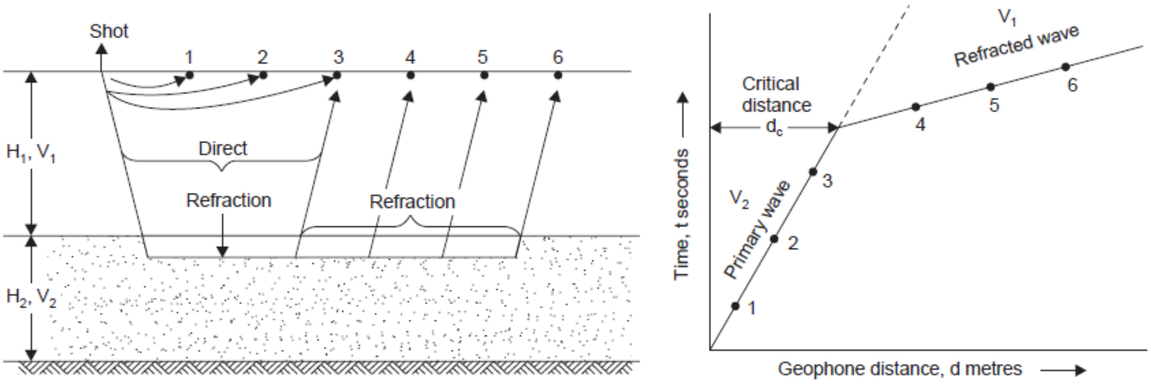
APPLIED GEOTECHNICAL ENGINEERING		
VTU Queestion Paper Solution - July - August 2022		
Module 1		
1a	What is soil exploration? What are the objectives of soil exploration?	7
	Soil exploration consists of determining the profile of the natural soil deposits at the site, taking the soil samples and determining the engineering properties of soils using laboratory tests as well as in-situ testing methods. Subsurface exploration depends upon the economy and importance of construction. Cost varies from 0.05 – 0.2% of the total cost of the entire structure and sometimes can go even upto 1%. Objectives 1. To select the type and depth of foundation of a structure 2. To determine the bearing capacity of soil 3. To determine the probable maximum and differential settlement 4. To determine the ground water table and to determine the properties of water 5. To predict lateral earth pressure against retaining walls and abutments 6. To select suitable construction techniques 7. To predict ad to solve potential foundation problems 8. To ascertain the suitability of soil as a construction material 9. To investigate safety of existing structure and to suggest remedial measures 10. To locate transportation routes 11. To know geological condition of rock and soil formation 12. To know the properties for design 13. To establish procedure for ground improvement 14. To observe the performance after construction	
1b	With a neat sketch explain seismic refraction method.	7
	Geophysical methods involve the technique of determining subsurface materials by measuring some physical property of the materials, and through correlations, using the values obtained for identifications. Seismic refraction method is one such method. When a shock or impact is made at a point on or in the earth, the resulting seismic (shock or sound) waves travel through the surrounding soil at speeds related to their elastic characteristics. The velocity is given by: $$v = C\sqrt{\frac{Eg}{\gamma}}$$ where, v = velocity of the shock wave, E = modulus of elasticity of the soil, g = acceleration due to gravity, γ = density of the soil, and C = a dimensionless constant involving Poisson's ratio. Principle: ➢ A shock may be created with a sledge hammer hitting a strike plate placed on the ground or by detonating a small explosive charge at or below the ground surface. ➢ The radiating shock waves are picked up by detectors, called 'geophones', placed in a line at increasing distances, d1, d2, ..., from the origin of the shock. ➢ The time required for the elastic wave to reach each geophone is automatically recorded by a 'seismograph'.	

- Some of the waves, known as direct or primary waves, travel directly from the source along the ground surface or through the upper stratum and are picked up first by the geophone.
- If the sub soil consists of two or more distinct layers, some of the primary waves travel downwards to the lower layer and get refracted as the surface.
- If the underlying layer is denser, the refracted waves travel much faster.
- As the distance from the source and the geophone increases, the refracted waves reach the geophone earlier than the direct waves.
- The distance of the point at which the primary and refracted waves reach the geophone simultaneously is called the 'critical distance' which is a function of the depth and the velocity ratio of the strata.
- The results are plotted as a distance of travel versus time graph, known as the 'time travel graph'.
- The reciprocal of the slope of the travel-time graph gives the velocity of the wave.
- Depth of the first layer is estimated as

$$H_1 = \frac{d_1}{2}\sqrt{\frac{v_2 - v_1}{v_2 + v_1}}$$

- Second layer thickness is estimated as:

$$H_2 = 0.85H_1 + \frac{d_2}{2}\sqrt{\frac{v_3 - v_2}{v_3 + v_2}}$$



1c	Determine the area ratios for the following soil sampler and comment the nature of samples obtained in each samplers (i) Core cutter 185 mm OD and 135 mm ID (ii) Split Barrel 51 mm OD 45 mm ID (iii) Shelby tube 51 mm OD 49 mm ID Which one you recommend to be used for getting good qualifier samples?	6
	(i) A_r = 87.79 % (ii) A_r = 28.44 % (iii) A_r = 8.33 % Shelby tube samplers are preferred since A_r is less than 10%	
2a	List the methods of dewatering technique used in the field and explain vaccum method.	6
	Drainage is the removal of removing gravity water from a soil mass to keep the soil in stable condition. Drainage includes sub surface drainage and surface drainage. Purpose of dewatering Construction stage	

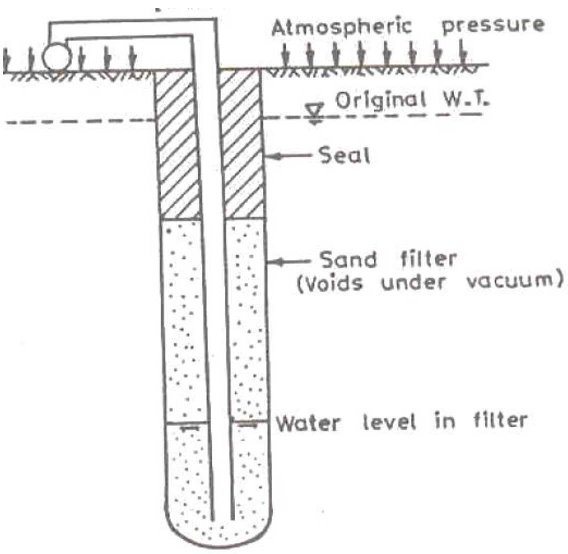
- Provide a dry excavation and permit construction and hence to proceed efficiently
- Reduce lateral pressure on sheeting and bracings
- Stabilize quick conditions, prevent heaving and piping
- Increase the supporting characteristics of foundation materials
- Increase stability of slope
- Cut off capillary rise and prevent piping and frost heaving in pavements

Post construction stage

- Reduce uplift pressure at the bottom of slabs, basements and canallinings
- Provide dry basements
- Reduce lateral pressure on retaining walls
- Control embankment seepage in all dams
- Control pore pressure beneath pavements

