (8.96) = 148.7$ kN/m²

$q_C = 130.4 - 1.03 \cdot (12.47) + 1.64 \cdot (8.96) = 132.2$ kN/m²

$q_D = 130.4 - 1.03 \cdot (12.47) - 1.64 \cdot (7.04) = 106$ kN/m²

$q_E = 134.9$ kN/m²

$q_F = 151.3$ kN/m²



CE 366 Foundation Engineering – I

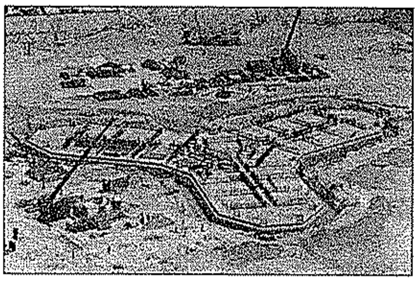
2012-2013 Spring Semester

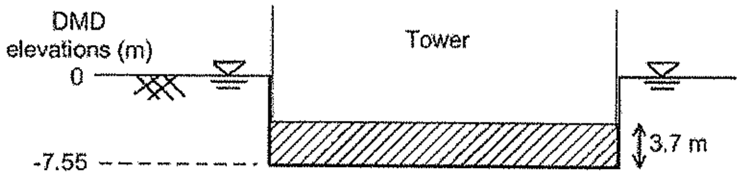
HOMEWORK I

Due on: 11.04.2013 12:00 p.m.

Question 1 (10%)

You are given Poulos and Bunce (2008)* paper describing the foundation design for the world's tallest building: Burj Dubai. The foundation of this building is a raft foundation with piles. However, for our CE366 homework question we will assume that the foundation is a raft foundation only (no piles).





You can see different soil layers and their modulus, E, values in Figure 2 of Poulos and Bunce (2008). Calculate the immediate settlement of the world's tallest building 1 year after construction, by making these assumptions:

-Assume a raft foundation only (no piles)

-Assume that, including the raft foundation, the building dead load and live load is 500.000 tons.

-Assume that the foundation is a circular shaped foundation with a diameter of 60 m (to represent the area of the flower-shaped** foundation of real Burj Dubai tower) and that the load is applied everywhere as a uniform pressure.

-Assume dry and saturated unit weights of all soils as 20 kN/m^3.

a) Using Schmertmann's method (Note: use the "Adopted small strain design E values" given in figure 2 of Poulos and Bunce (2008)).

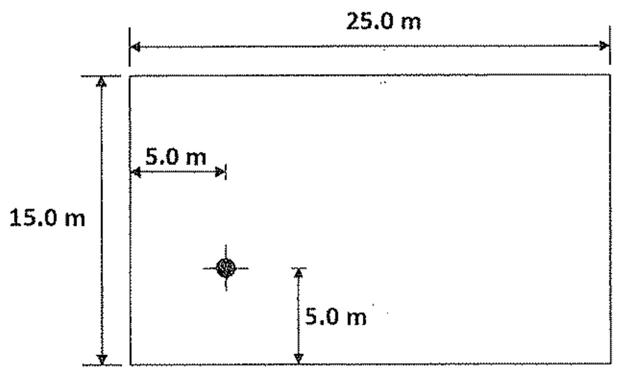
b) Under the center of the foundation, using the formula $s_i = \frac{qB}{E}(1 - v^2)I_s$

* Poulos and Bunce (2008) "Foundation Design For The Burj Dubai – The World's Tallest Building", Proc. 6th International Conference on Case Histories in Geotechnical Engineering, Virginia, USA, August 11-16, 2008.

** "...the triple lobed footprint of the building was inspired by the flower hymenocallis" http://en.wikipedia.org/wiki/Burj_Khalifa

Question 2 (10%)

Figure shows the plan of a rectangular foundation which transmits a uniform contact pressure of 120 kN/m^2. Determine the vertical stress induced by this loading at a depth of 10 m below the point.



Question 3 (10%)

A rectangular foundation (1.5 m x 1 m) is located at a depth of 1 m in a two-layered clay soil profile. Borings indicate that softer clay is located at a depth of 1 m from the bottom of the foundation. (see Figure 1)

a) Determine the gross allowable load for the foundation with a factor of safety of 3.

b) How would the result the change if the softer soil was placed at the top, i.e., $C_{u,1}$ = 48 kN/m^2 and $C_{u,2}$ = 120 kN/m^2 and the geometry of the foundation and soil profile remains the same.

Hint: For layered soils, the value of bearing capacity factor, N_c, is not a constant. It is a function of c_2/c_1, where c_2 belongs to the bottom layer, and z/B, where z is the depth measured from the bottom of the foundation to the interface of the two clay layers (see Figure 2). The ultimate bearing capacity is given as:

$$q_f = c_1 N_c S_c d_c + \gamma D$$

S_c =1.145, d_c=1.4, $FS = \dfrac{q_{nf}}{q_n}$

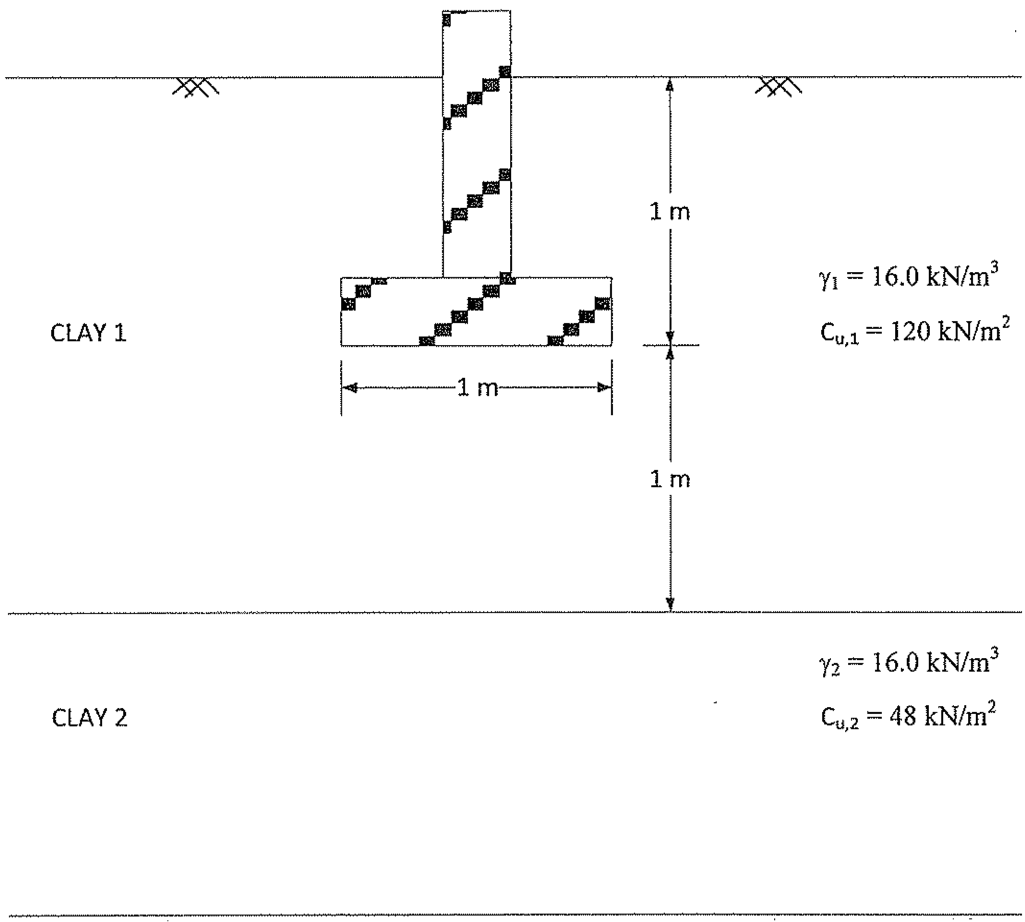


Figure 1

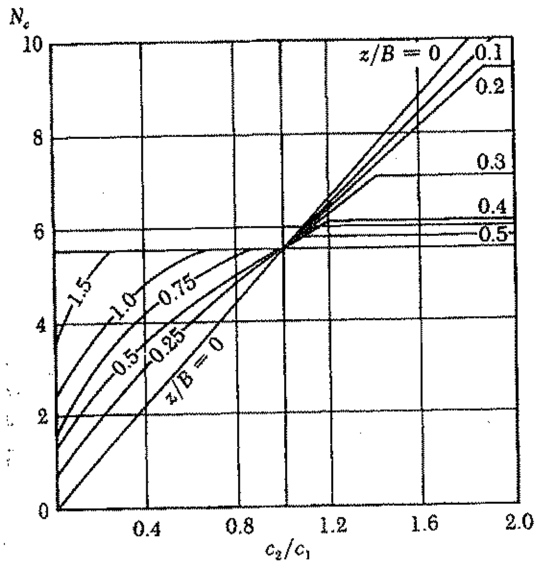


Figure 2. Bearing capacity on layered clay soils – q = 0 (Figure is redrawn after Reddy and Srinivasan, 1967)

Question 4 (10%)

A foundation with the dimensions of 25 m x 30 m in plan is to be constructed at a site where the unit weight of soil is 19 kN/m^3. The gross load to be transferred from the superstructure to the foundation is calculated to be 160 kPa and the foundation depth is planned as 4 m. Using De Beer's proposal, calculate the required depth of site exploration from the ground level.

Question 5 (10%)

A series of standard penetration tests (SPT) were carried out at a site where a deep silty sand deposit exists. The recorded blow counts and respective test depths are provided below. Ground water table is at 6.5 m from the surface. Drained and saturated unit weights for the sand are 18 kN/m^3 and 20 kN/m^3, respectively. Borehole diameter is 75 mm and standard sampler is used during testing.

i) Calculate $N_{1,60}$ values at depths of test assuming the energy efficiency is 45% for the SPT hammer mechanism.

ii) Estimate the internal angle of shear resistance at each depth for the tests in the sand layer

Soil Type	Depth (m)	SPT-N (measured)
	0	3 – 5 – 4
	3.2	7 – 6 – 8
	5.2	8 – 10 – 12
▽ GWT	6.5	
	7.2	7 – 11 – 18
	9.2	10 – 15 – 25

Question 6 (10%)

A vane shear test was carried out in a saturated clay layer using a vane of 100 mm length and 50 mm width. Peak value of the torque applied during the test was measured as 0.22 kNm. Calculate the undrained shear strength of the clay and apply the Bjerrum's correction factor assuming the plasticity index as 50.

Question 7 (10%)

The data given below is idealized from a cone penetration test (CPT) carried out at a specific site.

i) Complete the table by calculating the friction ratio (R_f) and plot variations of side friction, cone resistance and friction ratio as a function of depth.

ii) Identify the soil type at 1 m intervals using information provided in Figure 2.19 of CE-366 Lecture Notes.

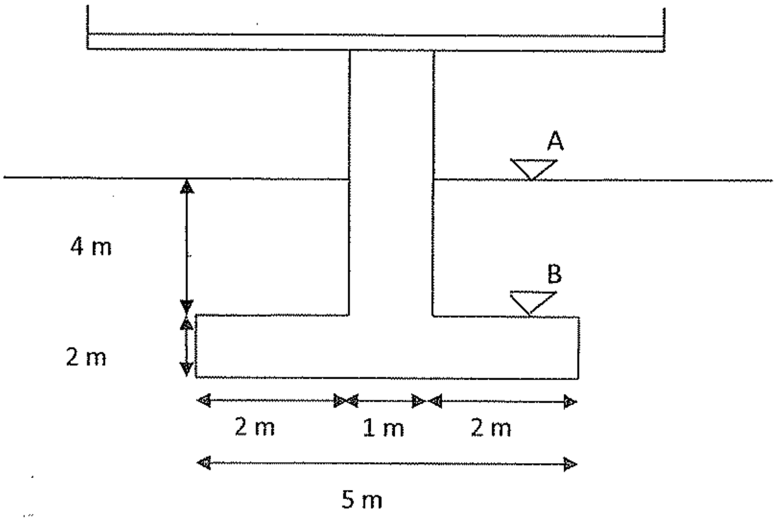
iii) Estimate the undrained shear strength of the soil at 9 m depth assuming the cone factor as N_k=16. Assume the soil has drained and saturated unit weights of 20 kN/m^3, with the water table being at 5 m depth from the ground surface.

Depth(m)	f_{sc}(kPa)	q_c(kPa)	RF= f_{sc}/ q_c (%)
1	20	500	
2	31	700	
3	55	1100	
4	42	1400	
5	35	800	
6	27	600	
7	27	0.6	
8	24	0.8	
9	45	0.9	
10	22	1.1	
11	15	1.5	
12	24	2.0	
13	20	2.2	
14	26	2.6	
15	32	2.7	

Question 8 (10%)

Section of an elevated highway, which is supported by a load bearing reinforced concrete wall on a continuous footing, is shown below. The total load per meter length of the structure (including dead load, live load, bearing wall and footing) is 1500 kN. The unit weights of soil above and below the water table are 18 kN/m^3 and 21 kN/m^3, respectively.

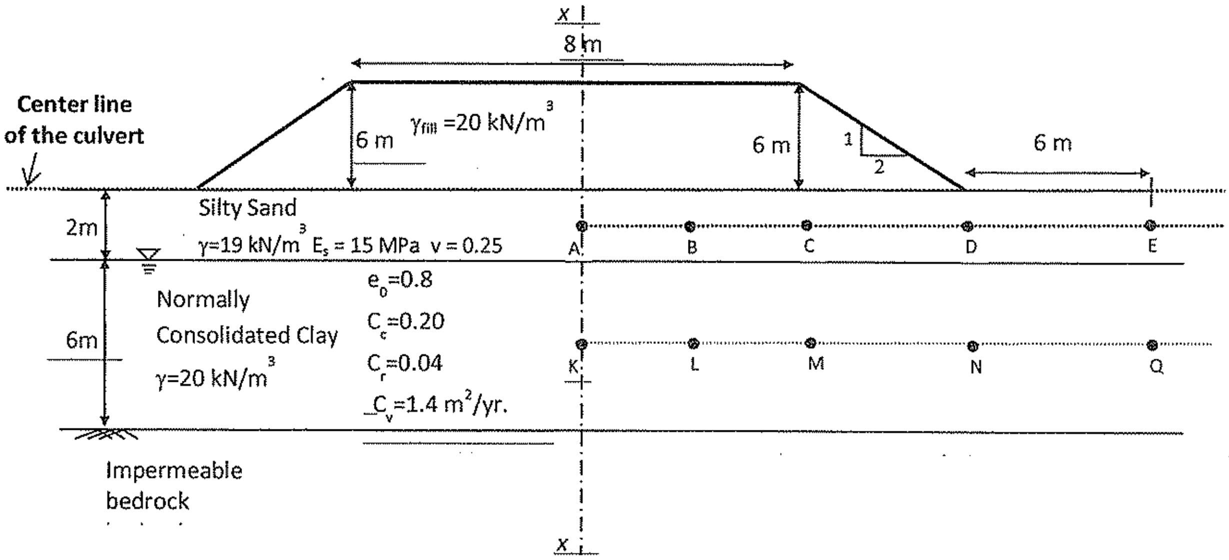
i) Calculate the net foundation pressure on the soil for the two positions of ground water table shown on the figure below.



ii) Calculate the factor of safety against uplift for the two positions of ground water table shown on the figure above.

Question 9 (10%)

A 6 m high highway embankment is to be rapidly constructed on the soil profile given below. A relatively small diameter culvert is placed beneath the embankment. Water table is located 2 m below the ground surface.



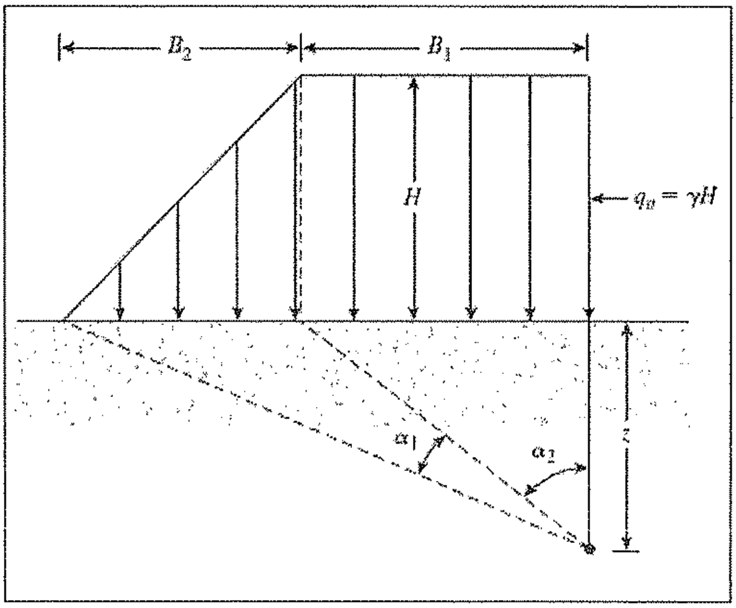
i) Estimate elastic settlements at points A, B, C, D and E by using Eqn 1. Assume Ip and B values are 0.03 and 20 m; respectively. Ignore elastic settlement due to clay and bedrock layers.

$\Delta H = \Delta q * B * (1-v^2)/E * I_p$ ....Eq(1).

ii) Estimate the ultimate consolidation settlement of the clay layer by considering the stresses at points K,L,M,N and Q.

iii) Estimate the settlement profile along culvert x-section corresponding to 3 years after the placement of the embankment. If a culvert with a settlement gradient of 1/250 is expected to crack, discuss if a crack is likely to occur.

<u>Hint:</u> Estimate vertical stress increases at points A (or K) through E (or Q) by using elastic stress distribution formulae given below.



$$\Delta\sigma = \frac{q_o}{\pi}\left[\left(\frac{B_1 + B_2}{B_2}\right)(\alpha_1 + \alpha_2) - \frac{B_1}{B_2}(\alpha_2)\right]$$

where

$q_o = \gamma H$.
γ = unit weight of the embankment soil
H = height of the embankment

$$\alpha_1 = \tan^{-1}\left(\frac{B_1 + B_2}{z}\right) - \tan^{-1}\left(\frac{B_1}{z}\right)$$

$$\alpha_2 = \tan^{-1}\left(\frac{B_1}{z}\right)$$

(Note that α_1 and α_2 are in radians.)

Question 10 (10%)

A building will be constructed on a clay deposit. The building will rest on a mat foundation at 2m depth and has 20m x 20m dimensions in plan. The clay deposit is 12m deep and overlies a rock layer. The ground water level is at 2m depth. The bulk unit weights are 18kN/m^3 and 20kN/m^3 above and below water table, respectively. The clay has c'=5kN/m^2, $\emptyset' = 20°$, c_u=48kN/m^2 and $\emptyset_u = 0°$. Take $\gamma_{water} = 10$kN/m^3.

$$q_{ult} = q_f = c_u N_c + \gamma D$$

$$q_{ult} = q_f = 0.4\gamma B N_\gamma + 1.2 c N_c + \gamma D N_q$$

(a) Calculate the ultimate bearing capacity of the foundation in the short and long terms. Comment on which is more critical?

CE 366 Foundation Engineering – I

2012-2013 Spring Semester

HOMEWORK I - SOLUTIONS

Due on: 11.04.2013 12:00 p.m.

Answer 1)

a) Using Schmertmann's method(Note: use the "Adopted small strain design E values" given in figure 2 of Poulos and Bunce (2008)).

$S_e = c_1 \cdot c_2 \cdot q \cdot (\sum_0^{Z_2} \frac{I_z}{E_s} \Delta z)$

$c_1 = 1-0.5\ [\sigma'_o / q]$

σ'_o = effective vertical stress at the depth of foundation, before construction

$\sigma'_o = \sigma_o - u_o = 7.55 \times 20 - 7.55 \times 10 = 75.7$ kPa

q = net foundation pressure

$q = \sigma'_f - \sigma'_o$ (or $q = \sigma_f - \sigma_o$).

σ'_f = effective vertical stress at the depth of foundation, after construction

Including the raft foundation, the building dead load and live load is 500.000 tons (1 ton = 10 kN)

Area of foundation = $\pi D^2 / 4 = \pi (60)^2 / 4 = 2827.4\ m^2$

$$\sigma'_f = \frac{5000000\ kN - 2827.4 \times 7.55 \times \gamma_{water}}{2827.4\ \ m^2} = 1693\ kPa$$

$q = \sigma'_f - \sigma'_o = 1693 - 75.7$ kPa = 1617.3 kPa

$c_1 = 1 - 0.5\ [\sigma'_o / q] = 1 - 0.5\ [75.7 / 1617.3] = 0.977$

$c_2 = 1 + 0.2 \log (t / 0.1) = 1 + 0.2 \log (1 / 0.1) = 1.2$

From Figure 2 of Poulos and Bunce (2008) the soil profile is given below (Note that DMD means Dubai Municipality Datum):

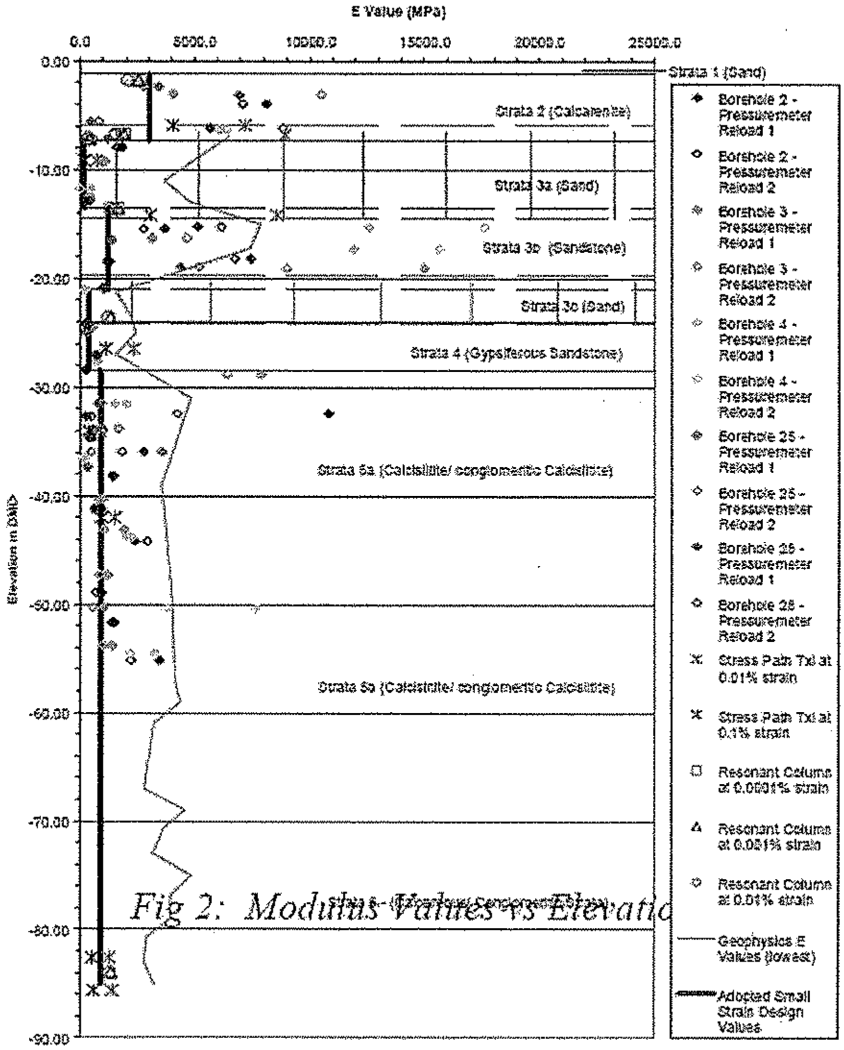


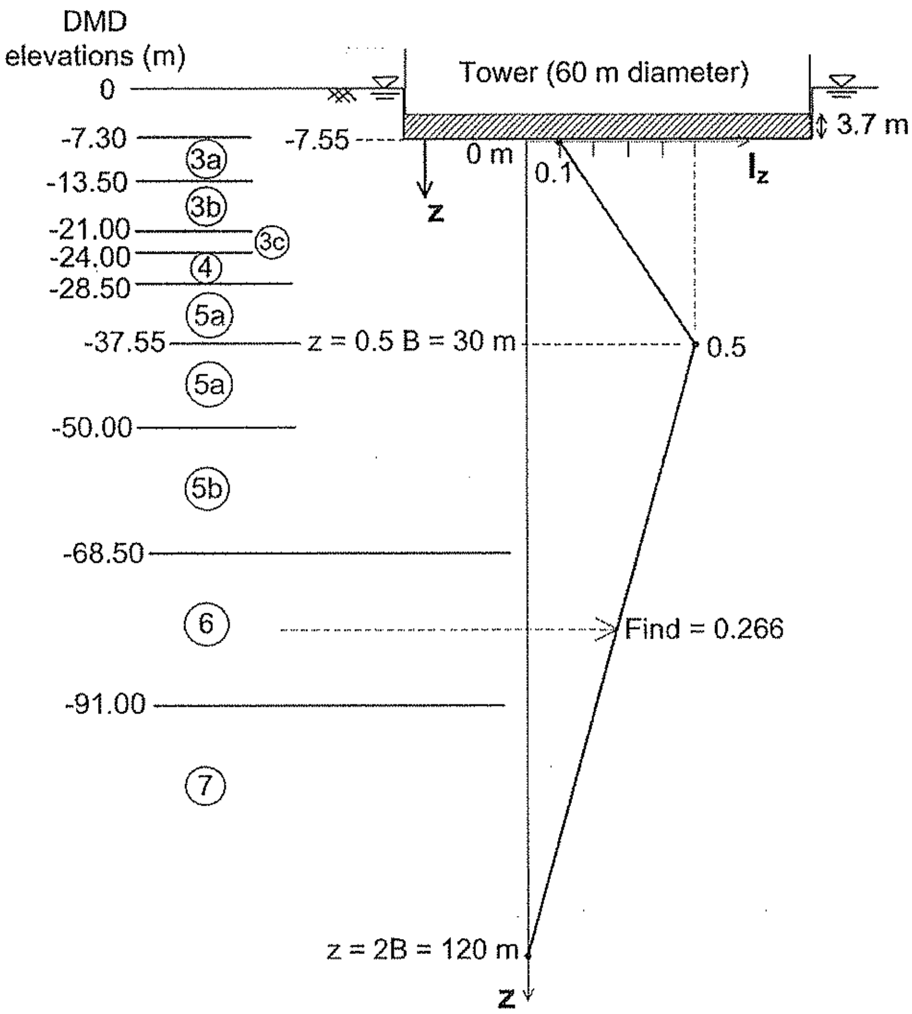
Fig 2: Modulus Values vs Elevatio

For circular foundations, Schmertmann's strain influence factors are given below (Note that z is measured from below the base of foundation):

