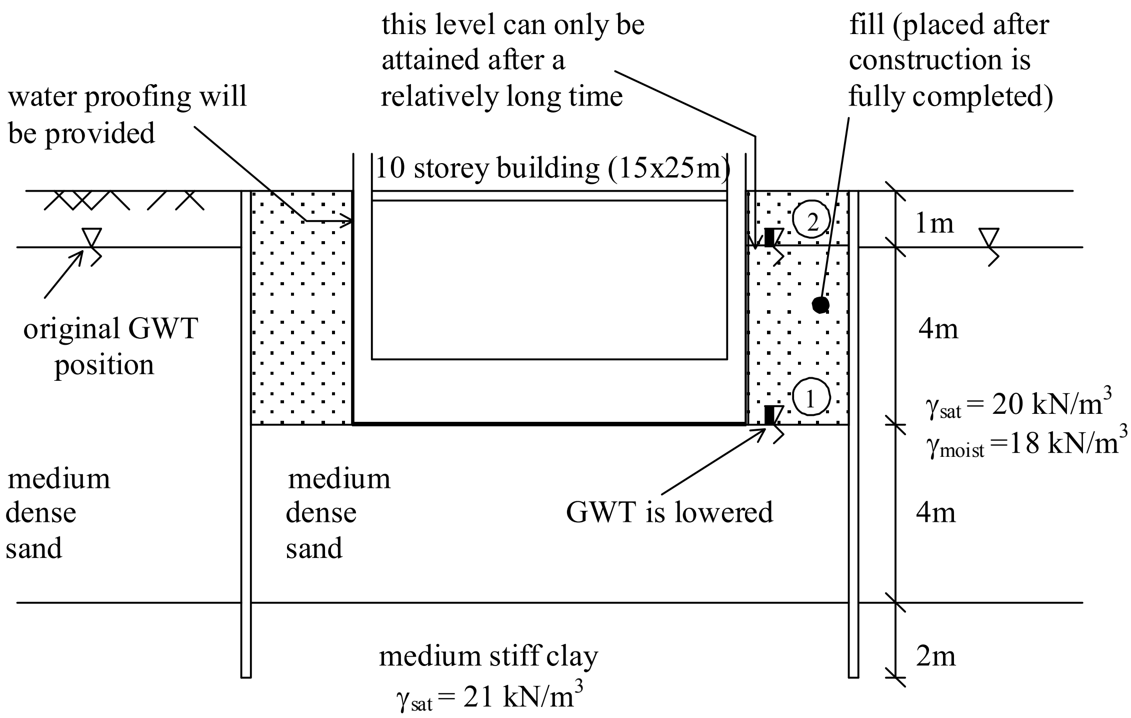
# CE 366 – BEARING CAPACITY (Problems & Solutions)

**P1**

***Question:***

An excavation will be made for a ten storey 15x25 m building. Temporary support of earth pressure and water pressure will be made by deep secant cantilever pile wall. The gross pressure due to dead and live loads of the structure and weight of the raft is 130 kPa (assume that it is uniform).

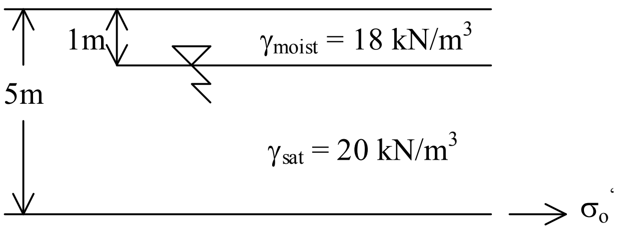



a) What is net foundation pressure at the end of construction but before the void space between the pile wall and the building has been filled, and *there is no water inside the foundation pit yet* (water level at the base level) (GWT position 1).

b) What is net foundation pressure long after the completion of the building, i.e. water level is inside the pile wall and the backfill between the building and the pile wall is placed (GWT position 2). What is the factor of safety against uplift?

***Solution:***

a) $q_{net} = \begin{bmatrix} \text{final effective stress} \\ \text{at foundation level} \end{bmatrix} - \begin{bmatrix} \text{initial effective stress} \\ \text{at foundation level} \end{bmatrix}$




$\sigma_o' = 18x1 + 4x(20\text{-}9.8) = 58.8\ kPa$

( gross pressure – uplift pressure) = final effective stress at foundation level, $\sigma_f'$

gross pressure =130 kPa (given)

uplift pressure = 0 kPa (Since GWT is at foundation level (1), it has no effect on structure load)

$\sigma_f' = 130 - 0 = 130\ kPa$

$q_{net} = 130 - 58.8$

$= \mathbf{71.2\ kPa}$

b) $\sigma_f' = 130 - 4x9.8 = 90.8\ kPa$

$4x9.8 \rightarrow$ uplift pressure

$\sigma_o' = 58.8$ kPa (same as above)

