**FIFTH SEMESTER B.E.DEGREE EXAMINATION, DEC 2019/JAN 2020**

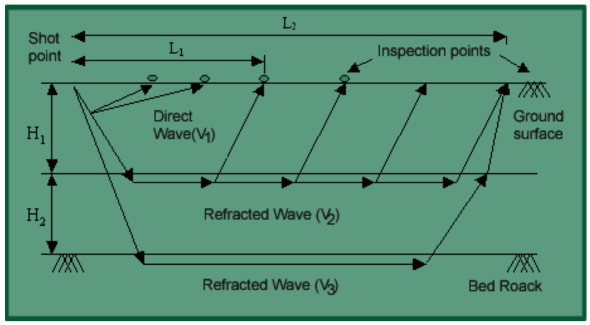
**APPLIED GEOTECHNICAL ENGINEERING (17CV53)**

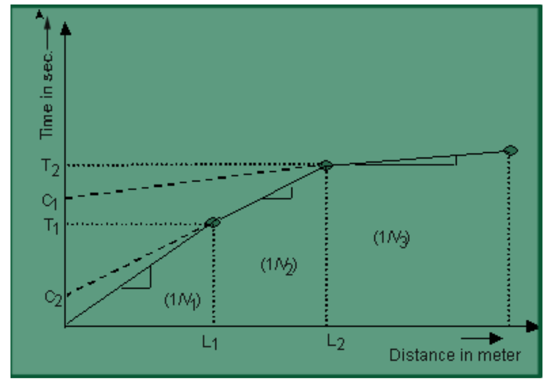
**1.a) What are the objectives of soil exploration ? [6 marks]**

**Ans:-** Soil investigations are done to obtain the information that is useful for one or more of the following purposes:
1. To know the geological condition of rock and soil formation.
2. To establish the groundwater levels and determine the properties of water.
3. To select the type and depth of foundation for proposed structure
4. To determine the bearing capacity of the site.
5. To estimate the probable maximum and differential settlements.
6. To predict the lateral earth pressure against retaining walls and abutments.
7. To select suitable construction techniques
8. To predict and to solve potential foundation problems
9. To ascertain the suitability of the soil as a construction material.
10. To determine soil properties required for design
11. Establish procedures for soil improvement to suit design purpose
12. To investigate the safety of existing structures and to suggest the remedial measures.

**1.b) With a neat sketch explain seismic refraction method of soil exploration. [8 marks]**

- ✓ **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

**<u>1.c)</u> What is the necessity of dewatering? Explain electro-Osmosis method of dewatering. [6 marks]**

**Ans:-** The objectives of dewatering are:

- ✓ To provide a dry excavation and permit construction to proceed efficiently
- ✓ To reduce lateral loads on sheeting and bracing in excavations
- ✓ Stabilize "quick" bottom conditions and prevent heaving and piping
- ✓ Improve supporting characteristics of foundation materials
- ✓ Increase stability of excavation slopes and side-hill fills
- ✓ Cut off capillary rise and prevent piping and frost heaving in pavements
- ✓ Reduce air pressure in tunneling operations

***<u>Electro-Osmosis method</u>***

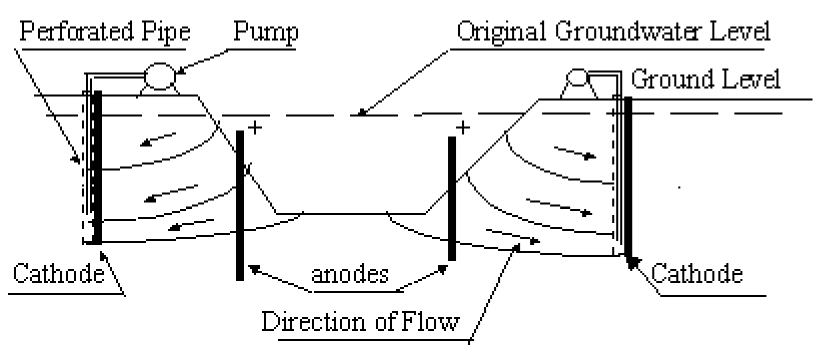



Fig. 8.8 Control of Groundwater by Electro-Osmosis Methods

Electro-osmosis is defined as "the movement of water (and whatever is contained in the water) through a porous media by applying a direct current (DC) field". It is the only effective method of dewatering in deep clay soils. As the surface of fine grained soil particles causes negative charge, the positive ions in solution are attracted towards the soil particles and concentrate near the surfaces. Upon application of the electro motive force between two electrodes in a soil medium the positive ions adjacent to the soil particles and the water molecules attached to the ions are attracted to the cathode and are repelled by the anode. The free water in the interior of the void spaces is carried along to the cathode by viscous flow. By making the cathode a well, water can be collected in the well and then pumped out.

**2.a) Define the following terms with reference to a sampling tube with a neat sketch: (1) inside clearance (2) outside clearance (3) area ratio (4) recovery ratio. [8 marks]**

**Ans:-** ***Inside Clearance***

$$Ci = \frac{D_3 - D_1}{D_1} x\,100$$

Where $D_3$ = inner diameter of the sample tube
It helps in reducing the frictional drag on the sample, and also helps to retain the core. For an undisturbed sample, the inside clearance should be between 0.5 and 3%.
***Outside Clearance***

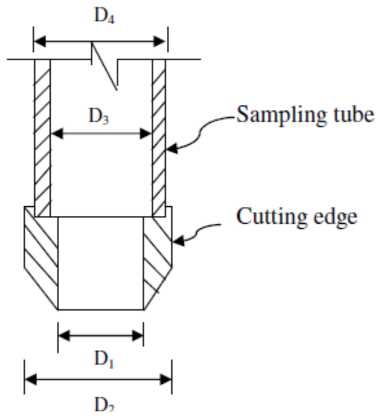
$$Co = \frac{D_2 - D_4}{D_4} x\,100$$

Where $D_4$ = outer diameter of the sample tube

Outside clearance facilitates the withdrawal of the sample from the ground. For reducing the driving force, the outside clearance should be as small as possible. Normally, it lies between zero and 2%.
***Area ratio***

$$\text{Area ratio } A_r = \frac{Max. Cross\ sectional\ area\ of\ the\ cutting\ edge}{Area\ of\ the\ soil\ sample}$$

$$A_r = \frac{{D_2}^2 - {D_1}^2}{{D_1}^2} x\,100$$




Where, $D_1$ = inner diameter of the cutting edge, $D_2$ = outer diameter of the cutting edge
For obtaining good quality undisturbed samples, the area ratio should be less than or equal to 10%.

***Recovery Ratio***

$$R_r = \frac{L}{H}$$

Where L = length of the sample within the tube, and H = Depth of penetration of the sampling tube

$R_r$ = 96 – 98 % for getting a satisfactory undisturbed sample.