$I_z = 0.1$ at $z = 0$

$I_z = 0.5$ at $z = z_1 = 0.5$ B

$I_z = 0$ at $z = z_2 = 2$B



Settlement calculations using Schmertmann's method:

DMD Elevations (m)	Layer Name	E* (10^3 kPa)	Layer thickness, Δz (m)	I_z	$\frac{I_z}{E_s} \cdot \Delta z$
-7.55 to -13.50	3a (sand)	200	5.95	0.1397	4.16 10^{-6}
-13.50 to -21.00	3b (sandstone)	1250	7.5	0.2293	1.38 10^{-6}
-21.00 to -24.00	3c (sand)	350	3	0.2993	2.57 10^{-6}
-24.00 to -28.50	4 (gypsiferous sandstone)	350	4.5	0.3493	4.49 10^{-6}
-28.50 to -37.55**	5a (calcisiltite)	900	9.05	0.4397	4.42 10^{-6}
-37.55 to -50.00	5a (calcisiltite)	900	12.45	0.4654	6.44 10^{-6}
-50.00 to -68.50	5b (calcisiltite)	900	18.5	0.3794	7.8 10^{-6}
-68.50 to -91.00	6 (calcerous conglomerate)	900	22.5	0.266	6.65 10^{-6}
-91.00 to127.55	7 (claystone/siltstone)	900	36.55	0.1225	4.97 10^{-6}
					$\sum_0^{z_2} \frac{I_z}{E_s} \Delta z = 4.29 \times 10^{-5}$

** E values are obtained from Figure 2 of Poulos and Bunce (2008)*
*** Since the Iz value is changing at that location, there needs to be a layer boundary at a depth where Iz=0.5*

$S_e = c_1 \cdot c_2 \cdot q \cdot (\sum_0^{z_2} \frac{I_z}{E_s} \Delta z) = (0.977) \cdot (1.2) \cdot (1617.3) \cdot (4.29 \times 10^{-5}) = 0.081\ m =$ **81 mm**

OR

Since the lower layers (5a-5b-6-7) all have the same E = 900 MPa, we may combine these layers (see the effect on settlement).

DMD Elevations (m)	Layer Name	E* (10^3 kPa)	Layer thickness, Δz (m)	I_z	$\frac{I_z}{E_s}\cdot\Delta z$
-7.55 to -13.50	3a (sand)	200	5.95	0.1397	4.16 10^{-6}
-13.50 to -21.00	3b (sandstone)	1250	7.5	0.2293	1.38 10^{-6}
-21.00 to -24.00	3c (sand)	350	3	0.2993	2.57 10^{-6}
-24.00 to -28.50	4 (gypsiferous sandstone)	350	4.5	0.3493	4.49 10^{-6}
-28.50 to -37.55**	5a(calcisiltite)	900	9.05	0.4397	4.42 10^{-6}
-37.55 to 127.55	5a-5b-6-7	900	90	0.25	2.5 10^{-5}
				$\sum_0^{Z_2}\frac{I_z}{E_s}\Delta z = 4.202\times10^{-5}$	

** E values are obtained from Figure 2 of Poulos and Bunce (2008)*
*** Since the Iz value is changing at that location, there needs to be a layer boundary at a depth where Iz=0.5*

$S_e = c_1\cdot c_2\cdot q\cdot(\sum_0^{Z_2}\frac{I_z}{E_s}\Delta z) = (0.977)\cdot(1.2)\cdot(1617.3)\cdot(4.202\times10^{-5}) = 0.0797\ m = 79.7$ mm

b) Under the center of the foundation, using the formula $s_i = \frac{qB}{E}(1-v^2)I_s$

$s_i = \frac{qB}{E}(1-v^2)I_s$

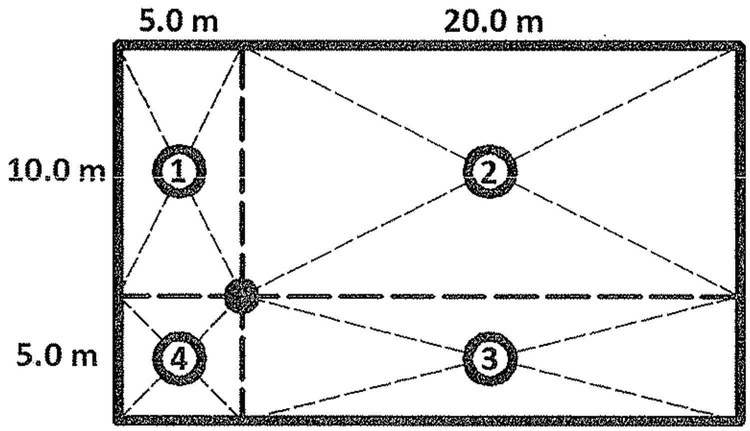
For E value, you can consider the depth of influence (stress bulbs) and consider the materials up to 2B depth below the base of foundation. In that zone, for E value to be used in the equation, you can select the smallest value, or the arithmetic average value of E values, or use different E values and do superposition, or use weighted average of E values with thickness of layers, up to the depth of 120 m below the base of foundation.

Weighted average E value (in the zone from the base of foundation to the depth of 120 m below the base of foundation)

Weighted average of E value= $\frac{200\times5.95+1250\times7.5+350\times7.5+900\times99.05}{120\ m} = 853$ MPa

$S_i = \frac{(1617.3)\ (60)}{853000}(1-0.2^2)(1.00) = 0.109$ m = **109 mm**

Answer 2)



Consider four rectangles (1, 2, 3, 4) each with a corner at the point; the vertical stress below the point is the sum of the stresses induced by each rectangle;

$\Delta\sigma_z = \Delta\sigma_{z(1)} + \Delta\sigma_{z(2)} + \Delta\sigma_{z(3)} + \Delta\sigma_{z(4)}$

$\Delta\sigma_z = q\ (I_{R(1)} + I_{R(2)} + I_{R(3)} + I_{R(4)})$

z = 10.0 m			
Rectangle	**m = B/z**	**n = L/z**	I_R
1	10/10 = 1.0	5/10 = 0.5	0.1202
2	10/10 = 1.0	20/10 = 2.0	0.1999
3	5/10 = 0.5	20/10 = 2.0	0.1350
4	5/10 = 0.5	5/10 = 0.5	0.0840

Then $\Delta\sigma_z = 120(0.1202 + 0.1999 + 0.1350 + 0.0840)$

$\Delta\sigma_z = 120 \times 0.5391 =$ **$65\ kN/m^2$**

Answer 3)

a) c_{u1}=120 kN/m^2

c_{u2}=48 kN/m^2

c_{u2}/ c_{u1}=48/120=0.4

z/B=1/1=1 → from graph (Figure 2);N_c=4.75

$q_f = c_1.N_c.S_c.d_c$

$S_c = 1.145, d_c = 1.4\ (given)$

$q_f = 120x4.75x1.145x1.4 + 16x1 = 913.7 + 16 \cong 930\ kN/m^2$

$q_{nf} = q_f - \gamma.D = 930 - 16x1 = 914\ kN/m^2$

$$q_{net} = \frac{q_{n,f}}{F.S.} = \frac{914}{3} \cong 305\ kN/m^2$$

$q_{gross,all} = q_{net} + \gamma.D = 305 + 16 = 321\ kN/m^2$

$P_{gross,all} = 321x1x1.5 = 481.5\ kN$

b) c_{u1}=48 kN/m^2

c_{u2}=120 kN/m^2

c_{u2}/ c_{u1}=120/48=0.4

z/B=1/1=1 → from graph (Figure 2);N_c=5.5

$q_f = c_1.N_c.S_c.d_c$

$S_c = 1.145, d_c = 1.4\ (given)$

$q_f = 48x5.5x1.145x1.4 + 16x1 = 423.2 + 16 \cong 439\ kN/m^2$

$q_{nf} = q_f - \gamma.D = 439 - 16x1 = 423\ kN/m^2$

$$q_{net} = \frac{q_{n,f}}{F.S.} = \frac{423}{3} = 141\ kN/m^2$$

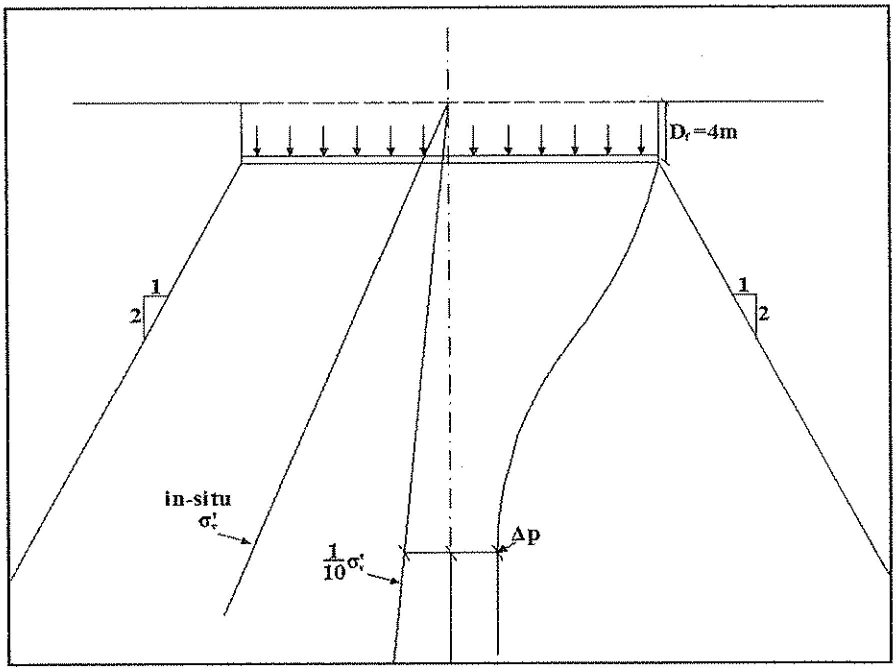
$q_{gross,all} = q_{net} + \gamma.D = 141 + 16 = 157\ kN/m^2$

$P_{gross,all} = 157x1x1.5 = 235.5\ kN$

Answer 4)

B X L= 25m X 30m

γ= 19 kN/m3



q_{gross} = 160 kPa

q_{net} = q_{gross} − γDf = 160 – 4*19

q_{net} = 84 kPa

<u>**By using the approximate method**</u>:

$$\Delta\sigma = \frac{q_{net}\,B\,L}{(B+z)(L+z)}$$

$$\Delta\sigma = 0.1\,\sigma_v{}'$$

$$\frac{84 * 25 * 30}{(25+z)(30+z)} = 0.1 * 19 * (z+4)$$

z = 14.7 m

<u>**Depth of boreholes from ground surface:**</u> L = z + D_f = 14.7 + 4 = 18.7 m

L =18.7 m

Answer 5)

$$N_{1,60} = N_{field} * \frac{ER}{0.60} * C_B * C_S * C_R * C_N$$

From Table 2.2 SPT Correction Factors(Robertson and Wride, NCEER 1997 Workshop):

Borehole diameter = 75 mm→$C_B = 1.00$

Standard sampler is used during the test →$C_S = 1.00$

$$C_N = 9.78 * \sqrt{\frac{1}{\sigma_v'(kN/m^2)}}$$

σ_v' (kPa): At Depth: 3.2 m → 3.2 * 18 = 57.6 kPa

At Depth: 5.2 m → 5.2 * 18 = 93.6kPa

At Depth: 7.2 m → 6.5 * 18 + 0.7 * (20 - 10) = 124kPa

At Depth: 9.2 m →6.5 * 18 + 2.7 * 10 = 144kPa

Soil Type	Depth (m)	SPT-N (measured)	(Field) SPT-N	C_R	N_{60}	σ_v'(kPa)	C_N	$N_{1,60}$	Φ(°)
____________	0	3 – 5 – 4	-	-	-	-	-	-	-
------------------------	3.2	7 – 6 – 8	14	0.75	8	57.6	1.29	10	30
------------------------	5.2	8 – 10 – 12	22	0.85	14	93.6	1.01	14	32
▽ GWT	6.5								
------------------------	7.2	7 – 11 – 18	29	0.95	20	124	0.88	18	33
------------------------	9.2	10 – 15 – 25	40	0.95	38	144	0.82	31	36

Friction angle values are estimated from the chart on page 37 of CE366 Lecture Notes using N60 values.

Answer 6)

T_{peak}= 0.22 kN.m

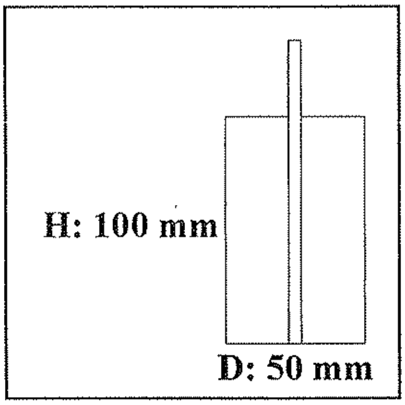
$$C_{u,field} = \frac{T}{\pi * D^2\left(\frac{H}{2}+\frac{D}{6}\right)}$$

$$C_{u,field} = \frac{0.22}{\pi * 0.05^2\left(\frac{0.1}{2}+\frac{0.05}{6}\right)}$$

$$C_{u,field} = 480\ kPa$$

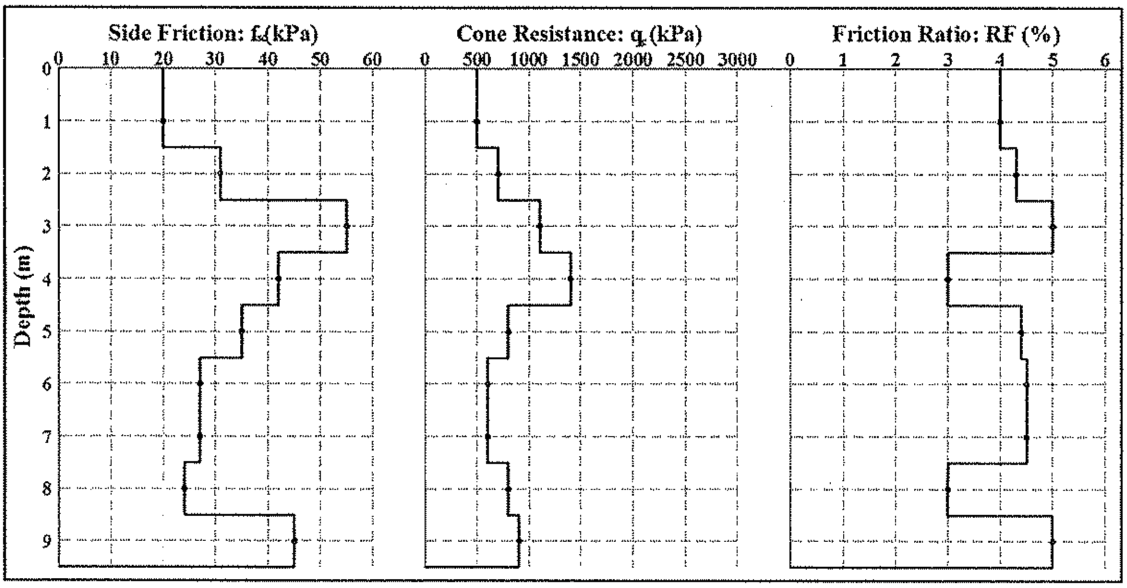
$$\lambda \cong 0.77\ for\ PI = 50\%$$

$$\mathbf{C_{u,corrected} = C_{u,field} * \lambda = 480 * 0.77 = 370\ kPa}$$

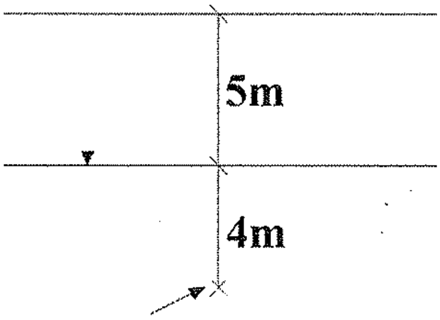


Answer 7)

Depth(m)	Side Friction f_{sc}(kPa)	Cone Resistance q_c(kPa)	Friction Ratio RF= f_{sc}/ q_c(%)	Soil Classification
1	20	500	4	Clay –Silty Clay
2	31	700	4.3	Clay – Silty Clay
3	55	1100	5	Clay – Silty Clay
4	42	1400	3	Clayey Silt – Silty
5	35	800	4.4	Clay – Silty Clay
6	27	600	4.5	Clay – Silty Clay
7	27	600	4.5	Clay – Silty Clay
8	24	800	3	Clay – Silty Clay
9	45	900	5	Clay – Silty Clay
10	22	1100	2	Sand Mixtures
11	15	1500	1	Sand Mixtures
12	24	2000	1.2	Sand Mixtures
13	20	2200	0.9	Sand Mixtures
14	26	2600	1	Sand Mixtures
15	32	2700	1.2	Sand Mixtures



iii)



$\sigma_v = 5 * \gamma_{drained} + 4 * \gamma_{saturated}$

$\sigma_v = 9 * 20$

$\sigma_v = 180\ kPa$

$$S_u(= C_u) = \frac{q_c - \sigma_v}{N_k}$$

Where, $N_k = 16$

$$S_u(= C_u) = \frac{900 - 180}{\mathbf{16}}$$

$S_u = 45\ kPa$

Answer 8)

a)

When water table is at point A:

$q_{net} = Q_{net}$ / (Foundation Area) $\quad\quad Q_{net} = Q_{gross} - W_{exc.soil}$

$W_{exc.soil} = (5 * 2 + 1 * 4) * 21 = 294$ kN/m

$Q_{net} = 1500 - 294 = 1206$ kN/m

$q_{net} = 1206 / 5 = 241.2$ kN/m^2

Alternatively;

$q_{net} = \sigma'_f - \sigma'_0 \quad\quad \sigma'_0 = 6 * (21 - 10) = 66$ kN/m^2

$\sigma'_f = q_{gross} + q_{soilonfound} - P_{uplift}$

$P_{uplift} = [5 * 6 * 10 - (2 + 2) * 4 * 10] / 5 = 140 / 5 = 28$ kN/m^2

$\sigma'_f = 1500 / 5 + 4 * 2 * 2 * (21 - 10) / 5 - 28 = 307.2$ kN/m^2

$q_{net} = 307.2 - 66 = 241.2$ kN/m^2

When water table is at point B:

$q_{net} = Q_{net}$ / (Foundation Area) $\quad\quad Q_{net} = Q_{gross} - W_{exc.soil}$

$W_{exc.soil} = (5 * 2) * 21 + (4 * 1) * 18 = 282$ kN/m

$Q_{net} = 1500 - 282 = 1218$ kN/m

$q_{net} = 1218 / 5 = 243.6$ kN/m^2

Alternatively;

$q_{net} = \sigma'_f - \sigma'_0 \quad\quad \sigma'_0 = 2 * (21 - 10) + 4 * 18 = 94$ kN/m^2

$\sigma'_f = q_{gross} + q_{soilonfound} - P_{uplift}$

$P_{uplift} = [5 * 2 * 10] / 5 = 100 / 5 = 20$ kN/m^2

$\sigma'_f = 1500 / 5 + 4 * 2 * 2 * (18) / 5 - 20 = 337.6$ kN/m^2

$q_{net} = 337.6 - 94 = 243.6$ kN/m^2

b)

Calculate the factor of safety against uplift:

When water table is at point A:

FS_{uplift} = [1500 + 4 * 4 * (21-10)] / 140

FS_{uplift} = 12

Calculate the factor of safety against uplift:

When water table is at point B:

FS_{uplift} = [1500 + 4 * 4 * (18)] / 100

FS_{uplift} = 17.9

Answer 9)

i) <u>Elastic settlements:</u>

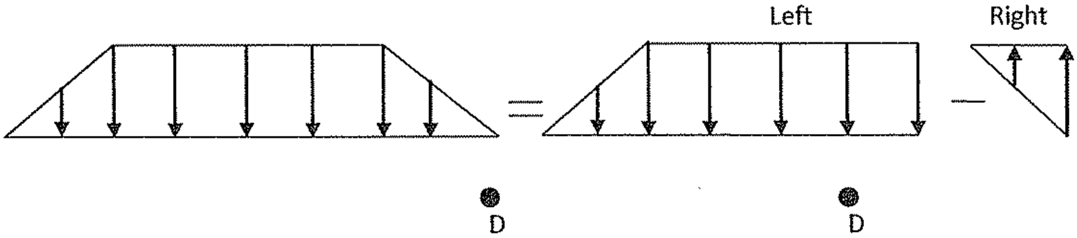
$S_e = \Delta q * B * (1 - v^2)/E * I_p$

B =20 m I_p = 0.03 v = 0.25 E = 15000 kPa

$(\Delta q)_A$ = 119.9 from table 1.

$(Se)_A = 119.9x20x(1-0.25^2)/15000x0.03 \approx 4.5x10^{-3}$ m = 4.5 mm

Sample calculation for $(\Delta\sigma)_D$:



$(B1)_{left}$ = 20 $(B2)_{left}$ = 12 z=1 $\alpha1_{left}$ = arctan ((20 + 12)/1) - arctan (20/1) = 0.0187 ≈ 0.02
$\alpha2_{left}$ = arctan (20/1) = 1.52

$(B1)_{right}$ = 0 $(B2)_{right}$ = 12 z=1 $\alpha1_{right}$ = arctan ((0 + 12)/1) - arctan (0) = 1.488 ≈ 1.49
$\alpha2_{right}$ = arctan (0) = 0

$(\Delta q)_{rigth} = q_0 /\pi * [(B1_{right} + B2_{right})/B2_{right} *(\alpha1_{right} + \alpha2_{right}) - B1_{right} /B2_{right} *\alpha2)] = 120/\pi * [(0+ 12)/12*(1.49+ 0)-0/12*0)]$ = 56.8 kPa

Similarly, $(\Delta q)_{left}$ = 60 kPa

$\Delta q_{total} = (\Delta q)_{left} - (\Delta q)_{rigth}$ = 60 – 56.8 = 3.2 kPa

All results are in given below:

Point	z (m)	q_o (kPa)	B1 (m) right	B2 (m) right	α1 (rad.) right	α2 (rad.) right	Δq_{rigth} (kPa)	B1 (m) left	B2 (m) left	α1 (rad.) left	α2 (rad.) left	Δq_{left} (kPa)	Δq_{total} (kPa)	Sett. (mm)
A	1	120	4	12	0.18	1.33	59.9	4	12	0.18	1.33	59.9	119.9	4.5
B	1	120	2	12	0.39	1.11	59.8	6	12	0.11	1.41	60.0	119.7	4.5
C	1	120	0	12	1.49	0.00	56.8	8	12	0.07	1.45	60.0	116.8	4.4
D	**1**	**120**	**0**	**12**	**1.49**	**0.00**	**56.8**	**20**	**12**	**0.02**	**1.52**	**60.0**	**3.2**	**0.1**
E	1	120	6	12	0.11	1.41	60.0	26	12	0.01	1.53	60.0	0.0	0.0

Table 1

Table 1

ii) <u>Consolidation settlement:</u>

İnitial effective stresses in Point K, L, M,N and Q:

σ'_o = 19x2 + 3x(20-10) = 68 kPa

To find the increase in vertical effective stresses at Points K, L, M,N and Q due to embankment load use formulae given as hint.

