حل شيت ١
٣ مدني
Soil

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Faculty of Engineering
Structure Engineering Dept.
3rd Year Civil
Geotechnical engineering (2) - CES 351
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جامعة عين شمس
كلية الهندسة
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ثالثة مدنى
هندسة جيوتقنية (2) – هدن 351
العام الجامعي 2013/2012

Assignment No.1 – Model answer

Soil Compaction

1- Discuss the differences between compaction and consolidation.

Solution of problem No.1:

Consolidation	Compaction
Reduction in soil voids by expulsion of water	Reduction in soil voids by expulsion of air
Occur under the effect of static loads	Occur under the effect of static loads, vibration loads or impact loads
Require long time to finish	Require relatively short time to finish
Phenomena that occur for cohesive soil	Can be applied to any soil type

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2- Compare between the standard and the modified Proctor test.

Solution of problem No.2:

Proctor test	Modified proctor test
Soil is compacted on three layers	Soil is compacted on five layers
Hammer fall height 30.5 mm	Hammer fall height 457 mm
Hammer mass 2.49 Kg	Hammer mass 4.50 Kg
No. of blows is 25 blows	No. of blows is 25 blows

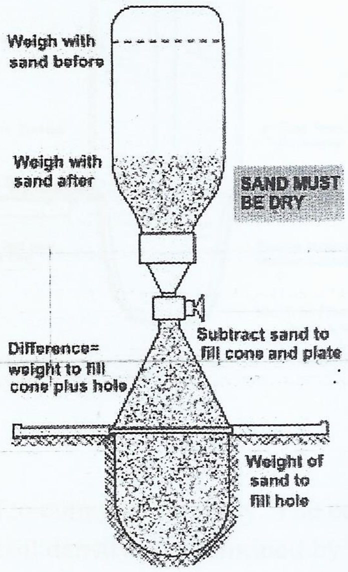
3- Show with aid of neat sketches when possible the different methods used to determine the soil density in field.

Solution of problem No.3:

- Sand cone method:

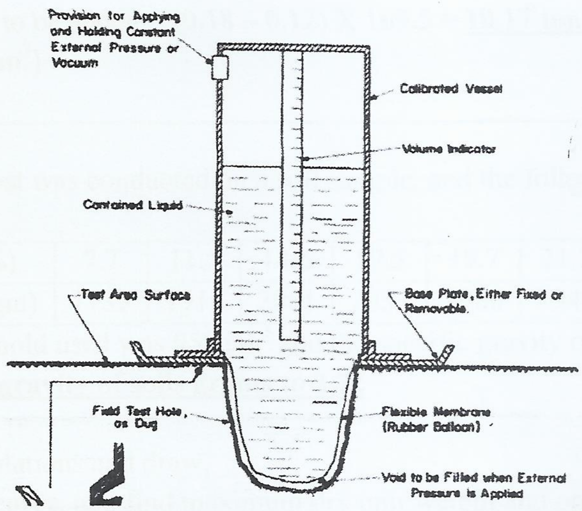
A small hole is dug in the compacted soil (the hole size depends on the soil grain size). The excavated material is then weighted in wet and dry conditions. The volume is determined using the sand cone apparatus. The apparatus consist of a jar filled with sand attached to a cone. A valve is attached on the cone to control sand fall. The apparatus is put above the dug

hole and the valve is open. The apparatus is weighted before and after the test. The difference in weight can be used to find the sand weight filling the excavated hole. The volume of the hole is found by dividing the sand in the hole weight to its density.



- Rubber balloon method:

A small hole is dug in the compacted soil (the hole size depends on the soil grain size). The excavated material is then weighted in wet and dry conditions. The volume is determined using the rubber balloon apparatus. The apparatus consist of a tank filled with water attached to a rubber balloon at its end. The apparatus is positioned above the dug hole with the balloon is in the hole. The water volume is tank is determined. The water is then allowed to flow into the balloon with aid of small pressure. The difference in water volume is equal to the hole volume.



- Core cutter method:

This method can only be applied to cohesive soil only. The compacted soil is cut by a core cutter with known volume. The soil density is determined by weighting the soil in the cutter a divide it on its volume.

4- The following data are available in connection with the construction of an embankment:
(a) **Soil from borrow pit:** Natural density = 1.75 g/cm^3, Natural water content = 12%
(b) **Soil after compaction:** density = 2 g/cm^3, water content = 18%.
For every 100 m^3 of compacted soil of the embankment, estimate:
(i) The quantity of soil to be excavated from the borrow pit, and
(ii) The amount of water to be added.

Solution of problem No.4:

(i) Soil after compaction dry weight (solid particles weight) for 100 m^3 = $\frac{2 \text{ X } 100}{1+0.18}$ = 169.5 ton (soil density = 2 t/ m^3)
Soil from borrow pit must have the same dry weight (same solid particles from borrow pit will be placed in the compacted volume) = 169.5 ton
Soil in borrow pit dry unit weight = $\frac{1.75}{1+0.12}$ = 1.5625 t/m^3
Volume of excavated soil = 169.5/1.5625 = **108.48 m^3**

(ii) Amount of water to be added = (0.18 – 0.12) X 169.5 = **10.17 ton = 10.17 m^3** (water unit weight = 1t/ m^3)

5- Proctor compaction test was conducted on a soil sample, and the following observations were made:

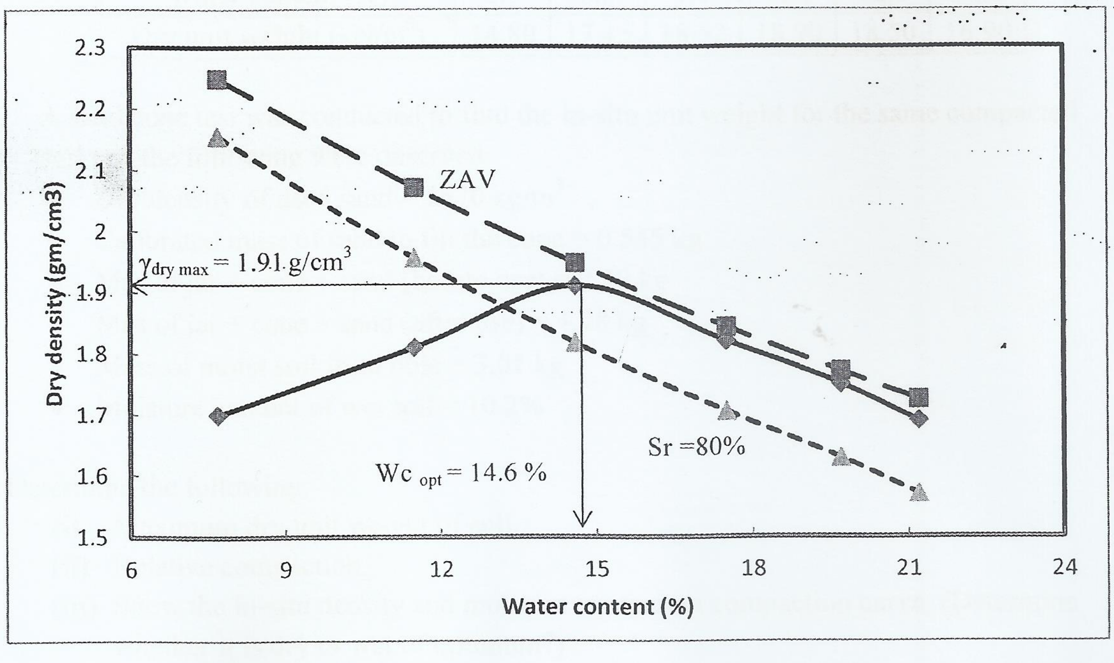
Water content (%)	7.7	11.5	14.6	17.5	19.7	21.2
Mass of wet soil (gm)	1739	1919	2081	2033	1986	1948

If the volume of the mold used was 950 cm^3 and the specific gravity of soils grains was 2.72
Correct the specific gravity of soils grains to 2.72

Make necessary calculations and draw,
(i) Compaction curve, and find maximum dry unit weight and optimum water content
(ii) 80% and 100% saturation lines.

Solution of problem No.5:

Water content (%)	7.7	11.5	14.6	17.5	19.7	21.2
Mass of wet soil (gm)	1739	1919	2081	2033	1986	1948
Wet unit weight (g/cm^3)	1.83	2.02	2.19	2.14	2.09	2.05
Dry unit weight (g/cm^3)	1.70	1.81	1.91	1.82	1.75	1.69
Sr =80% unit weight (g/cm^3)	2.16	1.96	1.82	1.71	1.63	1.57
Sr =100% unit weight (g/cm3)	2.25	2.07	1.95	1.84	1.77	1.73



6- The following observations were recorded when a sand cone test was conducted for finding the unit weight of a natural soil:

- Total density of sand used in the test = 1.4 g/cm^3.
- Mass of the soil excavated from hole = 950 g.
- Mass of the sand filling the hole = 700 g.
- Water content of the natural soil = 15 %.
- Specific gravity of the soil grains = 2.70

<u>Calculate</u>:

- The wet unit weight, the dry unit weight, the void ratio and the degree of saturation.

<u>Solution of problem No.6:</u>

Volume of the hole = 700 /1.4 = 500 cm^3

Wet unit weight of soil = 950 / 500 = **1.9 g/cm^3**

Dry unit weight of soil = $\frac{1.9}{1+0.15}$ = **1.65 g/cm^3**

Voids ratio (e): 1.65 = $\frac{2.70}{1+e}\gamma_w$: **e = 0.64**

The degree of saturation (Sr) = Gs Wc / e = 2.70 X 0.15 /0.64 = **0.63**

7- Laboratory compaction test results for clayey silt are given in the following table:

Water content (%)	6.0	8.0	9.0	11.0	12.0	14.0
Dry unit weight (kN/m^3)	14.80	17.45	18.52	18.90	18.50	16.90

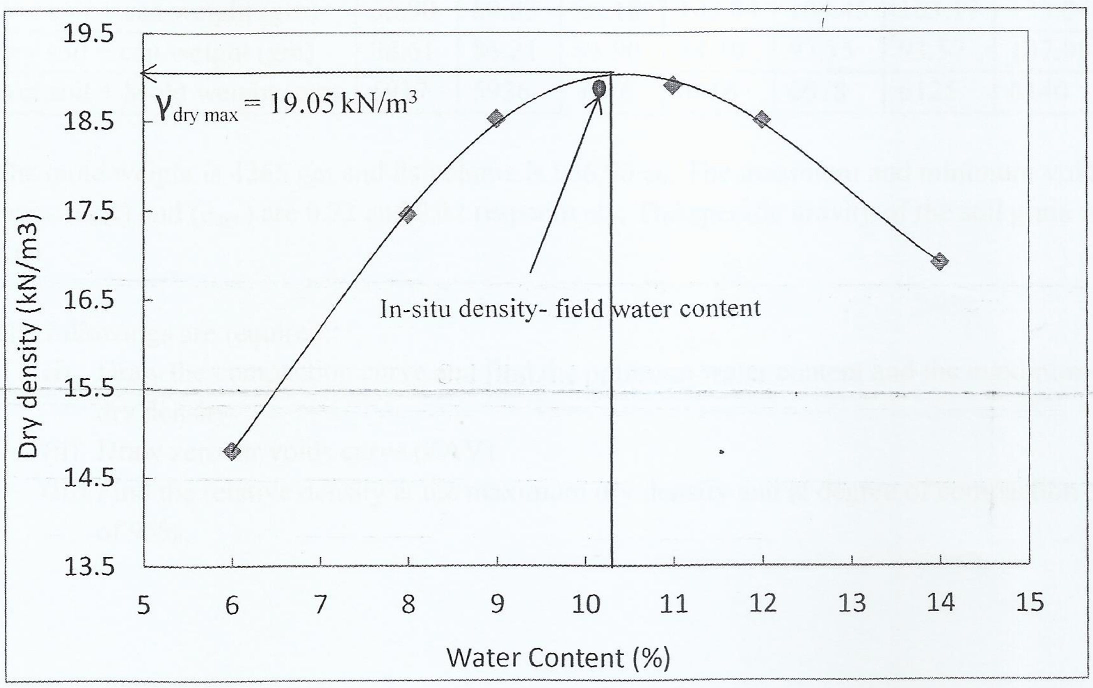
A sand cone test was conducted to find the in-situ unit weight for the same compacted soil and the following were observed:

- Dry density of used sand = 1570 kg/m^3
- Calibrated mass of sand to fill the cone = 0.545 kg
- Mas of jar + cone + sand (before use) = 7.59 kg
- Mas of jar + cone + sand (after use) = 4.78 kg
- Mass of moist soil from hole = 3.01 kg
- Moisture content of wet soil = 10.2%

Determine the following:

(i) Maximum dry unit weight of soil.
(ii) Relative compaction.
(iii) Show the in-situ density and moisture content on compaction curve. (Determine whether it is dry or wet of optimum?)

Solution of problem No.7:



Mass of soil in the hole = 7.59- 4.78- 0.545 = 2.265 kg
Volume of the hole = 2.265 /1570 = 0.00144 m^3
Wet density of the sand = 3.01 / 0.0144 = 2086.4 kg/m^3
Dry density of sand = 2086.4 / (1+0.102) = 1893.3 kg/m^3
Relative compaction = 18.93 / 19.05 = **0.99**

The in-situ density and moisture content are dry of the optimum

8- A rubber balloon test was used to find out the density of soil in field. The initial water volume in the container was 4500 cc. The mass of excavated soil was 2850 gm. If the water volume in the container after the test was 3050 cc and the weight of dry excavated soil was 2590 gm find the dry density of soil in field.