Vaccum method

Description



- Well points, cannot be used for draining silty sands and other fine sands with an effective size less than about 0.05 mm. such soil has a permeability between 1×10^{-5} to 1×10^{-7} m/s. such soils can be effectively drained using this method.
- A hole of 25 cm diameter is formed around the well point and the rising pipe by jetting water under pressure. When the water is still flowing, coarser sand is filled from the top upto 1 m from the top. The top 1 m is filled with clay or any other impervious material to form a seal.
- Then the header is connected to a vaccum pump and this will create a vaccum in the filter sand. As the pressure on water is at atmospheric pressure, head is created which decreases the flow resistance . Hence the ground water flows in the region of vaccum and dewatering occurs.
- As the effective pressure is reduced, consolidation occurs and it’s a slow process (take several weeks)

2b	List and explain different types of samples of soil	6

Samples are classified as

(i) Non-representative soil samples

(ii) Representative soil samples

Representative soil samples are further classified as

(a) Undisturbed and

(b) Disturbed soil samples

Non-representative Soil Sample

These are mixtures of soil from different soil strata. These samples are obtained by auger boring or sedimentation of wash boring. Such samples may help in determining the depth at which major changes in soil profile occur. In these samples, neither the structure, nor the moisture content nor the particles are preserved.

Representative soil sample - undisturbed

	The soil sample, in which the particle size distribution as well as the soil structure and the properties of the in situ stratum, remain preserved, is termed as undisturbed soil samples. Such soil samples are required for shear strength, consolidation tests, permeability and consolidation characteristics. This soil samples can be collected by stopping the boring process at a certain level and then inserting the appropriate sampler below the bottom of the bore. Natural water content remains unaffected. Representative soil sample - Disturbed soil sample The soil sample which contains the same particle size distribution as in the in situ stratum, but the natural structure of sample gets partly or entirely disturbed and modified, it is called a representative or disturbed soil sample. These soils represent composition and the natural content of the soil. can be used to determine the index properties of soils such as grain size, plasticity characteristics and specific gravity.	
2c	Estimate the position of ground water table with following data by Hvorslev's method. Depth upto which water is bailed out = 10.5 m Water rise in 1st day = 0.63 m; 2nd day = 0.57 m; 3rd day = 0.51 m	8
	$h_1 = 0.63$ m $h_2 = 0.57$ m $h_3 = 0.51$ m $H_o = \frac{h_1^2}{h_1 - h_2} = \frac{0.63^2}{0.63 - 0.57} = 6.615\ m$ $H_2 = \frac{h_2^2}{h_1 - h_2} = \frac{0.57^2}{0.63 - 0.57} = 5.415\ m$ $H_3 = \frac{h_3^2}{h_2 - h_3} = \frac{0.51^2}{0.57 - 0.51} = 4.335\ m$ Ground water table = 10.5 - h_w Based on base ground level $h_{w1} = (h_w + H_0) - H_0 = 10.5 - 6.615 = 3.885$ m Based on H_2 $h_{w2} = (h_w + H_0) - (H_2+h_1+h_2) = 10.5 - (5.415+0.63+0.57) = 3.885$ m Based on H_3 $h_{w3} = (h_w + H_0) - (H_3+h_1+h_2+h_3) = 10.5 - (4.335+0.63+0.57+0.51) = 4.455$ m $Average\ water\ table\ depth = \frac{1}{3} \times (3.885 + 3.885 + 4.455) = 4.075\ m$	
	Module 2	
3a	Distinguish between Boussinesq's theory with Westergaard's theory of stress distribution.	6
	Boussinesq's theory for vertical stresses due to a concentrated load Assumptions: 1. Soil mass is an elastic continuum, having constant modulus of elasticity 2. Soil mass is homogenous with identical properties at different point 3. Soil is isotropic 4. Soil mass is semi- infinite 5. Soil is weightless and is free from residual stresses before the application of the load Westergaard's solution Boussinesq's solution is isotropic. But actual sedimentary deposit are anisotropic. For eg. Thin layers of sand embedded in a homogenous soil mass and these thin sheets are assumed to be of infinite	

rigidity making it incompressible. Therefore, thin seams permit only downward movement of the soil mass without any lateral displacement.

Assumptions

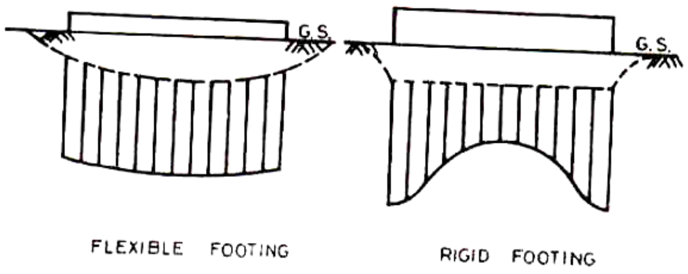
- Soil mass is anisotropic
- Seams/thin sheets of sand intercept homogenous soil deposits
- Seams of soil/sand that intercept have infinite rigidity and permit only downward displacement of soil mass
- No lateral displacement of seams
- Use Poisson's ratio in the impressions.

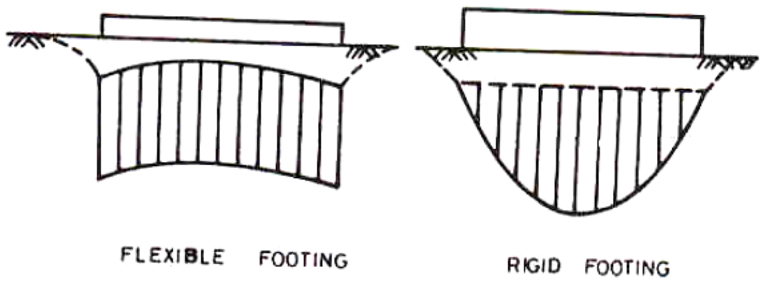
3b Explain contact pressure distribution of soil

The upward pressure exerted due to soil on the underside of the footing is termed as contact pressure. In the previous theories it is termed as contact pressure. In the previous theories it is assumed that the footing is flexible. But actual footings are not as flexible as assumed.