**2.b) What is stabilization of bore holes? Explain any one method. [6 marks]**

**Ans:-** Stabilization of bore holes necessary to prevent cohesion less soils against caving while drilling bore hole. Either of the following is used for bore hole support:
- **Steel casing –** hydraulically pushed
- **Drilling mud** – Circulation bentonite slurry

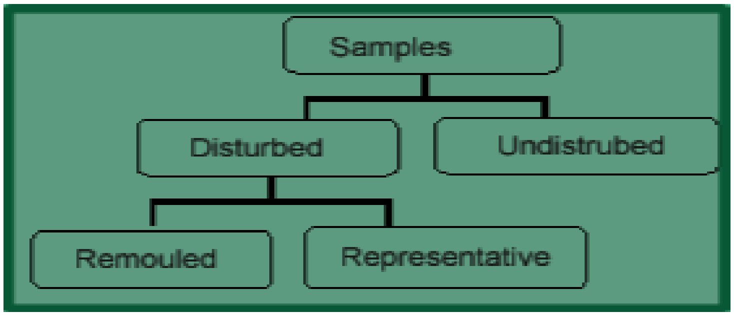
***Using drilling mud***: Bentonite mud or Drilling mud' is a thin mixture of water and bentonite clay, which can be mixed in powder form to the drilling water to create higher density suspension.

***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 re-circulated 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.

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

**Ans:-**



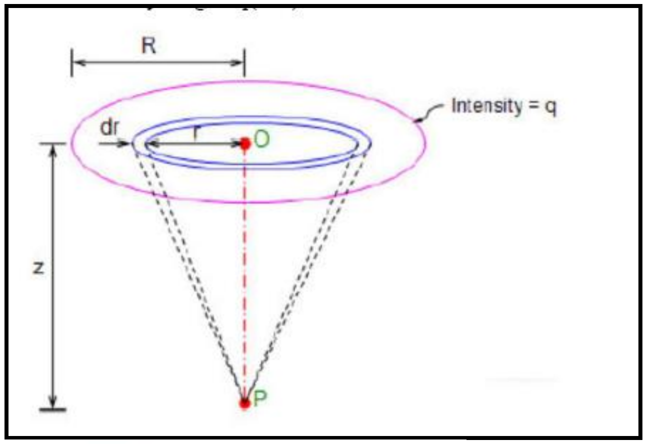

**Non-Representative samples:-** Non-Representative soil samples are those in which neither the in-situ soil structure, moisture content nor the soil particles are preserved. They are not representative**.** They cannot be used for any tests as the soil particles either gets mixed up or some particles 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.

**3.a) Derive the equation for vertical stress below the center of a circular area with uniform load intensity "q". [8 marks]**

**Ans:-** Let q = intensity of the load per unit area and R = the radius of the loaded area.
Let us consider an elementary ring of radius r and thickness „dr" of the loaded area. The load on the elementary ring = $q(2\pi r)dr$




$$\sigma_z = \frac{3Q}{2\pi}\frac{1}{Z^2}\left(\frac{1}{(1+(\frac{r}{Z})^2)^{\frac{5}{2}}}\right)$$

But we know that

$$\Delta\sigma_z = \frac{3(q2\pi r dr)}{2\pi}\frac{1}{Z^2}\left(\frac{1}{(1+(\frac{r}{Z})^2)^{\frac{5}{2}}}\right)$$

$$\Delta\sigma_z = \frac{3qrdr}{(r^2+Z^2)^{\frac{5}{2}}}Z^3$$

The vertical stress due to to full load is given by

$$\sigma_z = 3\,q\,z^3\int_0^R \frac{r\,dr}{(r^2+Z^2)^{\frac{5}{2}}}$$

$$Let\ r^2 + Z^2 = u^2$$

$$2r\,dr = du$$

when r = 0, $u = Z^2$

r = R, $u = R^2 + Z^2$

$$\sigma_z = 3\,q\,z^3\int_{Z^2}^{R^2+Z^2} \frac{du}{2(u)^{\frac{5}{2}}}$$

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

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

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

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

$$\sigma_z = I_c\, q$$

Where, Ic is the influence coefficient for the circular area and is given by

$$I_c = \left[1 - \left(\frac{1}{1+(\frac{R}{Z})^2}\right)^{\frac{3}{2}}\right]$$

Now $\tan\theta = R/Z$

$$I_c = \left[1 - \left(\frac{1}{1+(\tan)^2}\right)^{\frac{3}{2}}\right]$$

$$=(1-(\cos^2\theta)^{3/2}$$

$$I_c = 1 - \cos^3\theta \text{ --------}$$

**3.b) Define isobar. Construct an isobar for a vertical stress of 40 kN/m$^2$, when ground surface is subjected to a concentrated load of 1000 kN. [8 marks]**

**Ans:-**

**Isobar:-** 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. It is generally assumed that an isobar of 0.1Q forms a pressure bulb. The area outside the pressure bulb is assumed to have negligible stresses.

$$\sigma_z = I_B \times \frac{Q}{z^2}$$

$$40 = I_B \times \frac{\cancel{1000}^{25}}{z^2}$$

$$25 I_B = z^2$$

$$\therefore \boxed{I_B = 0.04 z^2}$$

For $r = 0$, $I_B = 0.4775$

$$\therefore \; z = \sqrt{\frac{0.4775}{0.04}} = \underline{3.455 m}$$

$$\sigma_z = \frac{3Q}{2\pi} \times \frac{1}{z^2} \left[ \frac{1}{1+\left(\frac{r}{z}\right)^2} \right]^{5/2}$$

$$\left[ \frac{\sigma_z \times 2\pi \times z^2}{3Q} \right]^{2/5} = \frac{1}{1+\left(\frac{r}{z}\right)^2}$$

$$\therefore \frac{r}{z} = \sqrt{\frac{1}{\left[\frac{\sigma_z \times 2\pi \times z^2}{3Q}\right]^{2/5}} - 1}$$

$$\frac{r}{z} = \sqrt{\frac{1}{(0.084 \times z^2)^{2/5}} - 1}$$

| Depth z(m) | $I_B$ | $r/z$ | $r$ |
|---|---|---|---|
| 0.75 | 0.0225 | 1.546 | 1.159 |
| 1.50 | 0.09 | 0.973 | 1.459 |
| 2.25 | 0.2025 | 0.640 | 1.44 |
| 3.00 | 0.36 | 0.344 | 1.032 |
| 3.45 | 0.4761 | 0.0087 | 0 |

**<u>3.c)</u> Estimate the immediate settlement of a footing of size 2m *3m resting at a depth of 2m in a sandy soil whose compression modulus is 10 N/mm$^2$ and footing is expected to transmit a unit pressure of 160 kN/m$^2$. Assume μ=0.28 and $I_f$ = 1.06. [4 marks]**

**Ans:-**

$$S_I = \left( \frac{1-\mu^2}{E} \right) q B I_\rho$$

= (1-.282/10*106)*160*2*1.06 =

**<u>4.a)</u> Explain the construction and use of Newmark's chart. [8 marks]**

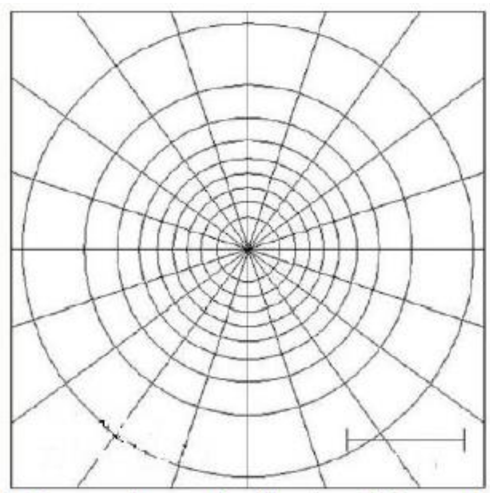
**Ans:- <u>Construction of Newmark's chart:-</u>**

For the specified depth z, say 10 m, the radii of the circles, R, are calculated from the equation

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

for relative radii given by, where, R/z is the relative radii, $\sigma_z$ = vertical stress at any depth, q = load intensity.