Solution of problem No.8:

Volume of the hole = 4500 – 3050 = 1450 cc
Dry density of the soil = 2590/ 1450 = 1.78 gm/cm^3

9- The following data were obtained from a modified proctor test on a sand specimen

Can weight (gm)	23.15	23.87	23.82	30.51	31.42	23.26	23.00
Wet soil + can weight (gm)	85.90	88.83	96.18	103.84	104.45	103.17	123.04
Dry soil + can weight (gm)	84.61	86.21	91.90	98.10	97.15	93.57	107.98
Wet soil + Mold weight (gm)	5917	5936	5976	6016	6078	6125	6140

The mold weight is 4265 gm and its volume is 936.90 cc. The maximum and minimum void ratios (e_{max}) and (e_{min}) are 0.72 and 0.42 respectively. The specific gravity of the soil grain is 2.64

The followings are required:

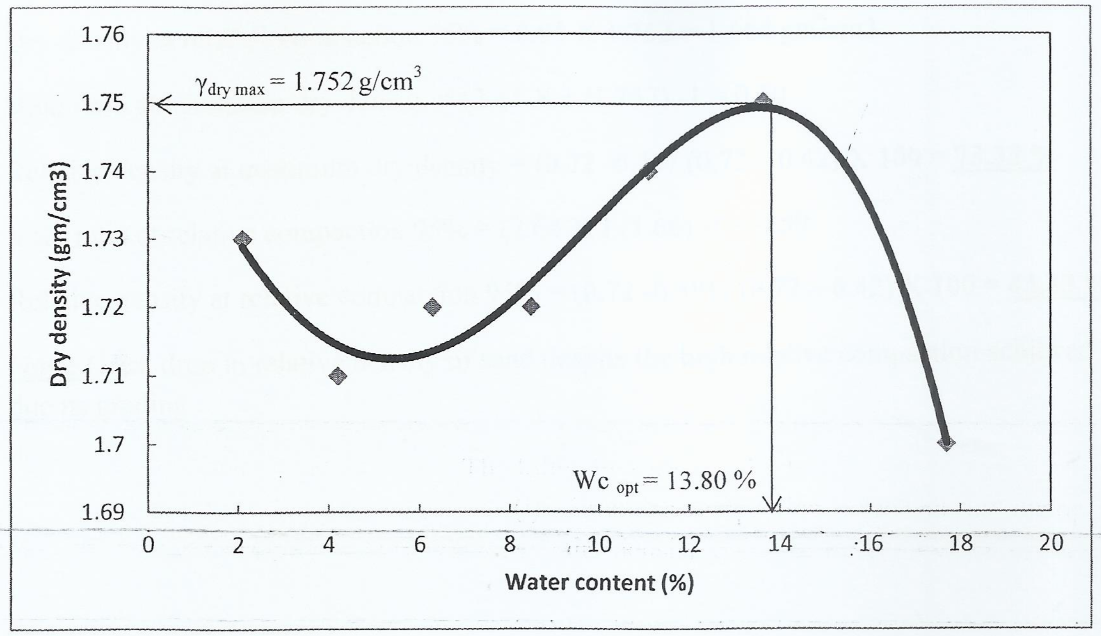
(i) Draw the compaction curve and find the optimum water content and the maximum dry density

(ii) Draw zero air voids curve (ZAV)

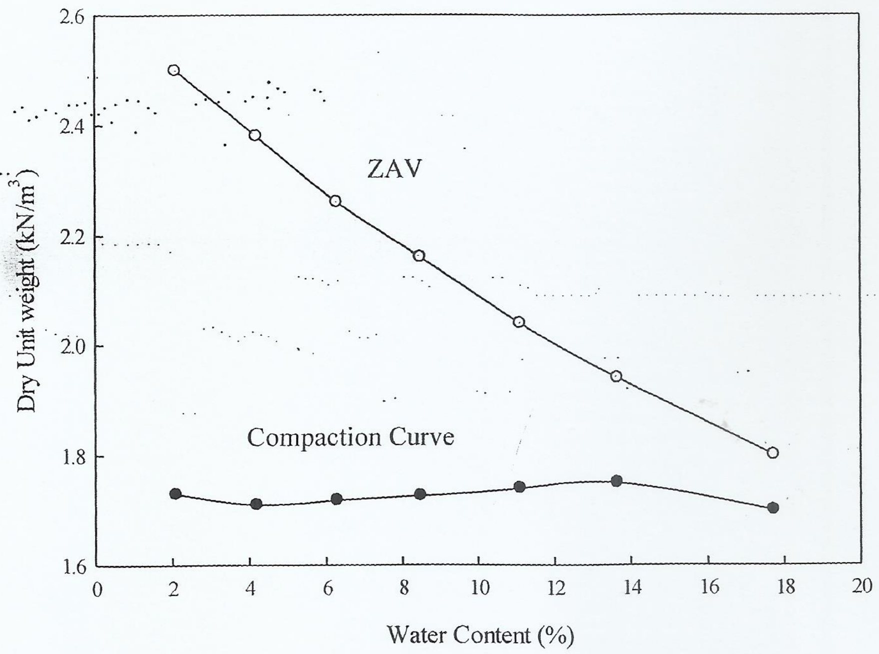
(iii) Find the relative density at the maximum dry density and at degree of compaction of 95%.

<u>Solution of problem No.8:</u>

Can weight (gm)	23.15	23.87	23.82	30.51	31.42	23.26	23.00
Wet soil + can weight (gm)	85.9	88.83	96.18	103.84	104.45	103.17	123.04
Dry soil + can weight (gm)	84.61	86.21	91.9	98.1	97.15	93.57	107.98
Wet soil + Mold weight (gm)	5917	5936	5976	6016	6078	6125	6140
Solid weight (gm)	61.46	62.34	68.08	67.59	65.73	70.31	84.98
Water weight (gm)	1.29	2.62	4.28	5.74	7.30	9.60	15.06
Water content	2.10%	4.20%	6.29%	8.49%	11.11%	13.65%	17.72%
Wet unit weight (gm/cm3)	1.76	1.78	1.83	1.87	1.94	1.99	2.00
Dry unit weight (gm/cm3)	1.73	1.71	1.72	1.72	1.74	1.75	1.70
ZAV (gm/cm3)	2.50	2.38	2.26	2.16	2.04	1.94	1.80



Curve is for poorly graded sand so the density is not affect greatly with change in water content,



Dry density at relative compaction 95% = 0.95 X 1.752 = 1.664 gm/cm3

Void ratio at maximum dry density = (2.64 X 1 /1.752) -1 = 0.50

Relative density at maximum dry density = (0.72 -0.5) / (0.72 – 0.42) X 100 = <u>73.33 %</u>

Void ratio at relative compaction 95% = (2.64 X 1 /1.66) -1 = 0.59

Relative density at relative compaction 95% = (0.72 -0.59) / (0.72 – 0.42) X 100 = <u>43.33 %</u>

Note: Great drop in relative density of sand despite the high relative compaction achieved due its grading

10-In a large project, the granular soil of the site was compacted to a moist unit weight of 18.0 kN/m^3. The compacted soil passed lab tests that showed that the water content was 11.6%, the maximum and minimum void ratios were 0.8 and 0.4 respectively and the specific gravity of soil particles was 2.7.

Determine the following:

(i) Degree of saturation.
(ii) Relative density.
(iii)dry unit weight.
(iv)relative compaction.
(v) ZAV unit weight.

➢ **Solution:**

- $\gamma_b = \frac{Gs(1+Wc)}{1+e} \gamma_w$
- $18 = \frac{2.7(1+0.116)}{1+e} * 10$
- $e = 0.674$
- $Gs.Wc = e.Sr$
- $2.7 * 0.116 = 0.674 * Sr$
- $\therefore Sr = 0.465$
- $Dr = \frac{e\max - e\,nat}{e\max - e\,min} = \frac{0.8-0.674}{0.8-0.4} * 100 = 31.5\%$
- $\gamma_b = \frac{Gs(1+Wc)}{1+e} \gamma_w$
- $\gamma_d = \frac{\gamma b}{1+Wc} = \frac{18}{1+.116} = 16.13$ kN/m3
- $\gamma_{d\,max} = \frac{Gs}{1+e\,min} \gamma_w = \frac{2.7}{1+0.4} * 10 = 19.28$ kN/m3
- $Rc = \frac{\gamma d\ field}{\gamma d\ max} * 100 = \frac{16.13}{19.28} * 100 = 83.66\ \%$
- $\gamma_{d\,ZAV} = \frac{Gs}{1+\frac{Gs.Wc}{Sr=1}} \gamma_w = \frac{2.7}{1+\frac{2.7*0.116}{1}} * 10 = 20.56$ kN/m3

11-For foundation work on a weak soil stratum, a soil volume of 15m x10m x5m is to be removed and replaced with a good soil. The weak soil has a bulk unit weight of 1.45 t/m^3 while the good soil has got maximum dry density of 1.8 t/m^3 at opt. water content of 16%. The specific gravity is 2.7. If the target of the compaction of the fill is to achieve 95% of the maximum dry density, what is the maximum Degree of saturation can be permitted and how much volume of new soil will be needed at this degree of saturation to fill the excavated volume. void ratio for the new soil before compaction = 0.75
Water content allowed range from 14.7% to 17% "to get Rc=95%"

➢ **Solution:**

- $\gamma_d = Rc * \gamma_{d\,max} = 0.95*1.8 = 1.71\ t/m^3$
- $\gamma_d = \frac{Gs}{1+e} \gamma_w$
- $1.71 = \frac{2.7}{1+e} *1$
- $\therefore e = 0.579$
- Max. degree of saturation is at max allowed water content = 17%
- Gs.Wc = e .Sr
- $2.7 * 0.17 = 0.579 * Sr_{max}$
- $\therefore Sr_{max} = 80\ \%$

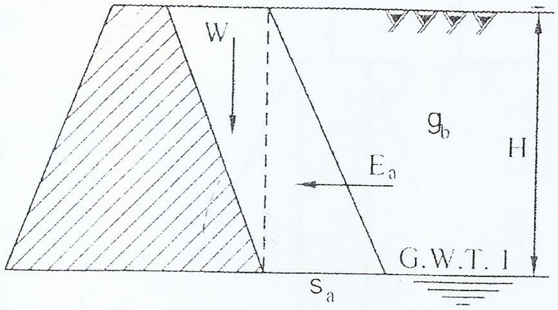
After Compaction : V_{t2}=15*10*5 = 750 m3 , e_2=0.579
Before Compaction: V_{t1}= ?? , e_2 = 0.75

- $V_t = V_s\,(1+e)$
- $750 = V_s\,(1+0.579)$
- $\therefore V_s = 475\ m^3$
- $\therefore V_{t1} = 475(1+0.75) = 831.25\ m^3$

LATERAL EARTH PRESSURE

1-a) Describe the effect of raising the ground water level on the horizontal and vertical forces acting on a retaining wall.

- For G.W.T.1:-



- assume $\gamma_b \cong \gamma_{sat}$

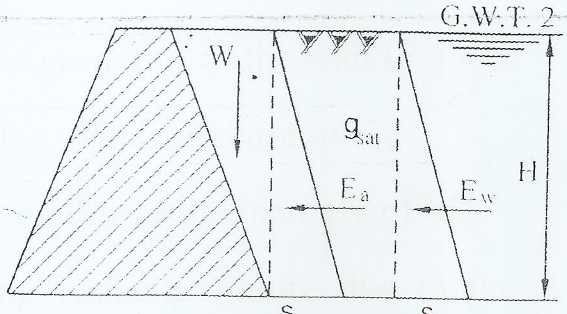
- $\sigma_a = k_a \cdot \gamma_{sat} \cdot H$

- $E_a = \frac{1}{2} \cdot \sigma_a \cdot H = \frac{1}{2} \cdot k_a \cdot \gamma_{sat} \cdot H^2$

$$\Rightarrow E_a = \frac{1}{2} \cdot H^2 (k_a \cdot \gamma_{sub} + k_a \cdot \gamma_w) \quad (1)$$

- $W = \gamma_{sat} \cdot V = \gamma_{sat} \cdot \left(\frac{1}{2} \cdot b \cdot H\right) \quad (2)$

- For G.W.T.2:-



- $\sigma_a = k_a \cdot \gamma_{sub} \cdot H$

- $E_a = \frac{1}{2} \cdot \sigma_a \cdot H = \frac{1}{2} \cdot k_a \cdot \gamma_{sub} \cdot H^2$

- $\sigma_w = \gamma_w \cdot H$

- $E_w = \frac{1}{2} \cdot \sigma_w \cdot H = \frac{1}{2} \cdot \gamma_w \cdot H^2$

$$\Rightarrow E_a + E_w = \frac{1}{2} \cdot k_a \cdot \gamma_{sub} \cdot H^2 + \frac{1}{2} \cdot \gamma_w \cdot H^2$$

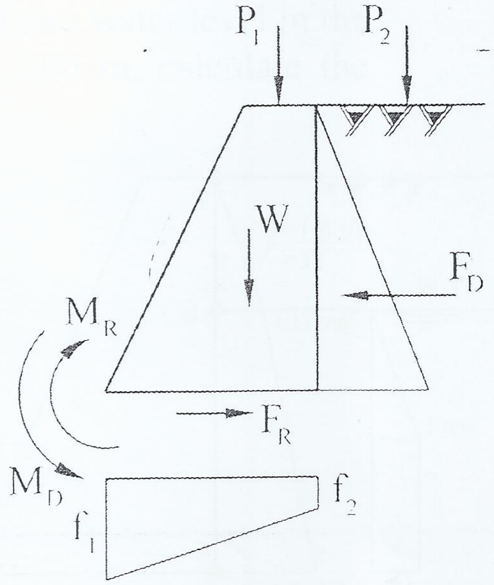
$$\Rightarrow E_a + E_w = \frac{1}{2} \cdot H^2 (k_a \cdot \gamma_{sub} + \gamma_w) \quad (3)$$

- $W = \gamma_{sat} \cdot V = \gamma_{sat} \cdot \left(\frac{1}{2} \cdot b \cdot H\right) \quad (4)$

- From eq. (1) and (3):-
  - When the ground water level raises the lateral force acting on retaining wall increases

- From eq. (2) and (4):-
  - When the ground water level raises the vertical force acting on retaining wall due to soil weight is approximately the same.

1-b) Discuss the influence of existing external loads on the body of the retaining wall or on the ground near the wall on the stability of the wall.



I- Influence of existing external loads on the body of the retaining wall (P_1):-

1- Increases the F.O.S. against sliding due to the increase of F_R.

2- Increases the F.O.S. against overturning due to the increase of M_R.

3- Decreases the F.O.S. against bearing capacity failure due to the increase of stresses under the base of the wall.

II- Influence of existing external loads on the ground near the wall (P_2):-

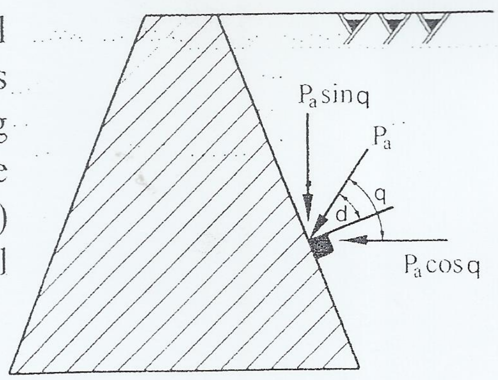
1- Decreases the F.O.S. against sliding due to the decrease of F_D.

2- Decreases the F.O.S. against overturning due to the increase of M_D.

3- Decreases the F.O.S. against bearing capacity failure due to the increase of (f_1) and the decrease of (f_2) under the base of the wall.

