FIFTH SEMESTER B.E.DEGREE EXAMINATION, DEC 2018/JAN 2019

APPLIED GEOTECHNICAL ENGINEERING (15CV53)

1.a) What is stabilization of bore holes? Mention various methods and explain any one method. [6 marks]

Ans:- This is necessary to prevent cohesion less soils against caving while drilling bore hole. Maintaining the integrity of the borehole is known as bore hole stabilization. The different methods of bore hole stabilization are (1) self supporting (2) by filling with water (3) by using drilling mud (4) by casing.

Using drilling mud: Bentonite mud or Drilling mud' is a thin mixture of water and bentonite clay, which can be mixed in powder form to the drilling water to create higher density suspension.

Advantages

- ✓ Firstly, it is more viscous and can therefore lift cuttings adequately at a lower velocity.
- ✓ Secondly it will cake the edges of the borehole, and the outside of the core, and will largely eliminate the seepage of water out of the borehole, thus reducing problems of loss of return.
- ✓ Hence, smaller volumes of flush fluid will be required and the fluid may he recirculated via a settling tank (where the cuttings are allowed to drop out of suspension).
- ✓ The cake formed on the outside of the borehole has the effect of considerably improving borehole stability and the prevention of softening of weak rock cores.

Disadvantages

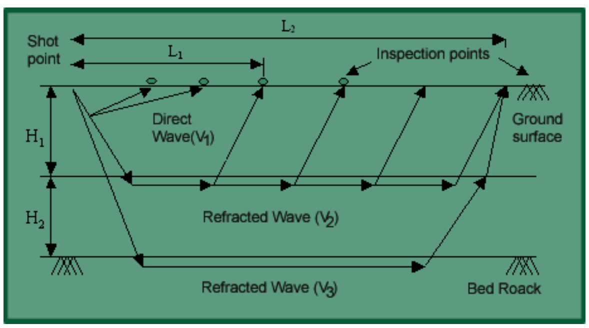
- ✓ The bentonite mud-soil cakes are difficult to dispose of, at the end of drilling a borehole. The mud cannot simply be tipped on the site, and it cannot be discharged into nearby sewers.
- ✓ Bentonite mud must be properly mixed, using appropriate equipment, in order to ensure that it is of the correct consistency and does not contain unmixed dry bentonite lumps, capable of clogging flush ports in the core barrel.

1.b) With the help of a neat sketch explain seismic refraction method of soil exploration. Using this method determine the velocity of waves in soil layers and thickness of the top stratum for the following details: Geophones are placed at a spacing of 40m in a staright line and the time taken for the last wave to be received at each geophone is given.
[8 marks]

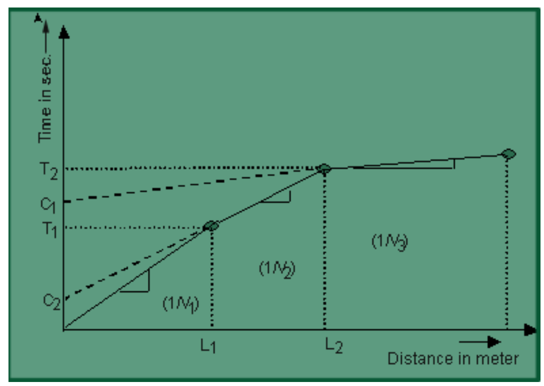
Time(s)	0.1	0.2	0.3	0.4	0.45	0.50	0.55
Distance (m)	40	80	120	160	200	240	280

Ans:- Based on the fact that seismic waves have different velocities in different types of soils (or rock) and besides the wave refract when they cross boundaries between different types of soils.

- ✓ Shock waves are created into the soil by exploding small charges or by striking a plate on the soil with a hammer. These waves are classified as direct, reflected and refracted waves.
- ✓ Radiating shock waves are picked up by geophones, where the time of travel gets recorded.
- ✓ Either a number of geophones are arranged along a line or shock producing device is moved away from the geophone.
- ✓ The direct wave travel in approximately straight line from the source of impulse. The reflected and refracted wave undergoes a change in direction when they encounter a boundary separating media of different seismic velocities.
- ✓ Results are plotted as a graph shown in figure below. Suited for the shallow explorations for civil engineering purpose.



Seismic refraction method



Graph of Time vs Distance

V1= L1/T1, V1 = velocity of direct waves.

Thickness of first layer H1= C2*V1/2 and H2 = (C1-C2)*V2/2

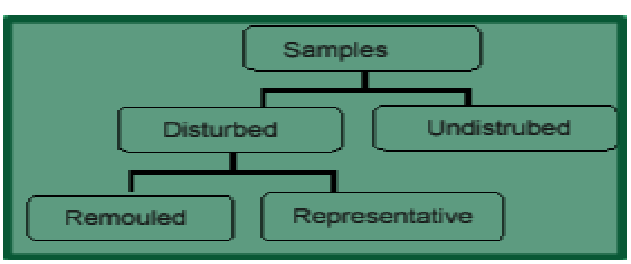
V1 = (160-40)/(0.4-0.1) = 400 m/s

V2 = (280-160)/ (0.55-0.4) = 800 m/s

Thickness Z1 = D1/2 * Sqrt((V2 – V1)/(V2+V1)) = 46.19m.

2.a) List and explain types of soil samples. [6 marks]

Ans:-



Non-Representative samples:- Non-Representative soil samples are those in which neither the in-situ soil structure, moisture content nor the soil particles are preserved. They are not representative**.** They cannot be used for any tests as the soil particles either gets mixed up or some particles may be lost.
E.g: Samples that are obtained through wash boring or percussion drilling.