$q_{net} = 90.8 - 58.8$

$= \mathbf{32.0\ kPa}$

OR

$q_{net} = q_{gross} - \gamma_{sat}D = 130\text{-}(18x1+4x20)$

$= \mathbf{32.0\ kPa}$

Factor of safety against uplift is:

$(FS)_{uplift}$ = weight of structure / uplift

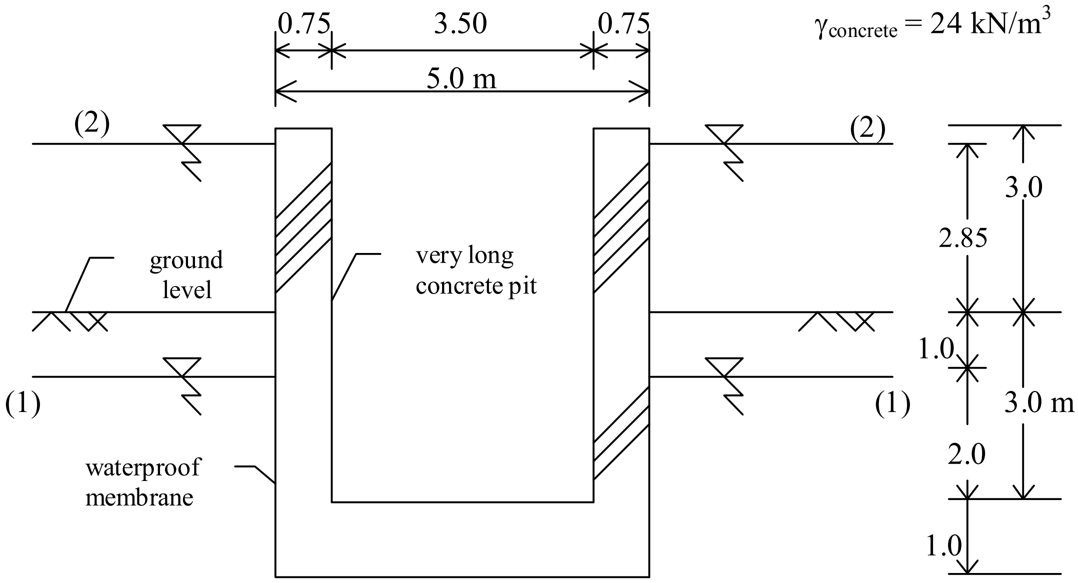
= (130x15x25) / (4x9.8x15x25)

= **3.3**

**P2**

***Question:***

Calculate the FS against uplift and calculate effective stress at the base level for water level at (1) and (2) for the canal structure given below. Note that the canal is very long into the page.




***Solution:***

- **water table at (1)**

Factor of Safety against uplift = $\underbrace{(2x6x0.75 + 5x1)x24}_{\Sigma\text{weight of pit}}$ / $\underbrace{(3x5)x9.8}_{\text{uplift}}$

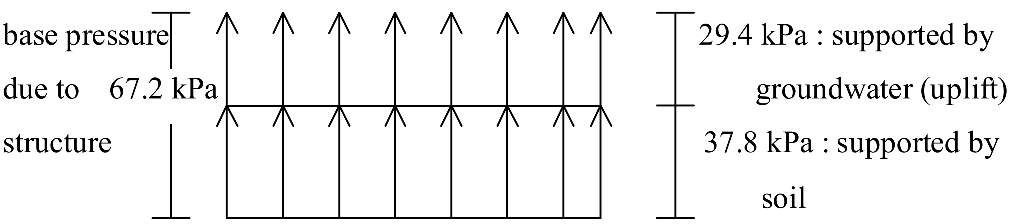
= 336 / 147

= **2.28**

Base pressure = 336 / 5 = 67.2 kN/m$^2$ due to weight of structure.(per meter of canal)

147 / 5 = 29.4 kN/m$^2$ is supported by groundwater

67.2 – 29.4 = 37.8 kN/m$^2$ is supported by soil (effective stress at the base)



- **<u>water table at (2)</u>**

FS = 336 / (6.85x5x9.8)
= 1.0 < 1.5 NOT OKEY

⇒ base pressure = 67.2 kPa is supported by ground water
uplift = weight of structure

<u>Soil does not carry any load, structure tends to float</u>

**P3**

***Question:***

A residential block will be constructed on a clay deposit. The building will rest on a mat foundation at 2m depth and has 20mx20m dimensions in plan.

The clay deposit is 26m deep and overlies limestone. The groundwater level is at 2m depth. The bulk unit weights are 18 and 20 kN/m$^3$ above and below water table respectively.