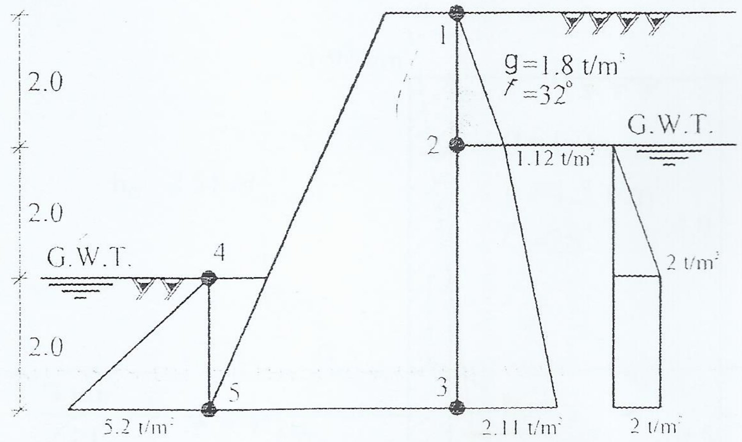
1-c) What is the effect of increasing the friction between the wall surface and the surrounding soil on the wall stability?

- When the friction between the wall surface and the surrounding soil increases (δ increases) the horizontal force acting on the wall ($P_a \cos \theta$) decreases and the vertical force acting on the wall ($P_a \sin \theta$) increases, hence the stability of the wall will increase.



2- A vertical smooth wall of 6.00 m height supports sand backfilling with horizontal surface. Compute the active earth pressure assuming water table to be at 2.00 m below the backfill surface. ($\varphi = 32°$, $\gamma = 1.80\ t/m^3$). If the soil and water level in the other side is above the wall footing by 2.00 m, calculate the passive earth pressure.

- Solution:-



$$- k_a = \frac{1-\sin 32}{1+\sin 32} = 0.31$$

$$- k_p = \frac{1}{k_a} = 3.25$$

$$- \sigma_a = k_a \cdot \sigma_v - 2c\sqrt{k_a}$$

$$- \sigma_{a_1} = 0.31(0) - 0 = 0$$

$$- \sigma_{a_2} = 0.31(2\times1.8) = 1.12\ t/m^2$$

$$- \sigma_{a_3} = 0.31(2\times1.8+4\times0.8) = 2.11\ t/m^2$$

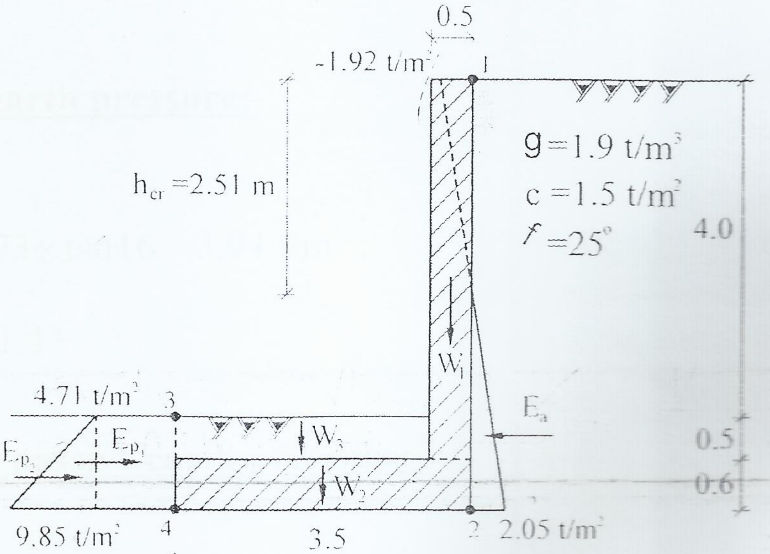
$$- \sigma_p = k_p \cdot \sigma_v + 2c\sqrt{k_p}$$

$$- \sigma_{p_4} = 3.25(0) + 0 = 0$$

$$- \sigma_{p_5} = 3.25(2\times0.8) = 5.2\ t/m^2$$

3- For the shown R.C. wall in Fig. (1), angle of friction between the footing and the soil is 16°, and density of R.C. is 2.5 t/m³. Calculate the factor of safety against sliding using Rankine theory for the following cases:

a. Neglecting the passive resistance
b. Taking the effect of the passive resistance



- Solution:-

1- L.E.P.:-

- $k_a = \dfrac{1-\sin 25}{1+\sin 25} = 0.41$

- $k_p = \dfrac{1}{k_a} = 2.46$

- $\sigma_a = k_a \cdot \sigma_v - 2c\sqrt{k_a}$

- $\sigma_{a_1} = 0.41(0) - 2\times 1.5 \times \sqrt{0.41} = -1.92$ t/m²

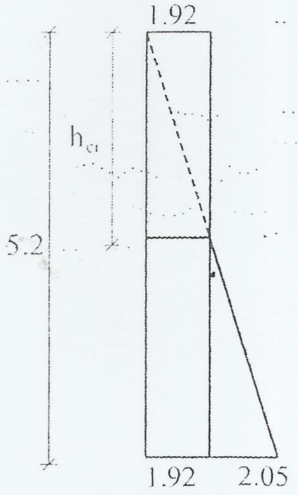
- $\sigma_{a_2} = 0.41(5.1\times 1.9) - 1.92 = 2.05$ t/m²

- $\sigma_p = k_p \cdot \sigma_v + 2c\sqrt{k_p}$

- $\sigma_{p_3} = 2.46(0) + 2\times 1.5\times\sqrt{2.46} = 4.71$ t/m²

- $\sigma_{p_4} = 2.46(1.1\times 1.9) + 4.71 = 9.85$ t/m²

2- Forces:-



- $\dfrac{h_{cr}}{5.2} = \dfrac{1.92}{1.92+2.05} \Rightarrow h_{cr} = 2.51$ m

- $E_a = \dfrac{1}{2}\times 2.05\times(5.2-2.51) = 2.76$ t/m'

- $E_{P_1} = 4.71\times 1.1 = 5.18$ t/m'

- $E_{P_2} = \dfrac{1}{2}\times 1.1\times(9.85-4.71) = 2.83$ t/m'

- $W_1 = 0.5 \times 4.5 \times 2.5 = 5.63 \text{ t/m}'$
- $W_2 = 0.6 \times 3.5 \times 2.5 = 5.25 \text{ t/m}'$
- $W_3 = 0.5 \times 3 \times 1.9 = 2.85 \text{ t/m}'$
- $\sum W = 5.63 + 5.25 + 2.85 = 13.73 \text{ t/m}'$

3- Check sliding:-

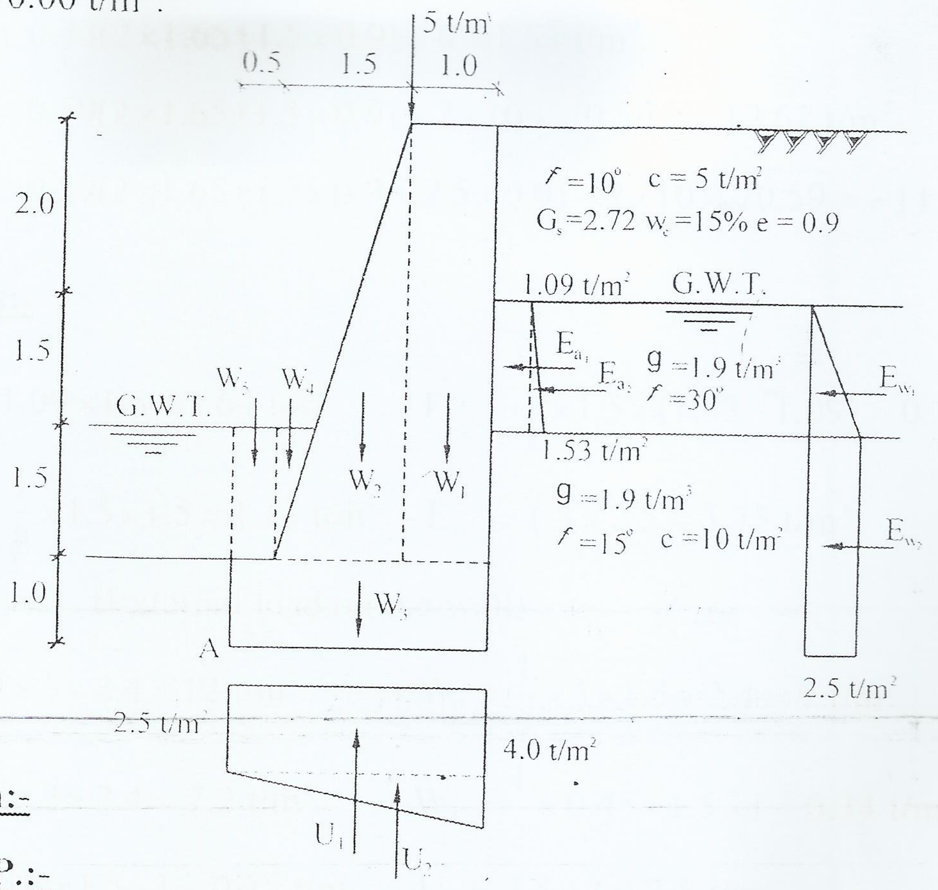
1- Neglecting the passive earth pressure:-

- $F_D = E_a = 2.76 \text{ t/m}'$
- $F_R = \sum W \tan\delta = 13.73 \times \tan 16 = 3.94 \text{ t/m}'$
- $\text{F.O.S.} = \dfrac{F_R}{F_D} = \dfrac{3.94}{2.76} = 1.43$

2- Taking the effect of the passive earth pressure:-

- $F_D = E_a = 2.76 \text{ t/m}'$
- $F_R = E_{p_1} + E_{p_2} + \sum W \tan\delta = 5.18 + 2.83 + 3.94 = 11.95 \text{ t/m}'$
- $\text{F.O.S.} = \dfrac{F_R}{F_D} = \dfrac{11.95}{2.76} = 4.33$

4- Calculate all the factors of safety for the wall shown in fig. (2). $q_{all} = 10.00\ t/m^2$.



- Solution:-

1- L.E.P.:-

- For the 1st soil layer:-

$$- \gamma_b = \frac{G_s(1+w_c)}{1+e} \cdot \gamma_w = \frac{2.72(1+0.15)}{1+0.9} \times 1 = 1.65\ t/m^3$$

$$- k_a = \frac{1-\sin 10}{1+\sin 10} = 0.7$$

- For the 2nd soil layer:-

$$- k_a = \frac{1-\sin 30}{1+\sin 30} = 0.33$$

- For the 3rd soil layer:-

$$- k_a = \frac{1-\sin 15}{1+\sin 15} = 0.59$$

$$- \sigma_{a_1} = 0.7(0) - 2 \times 5 \times \sqrt{0.7} = -8.37\ t/m^2$$

$$- \sigma_{a_2} = 0.7(2 \times 1.65) - 8.37 = -6.06\ t/m^2$$

- $\sigma_{a_3} = 0.33(2\times1.65) - 0 = 1.09\ t/m^2$

- $\sigma_{a_4} = 0.33(2\times1.65+1.5\times0.9) - 0 = 1.53\ t/m^2$

- $\sigma_{a_5} = 0.59(2\times1.65+1.5\times0.9) - 2\times10\times\sqrt{0.59} = -12.62\ t/m^2$

- $\sigma_{a_6} = 0.59(2\times1.65+1.5\times0.9+2.5\times0.9) - 2\times10\times\sqrt{0.59} = -11.29\ t/m^2$

2- Forces:-

- $E_{a_1} = 1.09\times1.5 = 1.64\ t/m$ - $E_{a_2} = \frac{1}{2}\times1.5\times(1.53-1.09) = 0.33\ t/m$

- $E_{w_1} = \frac{1}{2}\times1.5\times1.5 = 1.13\ t/m$ - $E_{w_2} = 1.5\times2.5 = 3.75\ t/m$

- $P = 5\ t/m$ (External load on the wall)

- $W_1 = 1\times5\times2.4 = 12\ t/m$ - $W_2 = \frac{1}{2}\times5\times1.5\times2.4 = 9\ t/m$

- $W_3 = 1\times3\times2.4 = 7.2\ t/m$ - $W_4 = \frac{1}{2}\times0.45\times1.5\times1 = 0.34\ t/m$

- $W_5 = 0.5\times1.5\times1 = 0.75\ t/m$ - $U_1 = 2.5\times3 = 7.5\ t/m$

- $U_2 = \frac{1}{2}\times1.5\times3 = 2.25\ t/m$

3- Check sliding:-

- $F_D = E_{a_1} + E_{a_2} + E_{w_1} + E_{w_2} = 6.85\ t/m$

- $F_R = \sum N \tan\delta + c_a \cdot B$

- $\sum N = P + W_1 + W_2 + W_3 + W_4 + W_5 - U_1 - U_2$

$\Rightarrow \sum N = 5 + 12 + 9 + 7.2 + 0.34 + 0.75 - 7.5 - 2.25 = 24.54\ t/m$

- $\delta = \frac{2}{3}\cdot\phi = \frac{2}{3}\times15 = 10^\circ$, $c_a = \frac{2}{3}\cdot c = \frac{2}{3}\times10 = 6.67\ t/m$

$\Rightarrow F_R = 24.54\times\tan10^\circ + 6.67\times3 = 24.34\ t/m$

- $F.O.S. = \frac{F_R}{F_D} = \frac{24.34}{6.85} = 3.55 > 1.5 \Rightarrow$ safe

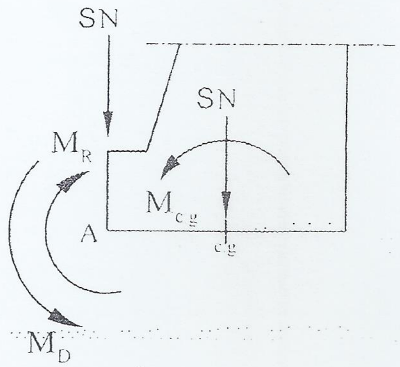
4- Check overturning:-

$$\text{- F.O.S.} = \frac{\sum M_{R_{@A}}}{\sum M_{D_{@A}}}$$

F_D t/m\`	Arm m	M_D m.t/m\`	F_R t/m\`	Arm m	M_R m.t/m
E_{a1}=1.64	3.25	5.33	P=5.0	2.0	10
E_{a2}=0.33	3.0	1.0	W_1=12.0	2.5	30
E_{w1}=1.13	3.0	3.39	W_2=9.0	1.5	13.5
E_{w2}=3.75	1.25	4.69	W_3=7.2	1.5	10.8
U_1=7.5	1.5	11.25	W_4=0.34	0.65	0.22
U_2=2.25	2.0	4.5	W_5=0.75	0.25	0.19
		30.16			64.71

$$\Rightarrow \text{F.O.S.} = \frac{64.71}{30.16} = 2.15 > 1.5 \Rightarrow \text{Safe}$$