The circles are then drawn to a convenient scale (say, 1 cm = 2 m or 1:200). A suitable number of uniformly spaced rays (to get required influence value) are drawn, emanating from the center of the circles. The resulting diagram will appear as shown in below. On the figure is drawn a line AB, representing the depth z to the scale used in drawing the circles. If the scale used is 1 cm = 2 m, then AB will be 5 cm. The influence value for this chart will be I = (1/c x s). The same chart can be used for other values of the depth “z” . The length AB is taken equal to the depth “z “of the given problem and to that scale the loaded diagram is plotted on a tracing sheet to be superimposed later on the Newmark’ s chart to obtain the vertical stress at the desired point.



**Application of Newmark’s Influence chart:-** Application of Newmark‟ s Influence chart in solving problems is quite easy and simple. The plan of the loaded area is first drawn on a tracing sheet to the same scale as the scale of the line segment AB on the chart representing the depth “z‟ . The location of the point where the vertical stress is required is marked on the plan, say as “P‟ . Now, the tracing sheet is placed over the chart, such that the point „P‟ comes exactly over the center of the chart from where the rays are emanating. Now the number of mesh covered by the plan is counted. In case of partly covered mesh an intelligent judgement of the fraction of mesh covered is required. Let the total number of mesh be equal to “n‟ . Then the vertical stress at the desired depth is given by: $\sigma_z = I \times n \times q$

Where I = Influence value = 1/(c x s)

n = Number of meshes under the loaded area

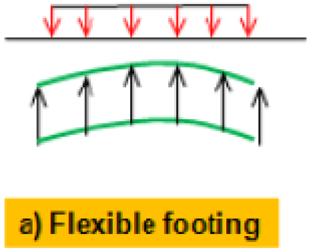
q = uniformly distributed load

c = No. of concentric areas

s = No. of radial line

**4.b) Explain contact pressure distribution in soils. [6 marks]**

**Ans:-** In sandy soils:-



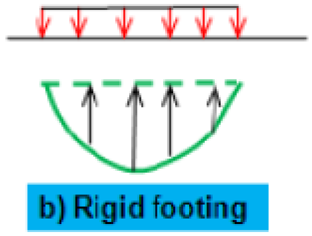
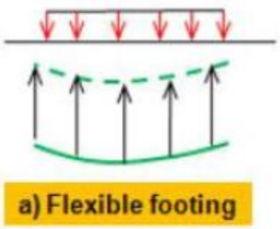
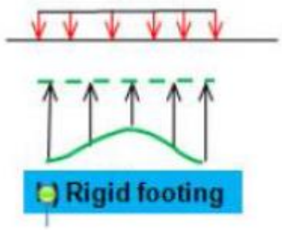




Fig a & b shows the qualitative contact pressure distribution under flexible and rigid footing resting on a sandy soil and subjected to a uniformly distributed load q. when the footing is flexible, the edges undergo a large settlement than at centre. The soil at centre is confined and therefore has a high modulus of elasticity and deflects less for the same contact pressure. The contact pressure is uniform. When the footing is rigid the settlement is uniform. The contact pressure is parabolic with zero intensity at the edge sand maximum at the centre.

In clayey soils:-




When the footing is flexible, it deforms into the shape of a bowel, with the maximum deflection at the centre. The contact pressure distribution is uniform If the footing is rigid, the settlement is uniform. The contact pressure distribution is minimum at the centre and the maximum at the edges(infinite theoretically). The stresses at the edges in real soil cannot be infinite as theoretically determined for an elastic mass.

**4.c)**

A square footing 1.2m × 1.2m rests on a saturated clay layer 4m deep. The soil properties are $W_L$ = 30%, $\gamma_{sat}$ = 17.8kN/m$^3$, w = 28% and G = 2.68. Determine primary consolidation settlement if the footing carries a load of 300kN. **(06 Marks)**

**Ans:-**

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

Cc = 0.009(30-10) = 0.18

$\sigma_0$ = 17.8* 4/2 = 35.6 kPa

$\Delta\sigma = 300/(1.2+4)^2 = 11.09$ kPa

e = 0.28*2.68 = 0.7504

$S_c$ = .0484 m

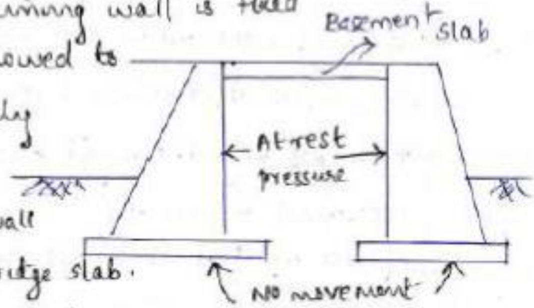
**<u>5.a)</u> Explain with a neat sketch at rest, active passive and Passive earth pressure. [6 marks]**

**Ans:-**

① <u>At rest pressure</u>

→ Soil mass is not subjected to any lateral movement

→ occurs when the retaining wall is fixed firmly on top & not allowed to rotate or move laterally




<u>Eg</u> ① Basement slab

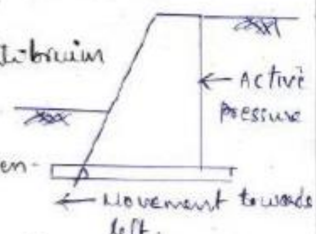
② Bridge abutment wall restrained at top by bridge slab.

→ Also known as state of elastic equilibrium as no part of soil mass has failed & attained the plastic eq bm.

② <u>Active Earth Pressure</u> :

→ Soil mass yields such that it tends to stretch horizontally.

→ This is in a state of plastic equilibrium as soil is on the verge of failure.




→ Develops on the right hand side when the wall moves towards the left.

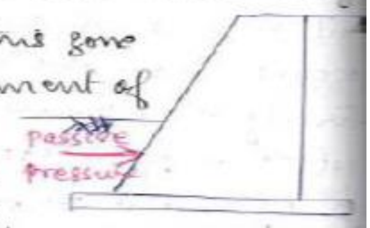
→ caused when there is an increase in the weight of the retained soil causing a substantial increase in the horizontal reaction

③ Passive Pressure

→ Occurs when the wall movement tend to compress the soil horizontally.

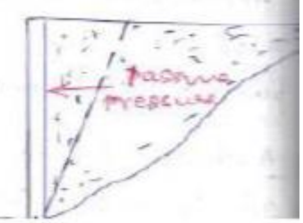
→ It is a condition of limiting equilibrium.

→ Develops on the left side of the wall below the g… level because the soil in this zone is compressed when the movement of the wall is towards left.




→ Develops on the right side of the wall when the movement of the wall is towards right.

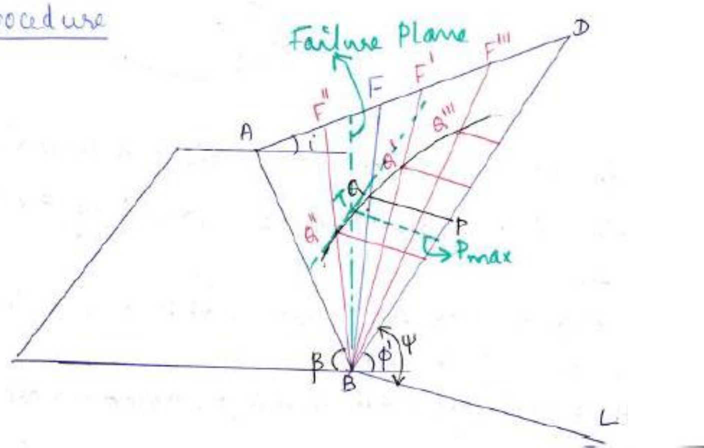
Eg. Pressure acting on anchor block.