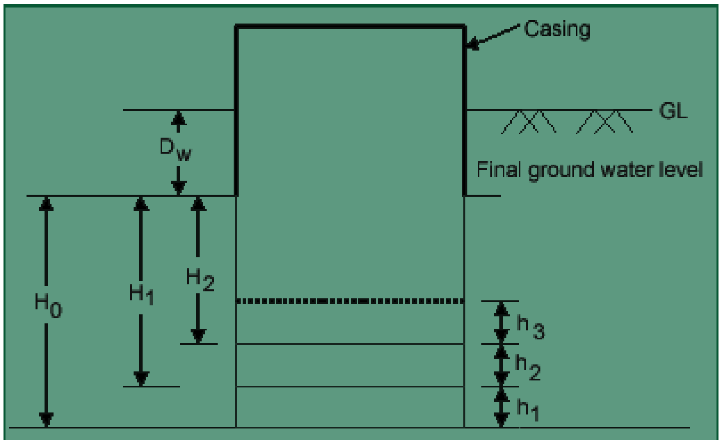
Disturbed soil samples:- Disturbed soil samples are those in which the in-situ soil structure and moisture content are lost, but the soil particles are intact. They are representative**.** They can be used for grain size analysis, liquid and plastic limit, specific gravity, compaction tests, moisture content, organic content determination and soil classification test performed in the lab.
E.g., obtained through cuttings while auguring, grab, split spoon (SPT), etc.

Undisturbed soil samples:- Undisturbed soil samples are those in which the in-situ soil structure and moisture content are preserved. They are representative and also intact. These are used for consolidation, permeability or shear strengths test. In sand, it is very difficult to obtain undisturbed sample. Obtained by using Shelby tube (thin wall), piston sampler, surface (box), vacuum, freezing, etc.

2.b) Explain the determination of ground water level by Hvorslev's method. Using this method estimate the ground water table by the following data: Depth up to which water is bailed out is 15 m, water rise in I day= 0.80 m, II day= 0.70 m, III day = 0.60 m.
[10 marks]

Ans:- Hvorslev's Method explanation:-

As per this method, the water table can be located in a borehole used for soil investigation. The borehole should have the same casing to stabilize the sites. This method most commonly referred to as the time lag method consists of bailing the water out of the casing and observing the rate of rise of water table in the casing at intervals of time until the rise of water table becomes negligible. The rate is observed by measuring the elapsed time and the depth of water surface below the top of the casing. The intervals at which the readings are required will vary somewhat with the permeability of the soil. In no case the elapsed time for the readings should be greater less than 5 minutes. Let the time be t_o, when the water table was at the depth of H_o, below the normal water table level . Let the successive rise in water table levels be h_1, h_2, h_3, etc. at times t_1, t_2, t_3 respectively wherein the difference in time is kept constant. Refer the figure below:



Hvorslev's method of locating ground water table

Now from the fig.

$H_o - H_1 = h_1$, $H_1 - H_2 = h_2$, $H_2 - H_3 = h_3$

Let $t_1 - t_o = t_2 - t_1 = t_3 - t_2$ **,etc.** $= \Delta t$

The depths H_1, H_2, H_3 of the water level in the casing from the normal water table D_W level can be computed as follows,

$H_o = h_1^2(h_1 - h_2)$ $H_1 = h_2^2(h_1 - h_2)$ $H_o = h_3^2(h_2 - h_3)$

let the corresponding depths of the water table level below the ground surface be h_{w1}, h_{w2}, h_{w3}, etc. we have

$h_{w1} = H_w - H_o$

$h_{w2} = H_w - (h_1 + h_2) - H_2$

$h_{w3} = H_w - (h_1 + h_2 + h_3) - H_3$

where, H_w is the depth of water table in the casing from the ground surface at the start of the test.

Normally $h_{w1} = h_{w2} = h_{w3} = h_w$; if not average value gives h_w.

Solution to the numerical:-

$H_o = h_1^2/(h_1 - h_2) = .0.8^2 / (.80 - .70) = 6.4$ m

$H_1 = h_2^2/(h_1 - h_2) = .70^2 / (.80 - .70) = 4.9$ m

$H_2 = h_3^2/(h_2 - h_3) = .60^2 / (.70 - .60) = 3.60$ m

1st day $h_{w1} = H_w - H_o$ = 8.6 m

2nd day $h_{w2} = H_w - (h_1 + h_2) - H_1$ = 8.6 m

3rd day $h_{w3} = H_w - (h_1 + h_2 + h_3) - H_2$ = 9.3 m

$h_w = (h_{w1} + h_{w2} + h_{w3})/3 = \underline{8.83\text{ m}}$

<u>3.a)</u> Explain the settlements with formulae. [6 marks]

Ans:- Total foundation settlement can be divided into three different components, namely Immediate or elastic settlement, consolidation settlement and secondary or creep settlement.
Immediate Settlement:-Immediate settlement is also called elastic settlement.

- It is determined from elastic theory.
- It occurs in all types of soil due to elastic compression.
- It occurs immediately after the application of load
- It depends on the elastic properties of foundation soil, rigidity, size and shape of foundation.

•Immediate settlement is calculated by the equation $S_I = \left(\frac{1-\mu^2}{E}\right) qBI_\rho$

Where, S_I = Immediate settlement, μ= Poisson's Ratio of foundation soil, E = Young's modulus of Foundation Soil, q = Contact pressure at the base of foundation, B = Width of foundation, I_f = Influence Factor.

<u>Consolidation Settlement:-</u>It occurs due to the process of consolidation.

- Clay and Organic soil are most prone to consolidation settlement.
- Consolidation is the process of reduction in volume due to expulsion of water under an increased load.
- It is a time related process occurring in saturated soil by draining water from void.
- Consolidation theory is required to predict both rate and magnitude of settlement.
- Since water flows out in any direction, the process is three dimensional.
- 7. But, soil is confined laterally. Hence, vertical one dimensional consolidation theory is acceptable.

Consolidation Settlement in normally consolidated clayey soil is given by the expression,

$$S_c = \left(\frac{C_c}{1+e_o}\right) H \log_{10}\left(\frac{\sigma_o + \Delta\sigma}{\sigma_o}\right)$$

Where, S_c = Consolidation Settlement, C_c = Compression Index, e_o = Initial Void Ratio, H = Thickness of clay layer, σ_o = Initial overburden pressure at the middle of clay layer ($\gamma_{sat}\frac{H}{2}$)Δσ = Extra pressure due to the new construction ($= \frac{P}{\left(2\frac{H}{2}+B\right)^2}$)

<u>Secondary Compression:-</u>This settlement starts after the primary consolidation is completely over.