The contact pressure distribution depends upon a number of factors such as

- Poisson's ratio of soil and footing material
- Modulus of elasticity
- Width of footing, thickness of footing etc

<table>
<tr><td colspan="2">Contact pressure on saturated clay</td></tr>
<tr><td>Flexible footing</td><td>Rigid footing</td></tr>
<tr><td>Settlement is max at the centre
Contact pressure is uniform</td><td>Settlement is uniform
Pressure is max at edges and min at centre</td></tr>
<tr><td colspan="2">for rigid footing plastic flow occurs and the pressure becomes finite

</td></tr>
<tr><td colspan="2">Contact pressure on sand</td></tr>
<tr><td>Flexible footing</td><td>Rigid footing</td></tr>
<tr><td>Edge of the footing undergoes a larger settlement than at the centre. The soil at the centreis confined and therefore has a high modulus of elasticity and deflects less for the same contact pressure.
Contact pressure is uniform</td><td>Settlement is uniform
Contact pressure increases from 0 at edges to a maximum at the centre. If the footing is embedded, there would be finite contact pressure at the edges. If the footing is embedded, contact pressure is maximum.</td></tr>
</table>

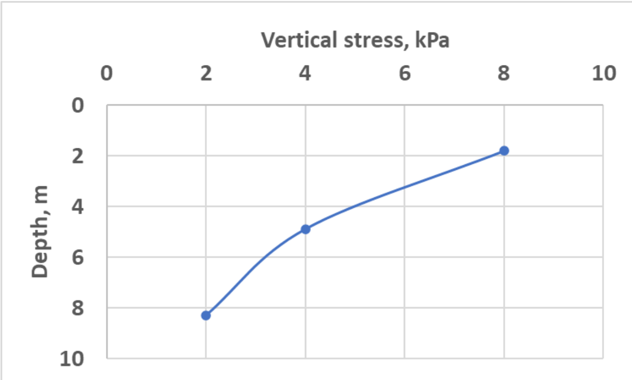


3c A circular area 6 m in diameter carries uniformly distributed load of 10 kPa. Determine the vertical stress at a depth of 2m, 4 m and 8 m. plot the variation of vertical stress with depth. 8

$$\sigma_z = 100\left[1 - \left[\frac{1}{1+\frac{3^2}{2^2}}\right]^{\frac{3}{2}}\right] = 8.293\ kPa$$

$$\sigma_z = 100\left[1 - \left[\frac{1}{1+\frac{3^2}{16}}\right]^{\frac{3}{2}}\right] = 4.88\ kPa$$

$$\sigma_z = 100\left[1 - \left[\frac{1}{1+\frac{3^2}{8^2}}\right]^{\frac{3}{2}}\right] = 1.79 kPa$$



4a Explain 6

(i) Pressure bulb
(ii) Pressure distribution on horizontal plane
(iii) Pressure distribution on vertical plane

(i) Isobars are useful for determining the effect of the load on the vertical stress at various points. The zone within which the stresses have a significant effect on the settlement is known as pressure bulb. It is generally assumed as an isobar of 0.1 Q. the area outsde the pressure bulb will have negligible stresses.

(ii) Stress distribution on a horizontal plane

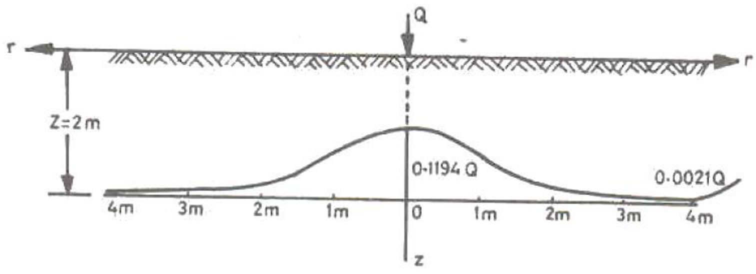
$$\sigma_z = I_B . \frac{Q}{z^2}$$

For a depth of 2 m,

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

$$\sigma_z = 0.25 \times I_B \times Q$$

r	0.0000	0.5000	1.0000	1.5000	2.0000	2.5000	3.0000	3.5000	4.0000
r/z	0.0000	0.2500	0.5000	0.7500	1.0000	1.2500	1.5000	1.7500	2.0000
I_B	0.4775	0.4103	0.2733	0.1565	0.0844	0.0454	0.0251	0.0144	0.0085
σ_z/Q	0.1194	0.1026	0.0683	0.0391	0.0211	0.0114	0.0063	0.0036	0.0021

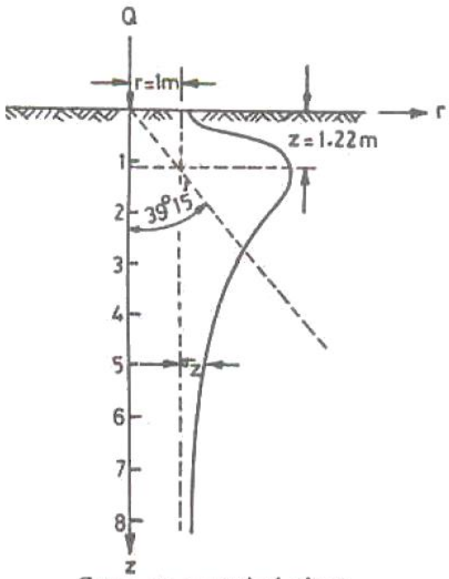


Vertical Stress on a horizontal plane.

(iii) Vertical stress distribution in a vertical plane

Let the radius be 1 m and for different depths the vertical stresses can be determined

z	0.2500	0.5000	1.0000	1.5000	2.0000	2.5000	3.0000	4.0000	5.0000
r/z	4.0000	2.0000	1.0000	0.6667	0.5000	0.4000	0.3333	0.2500	0.2000
I_B	0.0004	0.0085	0.0844	0.1904	0.2733	0.3295	0.3669	0.4103	0.4329
σ_z/Q	0.0064	0.0342	0.0844	0.0846	0.0683	0.0527	0.0408	0.0256	0.0173



Stress on a vertical plane

4b	What are the different types of settlements. Explain.	6

Foundation settlement can be classified into three types

- Immediate settlement
- Consolidation settlement
- Secondary settlement

Immediate settlement

- Takes place immediately after construction of the structure
- Called as distortion settlement because it is due to distortions/rearrangements within the foundation soil

Consolidation settlement

- This is due to gradual expulsion of water from the voids of the soil
- Because of the hydraulic gradient water starts flowing out and decrease in volume occurs
- This depends upon the permeability of the soil and its time dependent

Secondary settlement

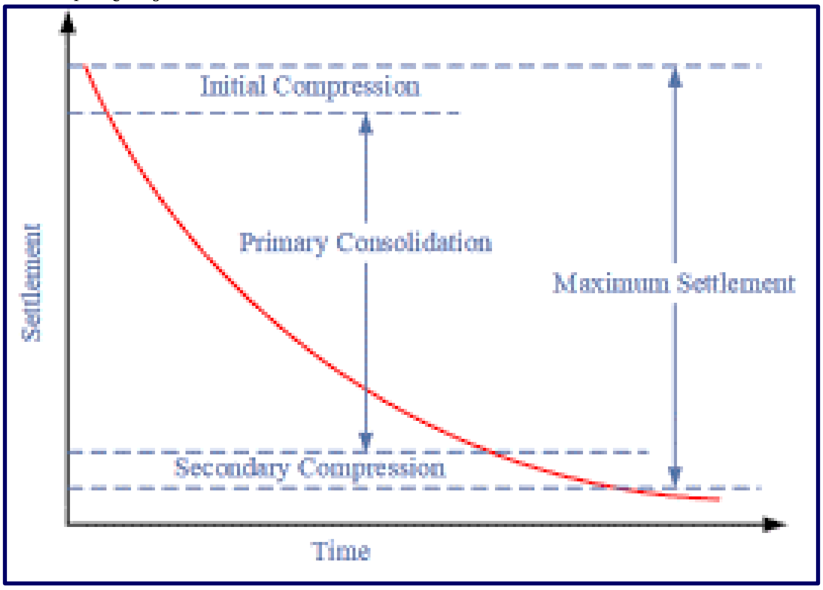
- This is due to secondary consolidation
- Excess pore water if its there gets dissipiated when the primary consolidation is complete.