**<u>5.b)</u> Explain Culmann's graphical method of finding out active earth pressures. [6 marks]**

**Ans:-**

Procedure



From B, a line BD is drawn at an angle $\phi'$ to the h[illegible]
As the weight of the wedge is plotted along this line, [illegible]
s also known as the weight line.
A line BL is drawn at an angle $\psi$ with the line BD, [illegible]
hat $\psi = \beta - \delta$
A failure surface BF is assumed & the weight W of
the failure wedge ABF is computed.

(4) The weight (W) of the wedge is plotted along BD such that $BP = W$.

(5) From P, draw a line PQ parallel to BL to intersect the failure surface BF at Q.

(6) The line PQ represents the magnitude of $P_a$ required to maintain equilibrium for the assumed failure plane.

(7) Similarly several other failure planes $BF''$, $BF'$, $BF'''$ etc are assumed & the procedure is repeated & thus the points

$Q''$, $Q'$, $Q'''$ etc are located.

(8) A smooth curve is drawn joining the points $Q''$, $Q$, $Q'$, $Q'''$. This curve is called culmann's line.

(9) A line (shown dotted) is drawn tangential to the culmann line & parallel to BD. Point T is the point of tangency.

(10) The magnitude of the largest value ($P_{max}$) of $P_a$ is measured from the tangent point T to the line BD and parallel to BL. It is equal to Coulomb's active pressure ($P_a$).

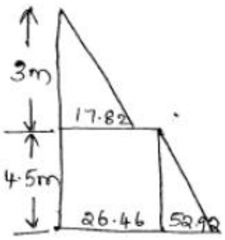
(11) The actual failure plane passes through the point T.

**5.c) A retaining wall retains a cohesionless backfill with a height of 7.5 m. The top 3m of the backfill has a unit weight of 18 kN/m³ and Φ=30°. Lower 4.5m of the backfill has a unit weight of 24 kN/m³ and Φ=20°. Obtain pressure distribution diagram and determine the total active pressure and its point of application. [10 marks]**

**Ans:-**

Ans: $K_{a_1} = \frac{1-\sin 30^\circ}{1+\sin 30^\circ} = \underline{0.33}$

$K_{a_2} = \frac{1-\sin 20^\circ}{1+\sin 20^\circ} = \underline{0.49}$




$p_1 = K_{a_1}\gamma_1 H_1 = 0.33 \times 18 \times 3 = 17.82\ kN/m^2$

$p_2 = K_{a_2}\gamma_1 H_1 = 0.49 \times 18 \times 3 = 26.46\ kN/m^2$

$p_3 = K_{a_2}\gamma_2 H_2 = 0.49 \times 24 \times 4.5 = 52.92\ kN/m^2$.

$P_1 = \frac{1}{2} \times 17.82 \times 3 = 26.73\ kN/m$

$P_2 = 26.46 \times 4.5 = 119.07\ kN/m$

$P_3 = \frac{1}{2} \times 52.92 \times 4.5 = 119.07\ kN/m$.

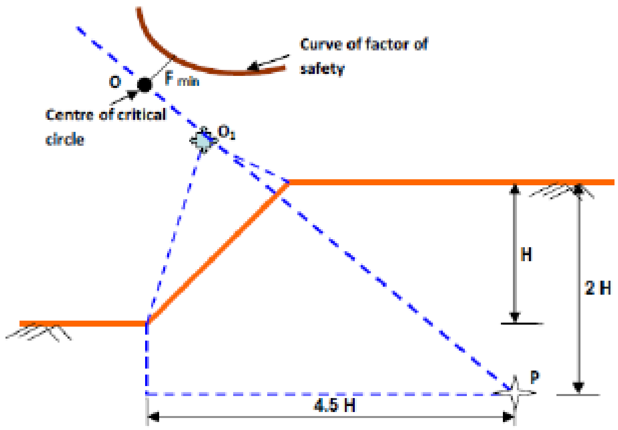
Total active pressure $= P_1 + P_2 + P_3 = \underline{264.87}\ kN/m$

$$\bar{x} = \frac{26.73\left(4.5 + \frac{3}{3}\right) + \left(\cancel{26.46}\ 119.07 \times \frac{4.5}{2}\right) + \cancel{52.92}\left(119.07 \times \frac{4.5}{3}\right)}{264.87}$$

$= \underline{2.24 m}$ from base

**6.a) Explain Fellineous method of obtaining critical slip surface in case of stability analysis of C-Φ soil. [8 marks]**

**Ans:-** For any given slope the corresponding direction angles a and g are set out from the base and the top as shown in figure. The point of intersection of these two lines is the centre of critical circle. In case of c-f 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.

**6.b) Explain the causes of slope failure and also list the types of slope failures. [6 marks]**

**Ans:-** Soil slope failures are generally of four types :

1. Translational Failure
2. Rotational Failure
3. Wedge Failure
4. Compound Failure

Causes of Slope Failure

① Erosion :-

→ Wind & flowing water cause erosion of top surface of slope & makes the slope steep.

→ Hence an increase in tangential component of force.

② Steady Seepage

→ Seepage forces in the sloping direction add to gravity & make the slope susceptible to instability.

→ Pore water pressure decrease the shear strength.

→ This condition is critical for d/s slope.

③ Sudden draw down

→ Reversal in the direction of flow.

→ Results in the instability of the side slope.

→ Shear stresses are more due to saturated unit weight while the shearing resistance decrease due to pore water pressure.

④ Rainfall

→ Saturates & erodes the soils and makes them soft.

⑤ Earthquake

→ A sudden built up of pore water pressure that reduces the available shear strength.

⑥ External loading

→ Increase the gravitational force & cause the slope to fail.

⑦ Construction activities at the toe of the slope

→ Increases the steepness of the slope & hence increase the gravitational force which will result in slope failure.

**6.c) A 5 m deep canal has side slope of 1:1. The properties of the soil are C = 20 kN/m2, Φ = 10◦, e = 0.8 & G = 2.8. If $S_n$ = 0.108, determine the FOS wrt cohesion when the canal runs full. Also find the same in case of sudden draw down, if Taylor's stability no: for this condition is 0.137. [6 marks]**