- During this settlement, excess pore water pressure is zero.
- This is creep settlement occurring due to the readjustment of particles to a stable equilibrium under sustained loading over a long time.
- This settlement is common in very sensitive clay, organic soils and loose sand with clay binders.

$$S_s = C_\alpha H \log_{10}\left[\frac{t_{sec} - t_{prim}}{t_{prim}}\right]$$

Where, C_α= Coefficient of secondary compression, H = Thickness of clay layer, t_{sec} = Time taken for secondary compression (usually life span of structure), t_{prim} = Time taken for primary consolidation to complete.

3.b) Define isobar. Using Boussinesq's equation construct isobar of intensity 0.25Q(25% isobar), where Q is the point load acting on the surface. [8 marks]

Ans:- Isobar is a curve joining the points of equal stress intensity. It is a spatial curved surface of the shape of an electric bulb or an onion. They are useful for determining the effect of the load on the vertical stress at various points.

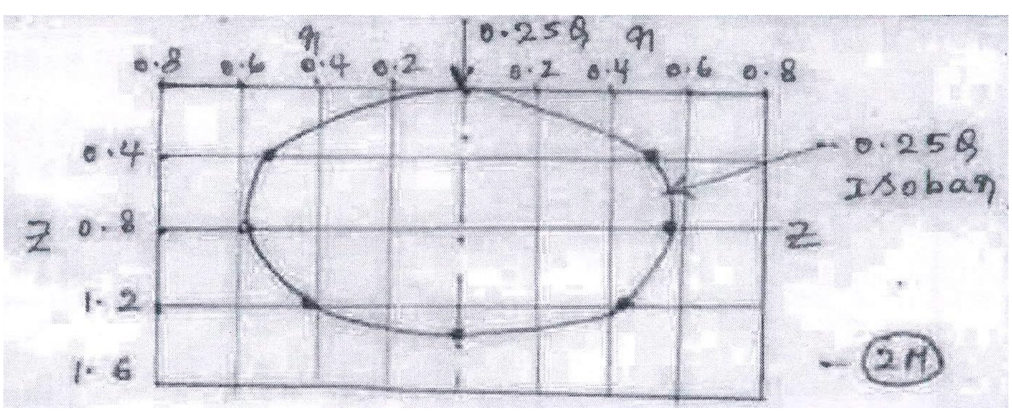
For Boussinesq's point load, $\sigma_z = \frac{Q}{z^2} I_B$ —(2M)

$\sigma_z = 0.25Q$ $\therefore I_B = 0.25 z^2$

For $r = 0$; $I_B = 0.4775$, $\therefore z = \sqrt{\frac{0.4775}{0.25}} = 1.382$ m

Isobar cross the line of action at 1.382 m —(2M)

Depth z (m)	Influence factor I_B	r/z	r	σ_z
0.4	0.04	1.3025	0.5208	0.25Q
0.8	0.16	0.7405	0.592	0.25Q
1.2	0.36	0.345	0.414	0.25Q
1.382	0.4775	0	0	0.25Q



4.a) A circular area 6m diameter carries a uniformily distributed load of 10KN/m^2. Determine the vertical stress at a depth of 2m, 4m and 8m. Plot the variation of vertical stress with depth. [6 marks]

Ans:-

$q = 10\ kN/m^2$

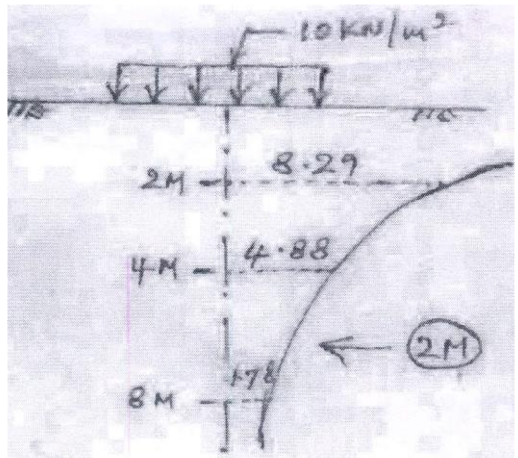
$R = 3\ m$ — (2M)

$$\sigma_z = q\left[1-\left(\frac{1}{1+\left(\frac{R}{z}\right)^2}\right)^{3/2}\right]$$

$z = 2m;\ \sigma_z = 8.29\ kN/m^2$

$= 4m;\ \sigma_z = 4.88\ kN/m^2$

$= 8m;\ \sigma_z = 1.78\ kN/m^2$



4.b) A square footing 1.2m * 1.2 m rests on a saturated clay layer 4m deep. W$_l$ = 30%, γ_{sat} = 17.8 10KN/m^3, w=28% and G=2.68. Determine the settlement if the footing carries a load of 300KN. [10 marks]

Ans:-

$$S_c = \left(\frac{C_c}{1+e_0}\right) H \log_{10}\left(\frac{\sigma_0+\Delta\sigma}{\sigma_0}\right) \text{ — (1M)}$$

$C_c = 0.009(W_L - 10) = 0.18$ — (2M)

$e_0 = wG\ (S_r = 1) = 0.75$ — (2M)

$\sigma_0 = \gamma_{sat} \cdot \frac{H}{2} = 35.6\ kN/m^2$ — (2M)

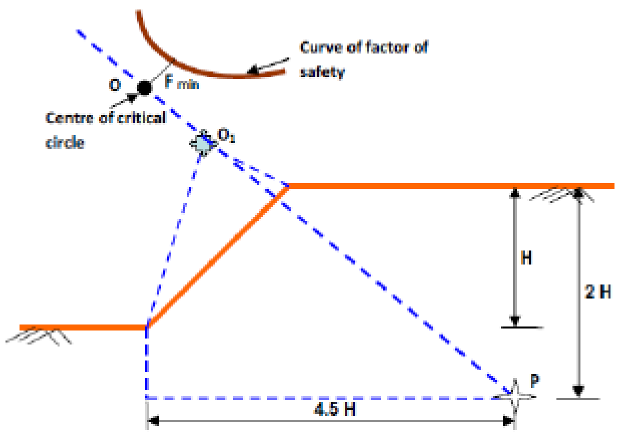
$$\Delta\sigma = \frac{P}{\left(2\frac{H}{2}+B\right)^2} = 11.09\ kN/m^2 \text{ — (2M)}$$