- ➢ Plastic readjustment of particles takes place and change in volume occurs which is very minimal

Settlement due to other causes

- ➢ Underground erosion -This will cause formation of cavities in the soil which when collapsed causes settlement
- ➢ Structural collapse of soil – due to dissolution of materials responsible for intergranular bond of grains
- ➢ Thermal changes – causes settlement in expansive soils
- ➢ Frost heave – occurs in structures founded in frost susceptible soils
- ➢ Vibrations and shocks -causes more settlement in loose, cohesionless soils
- ➢ Mining subsidence – due to removal of minerals and other materials from mines below
- ➢ Landslides – especially in unstable slopes
- ➢ Creep – especially on clayey slope (more settlement for no change in load)
- ➢ Changes in the vicinity – due to the construction of a new building near the existing foundation, settlement may occur

Total settlement = S_i+S_c+S_s



4c A normally consolidated clay layer is 18 m thick. Natural water content is 45%, saturated unit weight is 18 kN/m^3, specific gravity is 2.7 and liquid limit is 63%. The vertical stress increment at centre of clay layer due to foundation load is 9 kPa. Ground water table is at the surface. Determine the settlement. **8**

$$c_c = 0.009\,(63-10)\, for\ undisturbed\ soil$$
$$c_c = 0.009\,(63-10) = 0.477$$
$$\sigma_0 = 18 \times 9 = 162\ kPa$$
$$eS = wG$$
$$e = 0.45 \times 2.68 = 1.215$$
$$S_c = \frac{c_c}{1+e_0}.H.\log_{10}\left[\frac{\sigma_0+\Delta\sigma}{\sigma_0}\right]$$
$$S_c = \frac{0.477}{1+1.215}.18.\log_{10}\left[\frac{162+9}{162}\right] = 91.02\ mm$$

Module 3

5a With neat sketches explain types of earth pressure. **6**

At rest pressure

- ➢ Soil mass is not subjected to any yielding / movement.

➢ This case occurs when the retaining wall is firmly fixed at its to and is not allowed to rotate or move laterally.

➢ Figure above shows a brdge abutment slabwhich is restrained at its top by the bridge slab. This at rest condition is in elastic equilibrium.

Active earth pressure

➢ Soil mass yields such that it starts to stretch horizontally.

➢ This is in a state of pastic equilibrium.

➢ Soil mass is on the verge of failure

➢ This happens on the right hand side when the wall moves towards the left.

➢ An increase in weight of thevreatined soil cause a substaintial increase in horizontal reaction.

Passive earth pressure

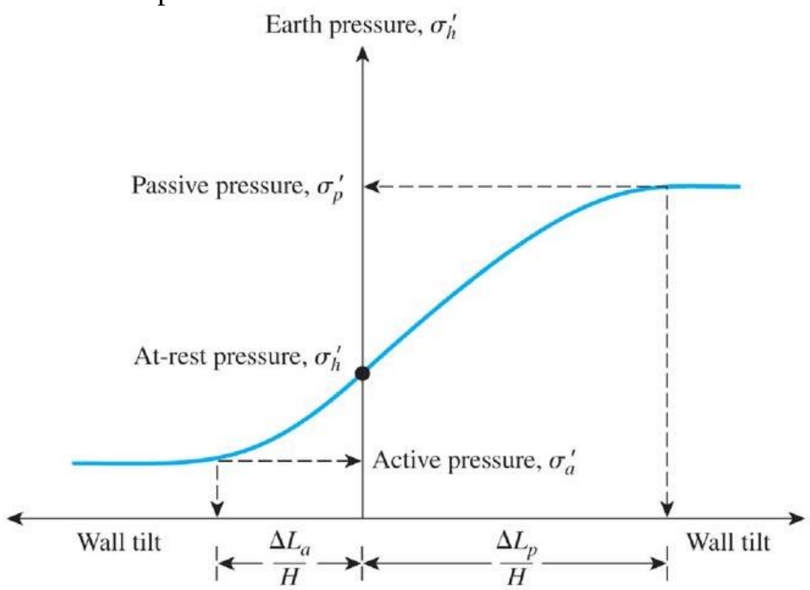
➢ When the movement of the wall is such that it tends to compress horizontally

➢ Condition of limiting equilibrium

➢ Develops on the left side of the wall below the ground level

➢ Because the soil in this zone is compressed when the movement of the soil is towards left.

Variation of pressure

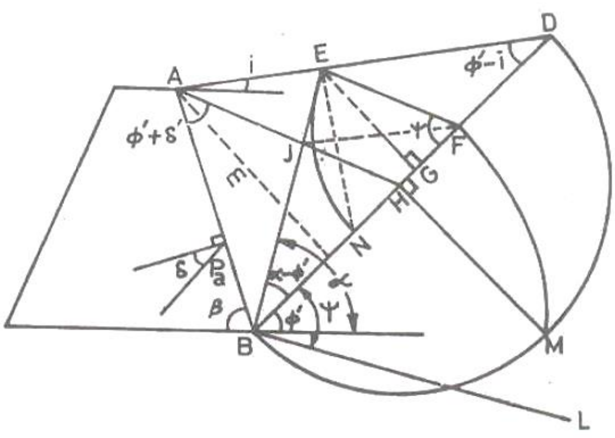


5b	Describe Rebhann's graphical method of determining the active earth pressure on retaining wall.	8

Rehbann gave a graphical method for the determination of the total active pressure according to Coulomb's theory. This is based on Poncelet's solution and hence its also known as Poncelet's method.