**Ans:-** When canal runs full

$\gamma_{sat}$ = (2.8+0.8)*10/ 1.8 = 20 kN/m$^3$

$\gamma_{sub}$ = 10 kN/m$^3$

Fc = 20/ (10*5*.108) = 3.704

Sudden draw down

Fc = 20/ (20*5*.137) = 1.46

**7.a) Define ultimate bearing capacity, net ultimate bearing capacity and safe bearing capacity. [6 marks]**

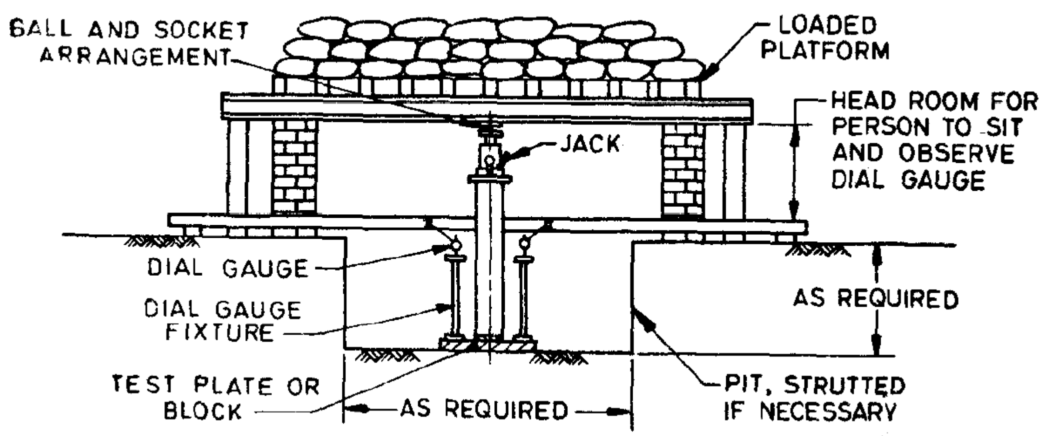
**Ans:-** Ultimate bearing capacity or Gross bearing capacity ( $q_u$): It is the least gross pressure which will cause shear failure of the supporting soil immediately below the footing.

Net ultimate bearing capacity ($q_{nu}$ ): It is the net pressure that can be applied to the footing by external loads that will just initiate failure in the underlying soil. It is equal to ultimate bearing capacity minus the stress due to the weight of the footing and any soil or surcharge directly above it. Assuming the density of the footing (concrete) and soil ( $\gamma$) are close enough to be considered equal, then, $q_{nu} = q_u - \gamma D_f$ where, $D_f$ is the depth of the footing.

Safe bearing capacity ($q_s$): It is the bearing capacity after applying the factor of safety (FS). These are of two types, Safe net bearing capacity ($q_{ns}$ ) : It is the net soil pressure which can be safety applied to the soil considering only shear failure. It is given by, $q_{ns} = q_{nu}/FS$

**7.b) Explain plate load test with a neat sketch. [8 marks]**

**Ans:-**




It is a field test for the determination of bearing capacity and settlement characteristics of ground in field at the foundation level. The test involves preparing a test pit up to the desired foundation level. A rigid steel plate, round or square in shape, 300 mm to 750 mm in size, 25 mm thick acts as a model footing. Dial gauges, at least 2, of required accuracy (0.002 mm) are placed on plate on plate at corners to measure the vertical deflection. Loading is provided either as gravity loading or as reaction loading. For smaller loads gravity loading is acceptable where sand bags apply the load. In reaction loading, a reaction truss or beam is anchored to the ground. A hydraulic jack applies the reaction load. At every applied load, the plate settles gradually. The dial gauge readings are recorded after the settlement reduces to least count of gauge (0.002 mm) & average settlement of 2 or more gauges is recorded. Load Vs settlement graph is plotted as shown. Load (P) is plotted on the horizontal scale and settlement (Δ) is plotted on the vertical scale. Red curve indicates the general shear failure & the blue one indicates the local or punching shear failure. The maximum load at which the shear failure occurs gives the ultimate bearing capacity of soil.

**7.c)**

A foundation 2.0m square is installed 1.2m below ground level in sandy soil having unit weight of $19.2 kN/m^2$ above water table and submerged unit weight of $10.1 kN/m^3$. If C = 0, and $\phi = 30°$, find ultimate bearing capacity when

i) Water table is well below the base of the foundation,
ii) Water table rises to foundation level,
iii) Water table rises to ground level.

Take $N_q = 22$ and $N_r = 20$. **(06 Marks)**

**Ans:-** B =2m, $D_f = 1.2m$, $\gamma = 19.2 kN/m^3$, $\gamma_{sub} = 10.1 kN/m^3$, c= 0, $N_q = 22$, $N_\gamma = 20$

**WT at ground level**

$q_u = 1.3*C*N_c + 0.4*B*\gamma_{sub}*N_\gamma + \gamma_{sub}*D_f*N_q \quad = 428.24 kN/m^2$.

**WT at foundation level**

$q_u = 1.3*C*N_c + 0.4*B*\gamma_{sub}*N_\gamma + \gamma*D_f*N_q \quad = 668.48 kN/m^2$.

**WT well below base of foundation level**

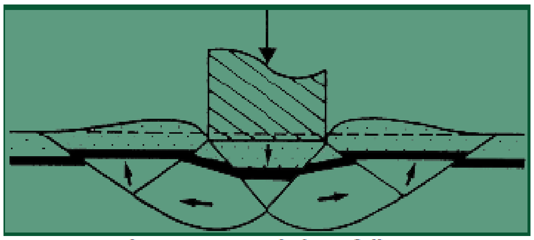
$q_u = 1.3*C*N_c + 0.4*B*\gamma*N_\gamma + \gamma*D_f*N_q \quad = 814.08 kN/m^2$.

**8.a) Distinguish between general shear failure and local shear failure. [6 marks]**

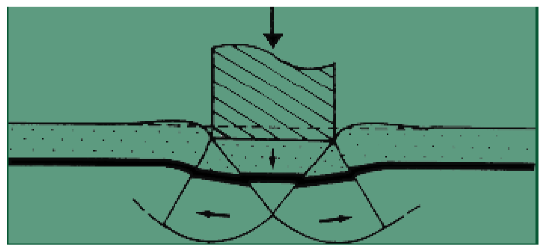
**Ans:-**

| **General Shear Failure** | **Local/Punching Shear Failure** |
|---|---|
| Occurs in dense/stiff soil<br>$\Phi>36^o$, N>30, $I_D>70\%$, $C_u>100$ kPa | Occurs in loose/soft soil<br>$\Phi<28^o$, N<5, $I_D<20\%$, $C_u<50$ kPa |
| Results in small strain (<5%) | Results in large strain (>20%) |
| Failure pattern well defined & clear | Failure pattern not well defined |
| Well defined peak in P-Δ curve. | No peak in P-Δ curve |
| Bulging formed in the neighbourhood of | No Bulging observed in the |

| | |
|---|---|
| footing at the surface | neighbourhood of footing |
| Extent of horizontal spread of disturbance at the surface large | Extent of horizontal spread of disturbance at the surface very small |
| Observed in shallow foundations | Observed in deep foundations |
| Failure is sudden & catastrophic | Failure is gradual |
| Less settlement, but tilting failure observed | Considerable settlement of footing observed |