$S_c = 0.0484\ m = 48.4\ mm$ — (1M)

5.a) Explain the Fellinious method of obtaining center of critical slip surface in case of stability analysis of C-φ soil. [6 marks]

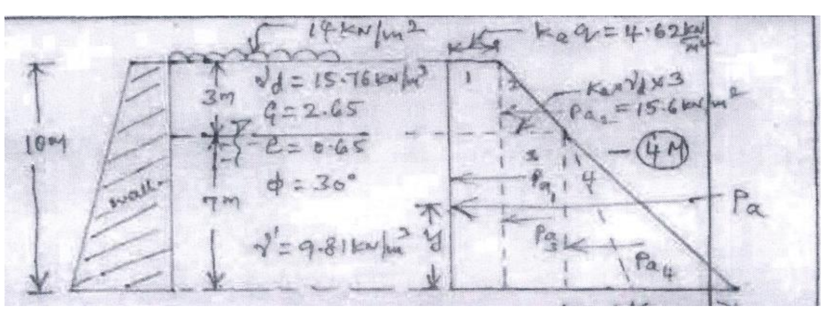
Ans:- In case of c-φ soils the procedure for locating critical slip surface is slightly different and is as given below:

Locate point O1 the centre of Fellenius circle. Locate point P at 2H below the top surface of the slope and 4.5H from the toe of the slope. Extend backwards the line PO1 beyond O1. Construct trial slip circles with centres located on the extended portion of the line PO1. For each of these trial slip circles find the F.S by the method of slices. Plot the F.S for each of these trial slip circles from their respective centre and obtain a curve of factor of safety. Critical slip circle is the one that has a minimum F.S.



5.b) A retaining wall of height 10m supports cohesionless soil with the following properties. G=2.65, e=0.65 and φ=30°. Water table lies ta 3m depth. Surface of back fill is horizontal and carries surcharge of intensity 14 KN/m2. Draw lateral earth pressure distribution diagram. Determine the total active earth pressure and its point of application. [10 marks]

Ans:-



$\gamma_d = \left(\frac{G}{1+e}\right)\gamma_w = 15.76\ kN/m^3$

$\gamma' = \left(\frac{G+e}{1+e}\right)\gamma_w - \gamma_w = 9.81\ kN/m^3$ (2M)

$K_a = \frac{1-\sin\phi}{1+\sin\phi} = 0.33$ — (1M)

$P_a = P_{a_1} + P_{a_2} + P_{a_3} + P_{a_4}$

$= 46.2 + 23.4 + 109.2 + 319.66$

$= 498.46\ kN/m^2$ — (2M)

$y = 3.1 m$. — (1M)

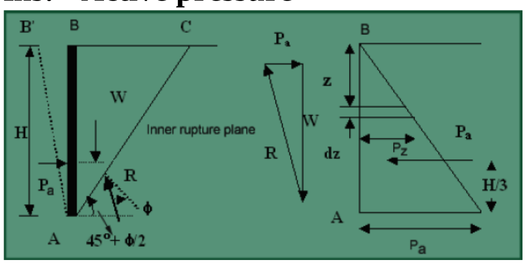
$= 88.67\ kN/m^2$

$K_a \gamma' \times y = 22.66\ kN/m^2$

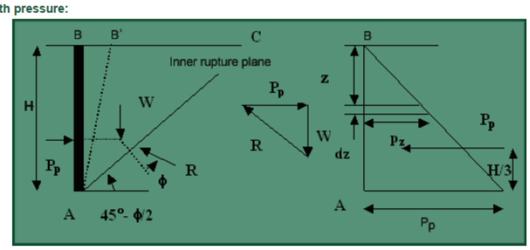
—10M—

<u>6.a)</u> Derive the equations for the earth pressure coefficients K_a and K_p by considering back fill with horizontal surface. Use Rankine's theory. [6 marks]

Ans:- Active pressure



Passive earth pressure:



The lateral pressure acting against a smooth wall AB is due to mass of soil ABC above the rupture line AC which makes an angle of $(45^0 + \phi/2)$ with the horizontal. The lateral pressure distribution on the wall AB of height H increases in same proportion to depth.

The pressure acts normal to the wall AB.

The lateral active earth pressure at A is $P_a = K_A \gamma H$, which acts at a height H/3 above the base of the wall. The total pressure on AB is therefore calculated as follows:

$$P_a = \int_0^H p_z dz = \int_0^H K_A \gamma z dz = 0.5 K_A \gamma H^2, \text{ where } K_A = \tan^2(45^0 + \phi/2)$$

If the wall AB is pushed into the mass to such an extent as to impart uniform compression throughout the mass, the soil wedge ABC in fig. will be in Rankine's Passive State of plastic equilibrium. The inner rupture plane AC makes an angle $(45^0 + \phi/2)$ with the vertical AB. The pressure distribution on the wall is linear as shown.

$$P_p = \int_0^H p_z dz = \int_0^H K_p \gamma z dz = 0.5 K_p \gamma H^2, \text{ where } K_p = \tan^2(45^0 + \phi/2)$$

<u>6.b)</u> An embankment is to be constructed with a soil having C=20 kN/m2, φ= 10◦ and γ=19 kN/m^3. The desired factor of safety with respect to cohesion as well as friction is1.5. determine (1) safe height of the desired slope if slope is 2H to 1V (2) Safe angle of slope if the desired height is 15m. For φ=10◦ Taylor's stability numbers are as follows: [10 marks]

Stability No:	**0.04**	**0.08**
Slope angle(i)	**20**	**30**

Ans:-

i) Safe Height

$F_c = 1.5$

$c = 20\ kN/m^2$

$\gamma = 19\ kN/m^3$

For slope 2H to 1V

$i = 26.56^\circ$

For $i = 26.56^\circ$ } From chart
$S_n = 0.066$

$$H_c = \frac{c}{F_a \gamma H}$$

$= 10.63\ m.$

(6M)