Usign the formulae, the incerase in stress in midlayer of the clay under embakment:

Point	z (m)	qo (kPa)	B1(m) right	B2(m) right	α1 (rad.) right	α2 (rad.) right	Δσ(kPa) rigth	α1 (rad.) left	α2 (rad.) left	B1(m) left	B2(m) left	Δσ(kPa) left	(Δσ)total (kPa)
K	5	120	4	12	0.59	0.67	56.0	0.59	0.67	4	12	56.0	112.0
L	5	120	2	12	0.85	0.38	52.3	0.42	0.88	6	12	57.7	110.0
M	5	120	0	12	1.18	0.00	44.9	0.31	1.01	8	12	58.6	103.5
N	5	120	0	12	1.18	0.00	44.9	0.09	1.33	20	12	59.8	14.9
Q	5	120	6	12	0.42	0.88	57.7	0.06	1.38	26	12	59.9	2.2

Table 2

$$s_c = \frac{C_c \log(\sigma'_1/\sigma'_0)}{1+e_0} H$$

Consolidation Settlements:

Point	Settlements (m)
K	0.282
L	0.279
M	0.268
N	0.057
Q	0.009

iii)

$$T_v = \frac{c_v t}{d^2}$$

Tv = 1.4x3/6^2 = 0.117

for $U < 0.60,\quad T_v = \frac{\pi}{4}U^2$

for $U > 0.60,\quad T_v = -0.933\log(1-U) - 0.085$

U =(4*Tv/ π)$^{0.5}$ = 0.39

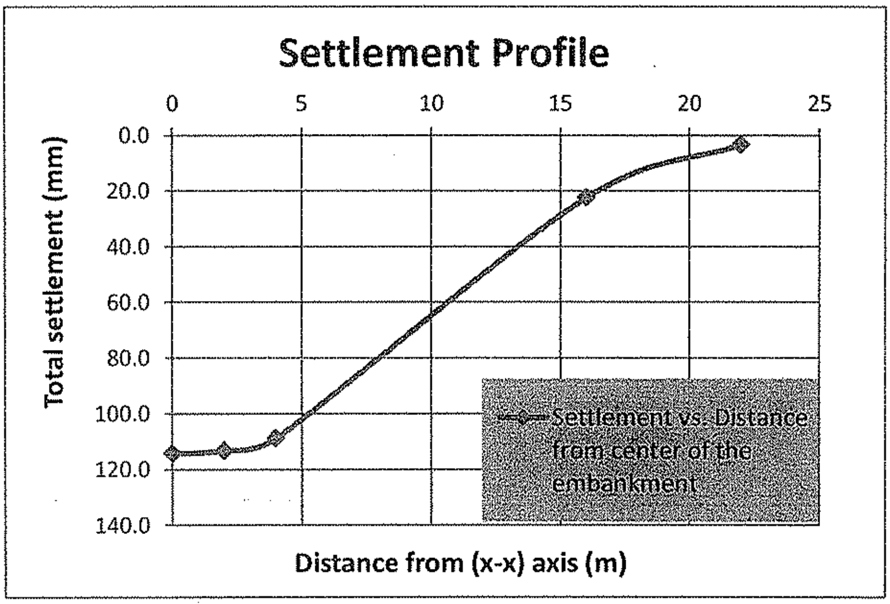
Summation of the elastic and consolidation settlements for <u>three years</u> gives the total settlements:

Point	S_c (mm)	S_e (mm)	S_{total} (mm)
A	109.9	4.5	114.4
B	108.7	4.5	113.2
C	104.5	4.4	108.9
D	22.4	0.1	22.5
E	3.5	0.0	3.5

The settlement gradients:

S_{total} (mm)	ΔH (mm)	ΔL (m)	ΔH/ΔL	Gradient
114.4	1.2	2	0.0006	1/1667
113.2	5.1	2	0.0022	1/465
108.9	85.4	12	0.0072	1/139
22.5	18.8	16	0.0032	1/316
3.5	---	---	---	---

> 1/250 (NOT OK). Cracking



Answer 10)

Long Term Ultimate Bearing Capacity:

$\emptyset' = 20° \Rightarrow N_\gamma = 3.5 \;\; N_q = 6.4 \;\; N_c = 14.8$

$$q_{ult} = 0.4\gamma' B N_\gamma + 1.2c' N_c + \gamma D N_q$$

$$q_{ult} = 0.4(20 - 10)x\ 20\ x\ 3.5 + 1.2\ x\ 5\ x\ 14.8 + 18\ x\ 2\ x\ 6.4$$

$$q_{ult} = 280 + 88.8 + 230.4 = 599.2\ kPa$$

Short Term Ultimate Bearing Capacity:

$\frac{D}{B} = \frac{2}{20} = 0.1 \Rightarrow N_c = 6.5$

$$q_{ult} = 48\ x\ 6.5 + 18\ x\ 2 = 348\ kPa$$

The ultimate bearing capacity of clay soil is more critical in the short term.

Release Date : **March 31, 2011, Thursday**
Version : **1.1**
Due On : **April 07, 2011, Thursday (5:00 PM – Strict)**

Rules for Homework:

1. Please submit your homework on the boxes labeled "**CE 366**" and located in the **Soil Mechanics Laboratory**. The deadline is strict and **NO EXTENSIONS** will be given.
2. Make sure that you check our website regularly. All announcements and corrections (if necessary) will be made available through our website.
3. Try to be clean, precise when you present your work. State your assumptions if you make any.
4. Discussion with your friends is strongly encouraged, however, homework needs to be solved and submitted individually.
5. Whenever you have a question about the homework, please contact your teaching assistant first. Remember that all TAs have office hours during the week. If you need further help, you can also contact your sections' instructor.

Version History:

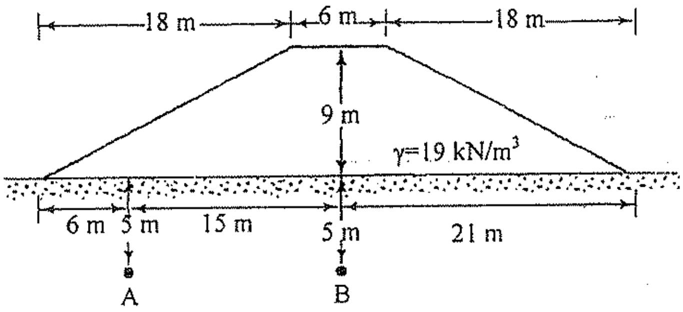
1.0. *(March 31, 2011)* Homework is released.
1.1. *(April 1, 2011)* The formulas for Question 1 is updated.

Question	Grade
1 (20%)	
2 (20%)	
3 (20%)	
4 (20%)	
5 (20%)	
Total (100%)	

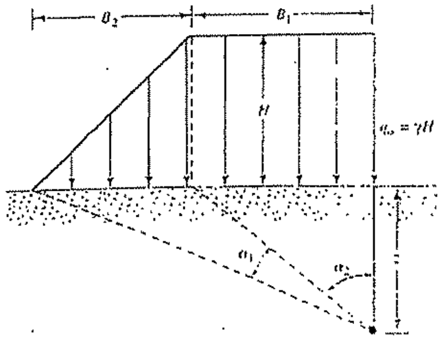
Surname, Name	
Signature	
Section You Are Registered For	

Question 1: (20%)

a. A highway embankment is shown in the figure given below. Estimate the stress increase under the embankment at points "A" and "B".



Hint:

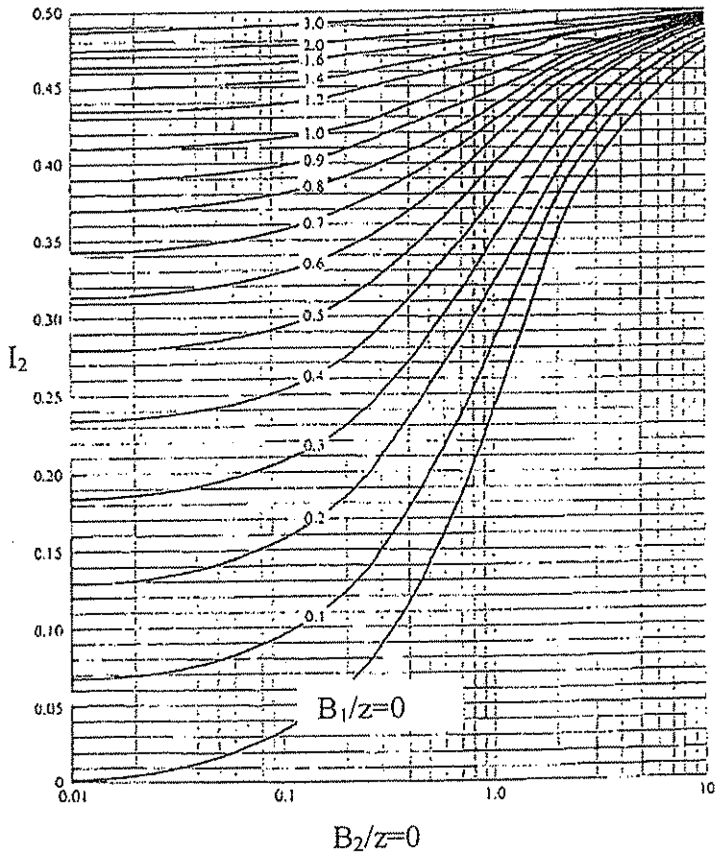


$$\Delta\sigma_z = \frac{q_0}{\pi}\left[\left(\frac{B_1+B_2}{B_2}\right)(\alpha_1+\alpha_2) - \frac{B_1}{B_2}(\alpha_2)\right]$$

where $q_o = \gamma H$
γ = unit weight of the embankment soil
H = height of the embankment

$$\alpha_1\,(\text{radians}) = \tan^{-1}\left(\frac{B_1+B_2}{z}\right) - \tan^{-1}\left(\frac{B_1}{z}\right)$$

$$\alpha_2 = \tan^{-1}\left(\frac{B_1}{z}\right)$$



or use simplified equation:
$\Delta\sigma_z = q_0 I_2$

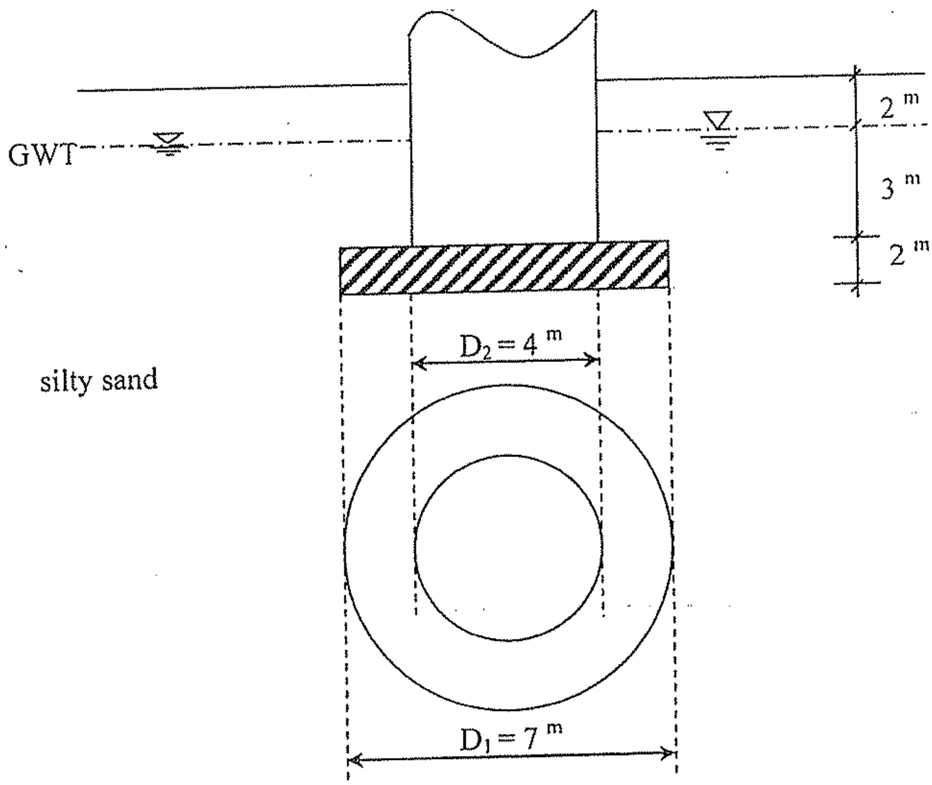
Ostenberg, 1957 chart for determination of vertical stress due to embankment loading.

b. Now transform the embankment load into a uniformly distributed load and estimate the average vertical stress increase at depth 5 m by using ii) 2:1 rule, ii) 30 degree rule.

Compare your findings with the values you have estimated in Question 1.a.

Question 2: (20%)

a. A chimney structure, having a diameter fo 4 m is supported by a 2 m thick circular foundation of diamater 7 m and the total weight of the structure including the foundation is 17500 kN. The ground water table is 2 m below the ground surface. If the foundation is seated at 5 m below the ground surface, determine the net foundation pressure. Drained and saturated unit weights of the soil are 17 kN/m^3 and 21 kN/m^3, respectively; and the unit weight of concrete is 24 kN/m^3.



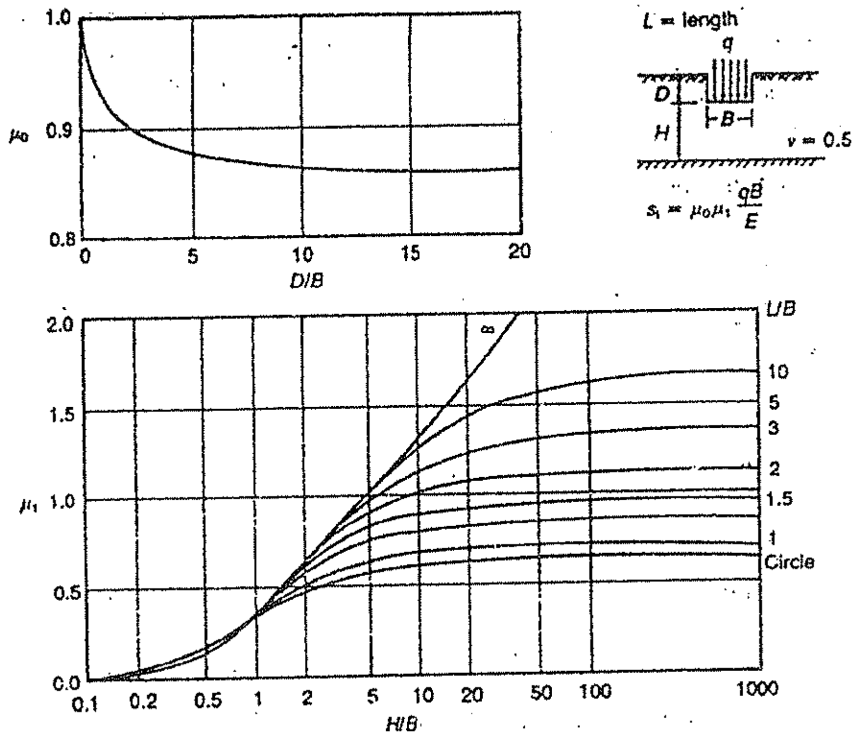
i. calculate the net foundation presure under the rigid mat.

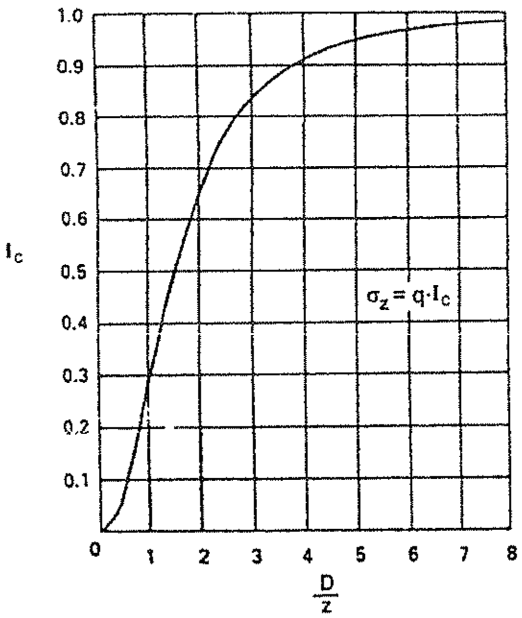
ii. Assuming the ground water table is at the ground surface, calculate the factor of safety against uplift.

b. A foundation of dimensions BxL = 3 m x 6 m is to be constructed over a thick sand deposit at a specific site. Foundation depth is 2 m, and the water table is at the ground surface. If the net foundation pressure is calculated to be 200 kPa, determine the required depth of subsoil investigation using De Beer's method. Saturated unit weight of the sand is 20 kN/m^3.

Question 3: (20%)

An oil storage tank 35m in diameter is located 2m below the surface of a deposit of clay 32m thick, the water table being at the surface: the net foundation pressure is 105 kN/m^2. A firm stratum underlies the clay. The average value of m_v for the clay is $0.14 m^2/MN$. The undrained value of Young's modulus (E_u) is estimated to be $40 MN/m^2$. Determine the total settlement (immediate+consolidation) under the centre of the tank.
HINT: The clay layer below the tank may be divided into 6 sublayers.

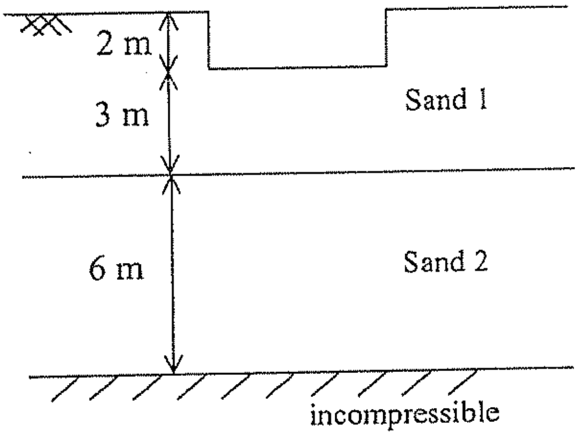




$$s_{OED} = m_v \Delta\sigma' H$$
$$s_c = \mu s_{OED}$$
$$\mu = 0.79$$

Question 4: (20%)

A 4 m x 4 m square footing with applied pressure, q (= q_{gross})= 210 kPa will be constructed at 2 m depth, on the given soil profile. According to the CPT test results, the average cone tip resistances for the first and second sand layers are 4 MPa and 5 MPa, respectively. The unit weight of both sand layers can be taken as 20 kN/m^3. No groundwater table is observed at the site.



a. Calculate the settlement of the footing 1 year after the construction using Schmertmann's method. Present a drawing of strain influnce factors and layers you used in calculations, together with a table showing your values.

b. Go to the website: *www.rocscience.com/settlecalc/schmertmann.html*. Read "Help" section and calculate the settlement of the problem in part (a), and compare in 1 sentence with your hand-calculated result. Show the results with "settlement versus depth below footing" plot in "Results" section. (Note: Do not select "Use Es", "Use Modified Schmertmann" "Subdivide Layers" options).

c. Using the website given above, keeping all other parameters constant:

i. What would be the settlement if the depth of foundation is 4 m instead of 2 m. Is this result expected/reasonable? Comment in 1-2 sentences.

ii. Prepare a plot of settlement versus time for up to 30 years.

Question 5: (20%)

A rectangular foundation (1.5 m x 1 m) is located at a depth of 1 m in a two-layered clay soil profile. Borings indicate that softer clay is located at a depth of 1 m from the bottom of the foundation. No ground water table is observed at the site. (see Figure 5.1)

a. Determine the gross allowable load for the foundation with a factor of safety of 3.
b. How would the result the change if the softer soil was placed at the top, i.e., $C_{u,1}$ = 48 kN/m^2 and $C_{u,2}$ = 120 kN/m^2 and the geometry of the foundation and soil profile remains the same.

Hint: For layered soils, the value of bearing capacity factor, N_c, is not a constant. It is a function of c_2/c_1, where c_2 belongs to the bottom layer, and z/B, where z is the depth measured from the bottom of the foundation to the interface of the two clay layers (see Figure 5.2). The ultimate bearing capacity is given as:

$$q_f = c_1 N_c S_c d_c + \gamma D$$

S_c =1.145, d_c=1.4, $FS = \frac{q_{nf}}{q_n}$

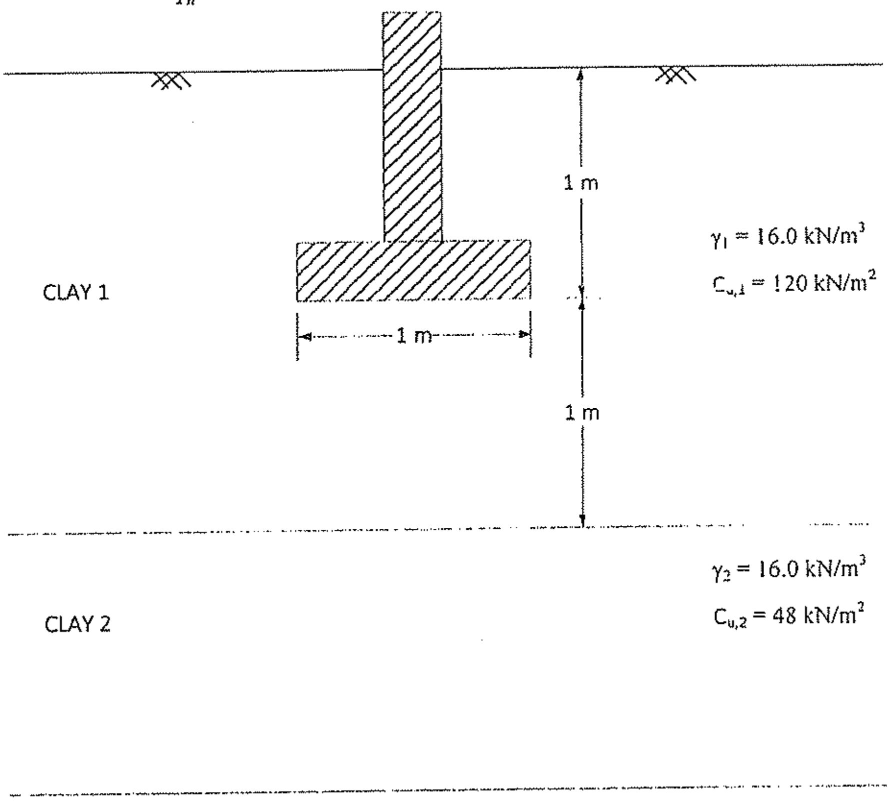


Figure 5.1 Soil profile for rectangular foundation to be built

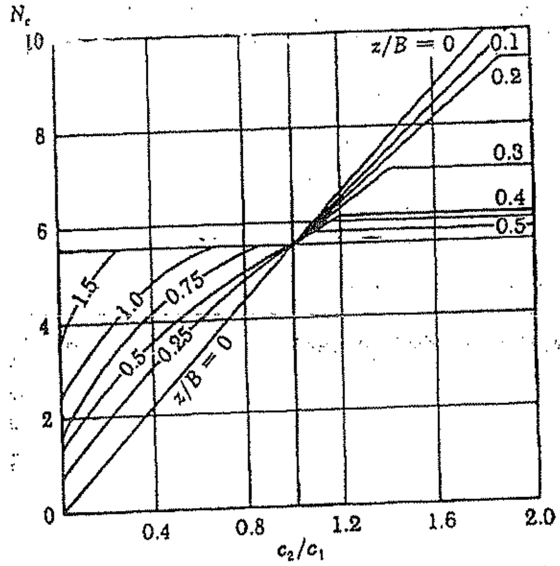
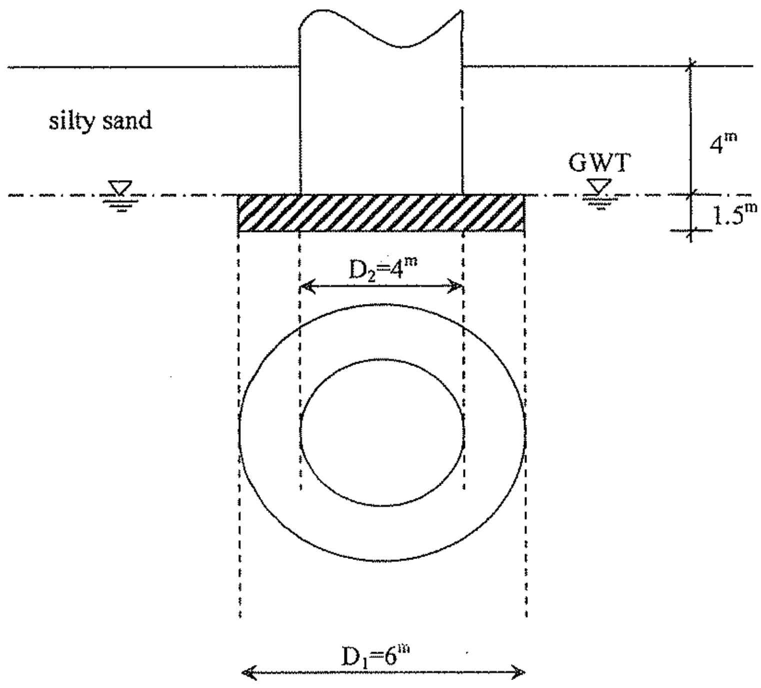


Figure 5.2. Bearing capacity on layered clay soils – q = 0 (Figure is redrawn after Reddy and Srinivasan, 1967)

Regulations:	
Due date:	*November 23, 2009, Monday at 17:00* (Not subject to postpone)
Submission:	You will submit the homework to **Ass. Zeynep Çekinmez (K1-102B)**
Help:	You can ask your questions about the homework to the assistant **during her office hours** listed on the web.
Team:	The homework has to be done individually. Improper collaborations will get zero.
Answers:	The answers to the homework questions will be posted on the bulletin board, next to the soil mechanics laboratory.
Remarks:	Midterm #1 is on 25th of November, Wednesday at 17:40!!!