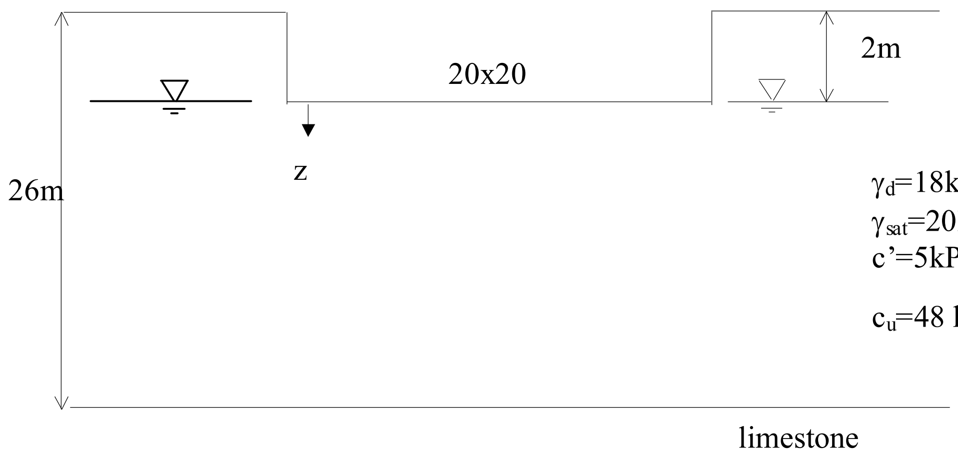
The clay has c'=5 kN/m$^2$, $\phi$'=20$^0$, $c_u$=48 kN/m$^2$, $\phi_u$=0. The coefficient of volume compressibility is 1.00x10$^{-4}$ m$^2$/kN at the ground surface and decreases with depth at a rate of 0.02x10$^{-4}$ m$^2$/kN per meter. Use $E_u/c_u$ = constant = 1250 and $I_s$ = 1.2

a) Calculate ultimate bearing capacity of the foundation in the short term?

b) For the foundation described above what is the (gross) allowable bearing capacity?

**NOTE:** For $\phi_u$=0 case use Skempton values, use a safety factor of 3.00 against shear failure of the foundation. Use sublayers. Maximum allowable total settlement of the building is 15 cm.

***Solution:***




Skempton expression for $\phi_u$=0 is : $q_f = c_u N_c + \gamma_{sat} D$ (total stress analysis)

$q_{nf} = c_u N_c$

**Short Term :**

$$\frac{D}{B}=\frac{2}{20}=0.1 \quad N_{c\ square}=6.4 \quad \text{(Skempton Chart, page 73 Fig.4.6 in Lecture Notes)}$$

$$q_f = 48x6.4 + 18x2 = 343.2 \text{ kPa}$$

$$q_{nf} = q_f - \gamma D = c_u N_c = 307.2 \text{ kPa}$$

**Settlement Check :**

$S_t = S_i + S_c$

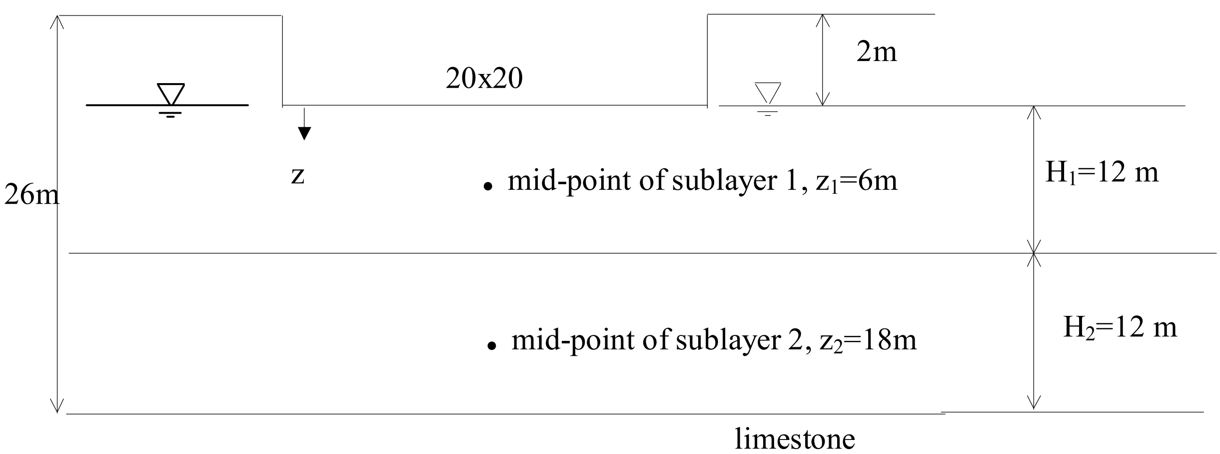
***IMMEDIATE SETTLEMENT IN CLAY, $S_i$:***

$$S_i = \frac{qB}{E}(1-\mu^2)I_s \quad \text{where } q = q_{net} \text{ (net foundation presure)} = \frac{q_{nf}}{FS} = \frac{307.2}{3} = 102.4 \text{ kPa}$$