(ii) Safe slope

$$S_n = \frac{c}{F_c \gamma H}$$

$= 0.047$

From chart

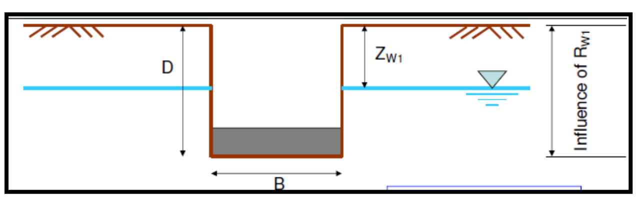
$i = 20 + 1.75$

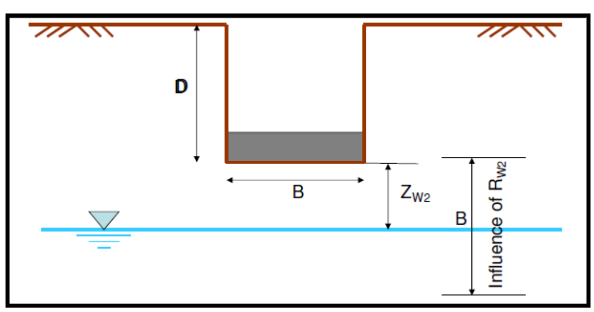
$= 21.75^\circ$

(4M)

__7.a)__ **With the help of sketches, explain the effect of water table and eccentric loading on bearing capacity of soil.** **[6 marks]**

Ans:- The position of ground water has a significant effect on the bearing capacity of soil. Presence of water table at a depth less than the width of the foundation from the foundation bottom will reduce the bearing capacity of the soil. If the ground water is located close to the footing, some changes have to be incorporated in the wedge and surcharge terms of bearing capacity equation. These changes are in the form of water table correction factors R_{w1} & R_{w2}.





Ultimate bearing capacity with the effect of water table is given by,

$$q_f = cN_c + \gamma DN_q R_{w1} + 0.5\gamma BN_\gamma R_{w2}$$

Here, $R_{w1} = \frac{1}{2}\left[1 + \frac{Z_{w1}}{D}\right]$

where Z_{W1} is the depth of water table from ground level.

1. $0.5<R_{w1}<1$
2. When water table is at the ground level ($Z_{w1} = 0$), $R_{w1} = 0.5$
3. When water table is at the base of foundation ($Z_{w1} = D$), $R_{w1} = 1$
4. At any other intermediate level, R_{w1} lies between 0.5 and 1

Here, $R_{w2} = \frac{1}{2}\left[1 + \frac{Z_{w2}}{B}\right]$

where Z_{W2} is the depth of water table from foundation level.

1. $0.5<R_{w2}<1$
2. When water table is at the base of foundation ($Z_{w2} = 0$), $R_{w2} = 0.5$
3. When water table is at a depth B and beyond from the base of foundation ($Z_{w2} >= B$), $R_{w2} = 1$
4. At any other intermediate level, R_{w2} lies between 0.5 and 1

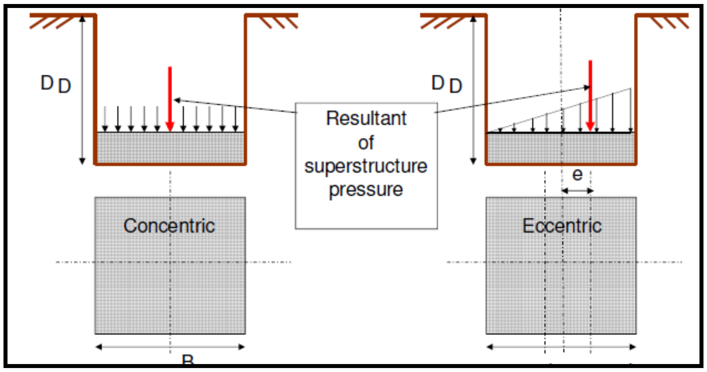
Effect of eccentric foundation base

The bearing capacity equation is developed with the idealization that the load on the foundation is concentric. However, the forces on the foundation may be eccentric or foundation may be subjected to additional moment. In such situations, the width of foundation B shall be considered as follows.

$B' = B - 2e$

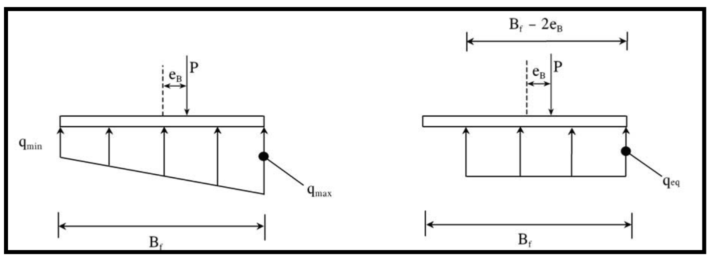
If the loads are eccentric in both the directions, then $B' = B - 2e_B$ & $L' = L - 2e_L$

Further, area of foundation to be considered for safe load carried by foundation is not the actual area, but the effective area as follows: $A' = B' * L'$



Effect of eccentric footing on bearing capacity

In the calculation of bearing capacity, width to be considered is B1 where B' < L'. Hence the effect of provision of eccentric footing is to reduce the bearing capacity and load carrying capacity of footing.



7.b) A square footing located at a depth of 1.3m below ground has top carry a safe load of 800 kN. Predict the size of the footing which is safe against applied load, if the desired FOS is 3.0. Assume e= 0.55, degree of saturation = 50%, G = 2.67, C= 8 kN/m^2. Use Terzaghi's analysis for general shear failure. Assume Φ= 30◦, N_c= 37.2, N_q= 22.5, N_γ= 19.7
[10 marks]

Ans:- e= 0.55, Sr= 0.5, G = 2.67, C= 8 kN/m^2, Φ= 30◦, N_c= 37.2, N_q= 22.5, N_γ= 19.7 , FOS =3, D_f = 1.3 m, γ_w = 10 kN/m^3.

q_u = 1.3*C*N_c + 0.4*B*ɣ*N_γ + ɣ*D_f*N_q

ɣd = (G* γ_w) / (1+e) = (2.67*10)/ (1+0.55) = 17.22 kN/m^3.