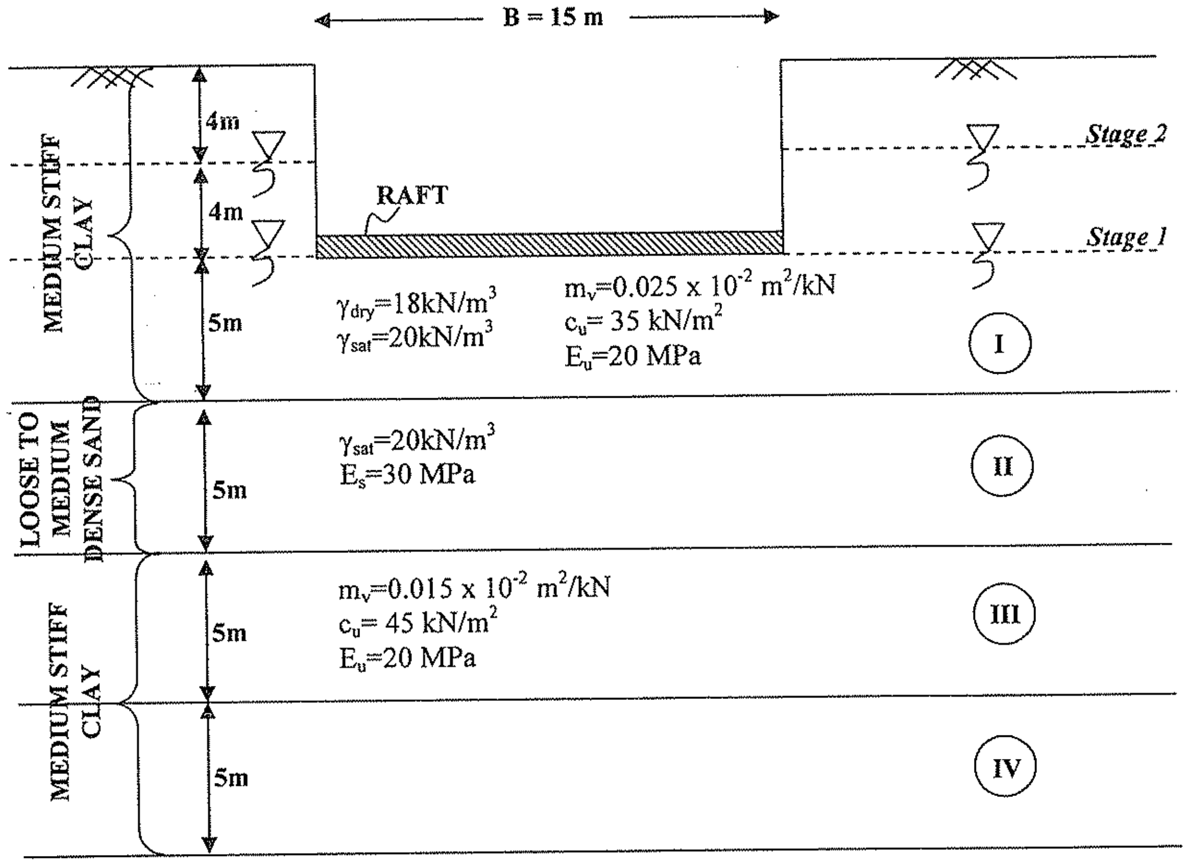
Q1) The section and plan views of a tower structure are given below. Dead weight of the tower excluding the foundation is 4500 kN. The dry and saturated unit weights of the soil are 19 kN/m^3 and 21 kN/m^3, respectively; and the unit weight of concrete is 24 kN/m^3.



(a) Calculate the net foundation presure under the rigid mat.

(b) Assuming the ground water table is at the ground surface, calculate the factor of safety against uplift.

Q2) A 16-storey apartment block is to be constructed at a site. The soil profile consists of a deep clay layer which contains a 5 m thick sand layer. The ground water table is at 4 m depth. The base of the raft under the building is 8 m deep from the ground surface. The profile and the soil properties are shown in the figure below.



The dimensions of the building and the raft are the same (15 m x 30 m). Total weight of the building (dead + live + raft) is 90000 kN.

Initially, water level is at Stage 2. Just after the construction water level is at Stage 1. After long time, water level again rises up to initial level, Stage 2.

Find the net foundation pressure for both Stage 1& Stage 2 and calculate the consolidation settlement of the building. Comment on the results.

Take the Skempton-Bjerrum correction factor $\mu = 0.75$. Consider the compressions of the soil within 20 m distance from the foundation level.

Q3) A machine has the plan dimensions of 8 m x 15 m. It has a 6 m deep basement and applies a gross pressure of 200 kPa. The soil at the site is deep, silty, gravelly sand and GWT lies at 3m depth. Two boreholes BH1 and BH2 were opened at the site and SPT blow numbers are given below. Using the Burland-Burbidge method calculate the expected settlement of the machine house in the long term. The depth of influence of the foundation is given as 5m in the chart provided by the authors. Take γ_{dry} = 17 kN/m^3, γ_{sat}=19 kN/m^3.

Depth (m)	**BH1 N**	**BH2 N**
1.5	15	20
3.0	19	18
4.5	20	16
6.0	20	19
7.5	17	20
9.0	26	24
10.5	22	28
12.0	25	26
13.5	29	32
15.0	31	33

Q4) The standard penetration test results for a sand deposit is given at the attached borehole log sheet below.

(a) Draw the corrected (only for overburden) N vs depth relationship.

(b) A shallow square foundation for a column is to be constructed. It must carry a net vertical load of 1000 kN. Assume that the depth of foundation is 2.0 m and the tolerable settlement is 35 mm. Determine the size of foundation using Peck-Hanson-Thornburn approach.

(c) Determine the total vertical load that 4 x 4 sqm column footing can support permissible settlement of 20 mm.

3

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Sondaj No / *Boring No:* **SK-1**
Sayfa No / *Sheet No* 1/2

SONDAJ LOGU / BORING LOG

PROJE ADI / *PROJECT NAME* : S.GÖKÇEN HAVAALANI / AIRPORT *(BAĞLANTI YOLLARI ve KÖPRÜLER/ CONNECTION ROADS AND BRIDGES)*

MÜHENDİS / *ENGINEER* : Mehmet GÖKÇEER

KOORDİNATLAR / *COORDINATES* Y: 441965.765 X: 4530464.755
SONDAJ KOTU / *ELEVATION* (m) 94.235
SONDAJ DERİNLİĞİ / *BORING DEPTH* (m) : 20.00
SONDAJ MAKİNASI / *DRILLING RIG* ACKER-2
SONDÖR / *FOREMAN* : Rasim ÇİÇEK
BAŞLAMA TARİHİ / *DATE STARTED* : 12.11.2007
BİTİŞ TARİHİ / *DATE COMPLETED* : 13.11.2007
MUHAFAZA BORU / CASING DIŞ ÇAPI / *O.D.* (mm) = 114 İÇ ÇAPI / *I.D.* (mm) : 101 UZUNLUĞU / *LENGTH* (m) : 12.00

YERALTI SUYU DURUMU / *GROUND WATER DATA*			
DERİNLİK *DEPTH (m)*	TARİH *DATE*	SAAT *HOUR*	AÇIKLAMA *REMARKS*

DERİNLİK *DEPTH (m)*	KAROT ve % si *CORE % RECOVERY*	R.Q.D. %	NUMUNE NO *SAMPLE NO.*	NUMUNE DERİNLİĞİ *SAMPLE DEPTH.*	ZEMİN TANIMLAMASI *SOIL DESCRIPTION*	ZEMİN PROFİLİ *SOIL PROFILE*	DAYANIKLILIK *STRENGTH*	AYRIŞMA *WEATHERING*	KIRIK / 30 cm *FRACTURE / 30 cm*	KIRIK AÇISI *FRACTURE ANGLE*	SU KAYBI *WATER LOSS* %	STANDART PENETRASYON DENEYİ Darbe Sayısı 0-15	15-30	30-45	N_{30}	GRAFİK / *GRAPH* 10 20 30 40 50
1																
2			SPT-1	1.50-1.95	SAND DEPOSIT										13	
3			UD-1	3.00-3.25												
			SPT-2	3.25-3.70											11	
4																
5			SPT-3	4.50-4.95											16	
6			UD-2	6.00-6.40												
7			SPT-4	6.40-6.85											9	
8			SPT-5	7.50-7.95											14	
9			UD-3	9.00-9.30												
10			SPT-6	9.30-9.75											8	
11			SPT-7	10.50-10.95											13	
12																
13	70	47	SPT-8	12.00-13.50											17	
14	47	41	SPT-9	13.50-15.00	SANDSTONE										38	
15																

KIVAM DURUMU / *STIFFNESS*		
N = 0 - 2	Çok yumuşak	*V.soft*
N = 3 - 4	Yumuşak	*Soft*
N = 5 - 8	Orta katı	M.stiff
N = 9 - 15	Katı	Stiff
N = 16 - 30	Çok katı	V.stiff
N > 30	Sert	Hard

SIKILIK / *DENSITY*		
N = 0 - 4	Çok gevşek	V.loose
N = 5 - 10	Gevşek	Loose
N = 11 - 30	Orta sıkı	M.dense
N = 31 - 50	Sıkı	Dense
N > 50	Çok sıkı	V.dense

ORANLAR / *PROPORTIONS*		
0 - 10 %	Pek az (Seyrek)	Trace
10 - 20 %	Az	Little
20 - 35 %	Sıfat	Adjective (Or some)
35 - 50 %	Ve	And

KIRIKLAR / 30 cm - *FRACTURES / 30 cm*			
> 1	Seyrek	Wide	(W)
1 - 2	Orta	Moderate	(M)
2 - 10	Sık	Close	(Cl)
10 - 20	Çok sık	Intense	(I)
> 20	Parçalı	Crushed	(Cr)

DAYANIMLILIK / *STRENGTH*		
I	Çok dayanımlı	Very strong
II	Dayanımlı	Strong
III	Orta	Medium
IV	Zayıf	Weak
V	Çok zayıf	Very weak

AYRIŞMA / *WEATHERING*		
I	Taze	Fresh
II	Az ayrışmış	Slightly weathered
III	Orta ayrışmış	Mod. weathered
IV	Çok ayrışmış	Highly weathered
V	Tamamen ayrışmış	Comp. weathered

KAYA KALİTESİ TANIMI / *RQD.*		
0 - 25 %	Çok zayıf	Very poor
25 - 50 %	Zayıf	Poor
50 - 75 %	Orta	Fair
75 - 90 %	İyi	Good
90 - 100 %	Çok iyi	Excellent

UD : Örselenmemiş Numune / Undisturbed Sample
D : Örselenmiş Numune / Disturbed Sample
SPT : Standart Pen Deneyi / Standard Pen. Test
VST : Vane Deneyi / Vane Shear Test
P : Pressiometre Deneyi / Pressuremeter Test
K : Karot Numunesi / Core Sample

4

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PROJE / PROJECT : S.GOKÇEN HAVALIMANI / AIRPORT (BAĞLANTI YOLLARI ve KÖPRÜLER / CONNECTION ROADS and BRIDGES)

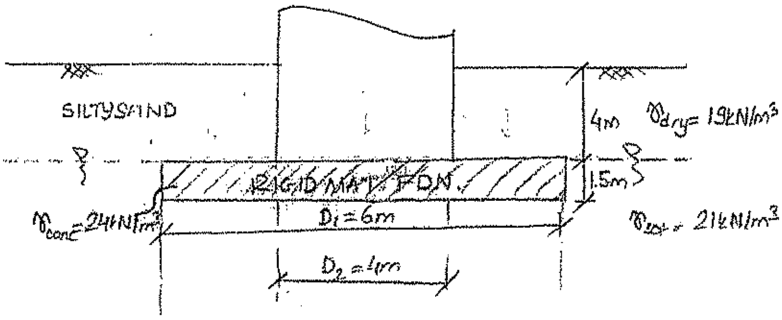
Sondaj No / *Boring No* : **SK-1**
Sayfa No / *Sheet No* : 2/2

DERİNLİK *DEPTH (m)*	KAROT ve % si *CORE % RECOVERY*	R.Q.D. %	NUMUNE NO. *SAMPLE NO.*	NUMUNE DERİNLİĞİ *SAMPLE DEPTH. from to*	ZEMİN TANIMLAMASI *SOIL DESCRIPTION*	ZEMİN PROFİLİ *SOIL PROFILE*	DAYANIKLILIK *STRENGTH*	AYRIŞMA *WEATHERING*	KIRIK / 30 cm *FRACTURE/30 cm.*	KIRIK AÇISI *FRACTURE ANGLE*	SU KAYBI *WATER LOSS %*	Darbe Sayısı *No. of Blows* 0-15	15-30	30-45	N_{30}	GRAFİK / *GRAPH* 10 20 30 40 50
16	27	-	SPT-10	15.00-16.50	SANDSTONE										47	
17	22	-	SPT-11	16.50-18.00											50/14	
18																
19	33	-	SPT-12	18.00-20.00											50/5	
20					20.00											
21					Kuyu Sonu / *End of Borehole*											
22																
23																
24																
25																
26																
27																
28																
29																
30																
31																
32																
33																
34																
35																

CE 366 FOUNDATION ENGINEERING I
2009-10 FALL SEMESTER
– HOMEWORK 1 –

~ SOLUTIONS ~

A.1)



$W_{superstructure} = 4500kN$

(a) $q_{net} = ?$

$$q_{gross} = \frac{\left\{\begin{array}{l}\text{Total weight of structure}\\ \text{-including fdn-}\end{array}\right\} + \left\{\begin{array}{l}\text{Effective weight of soil}\\ \text{on foundation}\end{array}\right\} - \left\{\begin{array}{l}\text{Uplift force}\\ \text{acting on structure}\end{array}\right\}}{\text{Foundation Area}}$$

$$q_{net} = q_{gross} - \sigma'_{v,0}$$

$$q_{gross} = \frac{\underbrace{\left\{4500 + (24)(1.5)(\pi 6^2/4)\right\}}_{5518} + \underbrace{\left\{(4)(19)(\pi/4)\left[6^2 - 4^2\right]\right\}}_{1194} - \underbrace{\left\{(10)(1.5)(\pi)(6^2/4)\right\}}_{424}}{(\pi)(6^2/4)}$$

$q_{gross} = 222.4kPa$

$q_{net} = 222.4 - [(4)(19) + (1.5)(21-10)] = \boxed{130kPa} \Rightarrow$ ANSWER

(b)

$$(F.S)_{uplift} = \frac{\left\{\begin{array}{l}\text{Total weight of structure}\\ \text{-including fdn-}\end{array}\right\} + \left\{\begin{array}{l}\text{Effective weight of soil}\\ \text{on foundation}\end{array}\right\}}{\left\{\begin{array}{l}\text{Uplift force}\\ \text{acting on structure}\end{array}\right\}}$$

$$= \frac{5518 + \overbrace{\left\{(4)(21-10)(\pi/4)\left[6^2 - 4^2\right]\right\}}^{691}}{\underbrace{\left\{424 + (10)(4)(\pi)(4^2/4)\right\}}_{926}} = \boxed{6.70} \Rightarrow \text{ANSWER}$$

(a) q_{net} for stage 1 = ?

$q_{net} = q_{gross} - \sigma'_{v,o}$ → initial effective overburden pressure

$$q_{gross} = \frac{\Sigma W_{fdn+superstructure} + \Sigma W_{soil\,on\,fdn} - F_{uplift}}{\text{Fnd. Area}} = \frac{90000 + 0 - 0}{(15 \times 30)} = 200\,kPa$$

$$q_{net} = 200 - \underbrace{[(4)(18) + (4)(20-9.8)]}_{112.8} = \boxed{87.2\,kPa} \Rightarrow \text{ANSWER}$$

(b) q_{net} for stage 2 = ?

$q_{net} = q_{gross} - \sigma'_{v,o}$

$$q_{gross} = \frac{90000 + 0 - (4)(15)(30)(9.8)}{(15 \times 30)} = 160.8\,kPa$$

$$q_{net} = 160.8 - 112.8 = \boxed{48\,kPa} \Rightarrow \text{ANSWER}$$

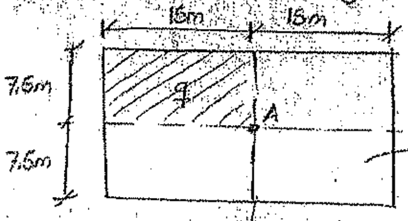
(c) S_{cons} = ?

$S_{oed} = m_v \Delta\sigma' H$

⇒ consolidation settlement proceeds in along time, thus gw level is @ stage 2 ⇒ $q_{net} = 48\,kPa$

⇒ since coefficient of volume compressibility, m_v, depends on σ'_v, by dividing the soil profile into more equal sublayers consolidation settlement is calculated in more accurate way.

⇒ $\Delta\sigma'$ is calculated by influence factor method based on the idea of:



$\Delta\sigma\,@A = \underbrace{4}_{\text{4 equal area}} q I_r = 4(48) I_r = 192 I_r$

★ For sandy soils, $m_v \approx 1/E'$

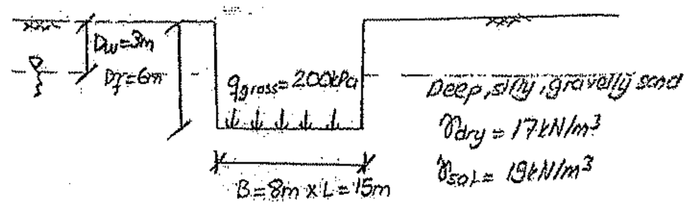
LAYER #	z_i (m)	$m = B/z$	$n = L/z$	I_r	$\Delta\sigma$ (kPa)	m_v (m²/kN)	ΔH (m)	S_{oed} (m)
I	2.5	3	6	0.247	47.42	0.025×10^{-2}	5	0.059275
II	7.5	1	2	0.200	38.40	$1/30000 = 3.3 \times 10^{-5}$	5	0.0064
III	12.5	0.6	1.2	0.142	27.26	0.015×10^{-2}	5	0.020445
IV	17.5	0.43	0.86	0.105	20.16	0.015×10^{-2}	5	0.01512

$\Sigma S_{oed} = 0.10124$

i) S_{oed} corrected by Skempton-Bjerrum correction → $\mu = 0.75$ (given) → is used since consolidation is not proceeded in 1-D but 2-D

$$S_c = \mu S_{oed} = 0.75 \times 10.1\,cm = \boxed{7.6\,cm} \Rightarrow \text{ANSWER}$$

A3)



$z_I = 5m$ (given) $\Rightarrow$ depth of influence of settlement.

① silty sand cor. → if all 3 are satisfied: i) Silty sand exists, ii) For saturated soils, iii) $N_{field} > 15$ $\Big\}$ $N = 15 + \frac{1}{2}(N_f - 15)$

Depth(m)	BH1 (N)	BH2 (N)	Nave	N	σ'_v (kPa)	C_N	N'
1.5	15	20	18	18	25.5	1.94	35
3.0	19	18	19	19	51	1.37	26
4.5	20	16	18	17	64.5	1.22	21
6.0	20	19	20	18	78	1.11	20
7.5	17	20	19	17	91.5	1.02	17
9.0	26	24	25	20	105	0.95	19
10.5	22	28	25	20	118.5	0.90	18
12.0	25	36	31	23	132	0.85	20
13.5	29	32	31	23	145.5	0.81	19
15.0	31	33	32	24	159	0.78	19

$\bar{N}$: average N within $\underset{6m}{(z=D_f)} \le z \le \underset{11m}{(z=D_f+z_I)} \Rightarrow I_c = \frac{1.71}{\bar{N}^{1.4}} = \frac{1.71}{19^{1.4}} = 0.028$

▶ Shape factor, f_s:

$$f_s = \left(\frac{1.25 L/B}{L/B + 0.25}\right)^2 = 1.216$$

▶ No need for f_l since $(H \to \infty) > (z_I = 5m)$; $f_l = 1.00$

▶ Time factor, f_t:

for long time → conservative way; after 30 years say ⇒ $f_t = 1.50$ (for static loading)

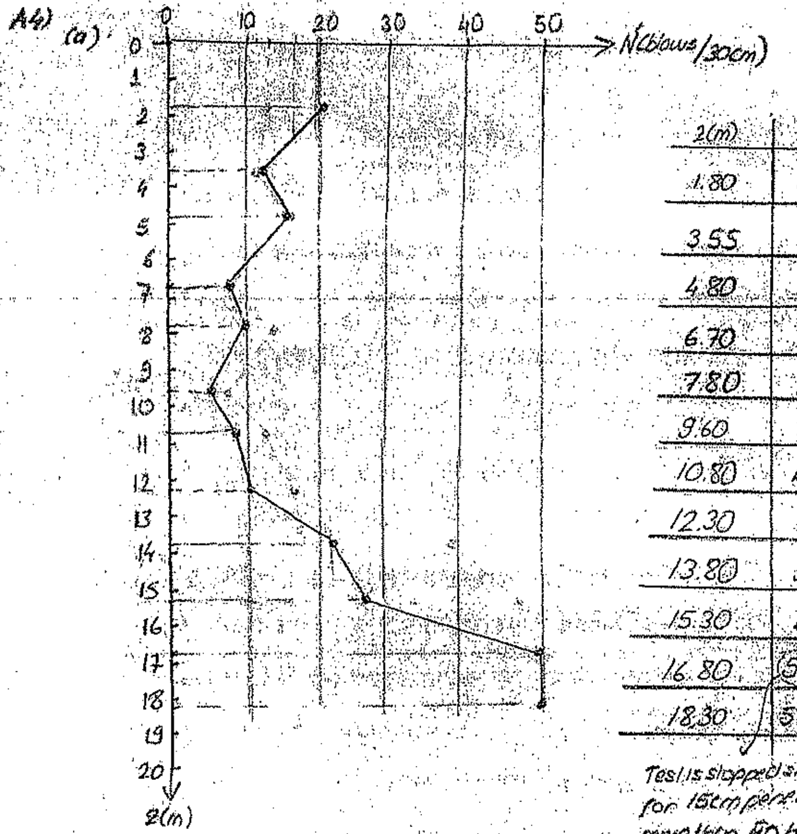
▶ $q_{net} = q_{gross} - \sigma'_{v,0} = 200 - [(3)(17) + (6-3)(19-10)] = 122 kPa$

$\therefore S_i = q_{net} B^{0.7} I_c f_s f_l f_t = (122)(8)^{0.7}(0.028)(1.216)(1.00)(1.50) = \boxed{26.4mm}$ ⇒ ANSWER