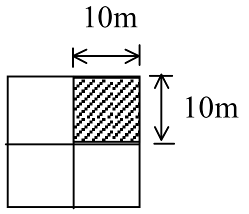
- Note that in clay for UNDRAINED CASE $\rightarrow \mu = 0.5$
- undrained modulus, $E_u = 60000$ kPa
- $I_s = 1.2$ (given)

$$S_i = \frac{102.4x20}{60x10^3}(1-0.5^2)x1.2 = 0.031m = 31mm$$

***CONSOLIDATION SETTLEMENT IN CLAY, $S_c$:***



- Vertical Stress due to $q_{net}$ should be determined at the mid-point of each sublayer




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

$\Delta\sigma = 4qI_r$ ; $q = q_{net} = 102.4$ kPa

$m_v = [1-0.2(2+z)]x10^{-4}$

| Layer no | z | m=n=10/z | $I_r$ | $\Delta\sigma$ | $m_v(m^2/kN)$ |
|---|---|---|---|---|---|
| 1 | 6 | 1.67 | 0.2 | 81.9 | $0.84x10^{-4}$ |
| 2 | 18 | 0.55 | 0.093 | 38.1 | $0.6x10^{-4}$ |

$S_{oed}$= ( $0.84x10^{-4}$x81.9x12)+( $0.6x10^{-4}$x38.1x12)=0.110m=110mm

$S_t$ = 31+110 ≅ 141mm<150mm (allowable) OK.

∴GENERALLY IN CLAY SHEAR FAILURE CONTROLS THE DESIGN, SETTLEMENT IS NOT CRITICAL. BUT IT SHOULD BE CHECKED ALSO

$(q_{all})_{net}$ = 102.4 kPa

$(q_{all})_{gross}$= 102.4+2x18 = 138 kN/$m^2$

**P4**

***Question:***

A footing of 4mx4m carries a uniform gross pressure of 300 kN/m$^2$ at a depth of 1.5m in a sand. 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, $\phi$'=32$^0$. Determine the factor of safety with respect to shear failure for the following cases;

a) The water table is at ground surface

b) The water table is 1.5m below the surface

***Solution:***

$$FS = \frac{(q_{ult})_{net}}{q_{net}} = \frac{q_{nf}}{q_n} = \frac{q_{ult} - \gamma D}{q_{gross} - \gamma D} = \frac{q_f - \gamma D}{q_n}$$

**For square footing:**

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

$c' = 0$ and $\phi' = 32^0$ $\quad N_\gamma = 26, N_q = 29$ (see page 69 Figure 4.3 in Lecture Notes)

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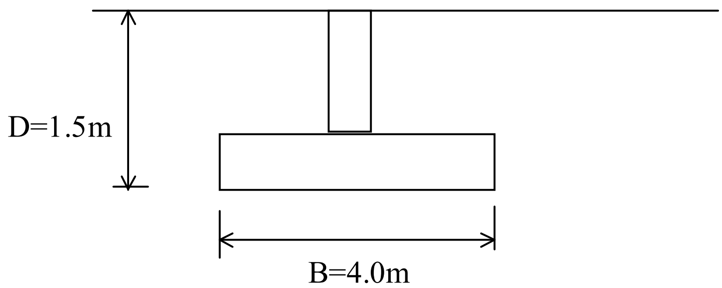


a)

$$q_f = 0.4B\gamma' N_\gamma + \gamma' D N_q = 0.4x4x(20-10)x26 + (20-10)x1.5x29 = 851\,kPa$$

$$q_{nf} = q_f - \gamma' D = 851 - (20-10)x1.5 = 836\,kPa$$

$$q_{gross} = 300\,kPa$$

i. $q_{net} = 300 - 20x1.5 = 270\,kPa$ OR

ii. $q_{net} = (300 - 1.5x10) - 1.5(20-10) = 270\,kPa$

$$FS = \frac{836}{270} = 3.1$$

b)

$$q_f = 0.4B\gamma' N_\gamma + \gamma_d D N_q = 0.4x4x(20-10)x26 + 17x1.5x29 = 1156\,kPa$$