q_u = 1.3*8*37.2 + 0.4*B*17.22*19.7 + 17.22*1.3*22.5

q_u = 890.565 + 135.69*B

q_{nu} = q_u - ɣ* D_f

$= 890.565 + 135.69*B – 17.22*1.3$

$= 868.179 + 135.69*B$

$q_{ns} = q_{nu} / FOS = (868.179 + 135.69*B)/3$

$= 289.393 + 45.23*B$

$q_s = q_{ns} + \gamma * D_f$

$= 289.393 + 45.23*B + 17.22*1.3$

$800/ B^2 = 311.779 + 45.23*B$

$\underline{B = 1.455 \text{ m.}}$

8.a) Explain Standard Penetration Test with suitable corrections. [6 marks]

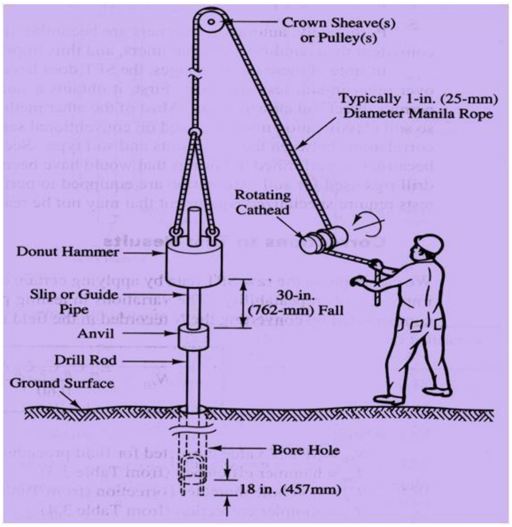
Ans:-

- The split-spoon sampler, attached to standard drill rods of required length is lowered into the borehole and rested at the bottom.
- The split-spoon sampler is driven into the soil for a distance of 450mm by blows of a drop hammer (monkey) of 65 kg falling vertically and freely from a height of 750 mm. The number of blows required to penetrate every 150 mm is recorded while driving the sampler. The number of blows required for the last 300 mm of penetration is added together and recorded as the N value at that particular depth of the borehole. The number of blows required to effect the first 150mm of penetration, called the seating drive, is disregarded.
- The split-spoon sampler is then withdrawn and is detached from the drill rods. The split-barrel is disconnected from the cutting shoe and the coupling. The soil sample collected inside the split barrel is carefully collected so as to preserve the natural moisture content and transported to the laboratory for tests. Sometimes, a thin liner is inserted within the split-barrel so that at the end of the SPT, the liner containing the soil sample is sealed with molten wax at both its ends before it is taken away to the laboratory.
- The SPT is carried out at every 0.75 m vertical intervals in a borehole. This can be increased to 1.50 m if the depth of borehole is large. Due to the presence of boulders or rocks, it may not be possible to drive the sampler to a distance of 450 mm. In such a case, the N value can be recorded for the first 300 mm penetration.
- SPT values obtained in the field for sand have to be corrected before they are used in empirical correlations and design charts.

Correction for overburden pressure :- Of two granular soils possessing the same relative density but having different confining pressures, the one with a higher confining pressure gives a higher N value. The corrected N values given by N’ = C_n * N;

C_n = Correction factor for overburden pressure.

Correction for dilatancy:- Di1atancy correction is to be applied when obtained after overburden correction, exceeds 15 in saturated fine sands and silts. The corrected equation is N’’ =15+0.5 (N’- 15)



8.b) A rectangular footing has a size of 1.8m*3m has to transmit the load of a column at a depth of 1.5m. Calculate the safe load which the footing can carry at a factor of safety of 3 against shear failure. Use IS code method. The soil has following properties: n= 40%, G=2.67, w=15%, C=8 KN/m^2 and φ=32.5°. **[10 marks]**

Ans:-

$$e = \frac{n}{1-n} = 0.667$$

$$\gamma_d = \frac{G}{1+e}\gamma_w = 15.71 kN/m^2$$

$$\gamma_b = \gamma_d(1+w) = 18.07 kN/m^2$$

$q_f = C N_c \cdot s_c d_c i_c + \gamma D N_q s_q d_q i_q + \frac{1}{2}\gamma B N_\gamma s_\gamma d_\gamma i_\gamma W'$

$i_c = i_q = i_\gamma = 1$ and $W' = 1$

For rectangular footi

$s_c = 1 + 0.2\frac{B}{L} = 1.12$

$s_q = 1 + 0.2\frac{B}{L} = 1.12$

$s_\gamma = 1 - 0.4\frac{B}{L} = 0.76$

— (2M)

Depth factor

$d_c = 1 + 0.2\frac{D}{B}\sqrt{N_\phi}$

$\sqrt{N_\phi} = \tan(45 + \frac{\phi}{2}) = 1.823$

$d_c = 1.364$

$d_q = d_\gamma = 1 + 0.1\frac{D}{B}\sqrt{N_\phi} = 1.152$ — (2M)

$q_n = q_f - \gamma D = 1815.89\ kN/m^2$

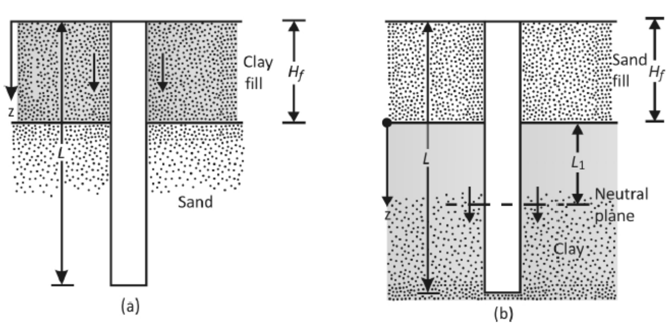
$q_s = \frac{q_n}{F} + \gamma D = 632.4\ kN/m^2$

Safe load $= q_s(B \times L) = 3415\ kN$. — (2M)

9.a) With the help of sketch, explain negative skin friction. [6 marks]

Ans:- Negative skin friction is a downward drag force exerted on the pile by the soil surrounding it. This action can occur under conditions such as the following:

1. If a fill of clay soil is placed over a granular soil layer into which a pile is driven, the fill will gradually consolidate. This consolidation process will exert a downward drag force on the pile (figure a) during the period of consolidation.
2. If a fill of granular soil is placed over a layer of soft clay, as shown in figure b, it will induce the process of consolidation in the clay layer and thus exert a downward drag on the pile.
3. Lowering of the water table will increase the vertical effective stress on the soil at any depth, which will induce consolidation settlement in clay. If a pile is located in the clay layer, it will be subjected to a downward drag force.