74

(8)

A4) (a)



Assume $\gamma_{dry} = \gamma_{sat} = 20\,kN/m^3$ for sand deposit
$\gamma_{dry} = \gamma_{sat} = 21\,kN/m^3$ for sandstone

z(m)	N_{field}	σ_v' (kPa)	C_N	N'
1.80	13	36	1.63	21
3.55	11	71	1.16	13
4.80	16	96	1.00	16
6.70	9	134	0.84	8
7.80	14	156	0.78	11
9.60	8	192	0.71	6
10.80	13	216	0.67	9
12.30	17	246	0.62	11
13.80	38	289.8	0.57	22
15.30	47	321.3	0.55	26
16.80	50/14	352.8		50/14
18.30	50/5	384.3		50/5

Test is stopped since for 15cm penetration more than 50 blows are needed (50 blows have already caused in 14cm penetration)

(b) $V_{net} = 1000\,kN$, $B = L$, $D_f = 2.0m$, $S_{all} = 35mm$

$q = 11 N_{ave} C_w$ for $S_{all} = 25mm$

N_{ave}: within $\boxed{D_f - 0.6B \le z \le D_f + 2.0B}$

C_w: groundwater effect → not out, take 1.0

According to Peck, Hanson & Thornburn (1974):

For $D_f/B = 1.0$ $(B = 2.0m) \rightarrow N_{ave} = 17 \rightarrow \left(q_{soil} = 200\,kPa \times \frac{35}{25} = 280\,kPa\right) > \left(1000/2^2 = 250\,kPa\right)$

For $D_f/B = 0.5$ $(B = 4.0m) \rightarrow N_{ave} = 13 \rightarrow \left(q_{soil} = 150 \times \frac{35}{25} = 210\,kPa\right) \gg \left(1000/4^2 = 62.5\,kPa\right)$

$\boxed{\therefore \text{ use } B = L = 2.0m}$ ⇒ ANSWER

(c) $q_{net,all} = 11 N C_w = \frac{11 \times 20}{25} \times N_{ave} \times C_w^{1.0} = 8.8 N_{ave}$

For $B = L = 4m \Rightarrow N_{ave} = 13$

$q_{net,all} \cong 115\,kPa = \frac{Q_{load}}{4^2}$

$\boxed{Q_{load} = 1840\,kN}$ ⇒ ANSWER

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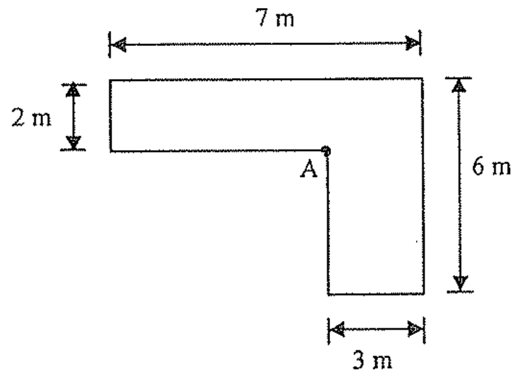
(9)

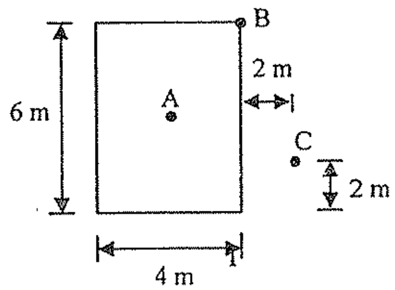
Regulations:

Due date:	*April 08, 2009, Wednesday at 17:00*
Submission:	You will submit the homework to **your own official sections' assistants**; Sec1: Abdullah Sandıkkaya (K1-102) Sec2: Selman Sağlam (K1-247) Sec3: Sevinç Ünsal (K1-115) Sec4: Menzer Pehlivan (K1-115) Sec5: Zeynep Çekinmez (K1-102)
Help:	You can ask your questions about the homework to the assistants during their office hours listed on the web.
Team:	The homework has to be done individually. Improper collaborations will get zero.
Answers:	The answers to the homework questions will be posted on the bulletin board, next to the soil mechanics laboratory.

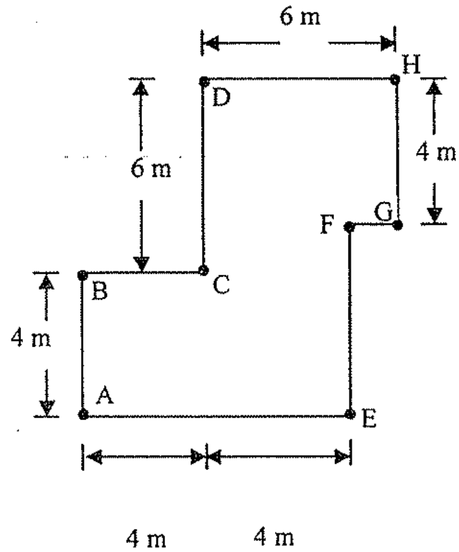
Q1) (10%) Figure 1 shows a plan view of a footing that applies a uniform pressure of 300 kPa on a horizontal ground surface. Calculate the vertical stress component at point A at a depth of 2 m using Newmark's influence chart.



Q2) (10%) A rectangular foundation 4 m x 6 m transmits a stress of 150 kPa on the surface of a soil deposit. Plot the distribution of increases of vertical stresses with depth under points A, B and C up to a depth of 20 m. At which depth is the increase in vertical stress below A less than 10 % of the surface stress?



Q3) (10%) Consider the foundation with the shape illustrated below transmitting a uniform pressure of q to the ground. Use Fadum's charts and estimate the resulting increase in vertical stresses due to q at a depth z =10 m beneath points A, F and H.



Q4) (10%) An SPT test has been performed with a standard sampler at depths 2m, 6m and 10m in a deep silty sand layer. The energy ratio can be assumed as 55 %. The blow counts reported are listed below. The water table is at 2 m from the ground surface. The dry and the saturated unit weights of the soil are 18 kN/m^3 and 20 kN/m^3 respectively. Calculate the corrected $N_{1,60}$ values for each depth.

Depth (m)	SPT – N
2	3 – 5 – 5
6	7 – 8 – 8
10	8 – 10 – 12

Q5) (10%) A vane shear test has been performed with a vane of 150 mm length and 75 mm width. The maximum torque measured during the test is 0.25 kNm. Calculate the undrained shear strength parameter and apply the Bjerrum's correction factor assuming the plasticity index of the soil as 50. Discuss the suitability of the test for this soil profile.

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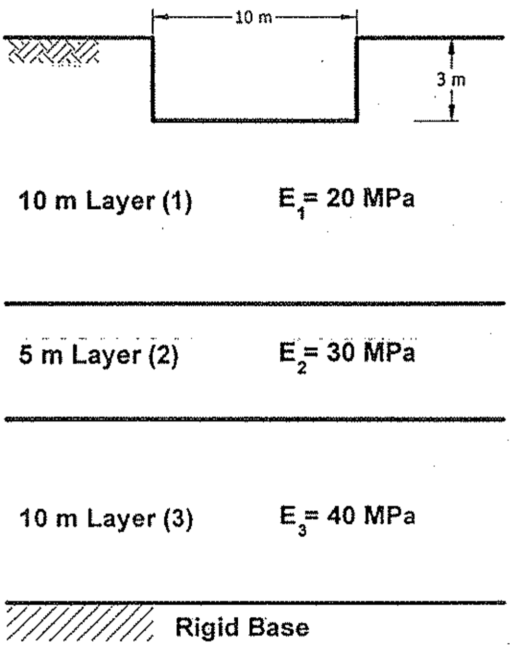
Q6) (10%) The following CPT data is given. Plot the given data with respect to depth and identify different soil layers. Estimate the undrained shear strength of the soil at the depth of 4.0 m assuming the cone factor as N_k=18. The soil is normally consolidated with a unit weight of γ=20kN/m^3 with the water table at 3 m depth from the ground surface.

Depth(m)	q_c(MPa)	q_s(kPa)
1	0.5	20
2	0.7	31
3	1.1	55
4	1.4	42
5	0.8	35
6	0.6	27
7	0.6	27
8	0.8	24
9	0.9	45
10	1.1	22
11	1.5	15
12	2.0	24
13	2.2	20
14	2.6	26
15	2.7	32
16	2.1	27
17	1.8	29
18	1.6	16
19	2.6	21
20	3.0	30

Q7) (5%) A load test is performed on a 0.3m by 0.3m square plate on a dense cohesionless sand (unit weight = 18 kN/m^3). The plate's base is located 0.6 m below the ground surface. If the plate fails at a load of 82 kN, what is the failure load per unit area of the base of a square footing 2m by 2m loaded with its base at the same depth in the same materials?

Q8) (10%) Calculate the immediate average settlement under the structure considering the figure given below. The footing is rectangular with 10 x 40 m and uniformly loaded with a net foundation pressure of 50 kPa. (Hint: use $\delta_i = \mu_0 \cdot \mu_1 \cdot \frac{q \cdot B}{E}$)



Q9) (10%) A 4 m square footing carries a uniform gross pressure of 300 kN/m^2 at a depth of 1.5 m in a sand layer. The saturated unit weight of the sand is 20 kN/m^3 and the unit weight above the water table is 17 kN/m^3. The shear strength parameters are c'= 0 and Ø'= 32°. Determine the factor of safety with respect to shear failure. The water table is at the ground surface.

Q10) (15 %) A foundation 3.5 m square is to be constructed at a depth of 1.2 m in a deep sand deposit, the water table being 3.0 m below the surface. The following uncorrected values of standard penetration resistance were determined at the location:

Depth (m)	0.70	1.35	2.20	2.95	3.65	4.40	5.15	6.00
N_{field}	6	9	10	8	12	13	17	23

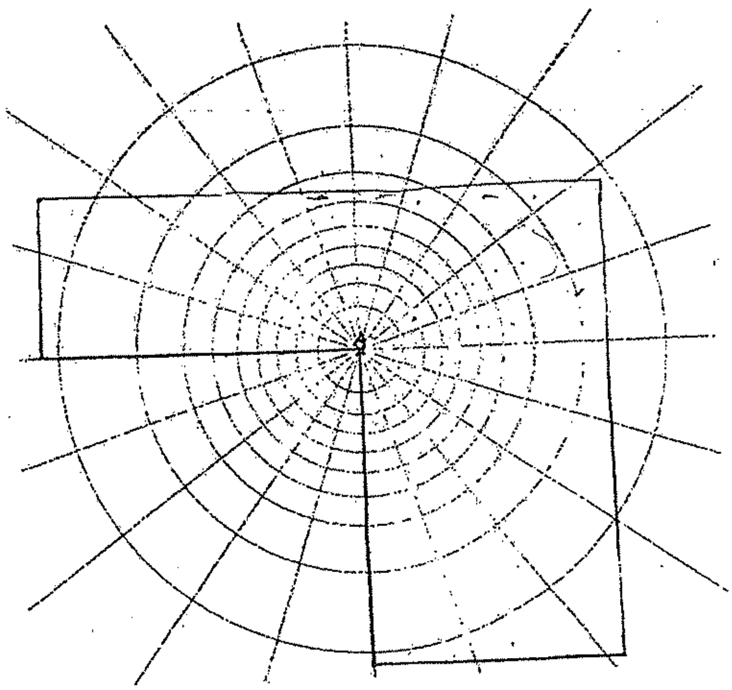
The saturated unit weight of the sand is 20 kN/m^3 and the unit weight above the water table is 17 kN/m^3. The shear strength parameters are c'= 0 and Ø'= 28°. If the settlement is not to exceed 25 mm, determine the allowable bearing capacity according to the following design procedures: (a) Terzaghi expression (1943), (b) Meyerhof (1956), (c) Burland, Broms and De Mello (1977), (d) Peck, Hanson and Thornburn (1974).

~ FOUNDATION ENGINEERING I ~

- HOMEWORK I -

1) a) By using Newmark's Influence Chart:

0,85



Scale line

According to Newmark (1942):

1°/ Length of scale line ≡ 2

2.1cm ≡ 2m

1cm ≡ 0.95m

2°/ Draw the given plan view of footing by scaling it (1cm ≡ 0.95m), by coinciding the asked point "A" with the centre of the influence chart.

3°/ Count the influence areas covered by the scale drawing of the loaded area, N.

$N = 124$

Note that, it should be approximately a unit area at that distance!

4°/ The vertical stress at point "A";

$$\sigma_z = 0.005 N q = 0.005(124)(300) = 186\ kPa$$

ANSWER: $\sigma_z = 186\ kPa$

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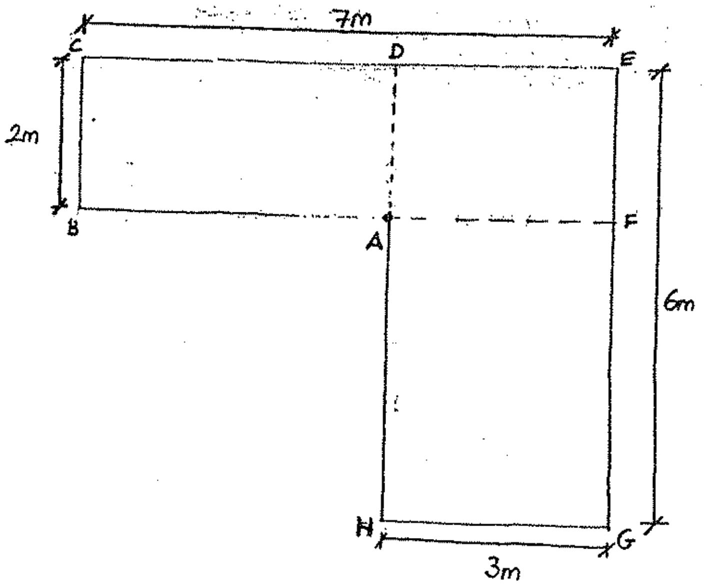
b) By using Influence Factor Method:

$$\sigma_z = q\, I_{r@A} = q\,[I_{ABCD} + I_{ADEF} + I_{AFGH}]$$



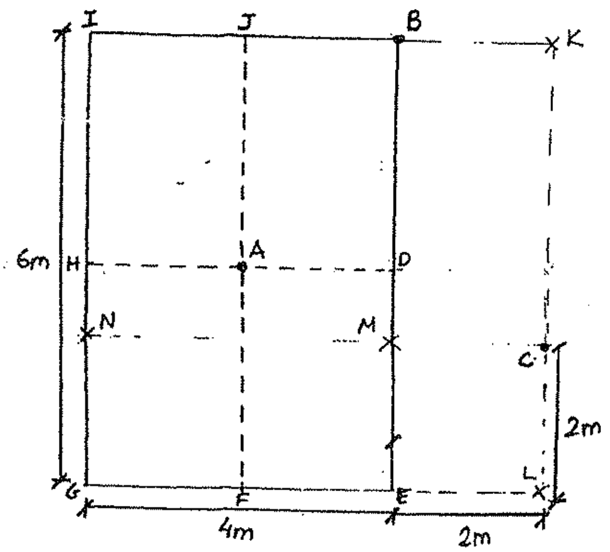
	$m = B/z$	$n = L/z$	I_r
ABCD	1	2	0.201
ADEF	1	1.5	0.196
AFGH	1.5	2	0.222

$$\sigma_z = (300)\,[0.201 + 0.196 + 0.222]$$

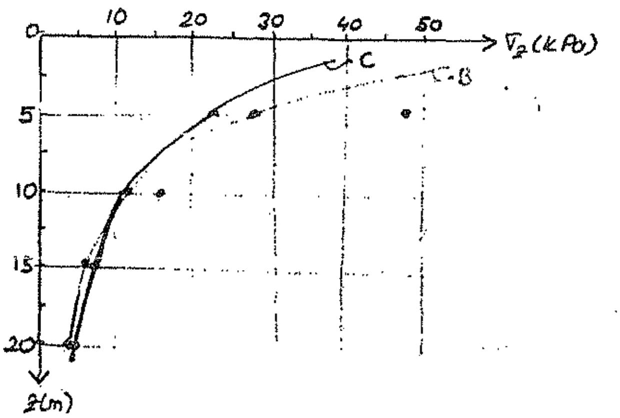
$$= 186\ kPa$$

ANSWER: $\sigma_z = 186\ kPa$

2) $q = 150\ kPa$



ANSWER:



For Point "A":
$\frac{150}{10} = 15\ kPa$

@ $z \cong 10.7m \Rightarrow \sigma_z = 15\ kPa$

For point "A": $\sigma_z = q\,(\overset{=I_r}{I_{AJBD}} + \overset{=I_r}{I_{ADEF}} + \overset{=I_r}{I_{AFGH}} + \overset{=I_r}{I_{AHIJ}})$

z(m)	$m = B/z$	$n = L/z$	I_r	σ_z (kPa)
5	0.4	0.6	0.08	48
10	0.2	0.3	0.027	16.2
15	0.13	0.2	0.012	7.2
20	0.10	0.15	0.006	3.6

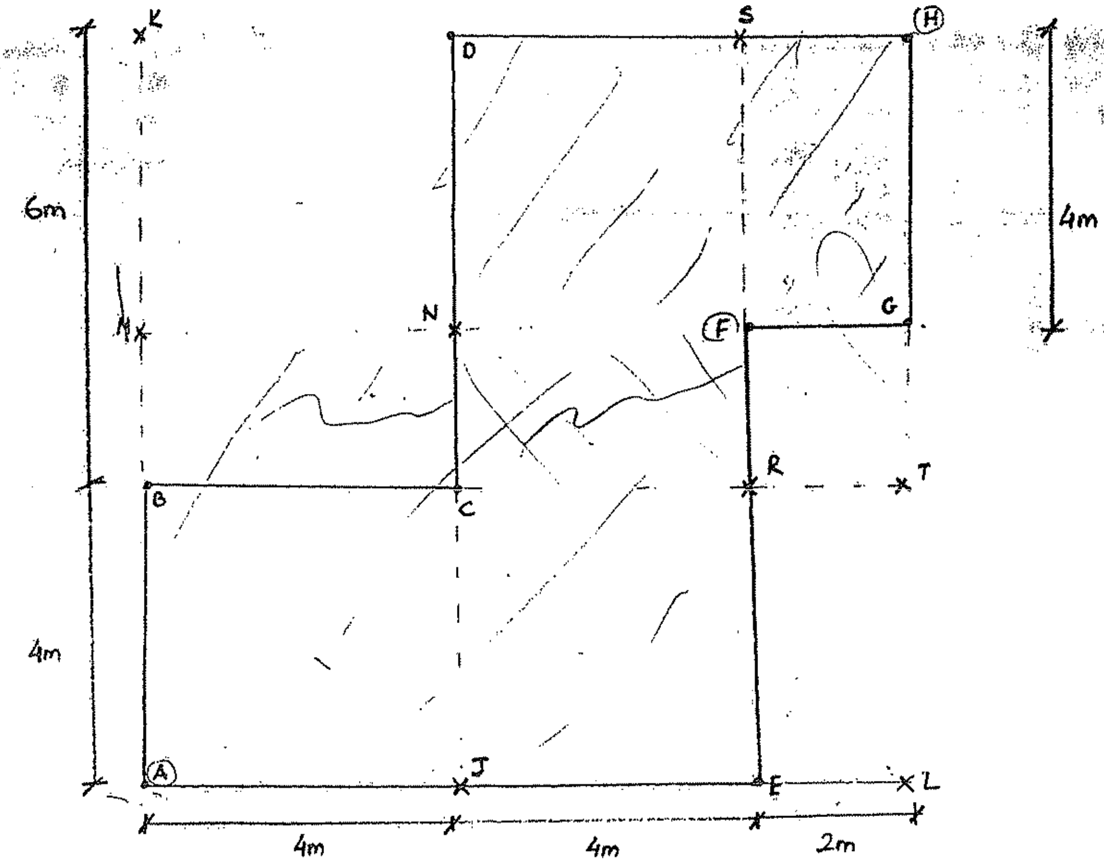
For point "B": $\sigma_z = q\,(\overset{=I_r}{I_{BEGH}})$

z(m)	$m = B/z$	$n = L/z$	I_r	σ_z (kPa)
5	0.8	1.2	0.185	27.8
10	0.4	0.6	0.08	12
15	0.27	0.4	0.043	6.5
20	0.2	0.3	0.027	4.1

For point "C": $\sigma_z = q\,(I_{CLGN} + I_{CNIK} - I_{CLEM} - I_{CMBK})$

z(m)	I_{CLGN}	I_{CNIK}	I_{CLEM}	I_{CMBK}	σ_z (kPa)
5	(0.4)(1.2) 0.117	(0.8)(1.2) 0.185	(0.4)(0.5) 0.06	(0.4)(0.8) 0.088	23.1
10	(0.2)(0.6) 0.045	(0.4)(0.6) 0.08	(0.2)(0.2) 0.0183	(0.2)(0.4) 0.0333	11.0
15	(0.13)(0.4) 0.019	(0.27)(0.5) 0.043	(0.13)(0.13) 0.005	(0.13)(0.27) 0.01	7.1
20	(0.1)(0.3) 0.013	(0.2)(0.3) 0.027	(0.1)(0.1) 0.0046	(0.2)(0.1) 0.0083	4.1

3)



By using Fadum's Charts (pg.: 1.8, Fig. 1.6 in Lecture Notes):

@ point "A": (@ z = 10m)

ANSWER:

$$\sigma_{z@A} = q\,[I_{AEFM} + I_{AJCB} + I_{ALHK} - I_{ALGM} - I_{AJDK}] = 0.128q$$

	AEFM	AJCB	ALHK	ALGM	AJDK
m =	0.6	0.4	1.0	0.6	0.4
n =	0.8	0.4	1.0	1.0	1.0
	0.127	0.06	0.176	0.135	0.10

@ point "F": (@ z = 10m)

ANSWER:

$$\sigma_{z,F} = q\,[I_{FEAM} + I_{FRCN} + I_{FGHS} + I_{FSDN} - I_{FRBM}] = 0.203q$$

	FEAM	FRCN	FGHS	FSDN	FRBM
m =	0.6	0.2	0.2	0.4	0.2
n =	0.8	0.4	0.4	0.4	0.8
	0.127	0.033	0.033	0.06	0.05

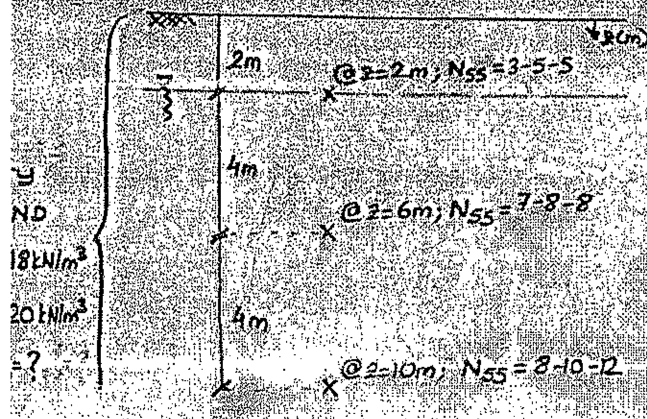
@ point "H": (@ z = 10m)

ANSWER:

$$\sigma_{z,H} = q\,[I_{ALHK} + I_{CTHD} + I'_{FGHS} - I'_{BTHK} - I'_{ELHS}] = 0.126q$$

	ALHK	CTHD	FGHS	BTHK	ELHS
m =	1.0	0.6	0.2	1.0	0.2
n =	1.0	0.6	0.4	0.6	1.0
	0.176	0.107	0.033	0.135	0.055

4)



1°/ Firstly, N_{55} values should be calculated at each given depth by regarding only 2^{nd} & 3^{rd} -15cm- counts, i.e. last 30cm (disregard the first 15cm blow counts at each depth)

z(m)	1^{st} 15cm	2^{nd} 15cm	3rd 15cm	N_{55}
2	~~3~~	5	5	10
6	~~7~~	8	8	16
10	~~8~~	10	12	22

2°/ It does not matter what the question asks, always $\underbrace{\text{silty sand correction}}_{N' = 15 + \frac{1}{2}(N-15)}$ should be done firstly if it is possible (if it satisfies all the following conditions:

i) If silty sand exists in soil profile

ii) If silty sand layer is under GWT

iii) if $N > 15$

$\Bigg\}$ in this question for $z \geq 2m$ all 3 conditions are satisfied.

Thus;

z(m)	N_{55}	N_{55}'	
2	10	10	⇒ (no silty sand correction)
6	16	16 *1	silty sand corrected.
10	22	19	

*1: $15 + \frac{1}{2}(16-15) = 15.5 \Rightarrow 16$

3°/

$$N_{1,60} = \frac{E_m C_B C_S C_R C_N . N}{0.60} \Rightarrow E_m : \text{hammer eficiency} \xrightarrow{\text{given as}} E_m = 0.55$$

page: 2.21 Table 2.2 from Lecture Notes.