The step-by step procedure of this graphical method is as follows:

1. Line BD is drawn at an angle ϕ' to the horizontal.



2. Line BL is drawn at an angle ψ with the line BD, is known as the earth pressure line. The angle ψ is equal to β – δ.
3. A semi circle BMD is drawn on BD as diameter.
4. Line AH is drawn parallel to B, intersecting line BD at H. A perpendicular HM is drawn at H, intersecting the semi-circle at M.
5. With B as centre and BM as radius, arc MF is drawn, intersecting BD at F. the line FE is drawn parallel to BL, intersecting the ground surface at E.
6. With F as centre and FE as radius, an arc is drawn to intersect BD at N. the line BE represents the critical failure plane.
7. The total active pressure P_a is given by

$$P_a = \gamma.(\,area\ of\ triangle\ NEF)$$

$$P_a = \gamma.\left(\frac{1}{2} \times NF \times x\right)$$

Where x is the perpendicular distance EG between E and BD

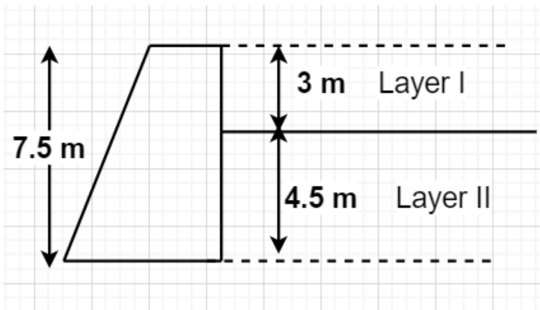
5c A retaining wall 7.5 m high retains cohesionless, horizontal backfill. The top 3 m of fill has aunit weight of 18 kN/m^3 and φ = 30° and the rest has a unit weight of 24 kN/m^3 and φ = 20°. Determine using Rankine's theory, the distribution of active earth pressure and total active earth thrust. 6

Layer I

$$k_a = \frac{1 - Sin30}{1 + Sin30} = 0.33$$

Layer II

$$k_a = \frac{1 - Sin20}{1 + Sin20} = 0.49$$

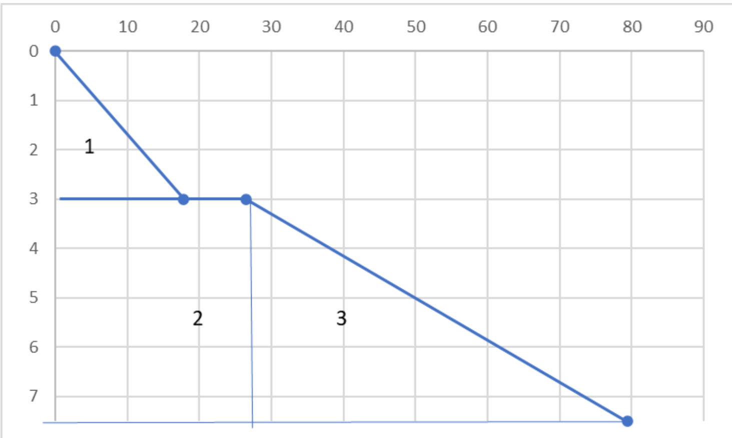


$$\sigma_z = k_a.\gamma z$$

z	Vertical stress	
0	0.5 ×	
3 in layer I	0.33 × 18 × 3	17.82
3 in layer II	0.49× 18 × 3	26.46
7.5 m	0.49× 24 × 4.5 +26.46	79.38

$\bar{x} = 2.24\ m\ from\ base$

Resultant is 264.87 kN acting at a distance of 2.24 m from base



6a With neat sketches, explain types of slope failures. 6

Soil slope failures are generally of four types :

1. Translational Failure

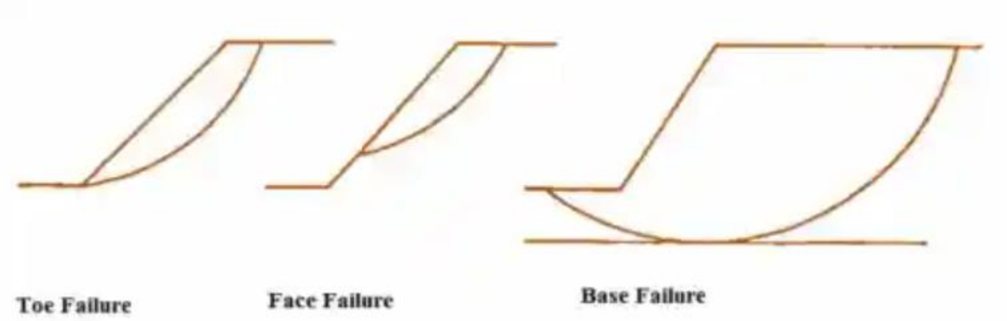
2. Rotational Failure
3. Wedge Failure
4. Compound Failure

Translational Failure

- Translation failure occurs in the case of infinite slopes and here the failure surface is parallel to the slope surface.
- A slope is said to be Infinite, when the slope has no definite boundaries and soil under the free surface contains the same properties up to identical depths along the slope.
- As said above, when the soil along the slope has similar properties up to a certain depth and soil below this layer is strong or hard stratum, the week topsoil will form a parallel slip surface when failed.
- This type of failure can be observed in slopes of layered materials or natural slope formations.

Rotational Failure

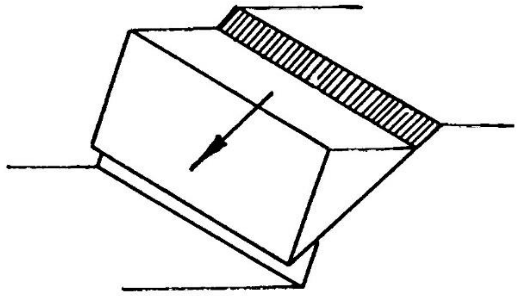
- In the case of rotational failure, the failure occurs by rotation along a slip surface and the shape thus obtained in slip surface is curved. Failed surface moves outwards and downwards.
- In homogeneous soils, the shape is circular while in case of non-homogeneous soils it is non-circular.
- Rotational failure may occur in three different ways :
  1. Face failure or slope failure
  2. Toe failure
  3. Base failure



- Face failure occurs when soil above the toe contains weak stratum. In this case the failure plane intersects the slope above toe.
- Toe failure is the most common failure in which failure plane passes through toe of slope.
- Base failure occurs when there is a weak soil strata under the toe and failure plane passes through base of slope.
- Rotational failure can be seen in finite slopes such as earthen dams, embankments, man-made slopes etc.

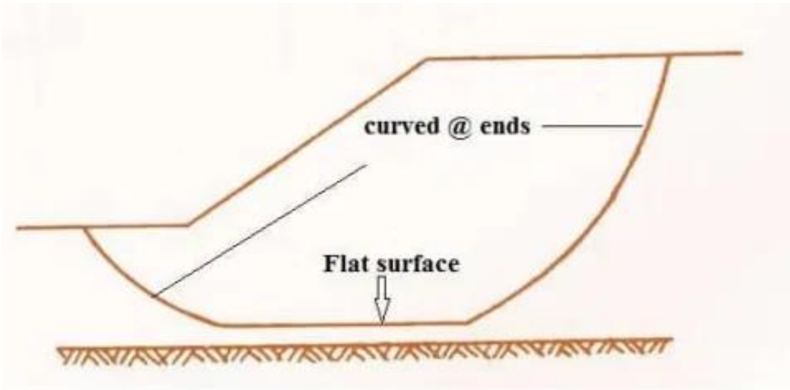
Wedge Failure

- Wedge failure, also known as block failure or plane failure, generates a failure plane that is inclined.
- This type of failure occurs when there are fissures, joints, or weak soil layers in slope, or when a slope is made of two different materials.
- It is more similar to translational failure but the difference is that translational failure only occurs in case of infinite slopes but wedge failure can occur in both infinite and finite slopes.