*General shear failure*

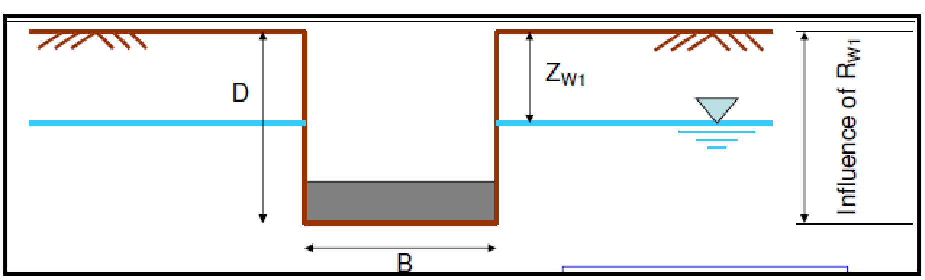


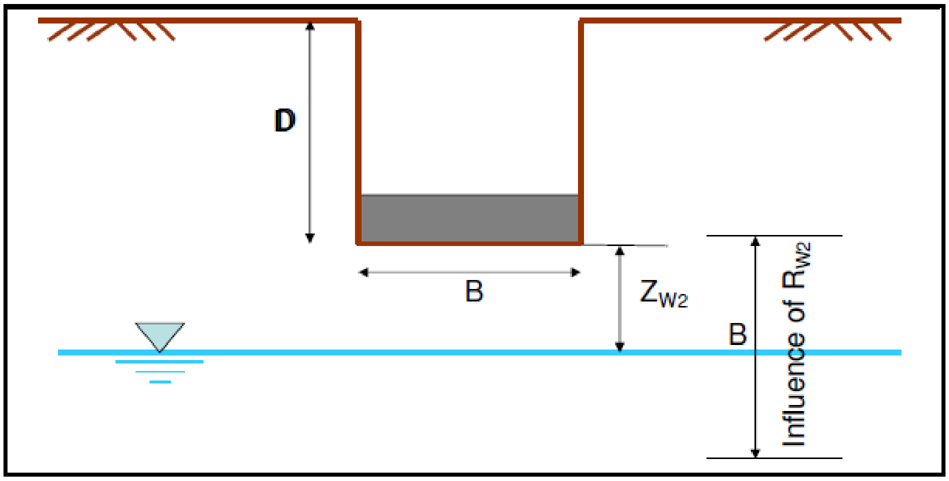
*Local shear failure*

**<u>8.b)</u> Explain with a neat sketch the effect of ground water table and eccentricity on bearing capacity.** **[8 marks]**

**Ans:- <u>Effect of water table on bearing capacity</u>**

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

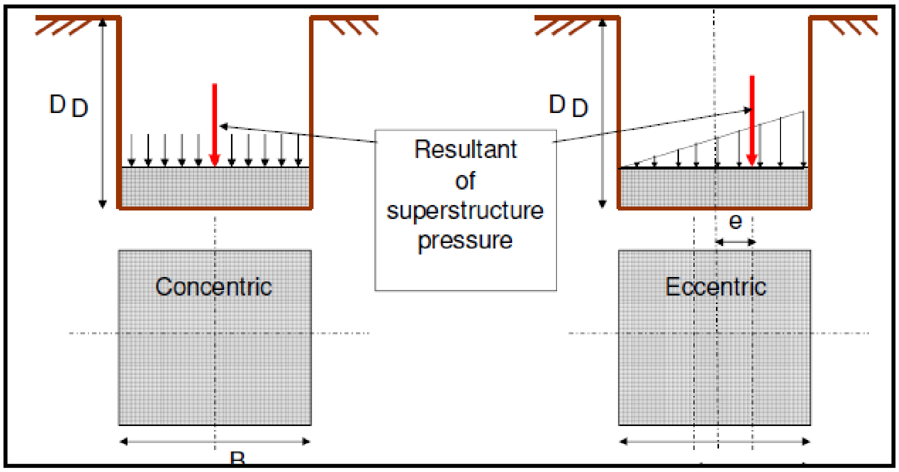
where $Z_{W2}$ is the depth of water table from foundation level.

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.

$B' = B - 2e$

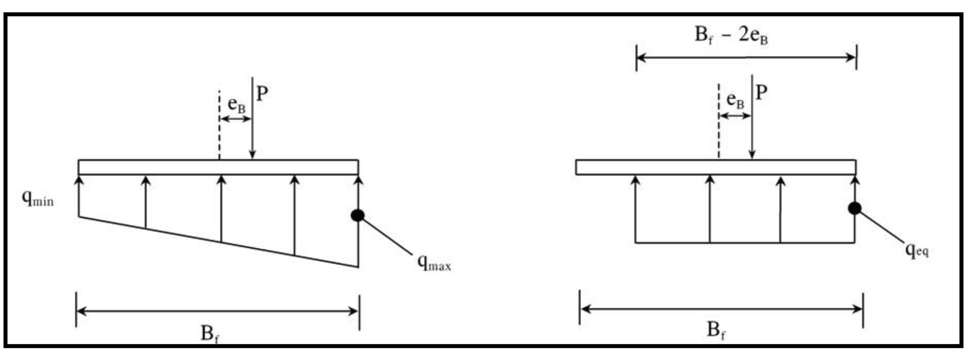



**Effect of eccentric footing on bearing capacity**

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: $A' = B' * L'$

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.




**Eccentrically loaded footing with (a) Linearly varying pressure distribution (structural design), (b) Equivalent uniform pressure distribution (sizing the footing).**

**8.c) How do you conduct SPT? What are the corrections applied to observed N values?**

**[6 marks]**

**Ans:-** The procedure for conducting SPT as per IS 2131 guidelines is as follows:

- The borehole is advanced to the required depth and the bottom cleaned.
- 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.

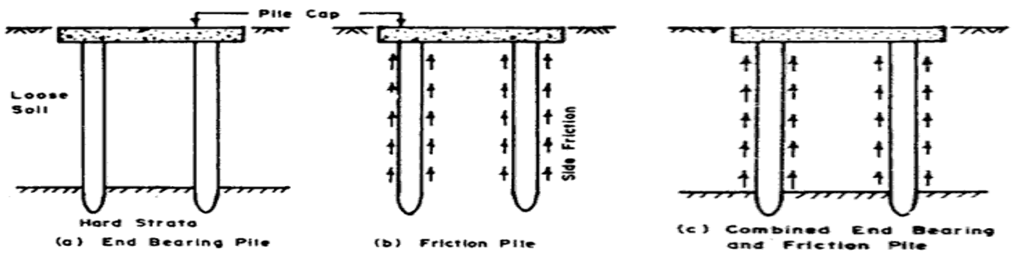
***Correction for dilatancy:-*** Di1atancy correction is to be applied when obtained after overburden correction, exceeds 15 in saturated fine sands and silts. The corrected equation is $N'' = 15 + 0.5\,(N' - 15)$

where $N''$= final corrected value to be used in design charts.

**9.a) Explain the classification of piles based on function.** **[6 marks]**

**Ans:-Classification based on the function:**

1. *End bearing piles*: Piles which transfer structural load to a hard and relatively incompressible stratum such as rock or dense sand are known as end bearing piles. These piles derive the required bearing capacity from end bearing at tip of the pile.

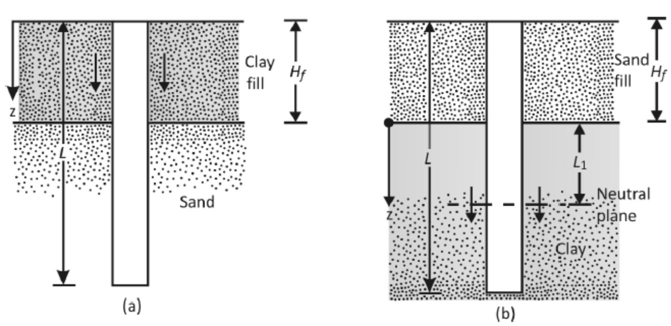



2. *Friction piles:* These are piles which derive carrying capacity from skin friction or adhesion between the pile surface and surrounding soil.

3. *Combined end bearing and friction piles*: These piles transfer loads by a combination of end bearing at the bottom of the pile and friction along the surface of the pile shaft.

**9.b) Explain negative skin friction in pile foundation.** **[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 $K_o = 1-\sin\Phi$

$\gamma_f$ = unit weight of the soil , $\delta$ = 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.