C_B : borehole diameter correction $\xrightarrow{\text{no info. assume } 65^{mm} \le B \le 115^{mm}}$ $C_B = 1.00$

C_S : sampler correction $\xrightarrow{\text{standard sampler}}$ $C_S = 1.00$

C_R : rod length correction $\xrightarrow{\text{since } z_{max}=10m \text{ so; rod length} \geq 12m}$ $C_R = 1.00$

C_N : overburden correction

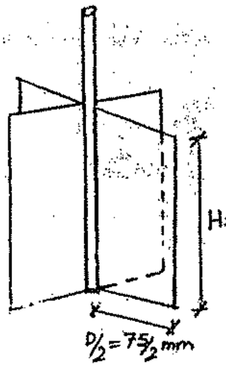
$$\left\{ C_N = 9.78 \sqrt{\frac{1}{\sigma_v' \text{ in kPa}}} \right\} \leqslant 2.0$$

z(m)	N_{55}	E_m	C_B	C_S	C_R	σ_v'(kPa)	C_N	$N_{1,60}$
2	10	0.55	1.00	1.00	1.00	36	1.63	15//
6	16					76 *2	1.12	16//
10	19					116	0.91	16//

ANSWER

*2: (2)(18)+(4)(20-10)=76kPa

5)



~VANE SHEAR TEST~

$T_{max} = 0.25kN.m$

$c_{u,field} = ?$

$c_{u,corr} = ?$ (if P.I.= 50)

__From equilibrium:__

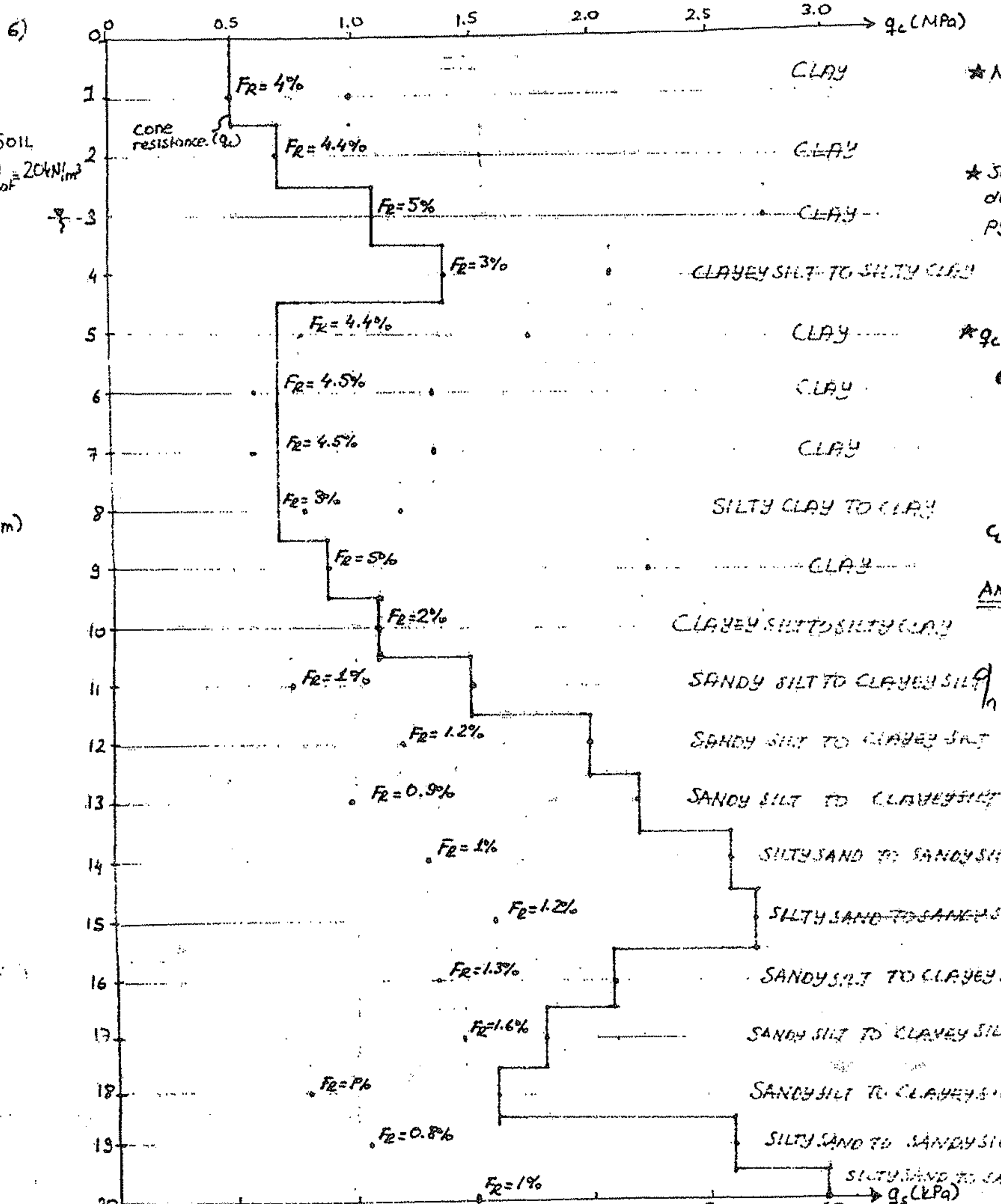
$$c_{u,field} = \frac{T_{max}}{\pi D^2\left(\frac{H}{2}+\frac{D}{6}\right)} = \frac{0.25*10^3}{\pi(75)^2\left(\frac{150}{2}+\frac{75}{6}\right)} * 10^6 = \underline{\underline{161.7\,kPa}} \rightarrow \text{ANSWER for } c_{u,f}$$

(10^3: to convert m → mm; 10^6: to convert /mm² to /m²)

$$c_{u,corr} = \underbrace{\lambda}_{\substack{\text{From pg.: 2.32}\\ \text{Fig.: 2.16 (b)}\\ \text{in Lec. notes}\\ \text{for PI}=50 \rightarrow \lambda = 0.78}} \cdot c_{u,field} = 0.78\,(161.7) = \underline{\underline{126.1\,kPa}} \rightarrow \text{ANSWER for } c_{u,corr}$$

$\frac{q_s}{q_c} \times 100$

6)



★ Note that;

$F_R = \frac{f_{sc}}{q_c} \times 100\%$

★ Soiltype can be determined from; Pg.: 2.31 Fig. 2.15

$c_u = \frac{q_c - P_o}{N_k}$

★ $q_c = c_u N_k + P_o$

@ z=4m ⇒ q_c = 1.4 = 1400

$N_k = 18$

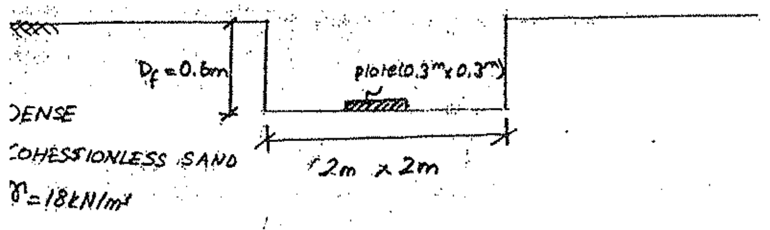
$P_o = (20)(3) + (20-9.8)$

$= 70.2\,kPa$

$c_u = \frac{1400 - 70.2}{18} = 73.$

__ANSWER: c_u = 73.9__

$q_{nt} = c_u N_c - \gamma$



$P_{u,plate} = 82kN$

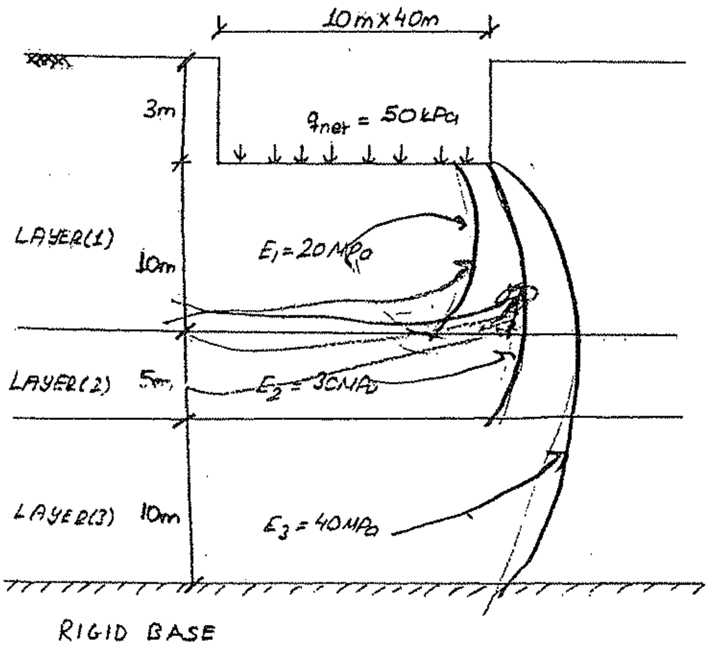
$P_{u,ftg(2m \times 2m)} = ?$

Since it is a cohesionless, sand soil:

$$q_{u,ftg} = q_{u,plate} \frac{B_F}{B_p}$$

$$q_{u,ftg} = \frac{(82)}{(0.3 \times 0.3)} \times \frac{2}{0.3} \Rightarrow q_{u,ftg} = 6074 \, kN/m^2$$

<u>ANSWER:</u> $q_{u,ftg} = 6074 kN/m^2$



$\delta_{i,ave} = ?$

$\delta_i = \mu_0 \mu_1 \frac{qB}{E}$ → $D_f \neq 0$ ✓ ; → assume $\nu = 0.5$ ✓

<u>From Figure 3.3 / page: 3.7 in Lect. Notes:</u>

$D/B = 3/10 = 0.3 \Rightarrow \mu_0 = 0.95$

$\left. \begin{array}{l} H/B = H/10 \\ L/B = 40/10 = 4 \end{array} \right\} \mu_1$

$$\mu_0 \cdot \mu_{1,1} \frac{qB}{E_1} + \mu_0 \cdot \mu_{1,2} \cdot \frac{qB}{E_2} + \mu_0 \cdot \mu_{1,3} \cdot \frac{qB}{E_3} - \mu_0 \cdot \mu_{1,1} \frac{qB}{E_2} \frac{qB}{} - \mu_0 \mu_{1,} \frac{qB}{E_3}$$

(1) <u>Layer (1) with H=10m:</u>

$H/B = 1 \Rightarrow \mu_1 = 0.33 \Rightarrow \delta_{i,1} = (0.95)(0.33)(50)(10)/20 = +7.84mm$

(2) <u>Layer (2) with H=15m:</u>

$H/B = 1.5 \Rightarrow \mu_1 = 0.58 \Rightarrow \delta_{i,2} = (0.95)(0.58)(50)(10)/30 = +9.18mm$

(3) <u>Layer (2) with H=10m:</u>

$H/B = 1 \Rightarrow \mu_1 = 0.33 \Rightarrow \delta_{i,3} = (0.95)(0.33)(50)(10)/30 = -5.23mm$

(4) <u>Layer (3) with H=25m:</u>

$H/B = 2.5 \Rightarrow \mu_1 = 0.75 \Rightarrow \delta_{i,4} = (0.95)(0.75)(50)(10)/40 = +8.91mm$

(5) <u>Layer (3) with H=15m:</u>

$H/B = 1.5 \Rightarrow \mu_1 = 0.58 \Rightarrow \delta_{i,5} = (0.95)(0.58)(50)(10)/40 = -6.89mm$

By assuming linear elasticity, superposition is applicable, such that:

$\delta_{i,ave} = \delta_{i,1} + \delta_{i,2} - \delta_{i,3} + \delta_{i,4} - \delta_{i,5}$

$= 13.81mm \approx 1.4cm$

<u>ANSWER:</u> $\delta_{i,ave} = 13.8 \, mm$

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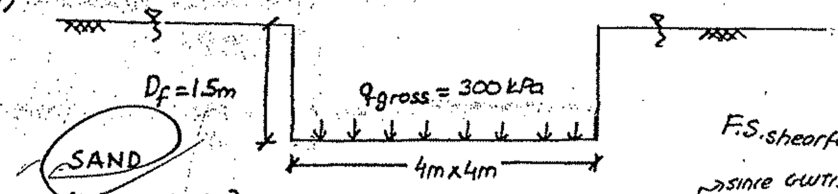
9)



$\gamma_{sat} = 20\,kN/m^3$

$\gamma_{dry/wet} = 17\,kN/m^3$

$c' = 0$

$\phi' = 32°$

$q = 18,75$

F.S.shearfailure = ?

$$F.S._{shearfailure} = \frac{q_{ult,net}}{q_{net}}$$

$$\frac{q_{nt}}{q_n} = \frac{0,4\,\phi\,\gamma\,N_\gamma\,\frac{N}{24}\,1{,}5\,\gamma}{q - \gamma D}$$

10,2 10ʲ

since GWT is @ ground surface

$$\Rightarrow q_{ult,gross} = \frac{1}{2}\gamma' B N_\gamma S_\gamma + \gamma' D N_q S_q$$

$$= \frac{1}{2}(20-9.8)(4)(24)(0.8) + (20-9.8)(1.5)(24)(1.0)$$

$$= 759\,kPa$$

From Fig.4.3, pg.: 4.3 in lecture notes:

since $\phi' = 32°$ } $N_\gamma = 24$, $N_q = 24$

From Table 4.1, pg.: 4.5 in Lect. notes:

since square fdn ⇒ } $S_\gamma = 0.8$, $S_q = 1.0$

$$q_{ult,net} = q_{u,gross} - \sigma_v'$$

$$= 759 - (1,5)(20-9.8)$$

$$= 743.7\,kPa \checkmark$$

$$\Rightarrow q_{net} = q_{gross} - \sigma_v'$$

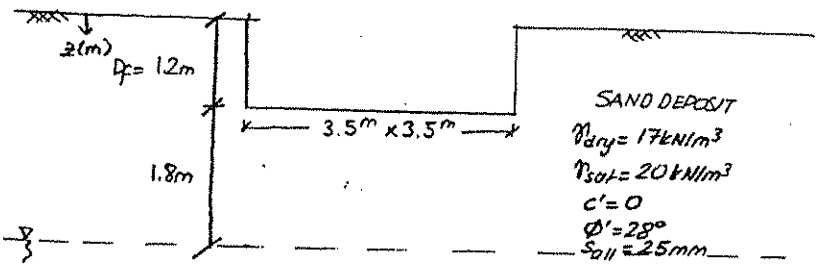
$$= 300 - (1.5)(20-9.8)$$

$$= 284.7\,kPa$$

Thus; $F.S._{shearfailure} = \frac{743.7}{284.7} = 2.61 > 2.5$ OK ✓

ANSWER: $F.S._{shear\ failure} = 2.61$ ✓

10)



ASSUME: $z \geq 6m \Leftrightarrow N_{field} = 23$

1°) SPT Data Table:

Assume $E_m = 0.45$ (since in Turkey generally donut-cathead system is used (Table 2.1, pg. 2.21 in lecture notes))

	DEPTH(m)	N_{field} (45)	σ_v'(kPa)	C_N	N'_{45} *1	N'_{55} *2	N'_{50} *3
1.075	0.70	6	11.9	2.00	12	10	11
1.775	1.35	9	22.95	2.00	18	15	16
2.575	2.20	10	37.4	1.60	16	13	14
3.3	2.95	8	50.15	1.38	11	9	10
4.075	3.65	12	57.63	1.29	16	13	14
4.775	4.40	13	65.28	1.21	16	13	14
5.575	5.15	17	72.93	1.15	20	16	18
	6.00	23	81.60	1.08	25	21	23

*1: $N' = N_{field} \times C_N$

*2: $N'_{55} = N'_{45} \times \frac{45}{55}$

*3: $N'_{50} = N'_{45} \times \frac{45}{50}$

(22)

q_{all} by Terzaghi expression etc.:

For sands $\Rightarrow$ $q_{ult,gross} = \frac{1}{2}\gamma B N_\gamma S_\gamma + \gamma_{dry} D N_q S_q$

$$= \frac{1}{2}(17)(3.5)(13)(0.8) + (17)(1.2)(15)(1.0)$$

$$= 615.4\,kPa$$

$$q_{ult,net} = 615.4 - \sigma'_{vo}$$

$$= 615.4 - (17)(1.2) = 595\,kPa$$

$$q_{all,net} = \frac{q_{ult,net}}{F.S.}$$

Assume $F.S. = 2.5$;

$$q_{all,net} = 238\,kPa$$

ANSWER (a): $q_{a,net} = 238\,kPa$

(b) q_{all} by Meyerhof (1956):

For ($B = 3.5^m > 1.22m$) $\rightarrow$ wide fdns:

$$q_{net,all} = 7.99\bar{N}'_{55}\left(\frac{3.28B + 1}{3.28B}\right)^2$$

$\bar{N}'_{55}$ is taken between $\underbrace{(D_f - 0.5B)}_{0} \le z \le \underbrace{(D_f + 2B)}_{8.2m}$

?

weighted average $\bar{N}_{55} = \dfrac{(1.025)(10) + (0.75)(15) + (0.8)(13) + (0.725)(9) + (1.6325)(13) + (0.75)(13) + (0.8)(16) + (2.625)(21)}{8.2}$?

$$= 17$$

$$q_{all,net} = 7.99(17)\left(\frac{(3.28)(3.5)+1}{(3.28)(3.5)}\right)^2 = 161\,kPa$$

ANSWER (b): $q_{a,net} = 161\,kPa$

(c) q_{all} by Burland, Broms and De Mello (1977):

$\Rightarrow$ Assume sand deposit is medium dense.

From figure 4.10, pg.: 4.16 in lecture notes:

For $B = 3.5m \Rightarrow S/q = 0.12 \xrightarrow{\text{for } S_{all} = 25mm} q = 208\,kPa$

ANSWER (c): $q = 208\,kPa$

(d) q_{all} by Peck, Hanson and Thornburn (1974):

$q_{n,all} = 11\bar{N}'_{50}C_w$ (kN/m²) for $S_{all} = 25mm$

$$C_w = 0.5 + 0.5\frac{D_w}{D_f + B} = 0.5 + 0.5\frac{3}{1.2+3.5} = 0.819$$

?

$\bar{N}'_{50} = \dfrac{(1.025)(11) + (0.75)(16) + (0.8)(14) + (0.725)(10) + (1.6325)(15) + (0.75)(15) + (0.8)(18) + (2.625)(23)}{8.2}$

? $= 18 \longrightarrow q_{all,net} = 11(18)(0.819) = 162\,kPa$

ANSWER (d): $q_{a,net} = 162\,kPa$

Middle East Technical University
Civil Engineering Department
Geotechnical Engineering Division

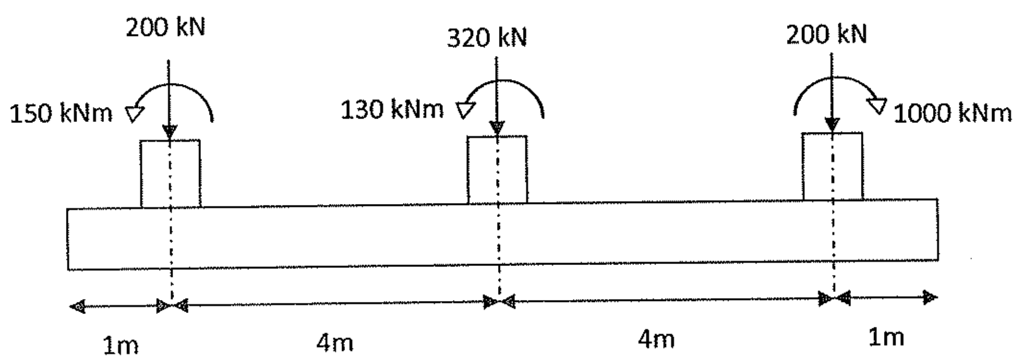
CE 366-FE1

(www.ce.metu.edu.tr/~ce366)
Fall' 2009-2010
Homework2

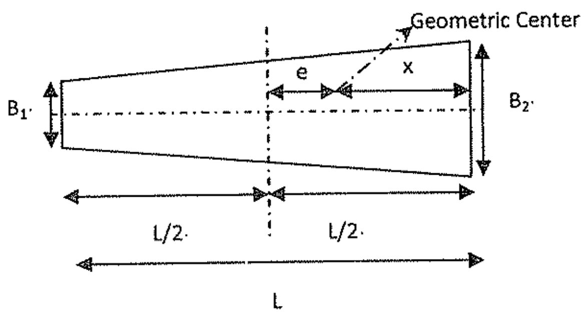
Regulations:	
Due date:	*December 25, 2009, Friday at 17:00* (Not subject to postpone)
Submission:	You will submit the homework to **Ass. Zeynep Çekinmez (K1-102B)**
Help:	You can ask your questions about the homework to the assistant **<u>during her office hours</u>** listed on the web.
Team:	The homework has to be done individually. Improper collaborations will get zero.
Answers:	The answers to the homework questions will be posted on the bulletin board, next to the soil mechanics laboratory.
Remarks:	Midterm #2 is on 29th of December, Tuesday at 17:40 in DR-1 !!!

Q1) For the foundation given in the figure below. Loads include the weight of the foundation.

B= 1m.



<u>Remember</u>:



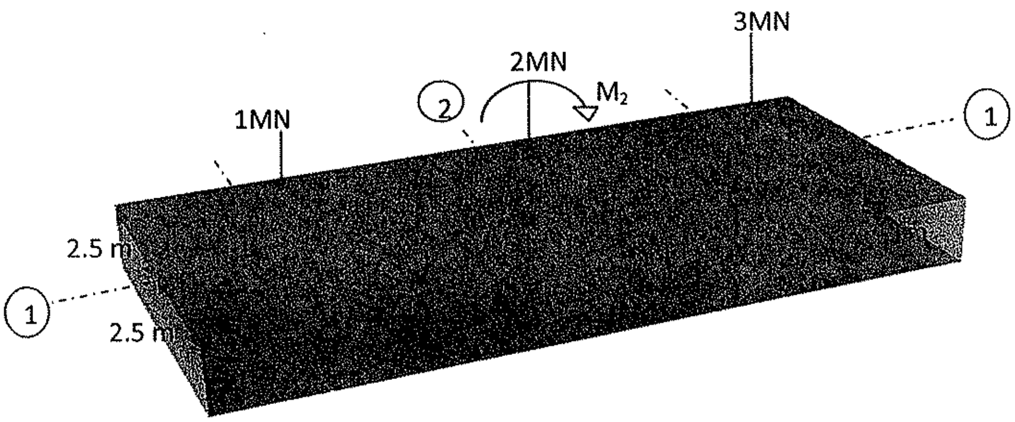
$$x = \frac{L}{3}\frac{B_2+2B_1}{B_1+B_2} = \frac{L}{2} - e$$

(a) Find eccentricity

(b) Assuming no area limitations, modify the foundation to obtain uniform contact stress distribution. Keep width of the footing constant (B=1m). Show what you propose on the figure, with dimensions clearly indicated.

(c) Assuming limitations are imposed due to area restrictions, design a trapezoidal footing (B_1 and B_2) such that a uniform soil pressure of q_{net}= 36 kPa acts beneath the footing. Keep the length of the footing constant (L=10m, as in the figure), such that the given figure will still be the side view of your final design. Present your design as a top view drawing with dimensions clearly indicated.

Q2)



Assume the combined footing is rigid.

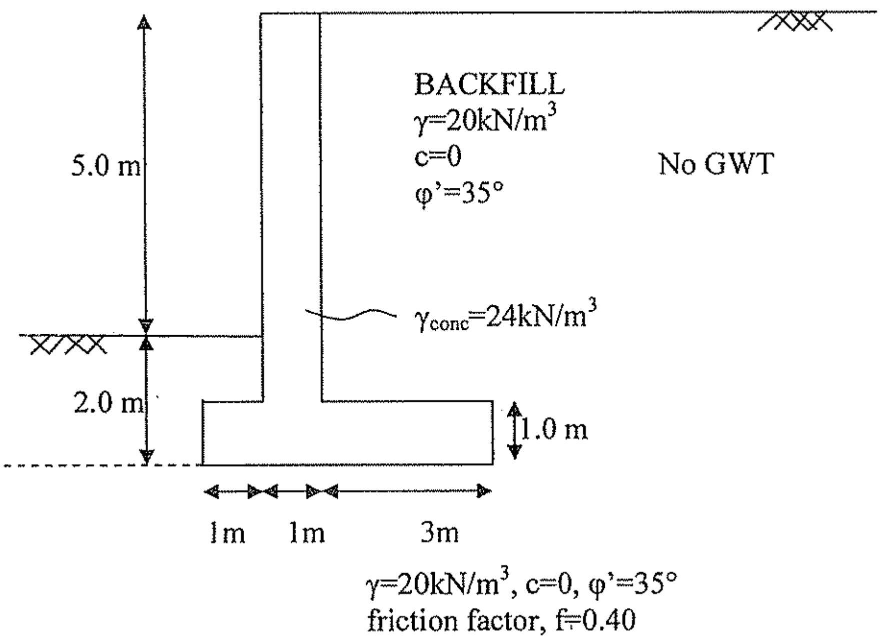
(a) Draw the stress distribution applied to the soil by the footing, for M_2=0.