Compound Failure

- A Compound failure is a combination of translational slide and rotational slide.
- In this case, the slip surface is curved at two ends like rotational slip surface and flat at central portion like in translational failure.
- The slip surface becomes flat whenever there is a hard soil layer at a considerable depth from toe.



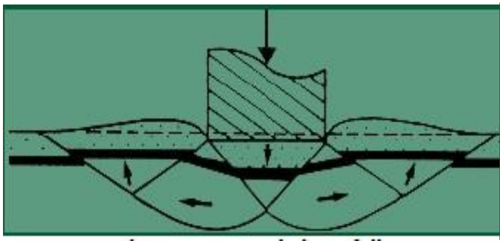
6b	Explain Swedish circle method of stability analysis of slopes for c – φ soils.	6

The actual shape of a slip surface in the case of finite slopes is curvilinear. However, for convenience its considered as circular. This methos was developed in Sweden. The step by step procedure is as follows.

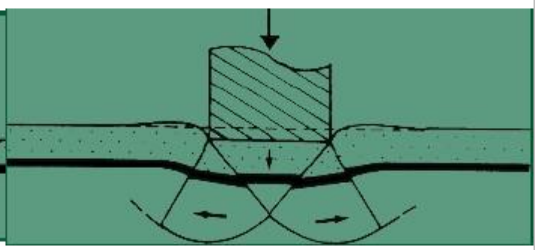
1. Let AB be the circular surface with radius r and centre O, the trial failre wedge above the slip surface is divided into vertical slices by drawing vertical lines as shown.
2. The slices are usually of equal width. In case of non- homogenous slopes where the slip surface passes through one type of material, a vertica line is always locaed at the point where the slip surface passes from one material to the other.
3. Each slice is acted upon by the following forces:
   - Weight (W) acting through its centre of gravity
   - Cohesive force C acting along the curved surface in the direction opposite to the direction of probable movement of the wedge
   - Reaction (R) at the base inclined at an angle ϕ
   - Reactions on the two vertical sides of the slice due to adjacent slides. However, reaction on the two sides are considered as equal and opposite and hence not considered for the analysis.
   - To compute weight of each wedge, the unit weight is multiplied by its volume. i.e., $W = \gamma bz$
4. The weight is resoled into two components viz., normal and tangential component. The moment dut to N component is zero as those components passthrough O. $Normal\ component = N = Wcosi$

	$Tangential\ component, T = WSin\ i$ $Overturning\ moment = T \times r = WSin\ i \times r$ $Resisting\ moment = (C\Delta L \times r + R \times rSin\emptyset)$--------------(1) Where R is the resultant reaction making an angle ϕ with the line passing through centre of rotation. From the geometry of the triangle, $N = RCos\emptyset$ $R = N/Cos\emptyset$ So equation (1) becomes: $Resisting\ moment = (C\Delta L \times r + N \times rtan\emptyset)$--------------($$Factor\ of\ Safety = \frac{Resisting\ moment}{Overturning\ moment}$$ $$Factor\ of\ Safety = \frac{(C\Delta L \times r + N \times rtan\emptyset)}{T \times r}$$ $$Factor\ of\ Safety = \frac{(C\Delta L + N \times tan\emptyset)}{T \times r}$$ This needs to be summed up for all slices: $$Factor\ of\ Safety = \frac{\left(C\hat{L} + \sum N \times tan\emptyset\right)}{\sum T}$$ For deposits wherein pore water pressure measurements are taken, FoS equation gets modified as: $$Factor\ of\ Safety = \frac{\left(C'\hat{L} + (\sum N - \sum U) \times tan\emptyset'\right)}{\sum T}$$ For purely cohesive, ϕ = o $Factor\ of\ Safety = \frac{C\hat{L}}{\sum T}$ $\hat{L} = r\ \theta$ $where\ \theta\ is\ the\ angle\ subtended\ by\ the\ slip\ circle$	
6c	An embankment is to be constructed with c – 20 kPa and φ = 20°.γ = 18 kN/m^3, FS = 1.25 and height is 10 m. Estimate the side slope required. Taylor's stability number are as follows below the table. Also find the FoS if the slope is 1V : 2H, φ = 20°. <table><tr><td>Slope angle</td><td>90</td><td>75</td><td>60</td><td>45</td><td>30</td><td>30</td><td>0</td></tr><tr><td>Sn</td><td>0.192</td><td>0.143</td><td>0.097</td><td>0,062</td><td>0.025</td><td>0.005</td><td>0</td></tr></table>	' 2 H give n
	$S_n = \frac{c}{F_c \gamma H}$ $S_n = \frac{20}{1.25 \times 18 \times 10} = 0.0888$ Slope = 56.57° 1V: 2H; β = 26.565° For that S_n = 0.01813 $0.01813 = \frac{20}{F \times 18 \times 10}$ F = 6.13	
	Module 4	
7a	Explain the types of shear failures with neat sketches	6
	<table><tr><th>General Shear Failure</th><th>Local/Punching Shear Failure</th></tr></table>	

Occurs in dense/stiff soil Φ>36°, N>30, I_D>70%, C_u>100 kPa	Occurs in loose/soft soil Φ<28°, 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 footing at the surface	No Bulging observed in theneighbourhood 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 failureobserved	Considerable settlement of footingobserved



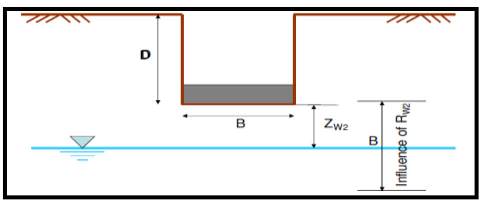
General shear failure

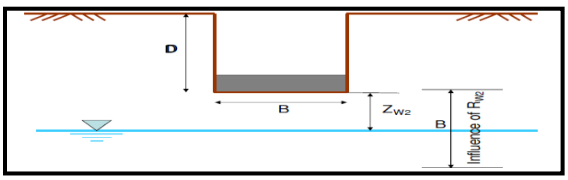


Local shear failure

7b	With the help of neat sketches, explain the effect of water table and eccentric loading on bearing capacity of soil.	8

The position of ground water has a significant effect on the bearing capacity of soil. Presence ofwater 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, $R_{w2} = \frac{1}{2}\left[1 + \frac{Z_{w2}}{B}\right]$