$$q_{nf} = q_f - \gamma D = 1156 - 17x1.5 = 1130\,kPa$$

$$q_{gross} = 300\,kPa$$

$$q_{net} = 300 - 17x1.5 = 275\,kPa$$

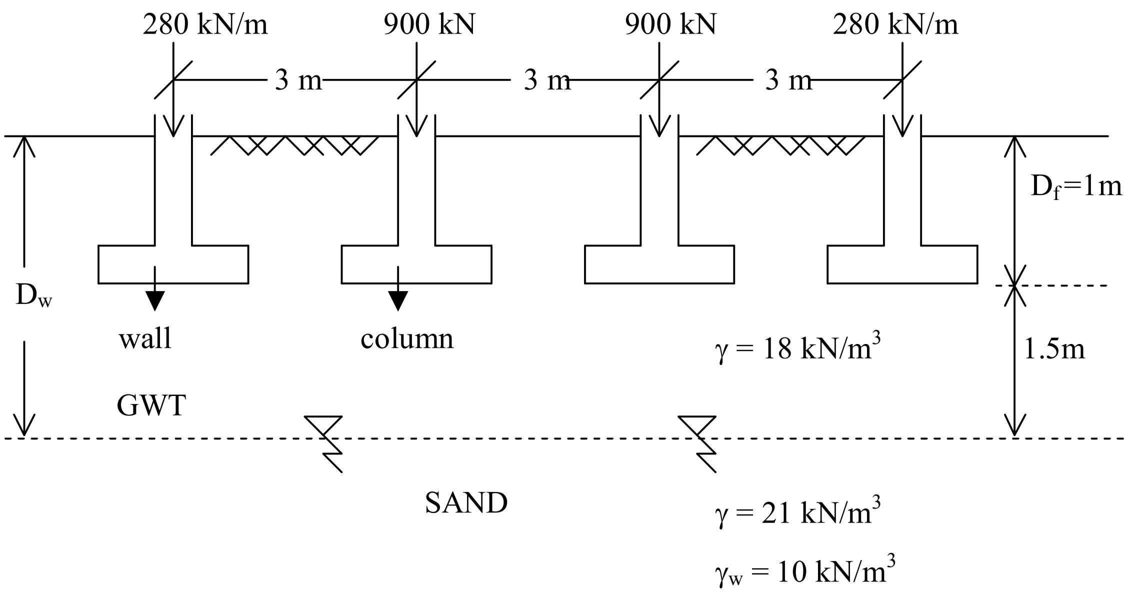
$$FS = \frac{1130}{275} = 4.1$$

## P5 FOOTING ON SAND

***Question:***

The column loads, wall loads and the pertinent soil data for a proposed structure is given below. Design the square column and wall footings for a permissible settlement of 30 mm, using Peck & Hanson & Thornburn charts. Make a reasonable assumption to obtain an average N value below the footing.

| depth | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| N | 8 | 14 | 11 | 16 | 18 | 11 | 9 | 13 | 18 | 20 | 50/11 | 50/7 |




**Footing on Cohesionless Soils:**

Assumptions:

- significant depth: 0.5 B above, 2 B below the footing
- weight of excavated soil ≅ weight of (footing + column) in the soil

  column load / area ≅ $q_{net}$
- footings to be designed for the largest $q_{net}$ (i.e. column ftg)

***Solution:***

NOTE: For Peck-Hanson-Thorburn, N values should be corrected for overburden stress

| Depth | $N_{field}$ | $\sigma_o'$ | $C_N$ | $N_1$ |
|---|---|---|---|---|
| 1 | 8 | 18 | 2.0 | 16 |
| 2 | 14 | 36 | 1.63 | 23 |
| 3 | 11 | 50.5 | 1.38 | 15 |
| 4 | 16 | 61.5 | 1.25 | 20 |
| 5 | 18 | 72.5 | 1.15 | 21 |
| 6 | 11 | 83.5 | 1.07 | 12 |
| 7 | 9 | 94.5 | 1.01 | 9 |
| 8 | 13 | 105.5 | 0.95 | 12 |
| 9 | 18 | 116.5 | 0.91 | 16 |
| 10 | 20 | 127.5 | 0.87 | 17 |
| 11 | 50/11 | | | - |
| 12 | 50/7 | | | - |

*$C_N$ (overburden correction) values are calculated by using eq.2.3 (page 31) in Lecture Notes.* ($C_N = 9.78x(1/\sigma_v^{1})^{0.5} \leq 2$)

**<u>Square column footings</u>** Peck & Hanson & Thornburn charts:Fig 4.8 in Lecture Notes

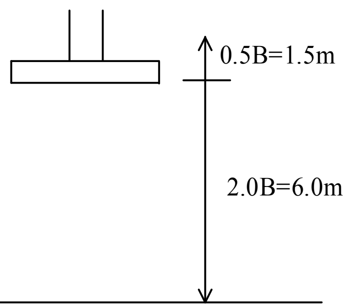
⇒ **assume B=3.0 m**

⇒ To obtain the average N value to be used in the calculations

Consider 0.5B=0.5x3=1.5m above

2.0B=2.0x3=6.0m below the foundation level

| Depth | $N_1$ |
|---|---|
| 1 | 16 |
| 2 | 23 |
| 3 | 15 |
| 4 | 20 |
| 5 | 21 |
| 6 | 12 |
| 7 | 9 |
| 8 | 12 |
| 9 | 16 |
| 10 | 17 |