(b) Draw the stress distribution for M_2= 8MNm.

Q3)

(a) Determine the factor of safety against base sliding and overturning for the retaining wall given below.

(b) Are the calculated factors of safety satisfactory? If not, suggest a modification to the design, for each unsatisfied criteria (if there is a danger of both sliding and overturning, suggest two separate modifications).



Q4) Following data is given for the design of an extensible reinforced earth wall of height 5.2m.

Reinforcing elements (steel strips)

Vertical spacing = 0.65m

Horizontal spacing = 1.2m

Cross-section = 65mm X 3mm

Ultimate strength = 340 N/mm^2

Friction angle between soil and reinforcing element = 35°

Length of the reinforcing element = 5m

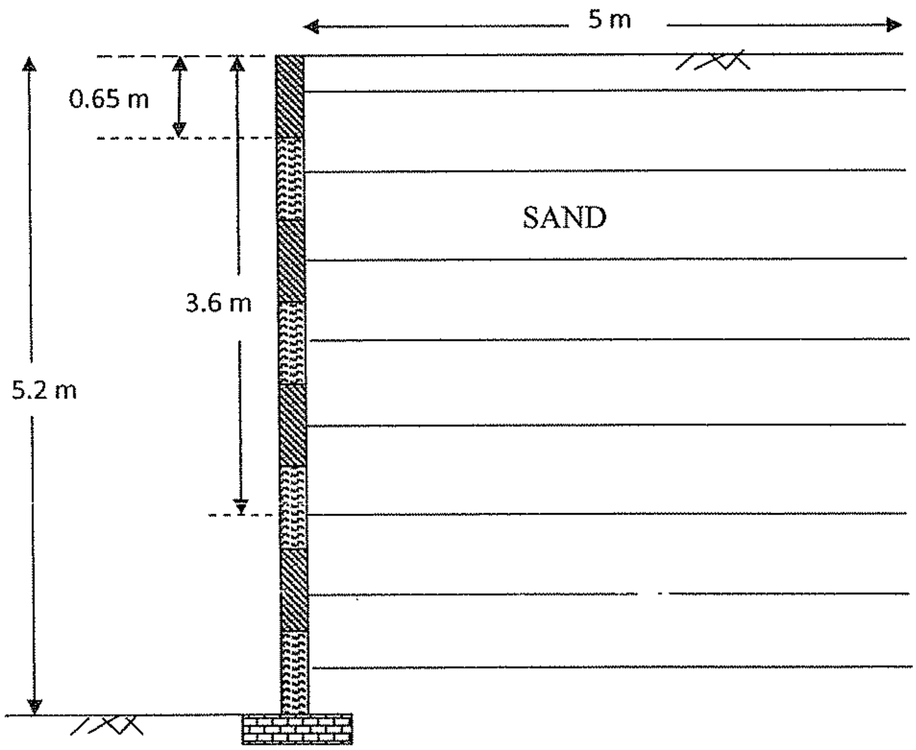
Fill material (sand)

$c=0$, $\varphi=30°$, $\gamma=18 kN/m^3$

Determine:

(a) the F.S. against bond failure and

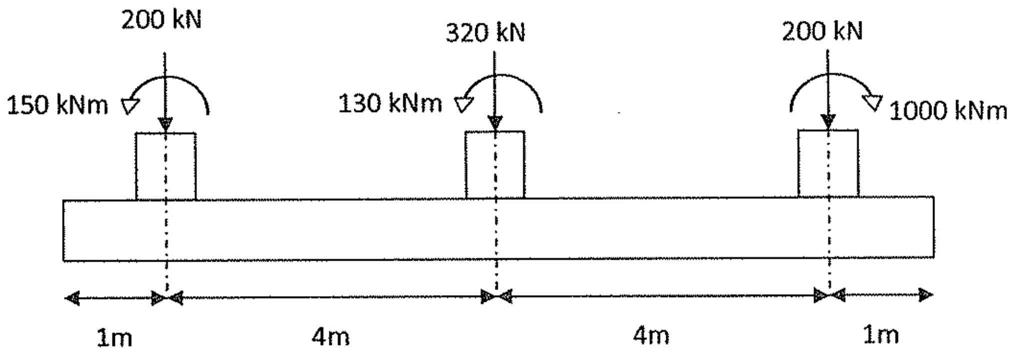
(b) the F.S. against tensile failure for an element 3.6m below the top of the wall, which is 5m in length.



CE 366 FOUNDATION ENGINEERING
2009-10 FALL SEMESTER
SOLUTIONS FOR HOMEWORK II

Question 1 :

Consider the foundation given in the figure below. Loads include the weight of the foundation. B= 1m.



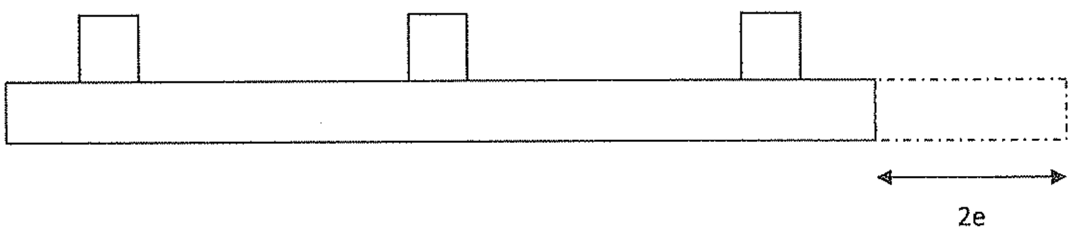
a) Find eccentricity

ΣM=-150-130+1000= 720 kNm ΣQ= 200+320+200=720

e=720/720= 1.0m

b) Assuming no area limitations, modify the foundation to obtain uniform contact stress distribution. Keep width of the footing constant (B=1m). Show what you propose on the figure, with dimensions clearly indicated.

Extend the footing by 2e as shown below.



c) Assuming limitations are imposed due to area restrictions, design a trapezoidal footing such that a uniform soil pressure of q_{net}= 36 kPa acts beneath the footing. Keep the length of the footing constant (L=10m, as in the figure). Present your design as a top view drawing with dimensions clearly indicated.

$$A = \frac{\Sigma Q}{q} = \frac{200 + 320 + 200}{36} = 20m^2$$

$$A = \frac{L}{2}(B_1 + B_2)$$

$$20 = \frac{10}{2}(B_1 + B_2)$$

$$4 = B_1 + B_2$$

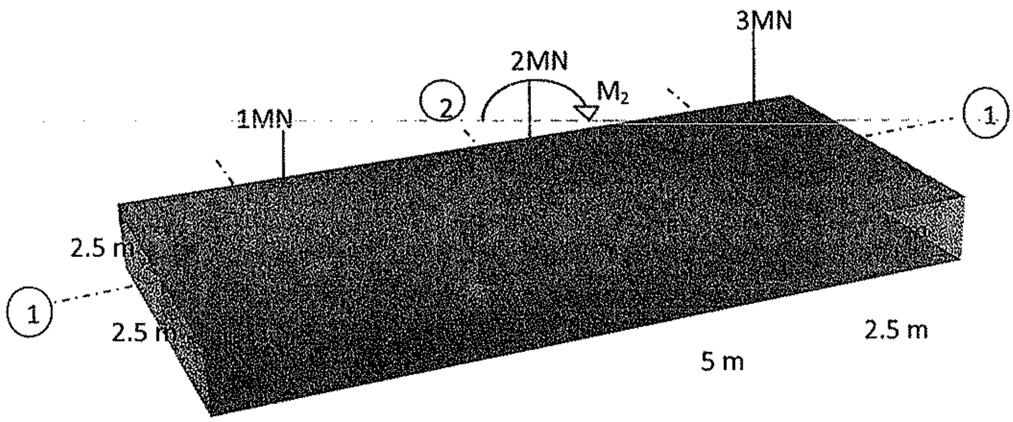
$$\frac{L}{3} \times \frac{B_2 + 2B_1}{B_1 + B_2} = \frac{L}{2} - e$$

$$\frac{10}{3} \times \frac{B_2 + 2 \times (4 - B_2)}{4} = \frac{10}{2} - 1$$

B_2= 3.2 m

B_1= 0.8 m

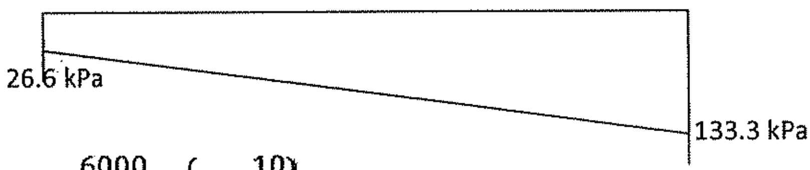
Question 2 :



Assume the combined footing is rigid.

a) Draw the stress distribution applied to the soil by the footing, for M_2=0.

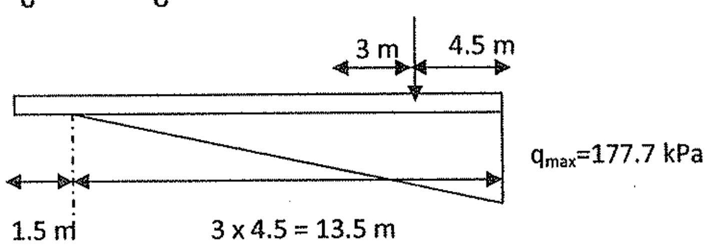
ΣF_v= 6 MN ΣM= 10 MN   e= 1.667 m   Area= 5*15= 75m^2



$$q = \frac{6000}{75} \times \left\{1 \pm \frac{10}{15}\right\}$$

b) Draw the stress disribution for M_2= 8MNm.

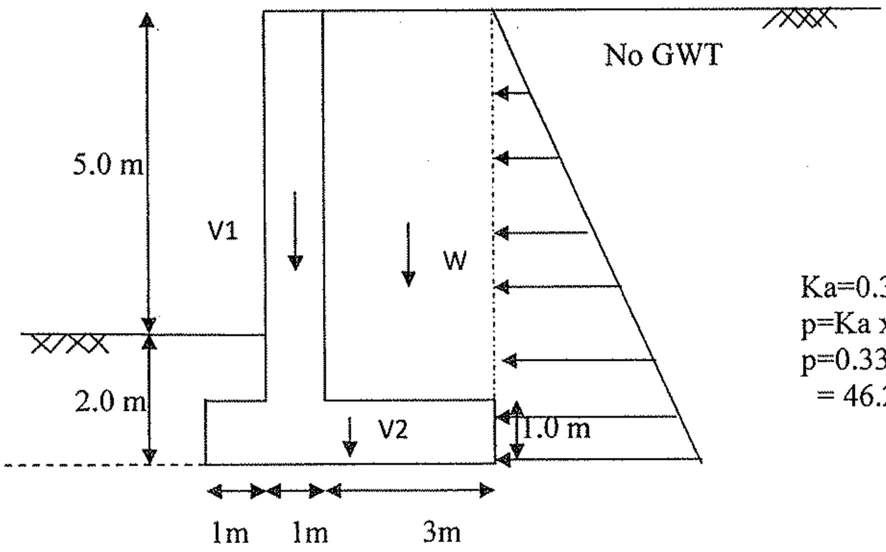
$$e = \frac{18}{6} = 3 > \frac{15}{6}$$



$$\frac{13.5 \times 5 \times q}{2} = 6\ MN$$
$$q = 177.8\ kPa$$

Question 3 :

Determine **(a)** the factor of safety against base sliding and overturning for the retaining wall given below. **(b)** Are the calculated factors of safety satisfactory? If not, suggest a modification to the design, for each unsatisfied criteria (if there is a danger of both sliding and overturning, suggest two separate modifications).



Ka=0.33
p=Ka x H x γ
p=0.33 x 7 x 20
= 46.2

γ=20kN/m^3, c=0, φ'=30°
friction factor, f=0.40

Force No	Force (kN/m)	Moment Arm	Moment (kNm/m)
V1	1 x 5 x 24 = 120	3.5	420
V2	5 x 24 x 1 = 120	2.5	300
W	3 x 6 x 20 = 360	3.5	1260
PA	0.5 x 46.2 x 7 = 161.7	7/3	377.3

$$FS\ overturning = \frac{420 + 300 + 1260}{377.3} = 5.24$$

$$FS\ sliding = \frac{600 \times 0.40}{377.3} = 1.48$$

An alternative solution for increase the FS is designing a base key.

Question 4 :

Following data is given for the design of an extensible reinforced earth wall of height 5.2m.

Reinforcing elements (steel strips)

Vertical spacing = 0.65m Horizontal spacing = 1.2m

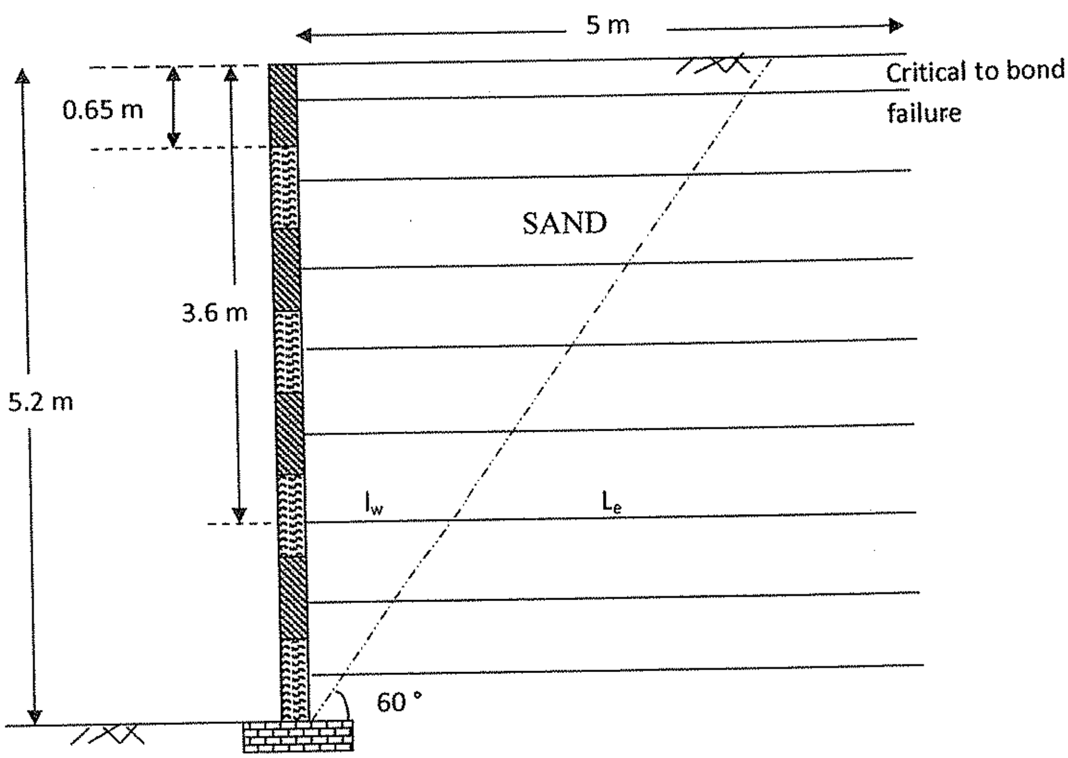
Cross-section = 65mm X 3mm Ultimate strength = 340 N/mm^2

Friction angle between soil and reinforcing element = 35°

Length of the reinforcing element = 5m

Fill material (sand) c=0, ø =30°, γ=18kN/m^3

Determine (a) the F.S. against bond failure and (b) the F.S. against tensile failure for an element 3.6m below the top of the wall, which is 5m in length.



ø =30°, $\theta = 45+30/2 = 60$ $K_a = \frac{1-\sin 30}{1+\sin 30} = 0.33$

a) $\tan 30 = \frac{l_w}{5.2-.65/2}$ => $l_w = 2.81m$ $l_e = 5-2.81 = 2.19m$

Q6) A mass housing construction requires a retaining structure to be built along its northern boundary, where two blocks will be residing nearby. You are provided the dimensions of the retaining structure for the preliminary design stage, as well as the results of the soil investigations. Your task is to investigate the stability of the retaining structure by considering overturning, sliding effects and bearing capacity failure. Assume that soil-wall interface is frictionless and the amount of lateral movement of the soil mass has fully mobilized the active earth pressure condition. Allowable bearing capacity is evaluated as $\square_{all}$ = 200 kPa. Do not consider the passive resistance of the 1m thick soil at the downhill section. Neglect the vertical component of the active earth pressure in your computations. Following these steps will guide you through the solution process:

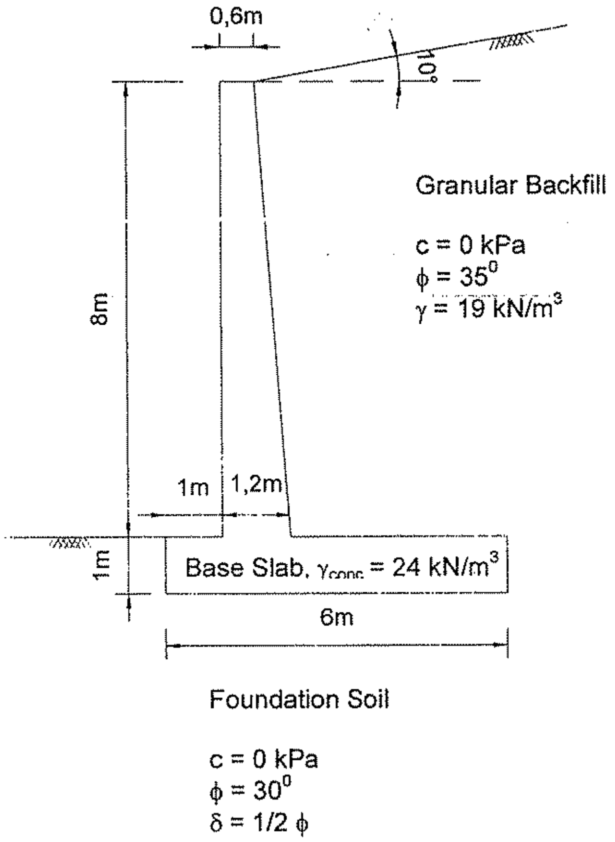
i) Prepare a sketch and show all the forces acting on the wall. Calculate the magnitudes of these forces.

ii) Investigate the overturning stability of the wall. Recall that FS_{min}=2.0.

iii) Compute the factor of safety against sliding. (FS_{min}=1.5)

iv) What are the maximum and minimum base pressures under the retaining structure? Are they within allowable limits? Sketch the pressure distribution under the base slab.

v) Based on these computations, what might be the likely failure mode(s) (if any) of the cantilever retaining wall ? Suggest a solution to overcome these stability issues, briefly discussing potential impacts of your modifications on sliding, overturning and bearing resistance.



Q7) The gravity retaining wall shown supports a highway cut. Geometry of the wall is given as presented in the sketch. Soil properties of the natural backfill and the foundation soil are presented. It is already known that the most critical failure wedge makes an angle of 60^0 with the horizontal. Investigate;

* F.S against overturning

* F.S against sliding.

Hints:

- Sketch all the forces, with the appropriate angles which they act.
- Calculate the mass of the mobilized soil wedge.
- Calculate the magnitude of the resultant active thrust on the wall by constructing a <u>scaled force polygon</u>. (Use Coulomb's active earth pressure theory).
- Neglect the passive resistance of the wall.

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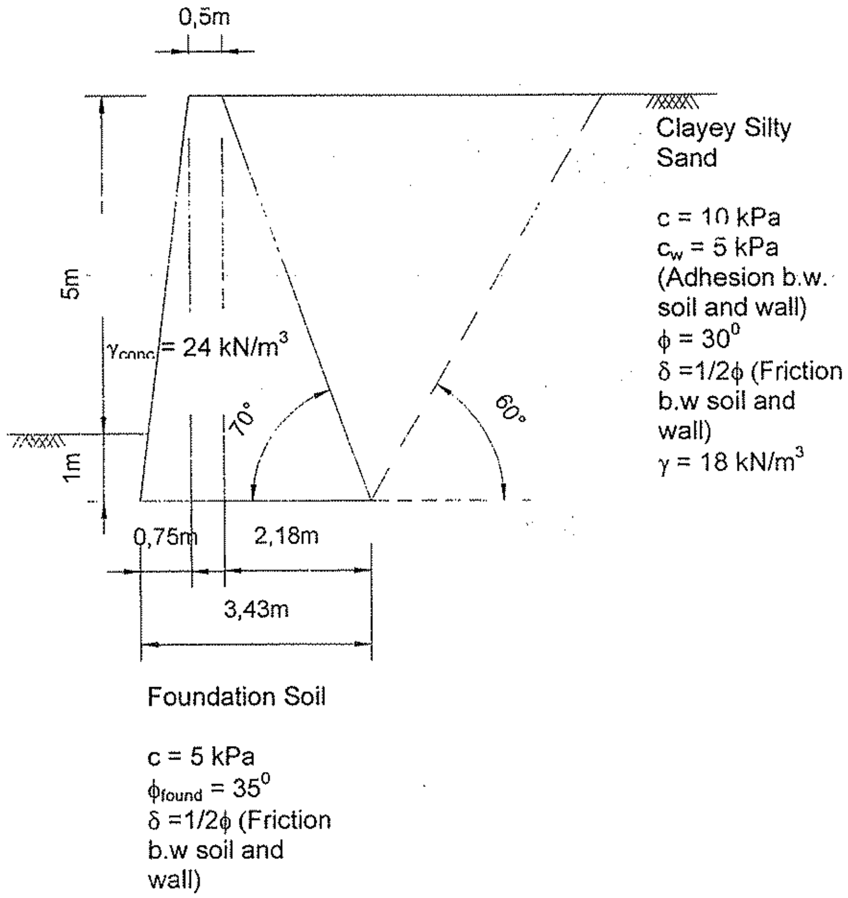
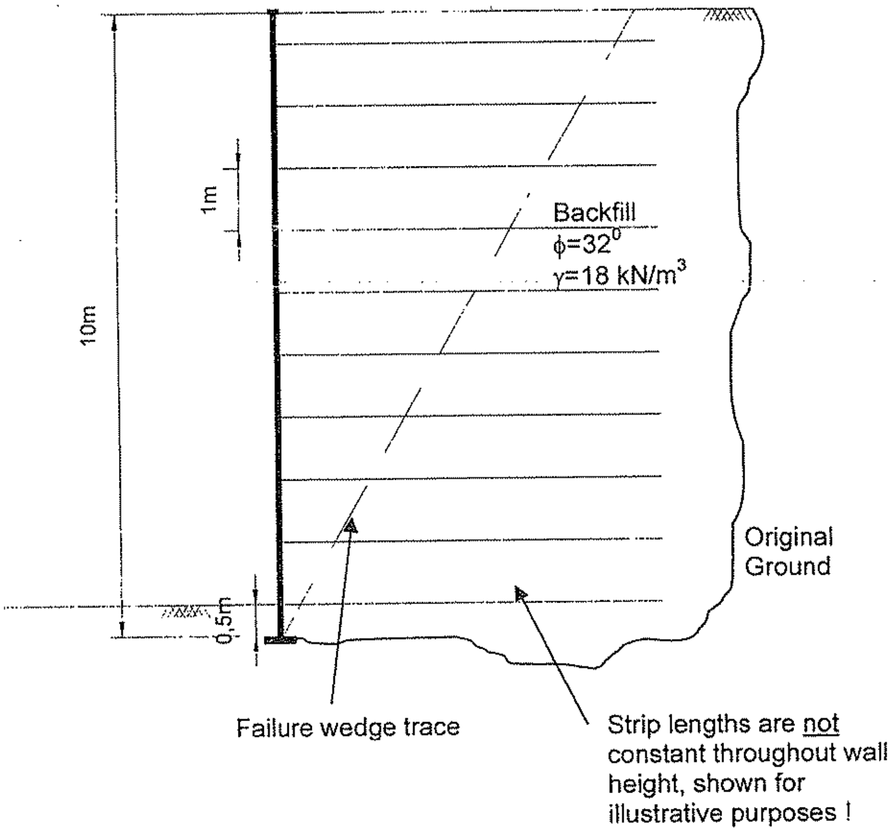


Q8) Analyze the reinforced earth wall using strip reinforcement. Place the strips at 1m spacings both vertically and horizontally. Ignore the weight of the concrete face panels.