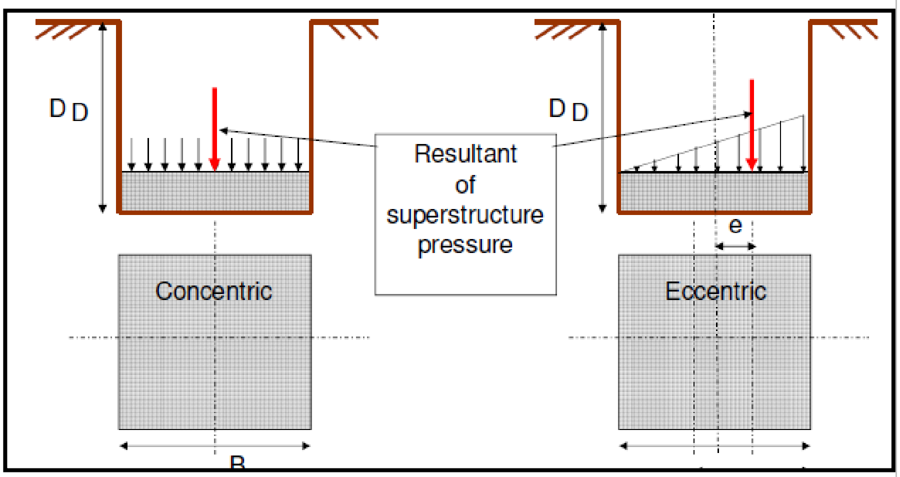
where Z_{W2} is the depth of water table from foundation level.

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 footing on bearing capacity

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 consideredas follows.

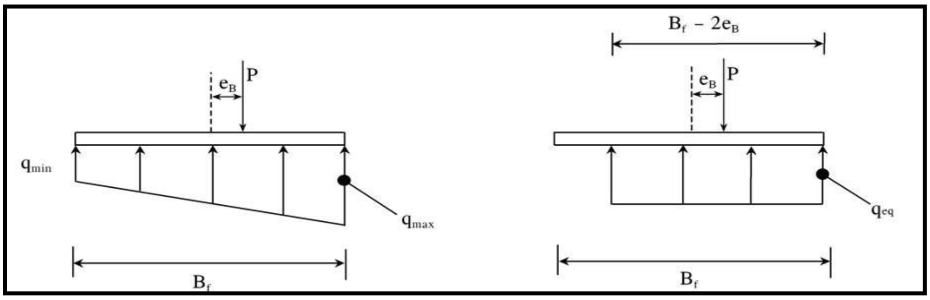
B' = B - 2e



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 actualarea, 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 theeffect 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).

7c	A square footing is to be constructed on a deep deposit of sand at a depth of 0.9 m to carry a design load of 300 kN with a factor of safety of 2.5. The ground water table may rise to the ground level during rainy season. Design the plan dimension of footing given γ_{sat} = 20.8 kN/m^3, N_c = 25, N_q = 34 and N_γ = 32.	6

$$\frac{300}{B^2} = \frac{1.3cN_c + 0.4\gamma BN_r W_\gamma + \gamma D_f(N_q - 1)W_q}{FoS} + \gamma D_f$$

$$\frac{300}{B^2} = \frac{0.4 \times 10.8 \times B \times 32 \times 0.5 + 10.8 \times 0.9 \times (34 - 1) \times 0.5}{2.5} + 18 \times 0.9$$

$$300 = 27.648\,B^3 + 64.152\,B^2 + 16.2B^2$$

For B = 1.55 m;

8a	List the assumptions and limitations made in Terzaghi's analysis.	6

Assumptions in Terzaghi's bearing capacity concept.

1) Soil is homogeneous and Isotropic.

2. The shear strength of soil is represented by Mohr Coulombs Criteria.
3. The footing is of strip footing type with rough base. It is essentially a two dimensional plane strain problem.
4. Elastic zone has straight boundaries inclined at an angle equal to Φ to the horizontal.
5. Failure zone is not extended above, beyond the base of the footing. Shear resistance of soil above the base of footing is neglected.
6. Method of superposition is valid.
7. Passive pressure force has three components (PPC produced by cohesion, PPq produced by surcharge and PPɣ produced by weight of shear zone).
8. Effect of water table is neglected.

9. Footing carries concentric and vertical loads.

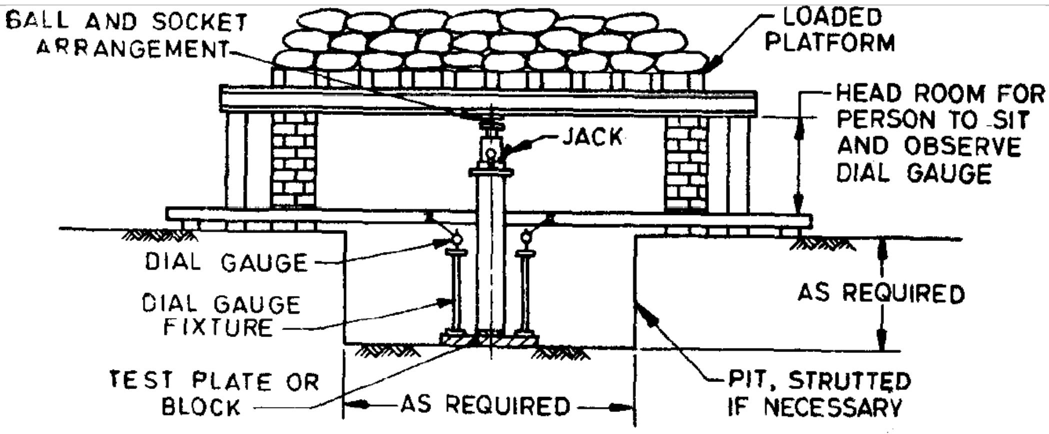
10. Footing and ground are horizontal.

11. Limit equilibrium is reached simultaneously at all points. Complete shear failure is mobilized at all points at the same time.

12. The properties of foundation soil do not change during the shear failure.

The assumptions itself are the limitations of this theory.

8b	With neat sketch, explain plate load test.	6



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 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.

Limitations of plate load test:-

(1) Size effect:- Since the size of the test plate and the foundation are very different, the results of plate load test do not directly reflect the bearing capacity of the foundation.

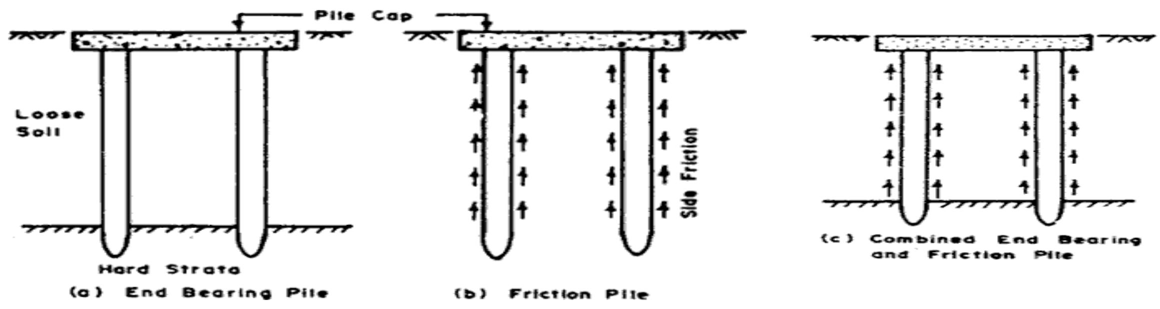
(2) Scale effect:- The ultimate bearing capacity of saturated clays is independent of the size of the plate but for cohesionless soils, it increases with the size of the plate.