$N_{1,av}$ = (16+23+15+20+21+12+9) / 7 = 17

$C_w = 0.5 + 0.5x[2.5/(1+3)] = 0.81$

$(q_n)_{all}$=11x $N_{1,av}$ x $c_w$ (kN/m$^2$) for 25 mm settlement (page 78 in Lecture Notes) $(q_n)_{all}$=11x17x0.81 = 151 kPa

$$(q_n)_{all} = (q_n)_{all}\, x \frac{S_{all}(mm)}{25}$$

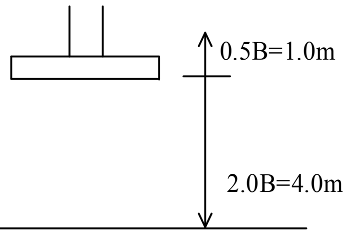
$q_{all}$ = 151x(30/25) = 181 kPa

$q_{net}$ = 900/(3x3) = 100 kPa

181 >>100 $\longrightarrow$ overdesign

$\Rightarrow$ **assume B = 2.0 m**

| Depth | $N_{cor}$ |
|---|---|
| 1 | 16 |
| 2 | 23 |
| 3 | 15 |
| 4 | 20 |
| 5 | 21 |
| 6 | 12 |
| 7 | 9 |
| 8 | 12 |
| 9 | 16 |
| 10 | 17 |




$N_{av}$ = (16+23+15+20+21) / 5 = 19

$C_w = 0.5 + 0.5x[2.5/(1+2)] = 0.92$

$(q_n)_{all}$=11x19x0.92 = 192 kPa

$q_{all}$ = 192x(30/25) = 230 kPa

$q_{net}$ = 900/(2x2) = 225 kPa

230 $\cong$ 225 OK

$\boxed{B = 2.0 \text{ m}}$

**Wall footings**

$\Rightarrow$ Use $q_{net}$ = 225 kPa

$$B = \frac{280}{225} = 1.25m$$

Check B value

$N_{av}$ = (16+23+15) / 3 = 18

$C_w$ = 0.5 + 0.5x[2.5/(1+1.25)] ≤ 1.0 ➔ $C_w$ = 1.0

$(q_n)_{all}$=11x18 = 198 kPa

$q_{all}$ = 198x(30/25) = 238 kPa

238 > 225 OK

## P6 FOOTING ON CLAY

### *Question:*

A public building consists of a high central tower which is supported by four widely spaced columns. Each column carry a combined dead load and representative sustained load of 2500 kN inclusive of the substructure (gross load). Trial borings showed that there is a 7.6m of stiff fissured Ankara clay ($c_u$=85 kPa, $E_u$ = 30 MN/m$^2$ and $m_v$ = 1x10$^{-4}$ m$^2$/kN) followed by dense sand which is relatively incompressible. Determine the required foundation width (assume square foundation) and allowable bearing pressure for the tower footings.

Assume $\gamma_{wet} = \gamma_{sat} = 18.6$ kN/m$^3$ (above and below GWT)

$\gamma_w = 10$ kN/m$^3$

Consider immediate and consolidation settlements. Divide the clay layer into 4 equal sublayers.

The foundation depth can be taken as 2m.

$\Rightarrow$ D=2.0m, $c_u$ = 85 kPa, $\mu$=0.5, $D_w$ = 1.2 m, F.S. = 2.5

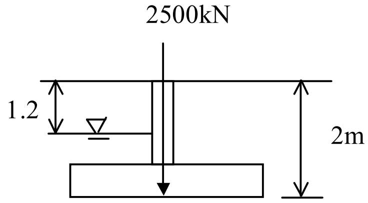
### *Solution:*

- **Assume B=2.0m**

$D_f/B=1 \quad \Rightarrow \quad N_c = 7.7$ (Skempton)

$q_{nf} = (q_{ult})_{net} = c_u N_c = 85x7.7 = 654.5$ kPa

for FS=2.5 $\quad (q_{net})_{safe} = 654.5/2.5 = 261.8$ kPa




$q_{net} = 2500/(2x2) - 2x18.6 = 587.8$ kPa

OR

$q_{net} = (2500/(2x2) - 0.8x10)-(1.2x18.6+0.8x8.6)$

$= 587.5$ kPa

$(q_{net})_{safe} << q_{net}$ **NOT ACCEPTED**

- **Assume B=3.0m**

$D_f/B = 0.67 \quad \Rightarrow \quad N_c = 7.4$ (Skempton)