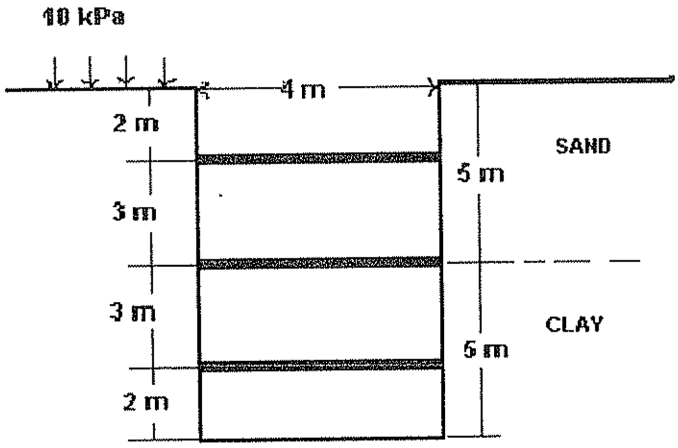
* Design for tie-breaking and pull-out of galvanized steel strips. Yield tensile strength of the strips are 240 MPa. Width of the strips are selected as 100 mm and corrosion rate is 0.0025 mm/year. Approximate service life of the wall is 40 years. Use a factor of safety of F.S=3.0 for both pullout and tie-break resistance. Backfill has an internal friction angle of 32^0 and unit weight of 18 kN/m^3. Assume that the friction angle between steel strips and soil is 70% of the soil-soil friction angle. Also assume that Rankine's lateral earth pressure distribution is valid. Prefer using variable strip lengths for each level.

* Compute the F.S against sliding of the wall.

49



Q9) An excavation is made and the site is supported by using braces.



The properties of the clay and sand layers are given below;

Sand layer ($\phi = 30^\circ$ and $\gamma = 19$ kN/m^3)

Clay layer ($\gamma = 19$ kN/m^3 and $c_u = 40$ kPa)

30

Do not consider ground water table.

Determine:

a) Factor of safety against bottom heave,

b) Strut loads for each level for the long braced system (Horizontal struts are spaced at every 5 meters).

Q10) An anchored sheet pile wall is constructed by driving a line of piling into a soil of saturated unit weight of 21 kN/m^3 and of shear strength parameters of c' = 10 kN/m^2 and Ø' = 27°. Backfilling to a depth of 8 m is placed behind the piling. The backfill has a saturated unit weight of 20 kN/m^3, a dry unit weight of 17 kN/m^3 and shear strength parameters of c' = 0 and Ø' = 35°. Tie rods are spaced at 2.5m centres, 1.5m below the surface of the backfill. The water level in front of the wall and the water table behind the wall are both 5m below the surface of the backfill. Using the free earth support method, determine the depth of penetration required and the force in each tie rod for a factor of safety of 2.0 with respect to passive resistance.

Q11) An excavation in sand is to extend to a depth of 8 m below the existing horizontal ground surface. Existing buildings near the site may be assumed to be equivalent to a uniform surcharge of 25 kN/m^2. Cantilever sheet piling will be used to support the vertical sides of the excavation. Coefficients of active and passive earth pressures have been estimated as 0.24 and 4.2 respectively. The groundwater table may be assumed to be below the expected depth of penetration, and the bulk unit weight of the soil is 17.5 kN/m^3.

With a factor of safety of 2.5 with respect to passive resistance, obtain an equation of the form,

$$d^3 + A.d^2 + B.d + C = 0$$

for the depth of penetration d of the sheet piling, where A, B and C are constants, and determine the depth of penetration.

HOMEWORK - 2 -
- SOLUTIONS -

1. Area of Foundation $= 8*14 + 2*4*6 = 160 m^2$

Total vertical load $= 2(100+400+300)+600+400+160*24*2.5 = 12200 kN$

Center of gravity of the base:

From left:

$$\frac{2*6*4+8*8*14+14*4*6}{160} = 8m \text{ from left}$$

From bottom → 7m from bottom.

Foundation is symmetrical.

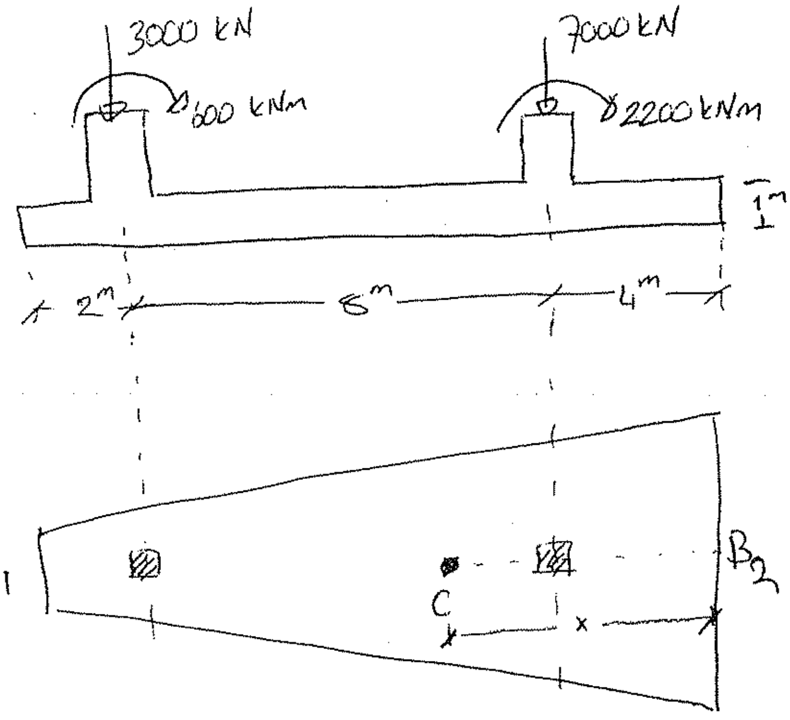
Location of ΣQ

Loading is symmetrical. → 8m from left
7m from bottom.

There is no eccentricity, therefore the base pressure is distributed equally.

$$q = \frac{12200}{160} = 76.25 < 100 \, kPa \rightarrow \text{safe}$$

ANSWER 2



$q_{all} = 400$ kPa

Area of the footing $= \frac{(B_1+B_2)}{2} \times 14$

Weight of the footing $= 1 \times 7 \times (B_1+B_2) \times 24$

$\Sigma V = 0$

$$3000 + 7000 + 168(B_1+B_2) = 400 \times 7 \times (B_1+B_2)$$

$$B_1+B_2 = 3.8\,m$$

$\Sigma M_C = 0$

$$3000(12-x) + 7000(4-x) - 600 - 2200 + W \times 0 = 400 \times 7 \times (B_1+B_2) \times 0$$

$$x = 6.12\,m$$

$$x = \frac{1}{3} \times 14 \times \frac{2B_1+B_2}{B_1+B_2} = \frac{1}{3} \times 14 \times \left(1 + \frac{B_1}{B_1+B_2}\right) = 6.12$$

$B_1 = 1.18 \Rightarrow 1.2\,m$

$B_2 = 2.62 \Rightarrow 2.7\,m$

5. Area of the foundation: $7 * 6 - 3 * 3 = 33 m^2$

Total vertical load: $2 * 300 + 2 * 350 + 400 = 1700 kN$

Center of gravity of the base:

From left:

$$\frac{3.5 * 7 * 6 - 5.5 * 3 * 3}{33} = 2.95 m \text{ from left}$$

From bottom:

$$\frac{3 * 7 * 6 - 4.5 * 3 * 3}{33} = 2.59 m \text{ from bottom}$$

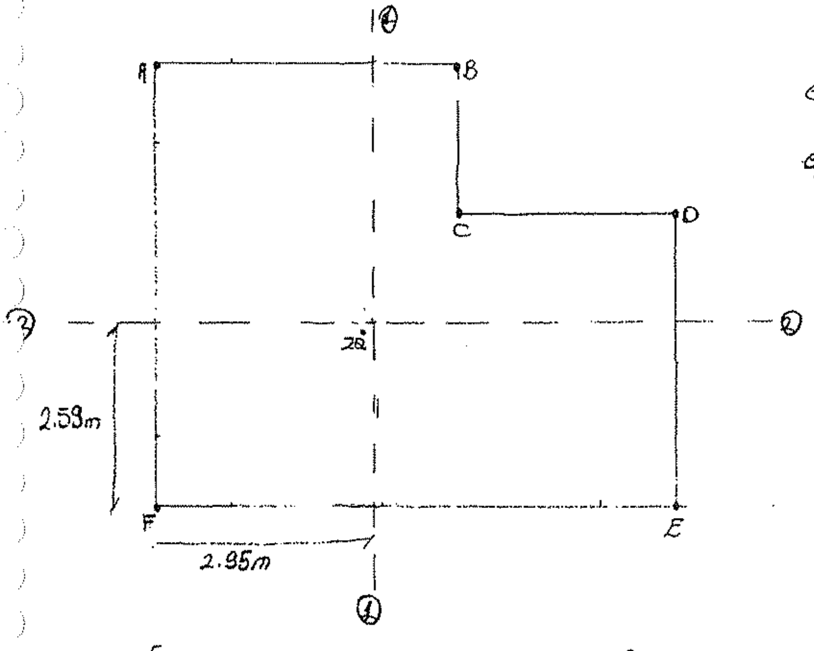
Location of ΣQ

From left:

$$\frac{L * (300 + 350) + 3 * (350 + 300) + 6 * 400}{1700} = 2.94 m \text{ from left}$$

From bottom

$$\frac{L (350 + 300 + 400) + 5 * (300 + 350)}{1700} = 2.53 m \text{ from bottom}$$



$e_1 = 2.9545 - 2.9411 = 0.013 m$

$e_2 = 2.59 - 2.53 = 0.06 m$

M_1 about 1-1 axis $\rightarrow$

$M_1 = 1700 \times 0.013 = 22.1 kN.m$

M_2 about 2-2 axis

$M_2 = 1700 * 0.06 = 102 kN.m$

$$I_{1-1} = \left[\frac{6 * 7^3}{12} + 6 . 7 . (3.5 - 2.95)^2\right] - \left[\frac{3 * 3^3}{12} + 3 * 3 (5.5 - 2.95)^2\right] = 206.28 m^4$$

$$I_{22} = \left[\frac{7 * 6^3}{12} + 6 . 7 . (3 - 2.59)^2\right] - \left[\frac{3 * 3^3}{12} + 3 * 3 (4.5 - 2.59)^2\right] = 146.18 m^4$$

$$q = \frac{\Sigma Q}{Area} \pm \frac{M_1 y_1}{I_{1-1}} \pm \frac{M_2 y_2}{I_{2-2}}$$

$$q = \frac{1700}{33} \pm \frac{22.1\, y_1}{206.28} \pm \frac{102\, y_2}{146.13}$$

$$q_F = 51.5 + \frac{22.1(2.95)}{206.28} + \frac{102(2.59)}{146.13} = 53.62\ kN$$

$$q_E = 51.5 - \frac{22.1(4.05)}{206.28} + \frac{102(2.59)}{146.13} = 52.87\ kN$$

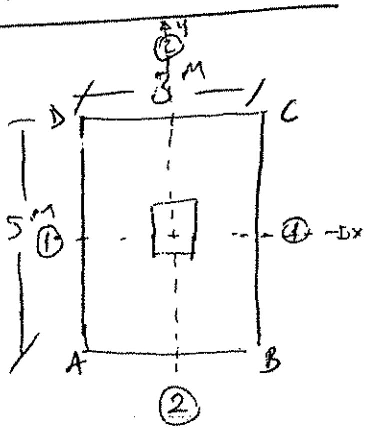
$$q_D = 51.5 - \frac{22.1(4.05)}{206.28} - \frac{102(0.41)}{146.13} = 50.78\ kN$$

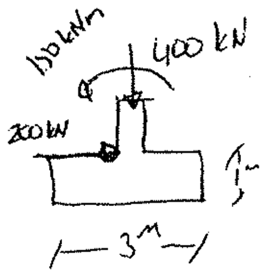
$$q_C = 51.5 - \frac{22.1(1.05)}{206.28} - \frac{102(0.41)}{146.13} = 51.10\ kN$$

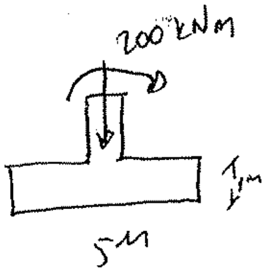
$$q_B = 51.5 - \frac{22.1(1.05)}{206.28} - \frac{102(3.41)}{146.13} = 49\ kN$$

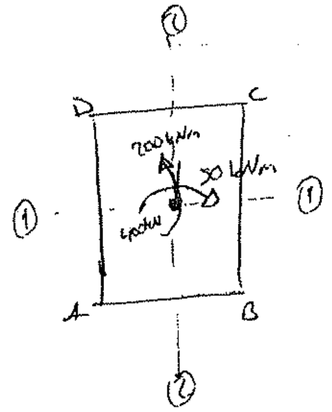
$$q_A = 51.5 + \frac{22.1(2.95)}{206.28} - \frac{102(3.41)}{146.13} = 49.44\ kN$$

ANSWER 4









$$\Sigma M_{2\text{-}2} = 200 \times 1 - 150 = 50 \text{ kNm}$$

$$\Sigma M_{1\text{-}1} = 200 \text{ kNm}$$

$$\Sigma V = 400 \text{ kN}$$

$$I_{11} = \frac{1}{12} \times 3 \times 5^3 = 31.25 \text{ m}^4$$

$$I_{22} = \frac{1}{12} \times 5 \times 3^3 = 11.25 \text{ m}^4$$

$$q = \frac{\Sigma V}{A} \mp \frac{M_{11} \cdot y}{I_{11}} \mp \frac{M_{22} \cdot x}{I_{22}}$$

$$q_A = \frac{400}{3 \times 5} - \frac{200 \times 2.5}{31.25} - \frac{50 \times 1.5}{11.25} = 4 \text{ kPa}$$

$$q_B = \frac{400}{3 \times 5} - \frac{200 \times 2.5}{31.25} + \frac{50 \times 1.5}{11.25} = 17.33 \text{ kPa}$$

$$q_C = \frac{400}{3 \times 5} + \frac{200 \times 2.5}{31.25} + \frac{50 \times 1.5}{11.25} = 49.33 \text{ kPa}$$

$$q_D = \frac{400}{3 \times 5} + \frac{200 \times 2.5}{31.25} - \frac{50 \times 1.5}{11.25} = 36 \text{ kPa}$$

$\frac{B}{2} = \frac{3.5}{2} = 1.75$ → center of gravity of the base

$e = 1.75 - 1.5 = 0.25$

$M_1 = 0.25 \times 2500 = 625$

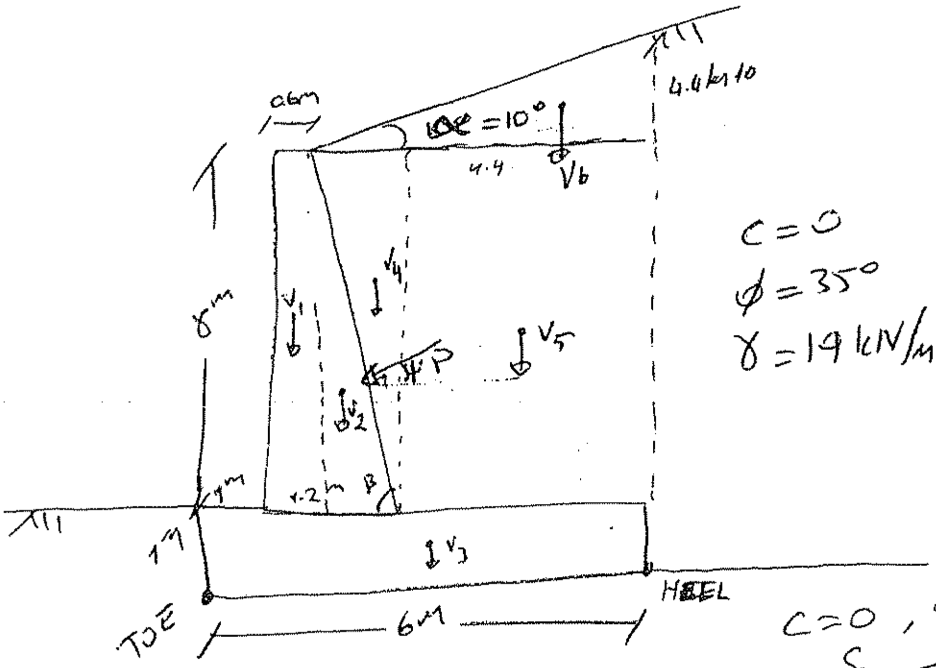
$M_T = 625 + 2500 = 3125$

$3125 = 2500e' \rightarrow e' = 1.25\,m$

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

$q_{max} = \frac{4 \cdot 2500L}{3(3.5-2.5)L} = 3333.\bar{3}$ kPa.

ANSWER 6



$c = 0$

$\phi = 35°$

$\gamma = 19\ kN/m^3$

$c = 0\ ,\ \phi = 30°$

$\delta_2 = \frac{1}{2}\phi = 15°$

$$K_A = \frac{\sin^2(\beta+\phi)}{\sin^2\beta\ \sin(\beta-\delta)} \times \frac{1}{\left[1 + \sqrt{\frac{\sin(\phi+\delta)\ \sin(\phi-\alpha)}{\sin(\beta-\delta)\ \sin(\alpha+\beta)}}\right]^2}$$

where

$\beta = \tan^{-1}\left(\frac{8}{0.6}\right) = 85°$

$\phi = 35°$ $\quad \alpha = 10°$

$\delta = 0$

$K_A = 0.334$

$\psi = \alpha + \delta = 10 + 0 = 10°$

$P = \frac{1}{2}\ 0.334 \times 19 \times 8^2 = 257\ kN/m$

moment ARM

$P_v = 257 \sin 10 = 44.63\ kN/m$ $\quad$ 2.025 m

$P_h = 257 \cos 10 = 253.10\ kN/m$ $\quad$ 3m

$M_v = 90.38\ kNm/m$ $\quad$ $M_h = 759.3\ kNm/m$

ANSWER 6 (cont'd)

		Moment ARM
$V_1 = 0.6 \times 8 \times 24$	$= 115.2$ kN/m	1.3 m
$V_2 = 0.6 \times 8 \times \frac{1}{2} \times 24$	$= 57.6$ kN/m	1.8 m
$V_3 = 6 \times 1 \times 24$	$= 144$ kN/m	3 m
$V_4 = 0.6 \times 8 \times \frac{1}{2} \times 19$	$= 45.6$ kN/m	2 m
$V_5 = 3.8 \times 8 \times 19$	$= 577.6$ kN/m	4.1 m
$V_6 = \frac{4.4^2 \times \tan 10}{2} \times 19$	$= 32.4$ kN/m	4.53 m

$M_1 = 149.76$ kNm/m $\quad M_4 = 91.2$ kNm/m

$M_2 = 103.68$ kNm/m $\quad M_5 = 2368.16$ kNm/m

$M_3 = 432$ kNm/m $\quad M_6 = 146.77$ kNm/m

$$FS_{overturning} = \frac{\Sigma M_{resisting}}{\Sigma M_{driving}} = \frac{3382}{759.3} = 4.45 > 2.0 \quad \underline{\underline{OK}}$$

$$FS_{sliding} = \frac{\Sigma V \tan 15}{\Sigma H} = \frac{1062.78 \tan 15}{253.10} = 1.1 < 1.5$$

$\underline{\underline{\text{NOT SAFE}}}$

$$q_{\substack{min\\max}} = \frac{\Sigma V}{B}\left(1 \mp \frac{6e}{B}\right)$$

$M = M_{resisting} - M_{driving}$
$= 2622.7$ kNm/m

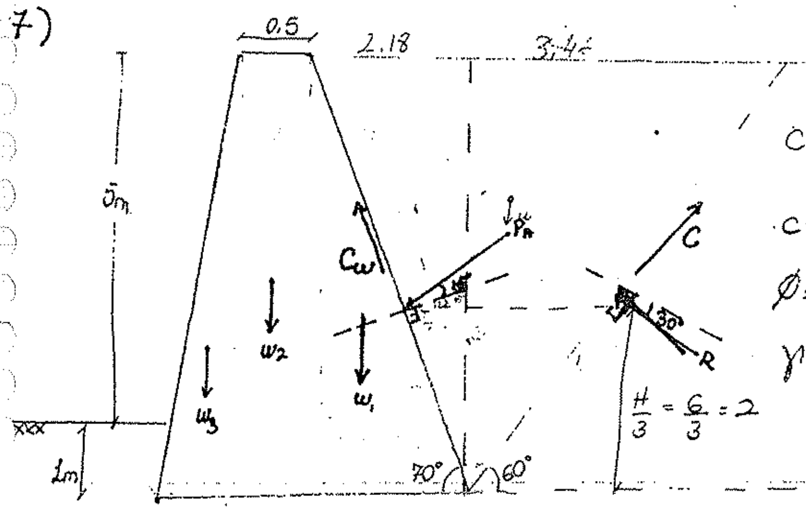
$X = \frac{M}{\Sigma V} = 2.48\text{ m} > B/6 \quad , \quad e = 3 - 2.48 = 0.52\text{ m}$

$q_{max} = \frac{1062.78}{6}\left[1 + \frac{6 \times 0.52}{6}\right] = 269 \text{ kPa} > 200 \quad \underline{\underline{\text{NOT SAFE}}}$

$q_{min} = 0$

> The retaining structure is not safe. In order to overcome its make adequate changes in design. For these changes, read related parts in either your Lecture Notes or Craig Soil Mechanics.

7)



Clayey Silt

$c = 10\,kPa$, $c_w = 5\,kPa$

$\phi = 30°$ $\delta = \frac{1}{2}\phi$

$\gamma = 18\,kN/m^3$

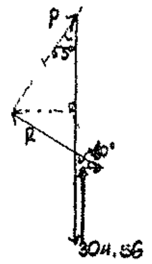
Foundation Soil $\rightarrow$ $c = 5\,kPa$ $\phi_{found} = 35°$

$\delta = \frac{1}{2}\phi$

$W = \frac{(2.18+3.46)*6}{2} \cdot 18 = 304.56$

$F_1 = \ell_1 C_w = 6.385 * 5 = 31.825$ $\rightarrow$ $F_{1,h} = 31.825 * \cos 70 = 10.92\ kN/m$; $F_{1,v} = 31.825 * \sin 70 = 30\ kN/m$

$F_2 = \ell_2\, C = 6.928 * 10 = 69.28$ $\rightarrow$ $F_{2,h} = 69.28 * \cos 60 = 34.64\ kN/m$; $F_{2,v} = 69.28 * \sin 60 \approx 60\ kN/m$



$P \cos 55 + R \sin 60 = 304.56 - 90 = 214.96$

$-P \sin 55 + R \cos 60 = 23.72$

$P = 87.27\ kN/m$

$P_v = 87.27 \sin 35 = 50.05\ kN/m$

$P_h = 87.27 \cos 35 = 71.49\ kN/m$

Force (kN/m)	Moment arm (m)	Moment (kN.m/m)
$W_1 = \frac{1}{2} * 2.18 * 6 * 24 = 156.96$	1.98	310.78
$W_2 = 0.5 * 6 * 24 = 72$	1.00	72
$W_3 = \frac{1}{2} 0.75 * 6 * 24 = 54$	0.5	27
$P_v = 50.05$	2.7	135.135 +
$\Sigma V = 333.01$		$\Sigma M_{resist} = 544.92$

$\Sigma M_{overturning} = 71.49 * 2 = 142.98$

$(F.S)_{overturning} = \frac{\Sigma M_r}{\Sigma M_o} = \frac{544.92}{142.98} = 3.8 > 2$ ✓

(cont'd)

$$(F.S)_{sliding} = \frac{\Sigma V \cdot \tan\delta + \left(\frac{2}{3} \cdot c_2 \cdot B\right) + \cancel{P_p}}{\Sigma H}$$ neglected

$$= \frac{333.01 * \tan 17.5 + \frac{2}{3} \cdot 5 \cdot 3.43}{71.49} = 1.63 > 1.5$$