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

<u>Negative skin friction in pile group</u>:- When a pile group passes through a soft unconsolidated stratum, the magnitude of $Q_{nsf}$ may be estimated from the following equations and the higher of the values obtained should be used in design.

$Q_{nsf}$ (block) = n* $Q_{nsf}$

$Q_{nsf}(block) = (C_u*H_f*P_b) + (\gamma*H_f*A_b)$

Where, n = no: of piles in the group, $P_b$ = perimeter of the block/group, $\gamma$ = unit weight of the soil within the pile group upto a depth $H_f$ , $A_b$ = area of the block/group.

**<u>10.)</u>**

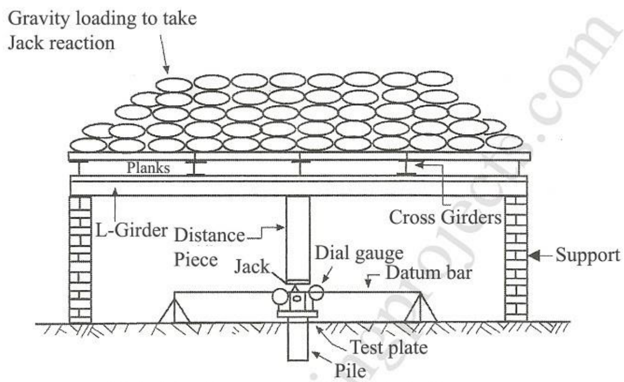
Write short notes on any four of the following:

a. Pile load test
b. Under reamed piles
c. Settlement of piles
d. Efficiency of pile group
e. Group capacity of piles. **(20 Marks)**

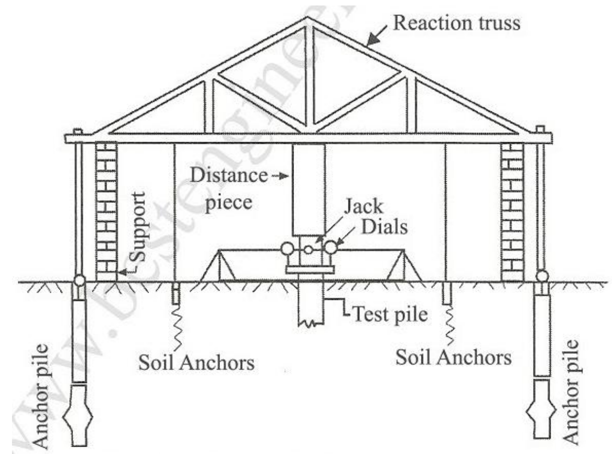
**Ans:-** <u>**Under reamed Piles**</u>

- 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.
- Provision of under reamed bulbs has the advantage of increasing the bearing and uplift capacities. It also provides better anchorage at greater depths.
- Indian Standard IS 2911 (Part III) - 1980 covers the design and construction of under reamed piles having one or more bulbs.
- According to the code the diameter of under reamed bulbs may vary from 2 to 3 times the stem diameter depending upon the feasibility of construction and design requirements.
- The code suggests a spacing of 1.25 to 1.5 times the bulb diameter for the bulbs.
- An angle of $45^0$ with horizontal is recommended for all under reamed bulbs.

## <u>Pile Load Test</u>




(a) Jack loading reaction by loaded platform




(b) Jack loading reaction by anchors

- The pile load test can be performed either on a working pile which forms the foundation of a structure or on a test pile. A test pile is the one used only in a load test and does not carry the load of the superstructure. The minimum test load on such piles should be 2*safe load from static formula or load corresponding to settlement of 0.1* pile dia. A working pile is a driven or cast in situ pile along with other piles to carry the load from the superstructure. The test load on such piles = 1.5*safe load or load corresponding to 12 mm settlement.
- According to IS 2911 part4, 1979, the test shall be carried out by applying a series of vertical downwards loads on a RCC cap over the pile. Load shall be applied by a remote controlled hydraulic jack taking reaction against a loaded platform. Load shall be applied at increments of 20% of safe load.
- Settlements shall be recorded with 3 dial gauges with a sensitivity of 0.02mm, symmetrically arranged over the test plate. Each load increment is kept for sufficient time till the rate of settlement becomes less than 0.02mm per hour.
- Test load is increased to 2.5* estimated load or load corresponding to settlement = 0.1*pile dia. Results are plotted in the form of load vs settlement curve.
- Allowable load on a single pile shall be lesser of the following:
    a. Two-thirds of the final load which cause a settlement of 12mm
    b. One-half to two-third of the final load which cause a settlement of 0.1*pile dia, or

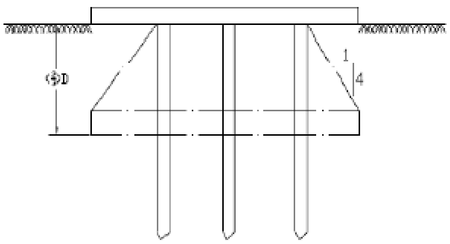
Two-thirds of final load which causes a net settlement of 6mm.

## Settlement of Pile Groups

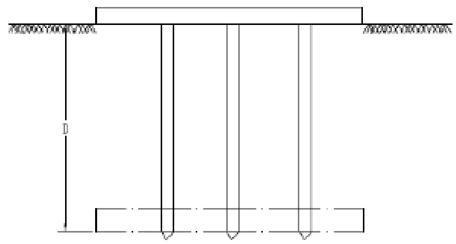
Due to group action, both immediate and consolidation settlement values of a pile group are greater than those for a single pile.

For bearing piles the total foundation load is assumed to act at the base of the piles on an imaginary foundation of the same size as the plan of the pile group as show in Fig 7.9 (b)

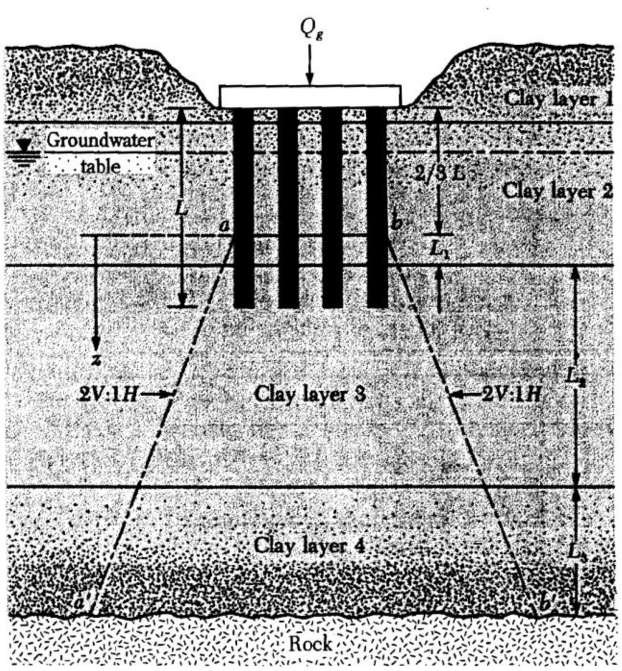
For friction piles it is virtually impossible to determine the level at which the structural load is effectively transferred to the soil. The level used in design is at a depth of two-thirds the penetration depth.




Friction piles



End bearing piles




Consolidation settlement of Pile group

***<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, $\sigma_o$ = initial overburden pressure at the middle of the clay strata

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

***Settlement of pile groups in sand:-***

*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.