5- Check bearing capacity:-

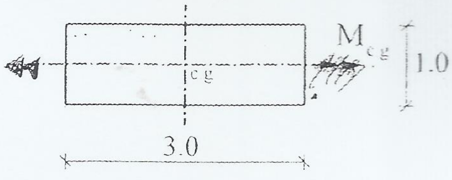


$$\text{-} \sum N = 24.54 \text{ t/m}`$$

$$\text{-} M_{c.g} = 24.54 \times 1.5 + 30.16 - 64.71$$

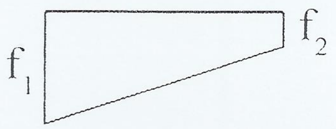
$$\Rightarrow M_{c.g} = 2.26 \text{ m.t/m}`$$

$$\text{- } e = \frac{M_{c.g}}{N} = \frac{2.26}{24.54} = 0.09 \text{ m}$$



$$\text{- } f_1 = \frac{N}{A}\left(1 + \frac{6e}{B}\right) = \frac{24.54}{3}\left(1 + \frac{6 \times 0.09}{3}\right)$$

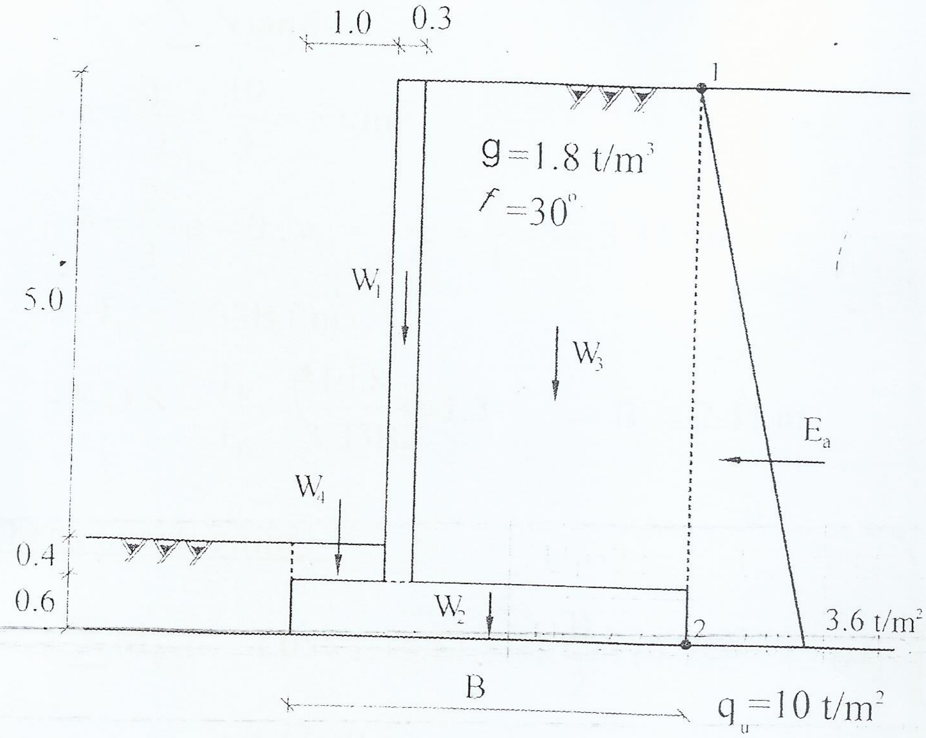
$$\Rightarrow f_1 = 9.65 \text{ t/m}^2 < q_{all} = 10 \text{ t/m}` \Rightarrow \text{Safe}$$



$$\text{- } f_2 = \frac{N}{A}\left(1 - \frac{6e}{B}\right) = \frac{24.54}{3}\left(1 - \frac{6 \times 0.09}{3}\right)$$

$$\Rightarrow f_2 = 6.71 \text{ t/m}^2 > \text{Zero} \Rightarrow \text{Safe}$$

5- Calculate the minimum breadth (B) for the smooth retaining wall in fig. (3) to have a factor of safety 1.30 against sliding, 1.50 against overturning, and the safe gross bearing capacity is 20.00 t/m2.



- Solution:-

1- L.E.P.:-

$$- k_a = \frac{1-\sin 30}{1+\sin 30} = 0.33$$

$$- \sigma_{a_1} = 0$$

$$- \sigma_{a_2} = 0.33(6 \times 1.8) = 3.6 \text{ t/m}^2$$

2- Forces:-

$$- E_a = \frac{1}{2} \times 3.6 \times 6 = 10.8 \text{ t/m}'$$

$$- W_1 = 0.3 \times 5.4 \times 2.5 = 4.05 \text{ t/m}'$$

$$- W_2 = 0.6 \times B \times 2.5 = 1.5B \text{ t/m}'$$

$$- W_3 = 5.4 \times (B - 1.3) \times 1.8 = 9.72B - 12.64 \text{ t/m}'$$

$$- W_4 = 1 \times 0.4 \times 1.8 = 0.72 \text{ t/m}'$$

$$- \sum W = 11.22B - 7.87 \text{ t/m}'$$

3- Check sliding:-

$$- F_D = E_a = 10.8 \text{ t/m}'$$

$$- F_R = \sum N \tan\delta + c_a \cdot B$$

$$- c = \frac{q_u}{2} = \frac{10}{2} = 5 \text{ t/m}^2$$

$$- \delta = \frac{2}{3} \cdot \phi = 0 \text{ , } c_a = \frac{2}{3} \cdot c = \frac{2}{3} \times 5 = 3.33 \text{ t/m}'$$

$$\Rightarrow F_R = 3.33B \text{ t/m}'$$

$$- \text{F.O.S.} = \frac{F_R}{F_D} = \frac{10.8}{3.33B} = 1.3 \qquad \Rightarrow B_1 = 2.31 \text{ m}$$

4- Check overturning:-

$$- \sum M_{R_{@A}} = 4.05 \times 1.15 + 1.5B \times \frac{B}{2} + (9.72B - 12.64) \times \left(\frac{B+1.3}{2}\right) + 0.72 \times 0.5$$

$$\Rightarrow \sum M_{R_{@A}} = 4.66 + 0.75B^2 + 4.86B^2 - 8.22 + 0.36$$

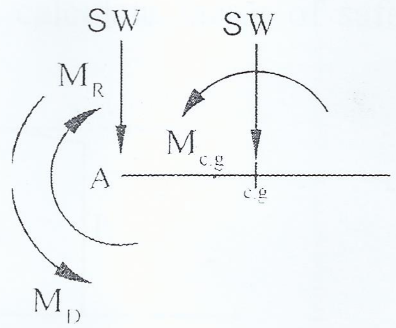
$$\Rightarrow \sum M_{R_{@A}} = 5.61B^2 - 3.2$$

$$- \sum M_{D_{@A}} = 10.8 \times 2 = 21.6 \text{ m.t/m}'$$

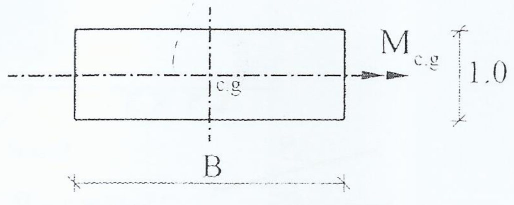
$$- \text{F.O.S.} = \frac{\sum M_{R_{@A}}}{\sum M_{D_{@A}}} = \frac{5.61B^2 - 3.2}{21.6} = 1.5$$

$$\Rightarrow 5.61B^2 - 35.6 = 0 \qquad \Rightarrow B_2 = 2.52 \text{ m}$$

5- Check bearing capacity:-



- $\sum W = 11.22B - 7.87$
- $M_R = 5.61B^2 - 3.2$
- $M_D = 10.8 \times 2 = 21.6$ m.t/m

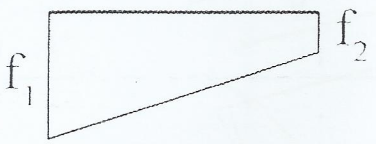


$$- M_{c.g} = (11.22B - 7.87) \times \frac{B}{2}$$

$$+ 21.6 - (5.61B^2 - 3.2)$$

$$\Rightarrow M_{c.g} = -3.94B + 18.4$$

$$- e = \frac{M_{c.g}}{N} = \frac{-3.94B + 18.4}{11.22B - 7.87}$$



$$- f_1 = \frac{N}{A}\left(1 + \frac{6e}{B}\right)$$

$$= \frac{11.22B - 7.87}{B}\left(1 + \frac{6(-3.94B + 18.4)}{B(11.22B - 7.87)}\right) = 20$$

$$\Rightarrow B_3 = 2.18 \text{ m}$$

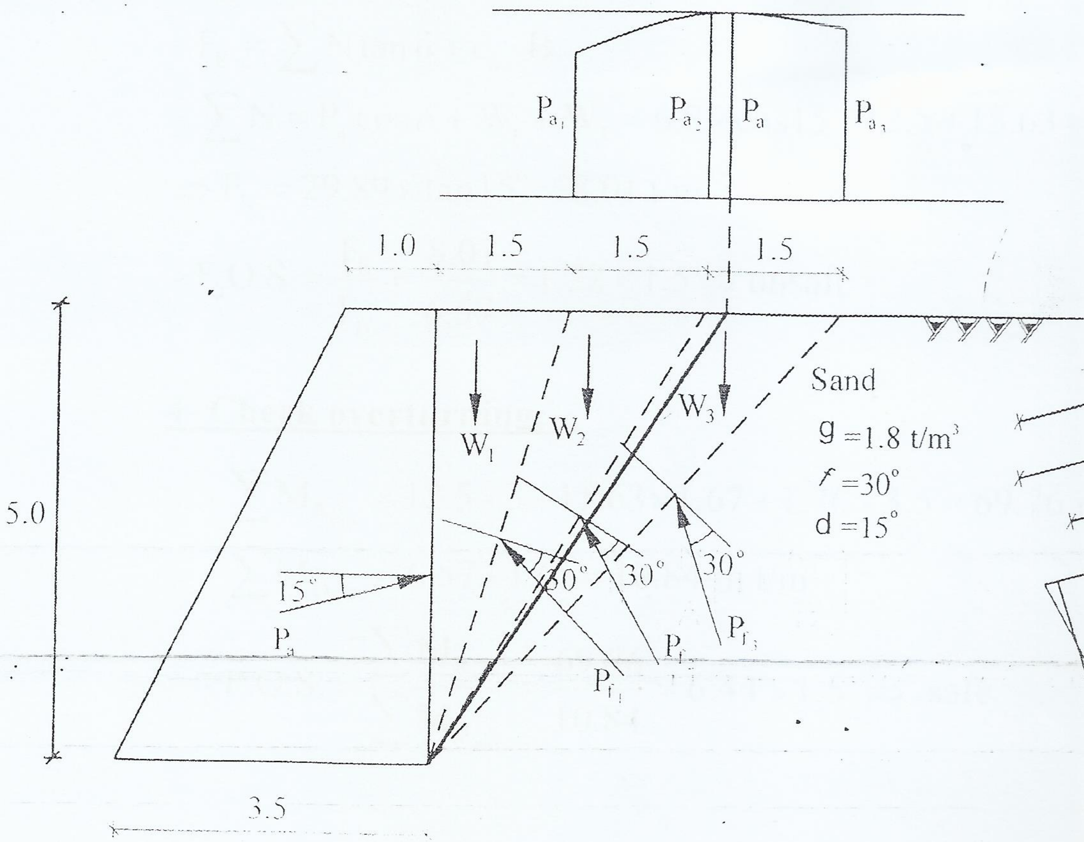
$$- f_2 = \frac{N}{A}\left(1 - \frac{6e}{B}\right) = \frac{11.22B - 7.87}{B}\left(1 - \frac{6(-3.94B + 18.4)}{B(11.22B - 7.87)}\right) = 0$$

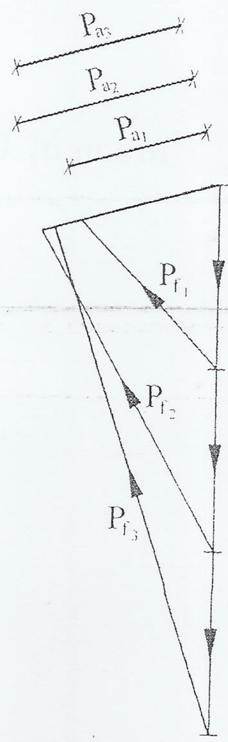
$$\Rightarrow B_4 = 2.51 \text{ t/m}^2$$

- From B_1 to B_4:-

$$\Rightarrow \text{Use} \quad B = 2.52 \text{ t/m}^2$$

6- For the given wall shown in fig (4), calculate factor of safety against sliding and overturning.