Clay Fill over Granular Soil:- The negative (downward) skin stress on the pile is

$$Q_{nsf} = \frac{pK'\gamma'_f H_f^2 \tan\delta}{2}$$

Where, p = perimeter of the pile , H_f = depth of fill/soil which is moving vertically

K = earth pressure coefficient $K_o = 1\text{-}\sin\Phi$

γ_f = unit weight of the soil , δ = soil-pile friction = 0.5 to 0.7Φ

If the fill is above the water table, the effective unit weight, $\gamma' f$, should be replaced by the moist unit weight.

Granular Soil Fill over Clay:- $Q_{nsf} = p*C*\alpha*H_f$, Where, C = Cohesion in the zone of H_f

9.b) A 200 mm diameter, 8m long piles are used as foundation for a column in a uniform deposit of medium clay having unconfined compressive strength of 100 KN/m2. The spacing between the piles is 500mm. There are 9 piles in the ground arranged in a square pattern. Calculate the ultimate load capacity of the group. Assume adhesion factor = 0.9 and N_c=9. [10 marks]

Ans:-

$q_u = 100\ kN/m^2 \therefore C_u = 50\ kN/m^2$

Ultimate capacity of single pile, Q_u

$Q_u = C_u N_c A_p + \alpha C_u A_s$

$= 50 \times 9 \times \frac{\pi}{4} \times (0.2)^2 + 0.9 \times 50 \times \pi \times 0.2 \times 8$

$= 240.33\ kN$ —(4M)

Ultimate capacity of 9 piles $= 9 \times 240.33$

$= 2163\ kN$

Width of pile group $= 2 \times 0.5 \times 0.2 = 1.2\ m$ (2M)

Ultimate load capacity of group by block failure, $Q_{ug} = C_u N_c A_b + P_b L C_u$

$= 50 \times 9 \times 1.2 \times 1.2 + 4 \times 1.2 \times 8 \times 50$

$= 2568\ kN.$ —(4M)

Take lower value ie $Q_u = \underline{2163\ kN}$

10.a) Write short notes on any four of the following:

(a) Efficiency of pile group (b) Group capacity of piles (c) Pile load test (d) Settlement of piles (e) Under reamed piles (f) Single loaded pile capacity. [16 marks]

Ans:-

(a) Efficiency of pile group:- Piles are generally used in groups with a common pile cap. A group may consist of two or three, or as many as ten to twelve piles depending on the design requirement. IS 2911 (Part1) 1979 recommends a minimum spacing of

- 2.5D – point bearing piles
- 3D – friction piles
- 2D- loose sands or fill deposits.

Spacing of piles in a group depends on (a) length, size & shape of piles (b) soil characteristics (c) magnitude and type of loads.

Generally center to center spacing between piles in a group is kept between 2.5 d and 3.5d where d is the diameter of the pile.

The efficiency of a pile group may be defined as $\eta_g = \frac{Q_{g(u)}}{NQ_u} \times 100$

$\eta = group\ efficiency$

$Q_{g(u)} = ultimate\ load - bearing\ capacity\ of\ the\ group\ pile$

$Q_u = ultimate\ load - bearing\ capacity\ of\ each\ pile\ without\ the\ group\ effect$

(b) Group capacity of piles:- Piles are generally used in groups with a common pile cap. A group may consist of two or three, or as many as ten to twelve piles depending on the design requirement. The load carrying capacity of a group of piles is given by

$$(Q_u)_g = Nq_u n$$

where,

$(Q_u)_g$= Load carrying capacity of pile group

N = number of piles

q_u= allowable load per pile

n = group efficiency

Its value for bearing or friction piles at sites where the soil strength increases with depth is found

to be 1. For friction piles in soft clays the value on *n* is less than 1. The actual value of *n* depends on soil type, method of pile installation, and pile spacing. When piles are driven in loose, sandy soils, the soil is densified during driving, and *n* >1 in such cases. It has been observed that if the spacing between piles is more than 2.5 times the pile diameter, the group efficiency is not reduced. The large pile to pile spacing will increase the overall cost of construction. The reduction in load capacity due to the group effect can be estimated empirically.

(c) Pile load test:-

The pile head is chipped off to natural horizontal plane till sound concrete is met. The projecting reinforcement is cut off suitably and the top is finished smooth & level with plaster of Paris. Loading platform of 6.2m x 6.2m is constructed by using 2nos. of ISMB 500 as main girders and 21nos of ISMB 300 as secondary girders. The CG of platform is made to coincide with centre of pile. Platform thus constructed is loaded with sand bags for required weight. A 20mm thick mild steel plate is placed on the top of pile head, Hydraulic jack of 250T Capacity is placed centrally on top of the plate. The gap between the top of jack and bottom of main girders is filled with steel packing materials. The Hydraulic pump is connected to jack by flexible pressure hose. Calibrated pressure gauge is connected to hydraulic pump. Datum bars of heavy sections were placed very near to pile head and are supported on ends at a distance of 2m on either side from face of the pile. Two numbers of settlement gauges are placed on pile head at diametrical opposite locations with the help of magnetic bases fixed on datum bars. The pump is operated till the ram of jack touches the bottom of main girders. At this stage the pressure gauge reading is zero and dial gauge reading are adjusted for zero loading. The loads are then applied in increments of 20% of safe load. For each increment of load the dial gauge reading are taken at intervals of 15 minutes, till the rate of settlement is less than 0.1 mm in the first half hour or 0.2 mm in one hour of for a maximum period of 2hrs. Then the next increment of load is applied and the procedure repeated till the test load is reached. This load is maintained for 24 hours and hourly settlement readings are noted. At the end of 24 hours, unloading is done gradually till the entire load is released.