$q_{nf} = (q_{ult})_{net} = c_u N_c = 85x7.4 = 629$ kPa

for FS=2.5 $\quad (q_{net})_{safe} = 629/2.5 = 251.6$ kPa

$q_{net} = 2500/3x3 - 2x18.6 = 241$ kPa

**$(q_{net})_{safe} \approx q_{net}$ OK**

**$\therefore$ B=3.0m**

**Settlements**

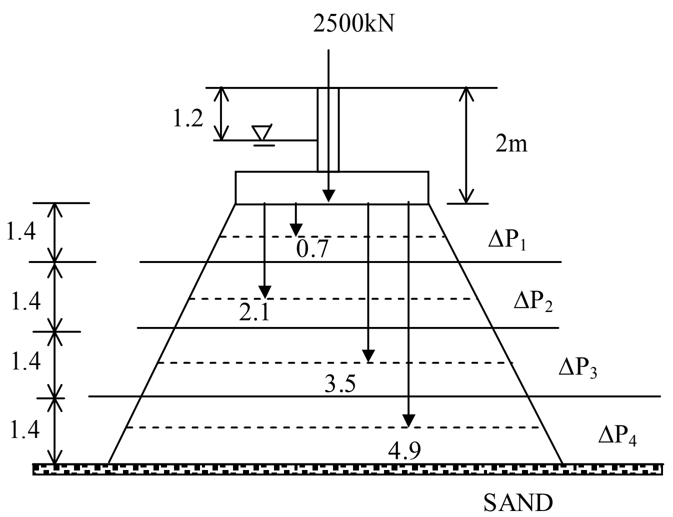
B=3.0m $E_u = 30000$ kPa $D_f$=2.0m

Compressible layer thickness H=7.6-2=5.6m

$$S_i = \mu_0 \mu_1 \frac{qB}{E_u}$$

$$\frac{H}{B} = 1.87 \quad \frac{D}{B} = 0.67 \quad \Rightarrow \quad \mu_0 = 0.95 \quad \mu_1 = 0.57$$

$$S_i = 0.57x0.95x\frac{241x3}{30000} = 0.013m = 13mm$$




Sand is relatively incompressible
(also = 2B)

$q_{net}$=241 kPa

$$\Delta P = \frac{q_{net} BL}{(B+z)(L+z)}$$

(Use 2:1 approximation)

| **Layer no** | **Thickness,H (m)** | **ΔP** |
|---|---|---|
| 1 | 1.4 | 158 |
| 2 | 1.4 | 83.4 |
| 3 | 1.4 | 51.3 |
| 4 | 1.4 | 34.8 |

Note that:

⇒ ΔP= vertical stress due to $q_{net}$ at the mid-point of each sublayer

$S_{oed}$=mv.Δσ.H

$S_{oed}=1x10^{-4}x1.4x(158+83.4+51.3+34.8)=4.585x10^{-2}m=45.85mm$

Apply Skempton-Bjerrum factor μ=0.5

$S_c = S_{oed}\ \mu = 45.85x0.5= 22.9mm$

$S_{total} = S_I + S_c = 13+22.9 = 35.9mm$

## P7 RAFT FOUNDATION ON DEEP CLAY LAYER

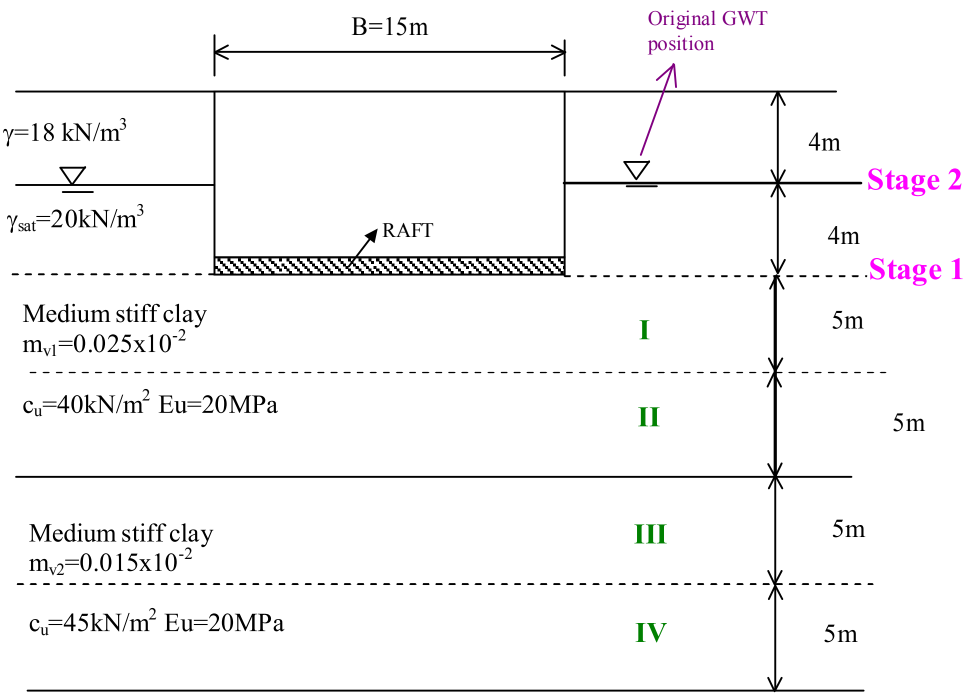
***Question:***

A 16-storey apartment block is to be constructed at a site. The soil profile consists of a deep clay layer which contains a 5m thick sand layer. The ground water table is at 4m depth. The base of the raft under the building is 8m 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 (15mx30m). Total weight of the building (dead+live+raft) is 90 000 kN.

Find the net foundation pressure and check the factor of safety against bearing capacity and calculate the total settlement of the building.

No secondary settlements are expected. Take the Skempton-Bjerrum correction factor μ= 0.75. Consider the compressions of the soil within 20m distance from the foundation level. The G.W.T. is at the "Stage 2" level prior to construction, lowered to "Stage1" level during the construction and rises back to "Stage 2" level in the long term.




Total weight of the building (dead+live+raft)=$Q_{gross}$=90000 kN

$q_{gross}$= 90000/(15x30) = 200 kPa

***Solution:***

**Stage 1** (GWT is lowered to the foundation level)

Uplift = 0

$\sigma_o' = 4x18+4(20-9.8) = 112.8$ kPa

$q_{net}=(200-0)-112.8 = 87.2$ kPa (net foundation pressure)

**Stage 2** (GWT is raised to its original position)

Uplift= 4x9.8= 39.2 kPa

$\sigma_o' = 4x18+4(20-9.8) = 112.8$ kPa

$q_{net}=(200-39.2)-112.8 = 48$ kPa

$q_{net}$= 87.2 kPa is MORE CRITICAL

Net bearing capacity of the foundation : $q_{nf} = q_f - \gamma D = c_u N_c + \gamma D - \gamma D = c_u N_c$

$c_u = 40$ kPa

$D_f/B=8/15=0.53$ $(N_c)_{square}=7.1$

$(N_c)_{rect.}=(N_c)_{square}\ (0.84+0.16B/L) = 7.1(0.84+0.16x15/30) = 6.5$

$q_{nf} = 6.5x40 = 260$ kPa

Safety factor against shear $FS = \dfrac{q_{nf}}{q_{net}} = \dfrac{260}{87.2} = 3.0$ OK

**Settlement Analysis:**

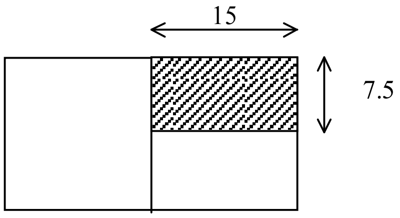
Total settlement= $S_t = S_I + S_c$

Consider the compressions of the soil within 20m distance from the foundation level.

Initial settlement $:S_i = \mu_0 \mu_1 \dfrac{qB}{E_u} = 0.95x0.5x\dfrac{87.2x15}{20000} = 3.1cm$

Consolidation Settlement : $S_c = m_v \Delta\sigma H$

For consolidation settlement; consider 5m thick sublayers.



$\Delta\sigma = 4qI_r$

**$q_{net}$ = 48 kPa since consolidation is a LONG TERM situation**

| n=B/z | m=L/z | $I_r$ | $\Delta\sigma = 4qI_r$ | $m_v$ ($m^2$/kN) |
|---|---|---|---|---|
| 7.5/2.5 | 15/2.5 | 0.245 | 47 | $0.025x10^{-2}$ |
| 7.5/7.5 | 15/7.5 | 0.2 | 38.4 | $0.025x10^{-2}$ |
| 7.5/12.5 | 15/12.5 | 0.145 | 27.8 | $0.015x10^{-2}$ |
| 7.5/17.5 | 15/17.5 | 0.102 | 19.6 | $0.015x10^{-2}$ |

$S_c = 0.025x10^{-2}x47x5 + 0.025x10^{-2}x38.4x5 + 0.015x10^{-2}x27.8x5 + 0.015x10^{-2}x19.6x5$

$S_c$ = 0.142m=14.2cm

$\Rightarrow$ $\mu$=0.75 (Skempton-Bjerrum)

$S_c$ = 14.2x0.75= 10.7cm

$S_t$ = 3.1+10.7 = 13.8 cm