- Solution:-

$$- W_1 = W_2 = W_3 = \frac{1}{2} \times 1.5 \times 5 \times 1.8 = 6.75 \text{ t/m}'$$

- From graph:-

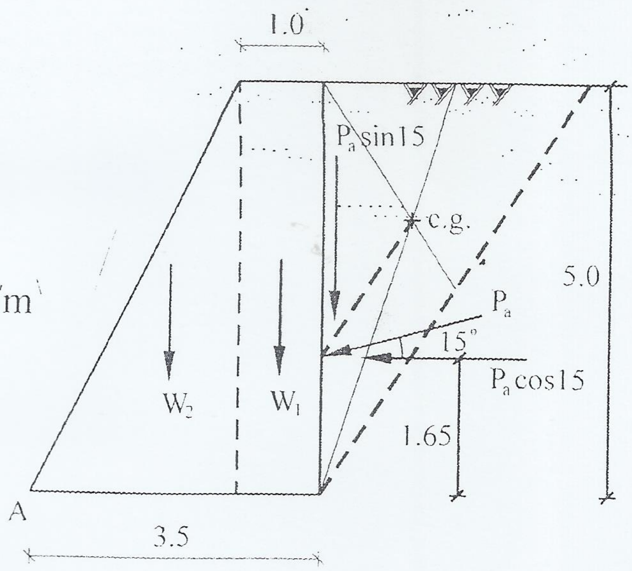
- $P_a = 6.8$ t/m'

- Forces:-

- $P_a = 6.8$ t/m'

- $W_1 = 1 \times 5 \times 2.5 = 12.5$ t/m'

- $W_2 = \frac{1}{2} \times 5 \times 2.5 \times 2.5 = 15.63$ t/m'



3- Check sliding:-

- $F_D = P_a \cos\delta = 6.8 \times \cos 15 = 6.57$ t/m'
- $F_R = \sum N \tan\delta + c_a \cdot B$
- $\sum N = P_a \cos\delta + W_1 + W_2 = 6.8 \times \cos 15 + 12.5 + 15.63 = 29.89$ t/m'

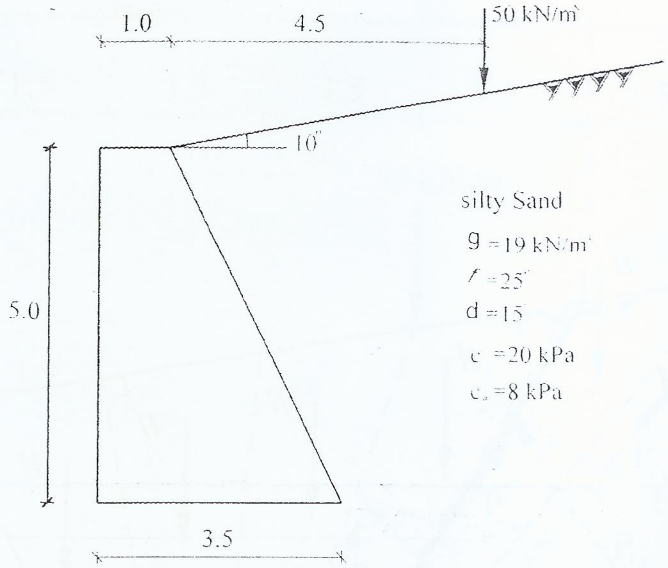
$\Rightarrow F_R = 29.89 \times \tan 15^\circ = 8.01$ t/m'

- $F.O.S. = \dfrac{F_R}{F_D} = \dfrac{8.01}{6.57} = 1.22 < 1.5 \Rightarrow$ unsafe

4- Check overturning:-

- $\sum M_{R_{@A}} = 12.5 \times 3 + 15.63 \times 1.67 + 1.76 \times 3.5 = 69.76$ m.t/m'
- $\sum M_{D_{@A}} = 6.57 \times 1.65 = 10.84$ m.t/m'
- $F.O.S. = \dfrac{\sum M_{R_{@A}}}{\sum M_{D_{@A}}} = \dfrac{69.76}{10.84} = 6.44 > 1.5 \Rightarrow$ safe

7- For the wall shown below, find the critical failure wedge using Coulomb active earth pressure theory. Also calculate the value of the earth pressure

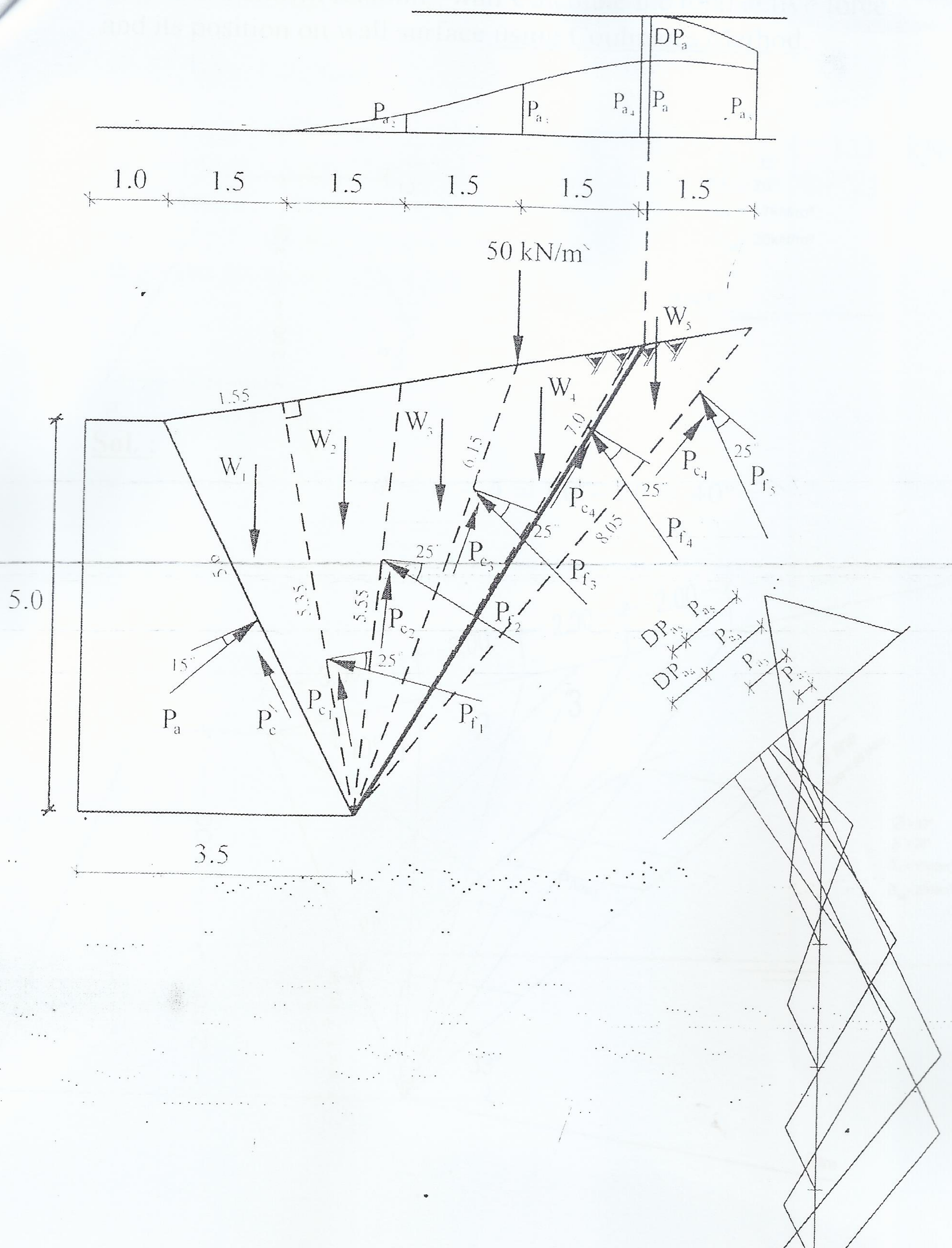


- Solution:-

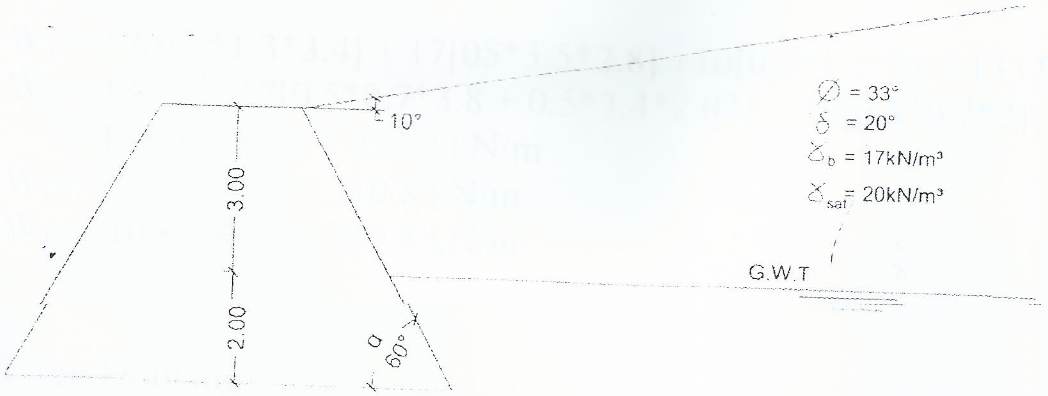
- $W_1 = W_2 = W_3 = W_4 = \frac{1}{2} \times 1.55 \times 5.35 \times 19 = 78.8$ kN/m'
- $P_c' = c_a \cdot L' = 8 \times 5.6 = 44.8$ kN/m'
- $P_{c_1} = c \cdot L_1 = 20 \times 5.35 = 107$ kN/m'
- $P_{c_2} = 20 \times 5.55 = 111$ kN/m'
- $P_{c_3} = 20 \times 6.15 = 123$ kN/m'
- $P_{c_4} = 20 \times 7 = 140$ kN/m'

- From graph:-

- $P_a = 48$ kN/m'
- $\Delta P_a = 30$ kN/m'

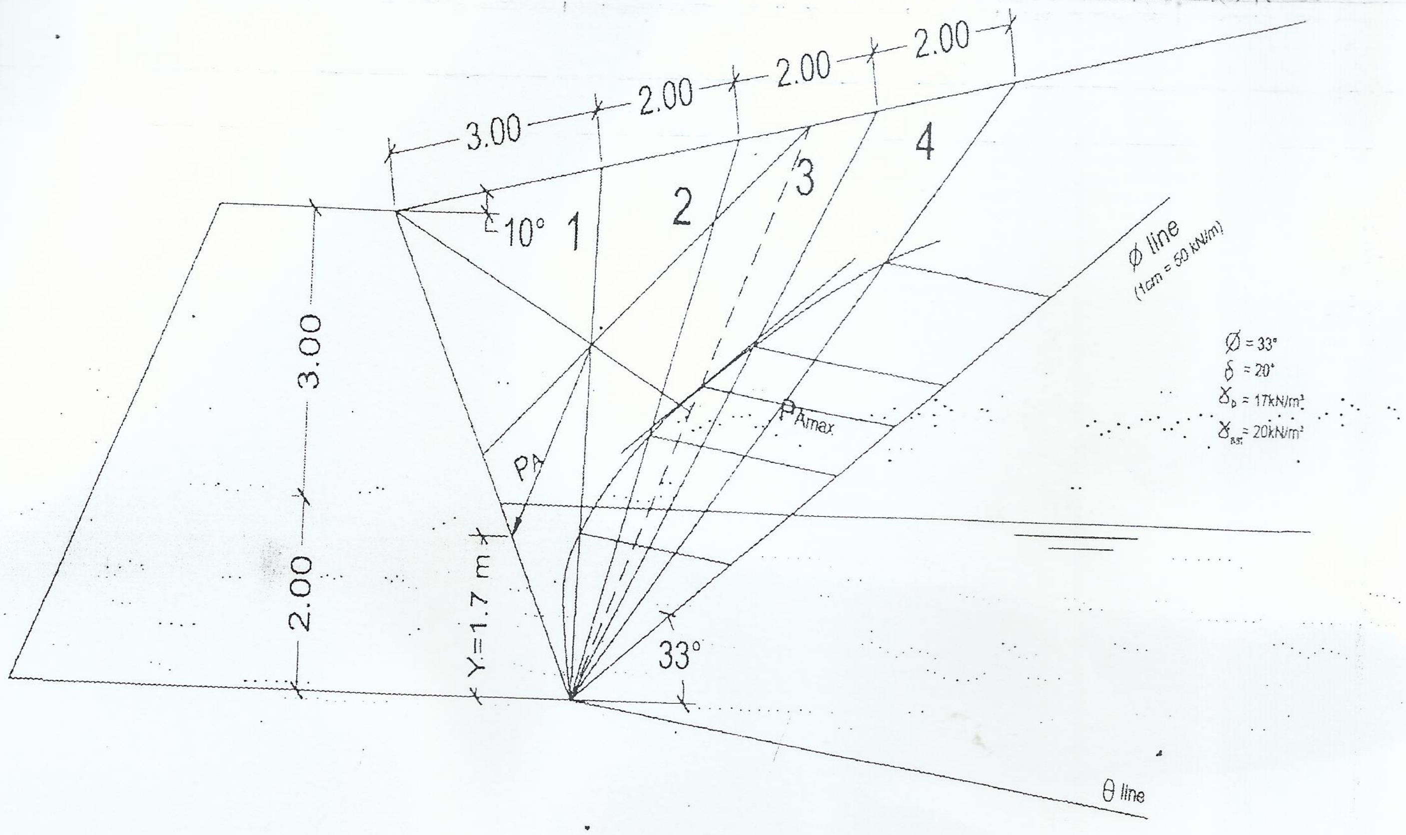

DPa
Pa2
Pa3
Pa4
Pa
Pa5
1.0
1.5
1.5
1.5
1.5
1.5
50 kN/m
W5
W4
W3
W2
W1
1.55
7.0
6.15
8.05
5.35
5.55
5.0
3.5
25°
15°
Pa
Pc
Pc1
Pc2
Pc3
Pc4
Pf1
Pf2
Pf3
Pf4
Pf5
DPa5
Pa5
Pa4
DPa4
Pa3
Pa2

8. For the shown retaining wall Calculate the total active force and its position on wall surface using Coulman's Method.