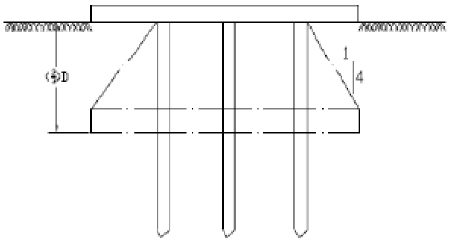
(d) Settlement of piles:- Pile settlement can be estimated as follows.

1. Compute the average pile axial force in each segment of length ΔL, average cross-section & A_{av} and shaft modulus of elasticity E_p from the pile butt to point.

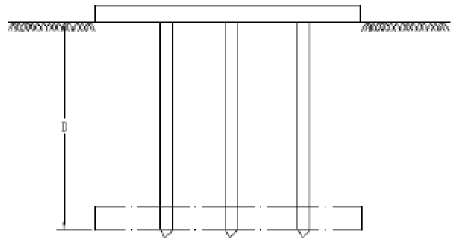
$$\Delta Hs, s = \frac{P_{av} \times \Delta L}{A_{av} \times E_p}$$

and sum the several values to obtain the axial total compression $\Delta H_a = \sum \Delta Hs, s$

Due to group action, both immediate and consolidation settlement values of a pile group are greater than those for a single pile.



Friction piles



End bearing piles

<u>Settlement of pile groups in clay:-</u> The consolidation settlement in pile group may be obtained from the expression given below:

$$S_c = [\ C_c * H * \log (\sigma_o + \Delta\sigma\ )/\ \sigma_o\ ]/\ [\ 1+e_o]$$

Where, $C_c = 0.009(w_L - 10)$ for undisturbed soils & $= 0.007(w_L - 10)$ for remolded soils

e_o = initial void ratio, σ_o = initial overburden pressure at the middle of the clay strata

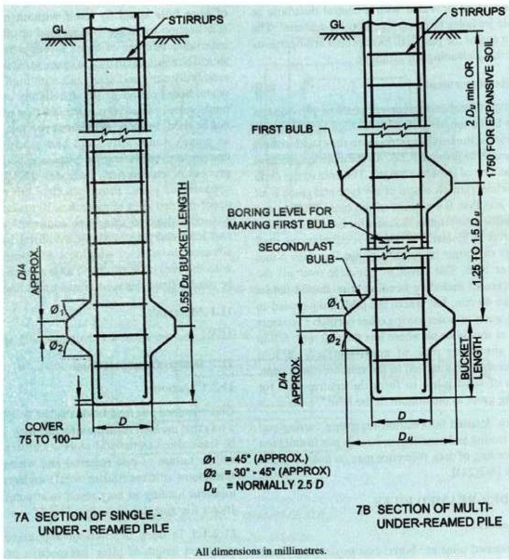
$\Delta\sigma = P/\ [B + (2*0.5H*\tan 30)]^2$

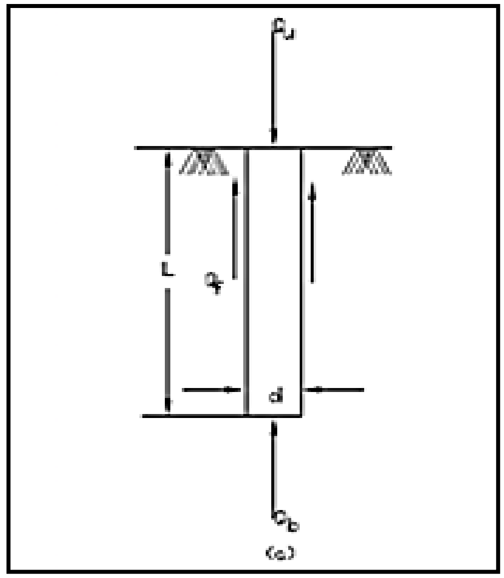
<u>Settlement of pile groups in sand:-</u>

Skempton's settlement ratio:- $S_g/S_i = \{[4B + 2.7]/\ [B + 3.6]\}^2$

Where, B = width of pile group in meters, S_i = settlement of a single pile obtained from pile load test data.

(e) Under reamed piles:- Under reamed piles are bored cast-in-situ concrete piles having one or more number of bulbs formed by enlarging the pile stem. These piles are best suited in soils where considerable ground movements occur due to seasonal variations, filled up grounds or in soft soil strata. They are suitable for loose and filled up sites, or where soils are weak or expansive like black cotton soil. The bulbs are located at depths where good bearing strata are available but they should not be placed too near the ground level. Bulb size is usually 2 to 3 times the pile stem diameter. The bulb provides a large bearing area, increasing the pile load capacity. They are also effective in resisting the downward drag due to the negative skin friction that arises in loose or expansive soils. Bulb spacing should not exceed 1.5 times the bulb diameter. Refer the figure below:





Under reamed Pile Load carrying capacity of pile

(f) Single loaded pile capacity:- The load carrying capacity of a single pile can be estimated using

1. Static formulae
2. Dynamic formulae
3. Correlations with penetration test data
4. Pile Load tests

Static Formulae

The static formulae for ultimate load carrying capacity of pile based on soil properties and pile geometry.

Piles in granular soils: ***Point bearing*** in granular soil, $q_{pu} = \bar{\sigma} N_q$ Where σ is the effective overburden pressure at the tip of the pile = ɣ*L; L is the length of the embedment of the pile.

Unit skin friction, $f_s = \sigma_h \tan \delta = K \bar{\sigma} \tan \delta$ Where K is the lateral earth pressure coefficient and δ is the angle of internal friction between the pile and the soil.

Ultimate skin friction resistance Qs $= K \bar{\sigma}_{av} \tan \delta$

The ultimate load Q_u is given by Qu = End bearing resistance Qp+ Skin resistance Qs

$= (q*N_q*A_p) + (k*\sigma_v*A_s*\tan\delta)$

Where, A_p = Cross section area of pile, N_q = Bearing capacity factor, σ_v= Effective overburden pressure ,k= Co-efficient of earth pressure, σ_v= Effective over burden pressure at middle of corresponding layer, δ = Angle of wall friction usually taken as ¾ ø of soil, A_s = Surface area of pile.