*Meyerhof's settlement ratio:-* $S_g/S_i = [s*(5- s/3)/ ( 1+ 1/r )^2]$

Where, s = ratio of pile spacing to pile diameter

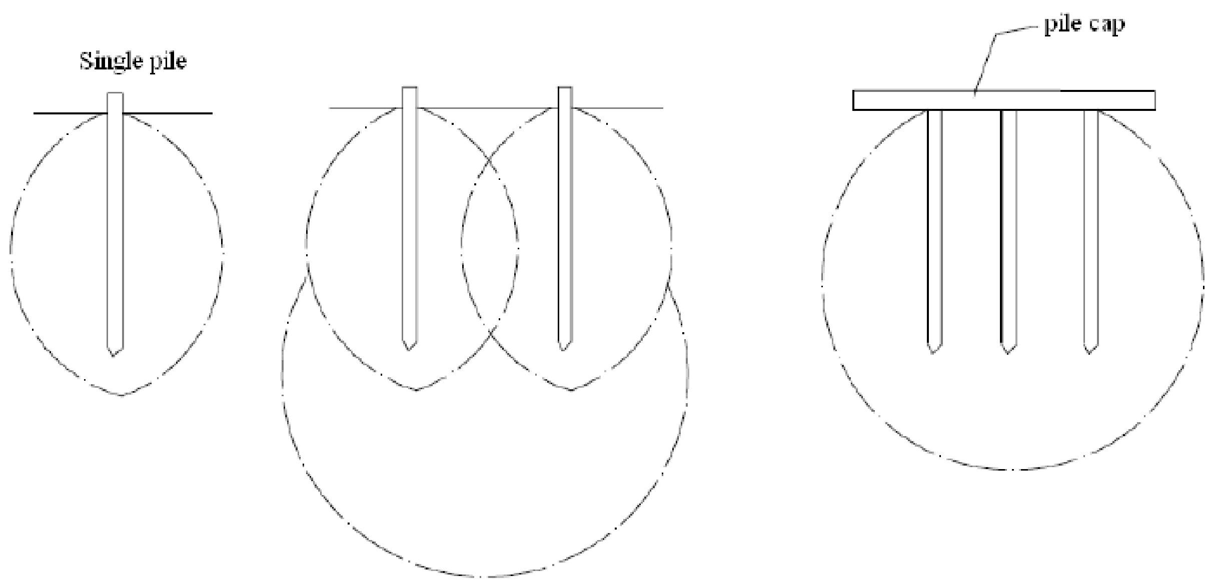
r = no: of rows in the pile group.

## Group Efficiency 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. 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 $\eta_g = \frac{Q_{g(u)}}{NQ_u} \times 100$

$\eta = group\ efficiency$

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

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

n = number of piles in the group

The efficiency of pile group depends on the following factors:

1. Spacing of piles

2. Total number of piles in a row and number of rows in a group, and

3. Characteristics of pile (material, diameter and length)

Efficiency of a pile group can also be obtained by using Converse – Lebarre formula:

$$\eta_g = 1 - \frac{\theta}{90}\left(\frac{(n-1)m + (m-1)n}{m \times n}\right)$$

Where, m = number of rows, n = number of piles in a row, $\theta = \tan^{-1}\frac{d}{s}$, d = diameter of pile and s = spacing of piles.

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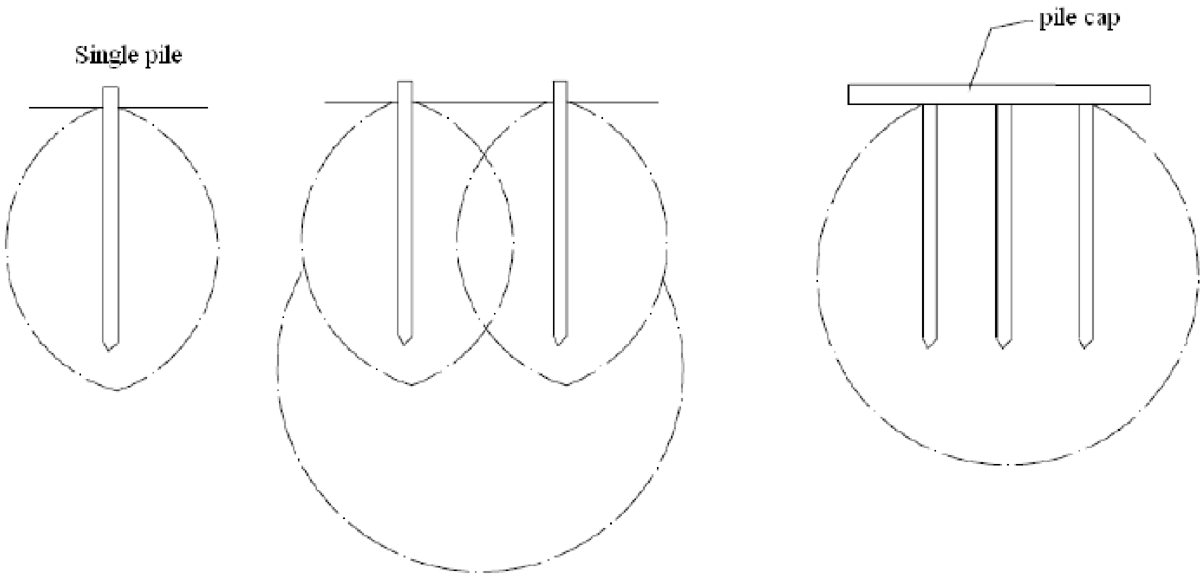


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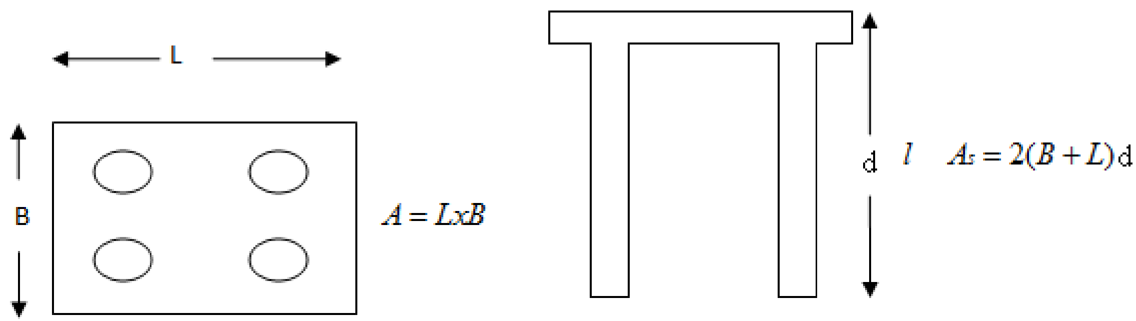
## <u>Ultimate Load Carrying Capacity for the Pile Group</u>

A group of piles may fail as a block, i.e., by sinking into the soil and rupturing it at the periphery of the group.




The ultimate load carrying capacity for the pile group taken as a block is given by

$$Q_{ug} = (C_{ub}*N_c*A_b) + (P_b*L*C_u')$$

Where $C_{ub}$= undrained strength of clay

$N_c$= Bearing capacity factor, taken as 9.0

$A_b$= c/s area of the block

$P_b$ = perimeter of the block

The load carrying capacity of the pile group may also be evaluated as

$$Q_u = (C_p*N_c*A_p) + (\alpha*C*A_s)$$

Here each pile is assumed to individually carry the same load whether in group or as a single pile. The lower of two values is taken as the load carrying capacity of pile group.