Sol. :

$$\theta = \alpha - \delta = 60 - 20 = 40°$$



Calculations of weights:

$$W = A_1\gamma_b + A_2\gamma_{sub}$$

W_1 = 17[0.5*1.3*3.4] + 17[05*3.5*2.8] +10[0.5*1.3*2] = 133.9 kN/m
W_2 = 133.9 + 17[0.5*0.7*3.8 + 0.5*3.4*2.03] + 10[0.5*0.7*2]
= 133.9 + 88.6 = 222.2 kN/m
W_3= 222.2+ 88.6 = 310.8 kN/m
W_4= 310.8+ 88.6 = 399.4 kN/m

From Drawing :
P_{Amax} = 150 kN/m

Y = 1.7 m

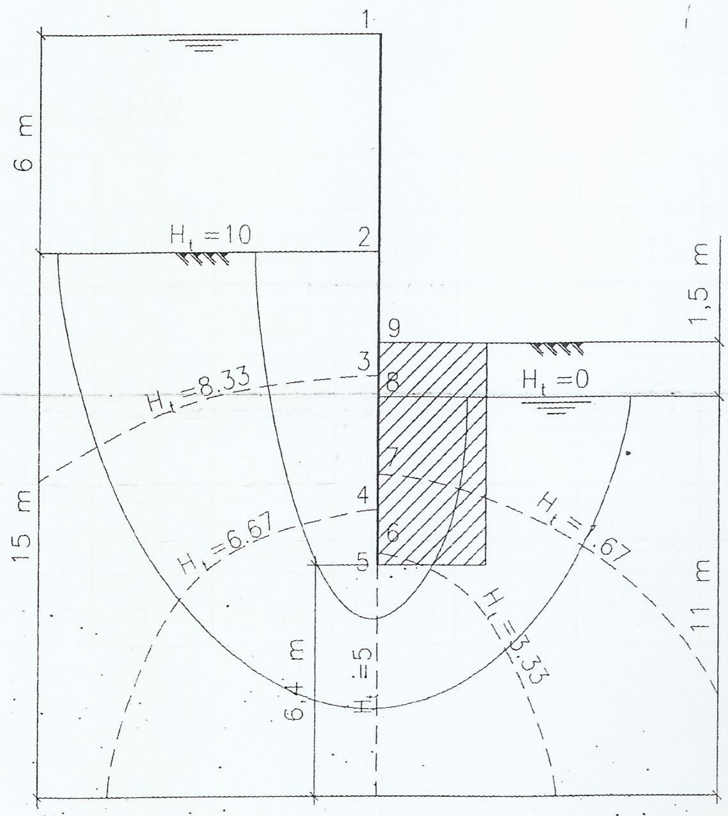
-No

Assignment (3)

Seepage through soil

Model Answer

Question (4):



From drawing: $N_F = 3$

$N_d = 6$

$q = KxHx(N_F / N_d) = (7.2x10^{-2}/100)x10x(3/6)x24x60x60$

$= 311.04 \; m^3/day/m$

Check Heave:

F.O.S. = Soil weight / Seepage force

$= (bxh_1 \, x\gamma_{sub} + bxh_2 \, x\gamma_{bulk}) / (bxH_{avg.} \, x\gamma_{sub})$

$= (3.05x4.6x1 + 3.05x1.5x2) / (3.05x((5+2.5)/2)x1)$

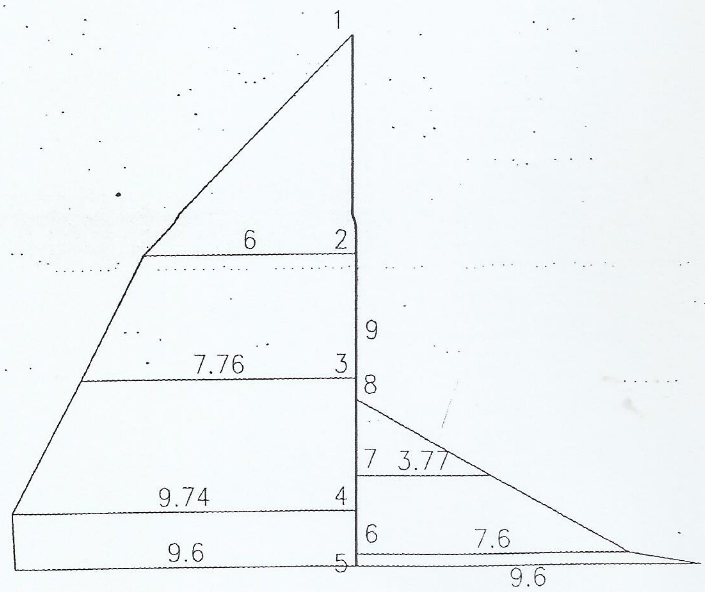
$= 2.02 > 1.5$ SAFE

Check Erosion:

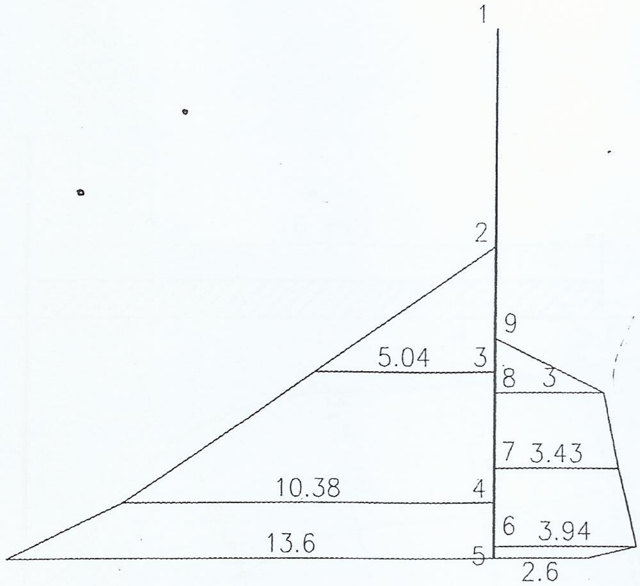
$$F.O.S. = i_{cr.} / i_{ex.}$$
$$= (\gamma_{sub} / \gamma_w) / (\Delta H / L_{ex})$$
$$= (1/1)/(1.67/2.11)$$
$$= 1.26 < 1.5 \quad \text{UnSAFE}$$

Calc. of stresses on S.P.W.:

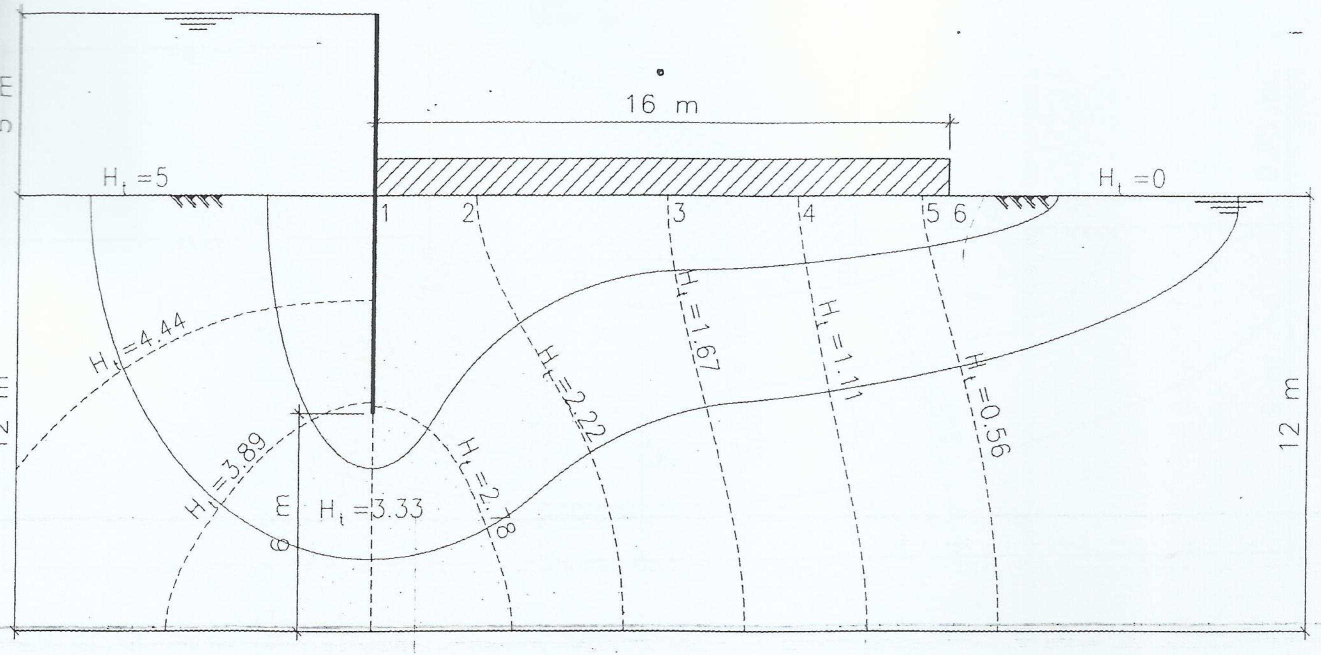
Point	1	2	3	4	5	6	7	8	9
H_t (m)	10	10	8.33	6.67	5	3.33	1.67	0	0
H_e (m)	10	4	0.57	−3.07	−4.6	−4.27	−2.1	0	1.5
H_p (m)	0	6	7.76	9.74	9.6	7.6	3.77	0	0
μ (t/m^2)	0	6	7.76	9.74	9.6	7.6	3.77	0	0
σ_t (t/m^2)	0	6	12.8	20.12	23.2 12.2	11.54	7.2	3	0
$\sigma_{eff.}$ (t/m^2)	0	0	5.04	10.38	13.6 2.6	3.94	3.43	3	0



Pore water pressure diagram



Effective stress diagram



From drawing: $N_F = 3$

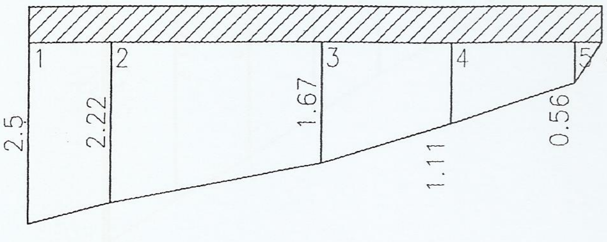
$N_d = 9$

$q = K \times H \times (N_F / N_d) = 7.5\times10^{-6} \times 5 \times (3/9) \times 24 \times 60 \times \cancel{60} \times 50$

$= 54.00 \ m^3/day/50m$

Calc. of Uplift on blanket:

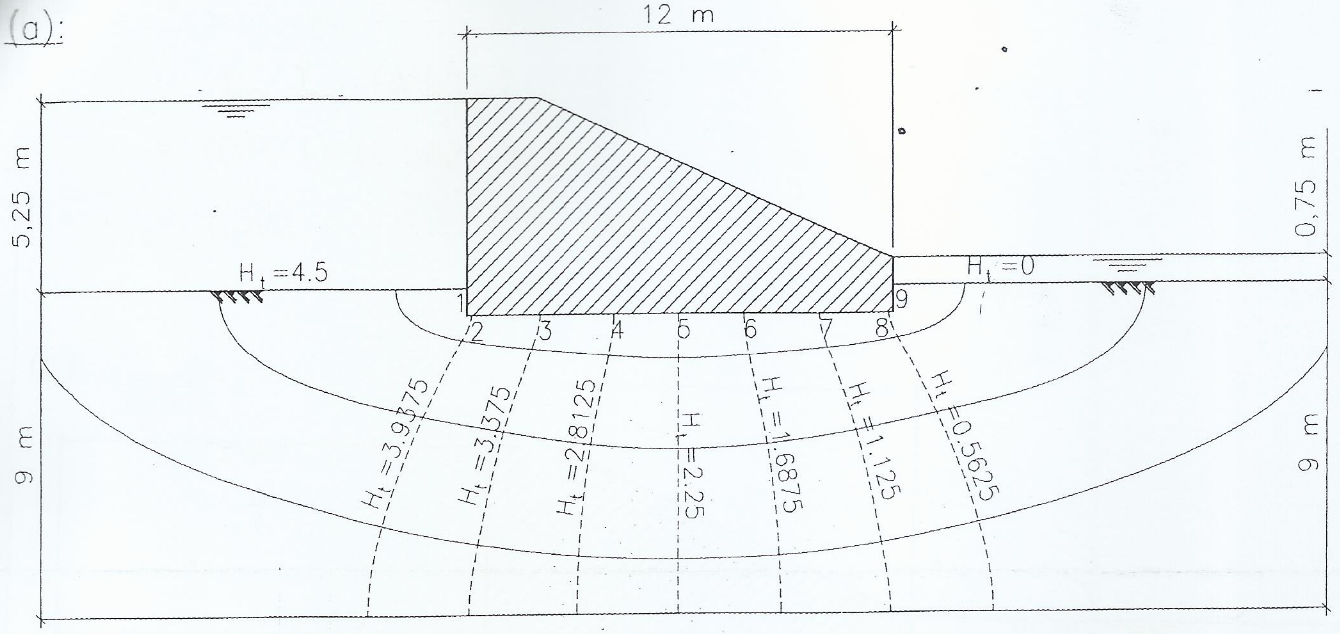
Point	1	2	3	4	5	6
H_t (m)	2.5	2.22	1.67	1.11	0.56	0
H_e (m)	0	0	0	0	0	0
H_p (m)	2.5	2.22	1.67	1.11	0.56	0
U (t/m^2)	2.5	2.22	1.67	1.11	0.56	0



Uplift pressure diagram

Question (6):

(a):



From drawing: $N_F = 4$

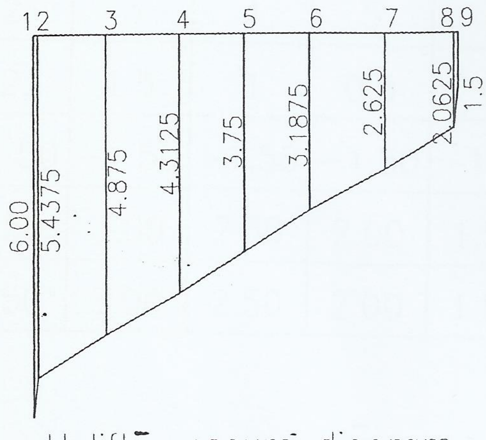
$N_d = 8$

$$q = K \times H \times (N_F / N_d) = (6.1 \times 10^{-2}/100) \times 4.5 \times (4/8) \times 24 \times 60 \times 60$$

$$= 118.584 \ m^3/day/m$$

Calc. of Uplift on Weir base:

Point	1	2	3	4	5	6	7	8	9
H_t (m)	4.5	3.9375	3.375	2.8125	2.25	1.6875	1.125	0.5625	0
H_e (m)	−1.50	−1.50	−1.50	−1.50	−1.50	−1.50	−1.50	−1.50	−1.50
H_p (m)	6.00	5.4375	4.875	4.3125	3.75	3.1875	2.625	2.0625	1.5
U (t/m^2)	6.00	5.4375	4.875	4.3125	3.75	3.1875	2.625	2.0625	1.5

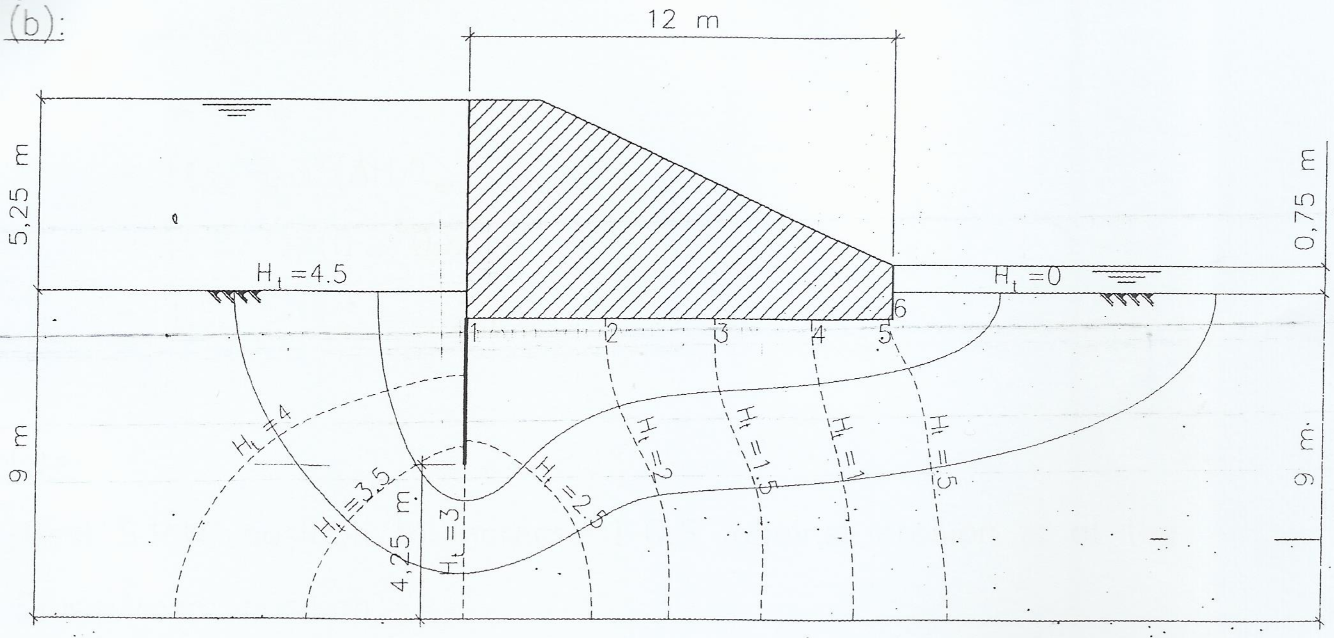


Uplift pressure diagram

Check Erosion:

$$F.O.S. = i_{cr.} / i_{ex.}$$
$$= (\gamma_{sub} / \gamma_w)/(\Delta H / L_{ex})$$
$$= (0.9/1)/(0.5625/0.87)$$
$$= 1.392 < 1.5 \quad \text{Un SAFE}$$

(b):



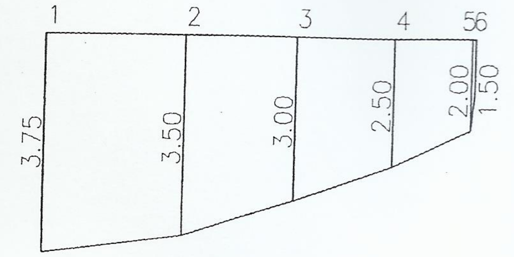
From drawing: $N_F = 3$

$N_d = 9$

$$q = K x H x (N_F / N_d) = (6.1 x 10^{-2}/100) x 4.5 x (3/9) x 24 x 60 x 60$$
$$= 79.056 \ m^3/day/m$$

Calc. of Uplift on Weir base:

Point	1	2	3	4	5	6
H_t (m)	2.25	2	1.5	1	0.5	0
H_e (m)	−1.50	−1.50	−1.50	−1.50	−1.50	−1.50
H_p (m)	3.75	3.50	3.00	2.50	2.00	1.50
U (t/m^2)	3.75	3.50	3.00	2.50	2.00	1.50



Uplift pressure diagram

Check Erosion:

F.O.S. $= i_{cr.} / i_{ex.}$

$= (\gamma_{sub} / \gamma_w)/(\Delta H / L_{ex.})$

$= (0.9/1)/(0.5/0.85)$

$= 1.53 \geq 1.5$ SAFE

Note:

-Best S.P.W. position to increase F.O.S. against erosion is at the Downstream position

-Best S.P.W. position to decrease the Uplift pressure on the wier's base is at the Upstream position

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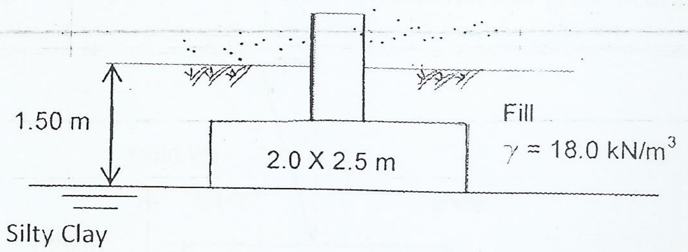
ثالثة مدنى
هندسة جيوتقنية (٢) – هدن ٣٥١
العام الجامعي ٢٠١٤ / ٢٠١٥

Assignment No.5

Bearing Capacity of Shallow Foundation

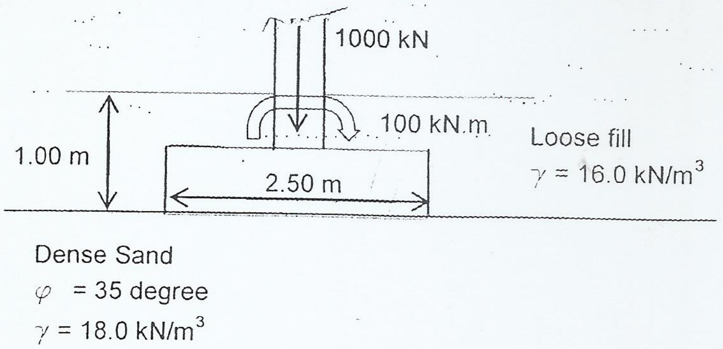
1- Discuss with neat sketches the different shear failure types of shallow foundation.

2- Determine the allowable net bearing capacity of the shown footing in both drained and undrained conditions if the footing is subject to normal load. Comment on the results (safety factor equals 3.0)

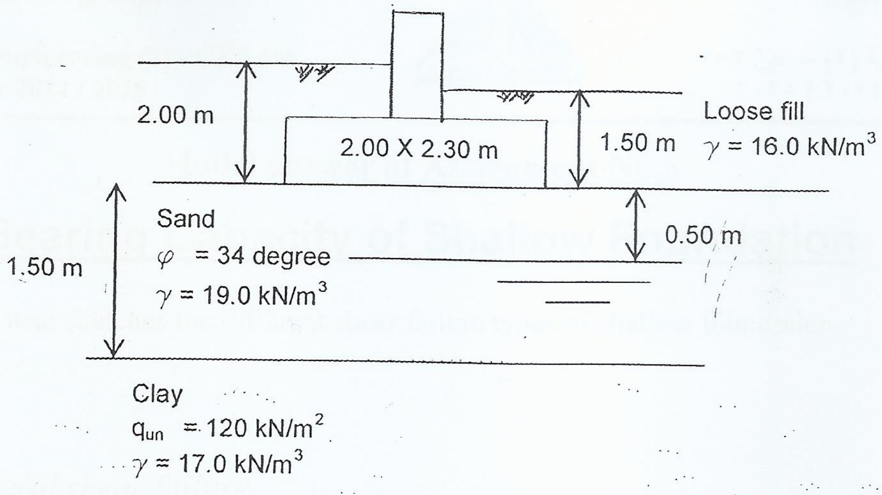


Condition	Cohesion (KPa)	Friction angle (deg)	Saturated Unit weight
Undrained	100	-	17.5 kN/m^3
Drained	30	15	

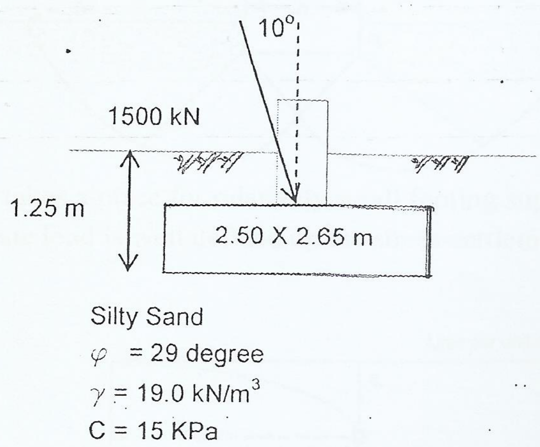
3- A footing 2.50m X 2.25m is subject to a normal load of 1000 kN and a bending moment of 100 kN.m. find the allowable net bearing capacity using a factor of safety 2.0



4- Find the allowable net bearing capacity for the shown below footing. Assume that only vertical force will be applied on the footing. Recalculate the problem if the clay is below the footing and underlain by the sand. compare between the results



5- For the shown footing it is required to find the allowable net and gross bearing capacity



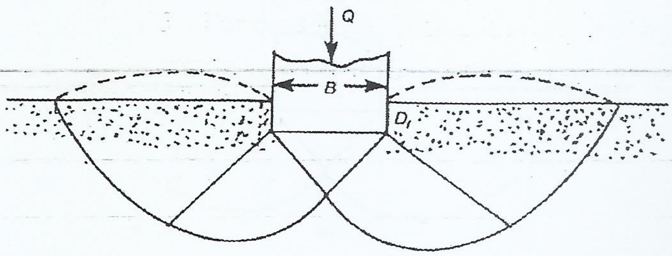
Model answer of Assignment No.5

Bearing Capacity of Shallow Foundation

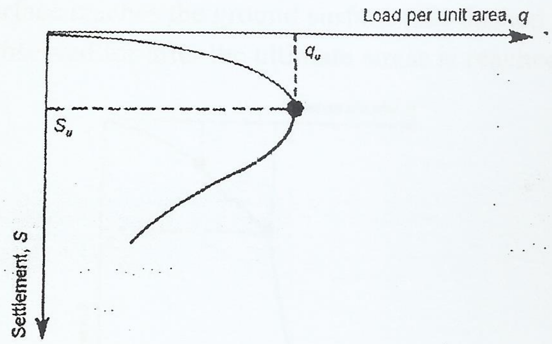
1- Discuss with neat sketches the different shear failure types of shallow foundation.

Problem 1:

- *General shear failure*

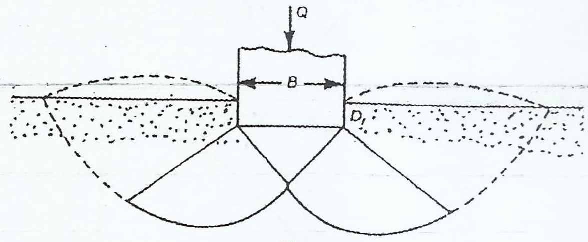


General shear failure takes a place for relatively small footing supported on stiff clay or dense sand, the ultimate load is well defined on the stress-settlement curve as shown below



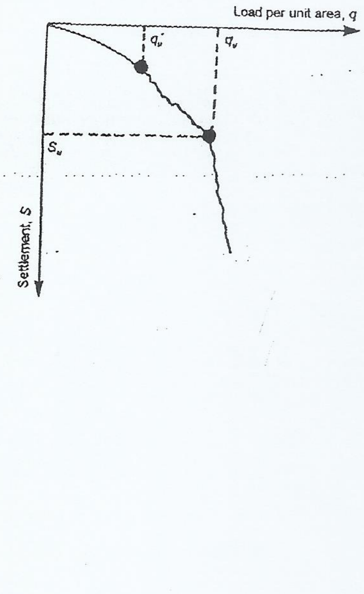
- *Local shear failure*

The local shear failure occur for larger footings supported on relatively medium dense sand or cohesive soil with medium consistency



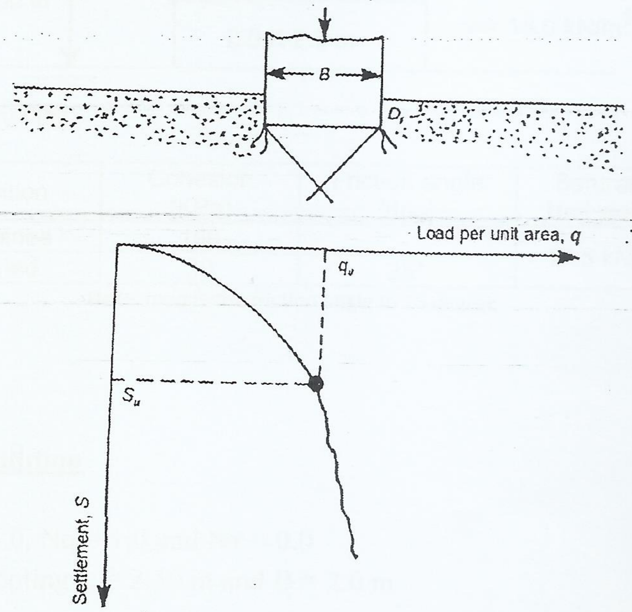
(a)

As stress below the footing increase the settlement increase till the first failure stress (qu') at this stress, the failure surface defined by solid lines is developed. With further increase in the stress the stress-settlement curve become steeper. When the stress reaches the ultimate capacity (qu) the failure surface reaches the ground surface (see dashed lines in the figure above). No peak stress is observed for after the ultimate stress is reached. See below figure

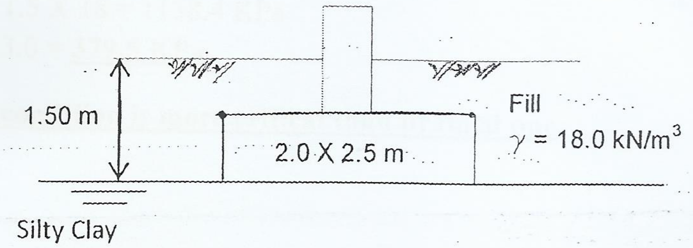


- *Punching shear failure*

'Takes place for footings on relatively weak soil. The peak stress is never observed. the ultimate stress is defined by the point which $\Delta S/\Delta q$ is maximum



2. Determine the allowable net bearing capacity of the shown footing in both drained and undrained conditions if the footing is subject to normal load. Comment on the results (safety factor equals 3.0)



Condition	Cohesion (KPa)	Friction angle (deg)	Saturated Unit weight
Undrained	100	-	17.5 kN/m³
Drained	30	25*	

Note: modify the friction angle to 25 degree

Problem 2:

- **Undrained condition**

φ = zero hence, Nc = 5.0, Nq = 1.0 and Nγ = 0.0
Shape factors for the footing L = 2.50 m and B = 2.0 m

$\lambda c = \lambda q = 1 + 0.3\ (2.0/2.5) = 1.24$
$\lambda\gamma = 1 - 0.3\ (2.0/2.5) = 0.76$
$\gamma_1 = 18\ kN/m^3$ and $\gamma_2 = 7.5\ kN/m^3$

$q_{ultg} = C\ \ Nc\ \ \lambda c + D_f\ \gamma_1\ \ Nq\ \ \lambda q + B\ \gamma_2\ \ N\gamma\ \lambda\gamma$
$q_{ultg} = 100\ X\ 5.0\ X\ 1.24 + 1.50\ X\ 18\ X\ 1.0\ X\ 1.24 + 0 = 653.48$ KPa
$q_{ultnet} = 653.48 - 1.5\ X\ 18 = 626.48$ KPa
$q_{allnet} = 626.48 / 3.0 =$ **208.8 KPa**

- **Drained condition**

φ = 25 hence, Nc 20.5, Nq = 10.5 and Nγ = 4.5
Shape factors for the footing L = 2.50 m and B = 2.0 m

$\lambda c = \lambda q = 1 + 0.3\ (2.0/2.5) = 1.24$
$\lambda\gamma = 1 - 0.3\ (2.0/2.5) = 0.76$

$\gamma_1 = 18$ kN/m^3 and $\gamma_2 = 7.5$ kN/m^3

$q_{ultg} = C\ Nc\ \lambda c + D_f\ \gamma_1\ Nq\ \lambda q + B\ \gamma_2\ N\gamma\ \lambda\gamma$

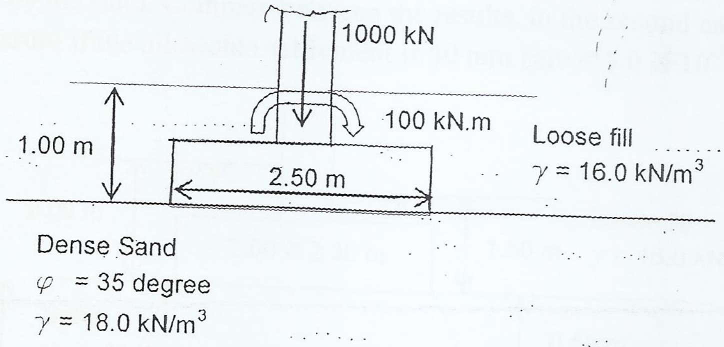
$q_{ultg} = 30 X 20.5 X 1.24 + 1.50 X 18 X 10.5 X 1.24 + 2 X 7.50 X 4.5 X 0.76 = 1165.4$ KPa

$q_{ultnet} = 1165.4 - 1.5 X 18 = 1138.4$ KPa

$q_{allnet} = 1138.4 / 3.0 =$ **379.5 KPa**

The undrained condition is more critical than drained one

3. A footing 2.50m X 2.25m is subject to a normal load of 1000 kN and a bending moment of 100 kN.m. find the allowable net bearing capacity using a factor of safety 2.0



Problem 3:

$\varphi = 35$ hence, $Nc = 46.0$, $Nq = 33.0$ and $N\gamma = 23.0$

Shape factors for the footing $L = 2.50$ m and $B = 2.25$ m

$e_L = M/N = 100/1000 = 0.1$

$L' = L - 2e_L = 2.50 - 0.1 \times 2 = 2.30$

$\lambda c = \lambda q = 1 + 0.3\ (2.25/2.3) = 1.29$

$\lambda\gamma = 1 - 0.3\ (2.25/2.3) = 0.71$

$\gamma_1 = 16$ kN/m^3 and $\gamma_2 = 18.0$ kN/m^3

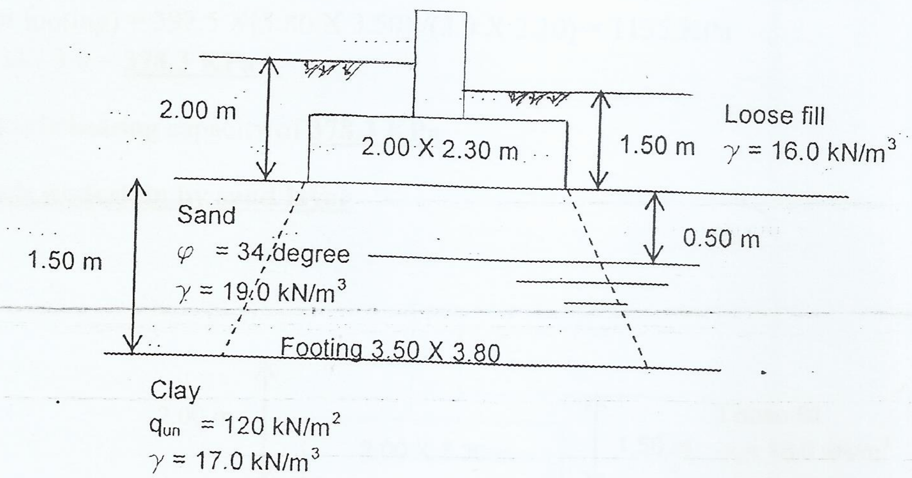
$q_{ultg} = C\ Nc\ \lambda c + D_f\ \gamma_1\ Nq\ \lambda q + B\ \gamma_2\ N\gamma\ \lambda\gamma$

$q_{ultg} = 0 + 1.00 \times 16 \times 33.0 \times 1.29 + 2.25 \times 18 \times 23 \times 0.71 = 1342.5$ KPa

$q_{ultnet} = 1342.5 - 1.0 \times 16 = 1326.5$ KPa

$q_{allnet} = 1326.5 / 2.0 =$ **<u>663.24 KPa</u>**

4. Find the allowable net bearing capacity for the shown below footing. Assume that only vertical force will be applied on the footing. Recalculate the problem if the clay is below the footing and underlain by the sand. Compare between the results. in the second case find the allowable bearing pressure if the allowable settlement is 50 mm ($m_v = 5.0 \times 10^{-5} Kpa^{-1}$)



Problem 4:

- **Sand layer**

$\varphi = 34$ hence, $N_c = 42.0$, $N_q = 29.5$ and $N_\gamma = 20.5$
Shape factors for the footing L = 2.30 m and B = 2.00 m
$\lambda_c = \lambda_q = 1 + 0.3\ (2.0/2.3) = 1.26$
$\lambda_\gamma = 1 - 0.3\ (2.0/2.3) = 0.74$
$\gamma_1 = 16.0\ kN/m^3$ and $\gamma_2 = 9.0 + 10\ (0.5/2.0) = 11.5\ kN/m^3$

$q_{ultg} = C\ N_c\ \lambda_c + D_f\ \gamma_1\ N_q\ \lambda_q + B\ \gamma_2\ N_\gamma\ \lambda_\gamma$
$q_{ultg} = 0 + 1.50 \times 16 \times 29.5 \times 1.26 + 2.00 \times 11.5 \times 20.5 \times 0.74 = 1241$ KPa
$q_{ultnet} = 1241 - 1.5 \times 16 = 1217$ KPa
$q_{allnet} = 1217 / 3.0 =$ **405.6 KPa**

- **Clay layer**

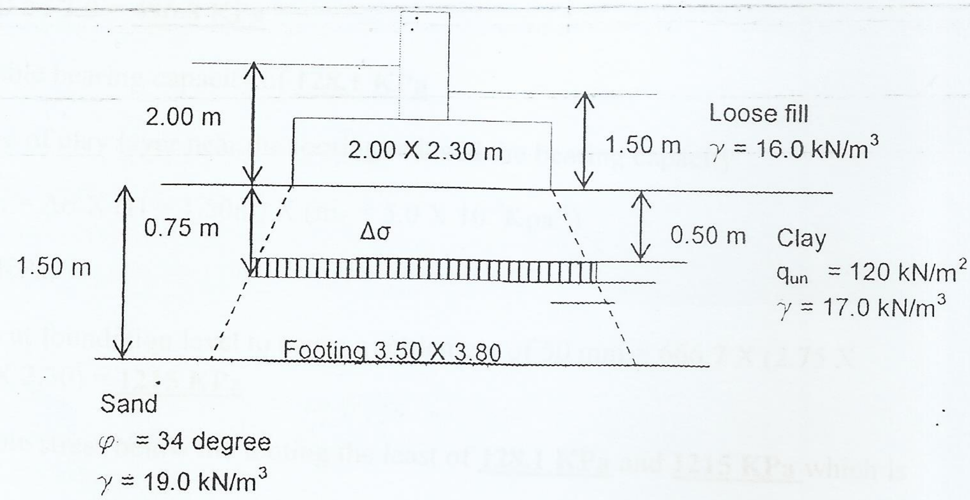
$\varphi =$ zero hence, $N_c = 5.0$, $N_q = 1.0$ and $N_\gamma = 0.0$
Shape factors for the footing L = 3.80 m and B = 3.50 m
$\lambda_c = \lambda_q = 1 + 0.3\ (3.5/3.8) = 1.27$
$\lambda_\gamma = 1 - 0.3\ (3.5/3.8) = 0.73$

$\gamma_1 = (16 X 1.50 + 19 X 0.5 + 9 X 1.0) / 3 = 14.20$ kN/m^3 and $\gamma_2 = 7.0$ kN/m^3

$q_{ultg} = C\ Nc\ \ \lambda c + D_f\ \gamma_1\ Nq\ \lambda q + B\ \gamma_2\ N\gamma\ \lambda\gamma$
$q_{ultg} = 60 X 5.0 X 1.27 + 3.0 X 14.20 X 1.0 X 1.27 + 0 = 435.1$ KPa
$q_{ultnet} = 435.1 - 3.0 X 14.2 = 392.5$ KPa
q_{ultnet} (from footing) = 392.5 X(3.80 X 3.50) /(2.0 X 2.30) = 1135 KPa
$q_{allnet} = 1135 / 3.0 =$ **378.3 KPa**

Take allowable bearing capacity of **378.3 KPa**

Clay layer is underlain by sand layer



- **Clay layer**

φ = zero hence, Nc = 5.0, Nq = 1.0 and Nγ = 0.0
Shape factors for the footing L = 2.30 m and B = 2.00 m
$\lambda c = \lambda q = 1 + 0.3 (2.0/2.3) = 1.26$
$\lambda\gamma = 1 - 0.3 (2.0/2.3) = 0.74$
$\gamma_1 = 16.0$ kN/m^3 and $\gamma_2 = 7.0 + 10 (0.5/2.0) = 9.50$ kN/m^3

$q_{ultg} = C\ Nc\ \ \lambda c + D_f\ \gamma_1\ Nq\ \lambda q + B\ \gamma_2\ N\gamma\ \lambda\gamma$
$q_{ultg} = 60 X 5.0 X 1.26 + 1.50 X 16.0 X 1.0 X 1.26 + 0 = 408.2$ KPa
$q_{ultnet} = 408.2 - 1.5 X 16 = 384.2$ KPa
$q_{allnet} = 384.2 / 3.0 =$ **128.1 KPa**

- **Sand layer**

$\varphi = 34$ hence, $Nc = 42.0$, $Nq = 29.5$ and $N\gamma = 20.5$

Shape factors for the footing L = 3.80 m and B = 3.50 m
$\lambda c = \lambda q = 1 + 0.3\ (3.5/3.8) = 1.27$
$\lambda\gamma = 1 - 0.3\ (3.5/3.8) = 0.73$
$\gamma_1 = (16 \text{ X } 1.50 + 17 \text{ X } 0.5 + 7 \text{ X } 1.0) / 3 = 13.2$ kN/m^3 and $\gamma_2 = 9.0$ kN/m^3

$q_{ultg} = C\ Nc\ \ \lambda c + D_f\ \gamma_1\ \ Nq\ \ \lambda q + B\ \gamma_2\ \ N\gamma\ \lambda\gamma$
$q_{ultg} = 0 + 3.0 \text{ X } 13.2 \text{ X } 29.5 \text{ X } 1.27 + 3.50 \text{ X } 9.0 \text{ X } 20.5 \text{ X } 0.73 = 1530.1$ KPa
$q_{ultnet} = 1530.1 - 3.0 \text{ X } 13.2 = 1490.5$ KPa
$q_{allner} = 1490.5 / 3.0 =$ **496.8 KPa**

Take allowable bearing capacity of **128.1 KPa**

The presence of clay layer near the footing reduced the bearing capacity

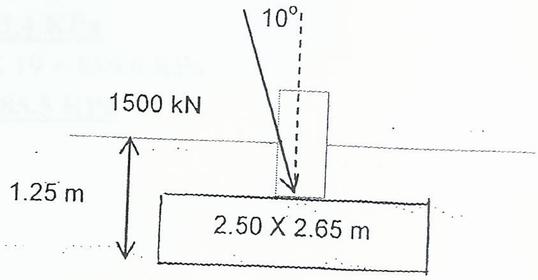
$S_{all} = 50 \text{ mm} = \Delta\sigma \text{ X } (H = 1.50\text{m}) \text{ X } (m_v = 5.0 \text{ X } 10^{-5} \text{Kpa}^{-1})$

$\Delta\sigma = 666.7$ KPa

Hence stress at foundation level to cause a settlement of 50 mm = 666.7 X (2.75 X 3.05)/(2.00 X 2.30) = **1215 KPa**

Take allowable stress below the footing the least of **128.1 KPa** and **1215 KPa** which is **128.1 KPa**

5. For the shown footing it is required to find the allowable net and gross bearing capacity



Silty Sand
φ = 29 degree
γ = 19.0 kN/m^3
C = 15 KPa

Problem 5:

Vertical force (V) = 1500 Cos 10 = 1477 kN
Horizontal force (H) = 1500 Sin 10 = 260 kN
Vertical force at soil failure (V_b) = 1477 X (Fs = 3.0) = 4431 kN
Horizontal force at soil failure (H_b) = 260 X (Fs = 3.0) = 780 kN

φ = 29 hence, Nc = 27.5, Nq = 16.5 and Nγ = 9.5
Shape factors for the footing L = 2.65 m and B = 2.50 m
$\lambda c = \lambda q = 1 + 0.3\ (2.5/2.65) = 1.28$
$\lambda\gamma = 1 - 0.3\ (2.5/2.65) = 0.72$
γ_1 = 19.0 kN/m^3 and γ_2 = 19.0 kN/m^3

$$q_{ultg} = C\ Nc\ \lambda c\ ic + D_f\ \gamma_1\ Nq\ \lambda q\ iq + B\ \gamma_2\ N\gamma\ \lambda\gamma\ i\gamma$$

Case of C-ϕ soil:

$$iq = \left[1 - 0.7\ \frac{H_b}{V_b + A\ C\ Cot\ (\varphi)}\right]^3 = \left[1 - 0.7\ \frac{780}{4431 + (2.50\ X\ 2.65)\ 15\ Cot\ (29)}\right]^3 = 0.685$$

$$i\gamma = \left[1 - \frac{H_b}{V_b + A\ C\ Cot\ (\varphi)}\right]^3 = \left[1 - \frac{780}{4431 + (2.50\ X\ 2.65)\ 15\ Cot\ (29)}\right]^3 = 0.573$$

$$ic = \left[i_q - \frac{1 - i_q}{N_q - 1}\right] = \left[0.685 - \frac{1 - 0.685}{16.5 - 1}\right] = 0.664$$

q_{ultg} = 15 X 27.5 X 1.28 X 0.664+ 1.25 X 19 X 16.5 X 1.28 X 0.685 + 2.50 X 19 X 9.5 X 0.72 X 0.573 = 880.4 KPa

q_{allg} = 880.4 / 3.0 = **293.4 KPa**

q_{ultnet} = 880.4 – 1.25 X 19 = 856.6 KPa

q_{allnet} = 856.6 / 3.0 = **285.5 KPa**