(3) Time effect:- A plate load test is essentially for a short duration. For clayey soils it does not give the ultimate settlement.

(4) Interpretation of failure load:- The failure load is not well defined, except in case of general shear failure.

(5) Reaction load:- It is not practical to provide a reaction of more than 250 KN. Hence test on a plate size larger than 0.6m width is difficult.

	(6) Water table:- The level of WT affects bearing capacity of the sandy soils. If WT is above the level of footing, it has to be lowered by pumping before placing the plate.	
8c	A square footing 2.8 m × 2.8 m is built on a homogenous bed of sand of density 18 kN/m^3 and φ = 36°. If depth of foundation is 1.8 m. Determine the safe load on footing. Take F = 2.5, N_c = 27, N_q = 36 and N_γ = 35.	8
	General shear failure $\boldsymbol{q_u} = \mathbf{1.3}c\boldsymbol{N_c} + \mathbf{0.4}\boldsymbol{B\gamma N_\gamma} + \boldsymbol{\gamma D_f N_q}$ $\boldsymbol{q_u} = \mathbf{1.3} \times 0 \times \boldsymbol{N_c} + \mathbf{0.4} \times 2.8 \times 18 \times 35 + 18 \times 1.8 \times 36 = 1872$kPa $q_{nu} = q_u - \gamma D_{f=}$ 1872 – 18 ×1.8 = 1839.6 kPa Safe BC = 1839.6/2.5 Safe BC = 735.84 kPa Safe load = 735.84×2.8×2.8 = 5768.9 kN	
	Module 5	
9a	Explain the classification of piles based on the material and function.	8
	Classification based on materials or composition: 1. Timber piles: Timber piles are made from tree trunks and are well seasoned, straight and free from all defects. Usually available length will be 4 to 6m. Timber piles are used where good bearing stratum is available at a relatively shallow depth. 2. Concrete piles: Concrete piles are either precast or cast in-situ. Precast piles are cast and cured at the casting yard and then transported to the site for installation. These piles are adequately reinforced to withstand handling stresses along with working stress. Precast piles are generally used for short lengths. Cast-in-situ piles are constructed by drilling hole in the ground and then filling that hole with freshly prepared concrete after placing the reinforcement. 3. Steel Piles: Steel piles are usually of rolled H-sections or thick pipe sections. These piles are used to withstand large impact stresses and where fewer disturbances from driving is desired. These piles are also used to support open excavations and to provide seepage barrier. 4. Composite piles: A pile made up of two different materials like concrete and timber or concrete and steel is called composite pile. Composite piles are mainly used where a part of the pile is permanently under water. The part of the pile which will be under water can be made of untreated timber and the other part can be of concrete. 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.	

		
9b	Mention the situations where the pile foundation is necessary.	4
	Necessity of pile foundation:- Pile foundations are used in the following conditions: 1. When the strata at or just beneath the ground surface is greatly compressible and very weak or loose for carrying the load transferred from the structure above. 2. When the plan of the structure is irregular relative to its outline and load distribution. This may lead to the unpleasant non-uniform settlement when the shallow foundation is constructed. 3. For the transmission of different structural loading/forces via deep water to the firm stratum below. 4. To withstand the horizontal forces as well as the vertical forces/loads in the earth retaining structures and tall structure that are subjected to horizontal forces of wind and earthquake. 5. When the soil conditions are in very loose/weak conditions that a washout, erosion and scour of soil underneath leads to the collapse of shallow foundations. 6. For the foundations of some structures, such as transmission towers, off-shore platforms, which are subjected to uplift. 7. In expansive soils like black cotton soil, that swells or shrinks when the water content varies, the piles are used to transfer the load below the active zone.	
9c	In a group of 16 pile diameter is 450 mm and centre to centre spacing of the square group is 1.5 m. If c = 50 kPa, determine whether the failure would occur with the pile acting individually, or as a group? Neglect bearing at the tip of the pile. All piles are 10 m long. Take adhesion factor as 2 and Factor of safety 2.5. also find safe allowable load.	8
	$Q_{ug} = 4.95 \times 4 \times 10 \times 50 = 9900\ kN$ $Q_{uindividual} = 16(\pi \times 0.45 \times 2 \times 10 \times 50) = 22619\ kN$ Capcity of pile is lowest of the two… 9900 kN	
10 a	Write a short notes on : (i) Group capacity of piles (ii) Negative skin friction (iii) Under reamed piles (iv) Settlement of piles	20
	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 Qug = (Cub×Nc×Ab) + (Pb×L×Cu’) Where Cub= undrained strength of clay Nc= Bearing capacity factor, taken as 9.0 Ab= c/s area of the block Pb = perimeter of the block	

The load carrying capacity of the pile group may also be evaluated as
Qu = (Cp×Nc×Ap) + (α×C×As)

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.

Negative skin friction

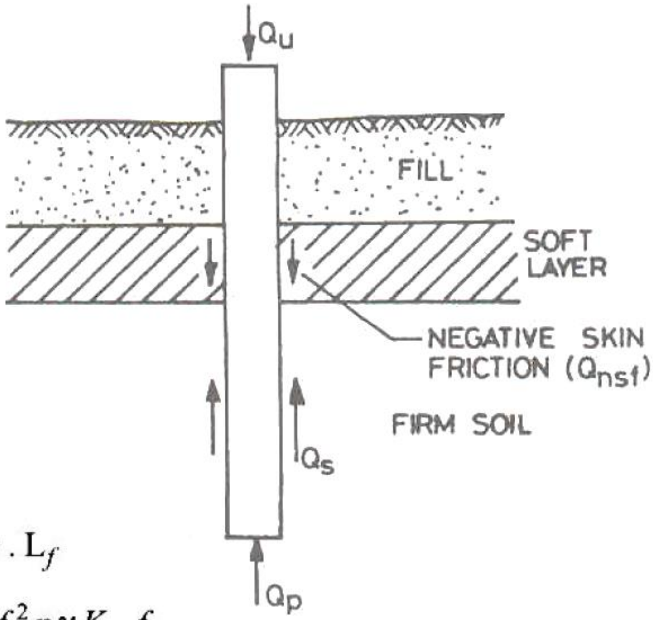
When the soil layer surrounding a portion of the pile shaft settles more than the pile, a downward drag occurs on the pile. The drag is known as negative skin friction.

Negative skin friction develops when a soft or loose soil surrounding the soil settles after the pile has been installed. The negative skin friction occurs in the soil zone which moves downward relative to the pile. The negative friction imposes an extra downward load on the pile.

The net ultimate load -carrying capacity of the pile is given by

$$Q_u' = Q_u - Q_{nf}$$

Q_u' – *net ultimate load*
Q_{nf} – *negative skin friction*

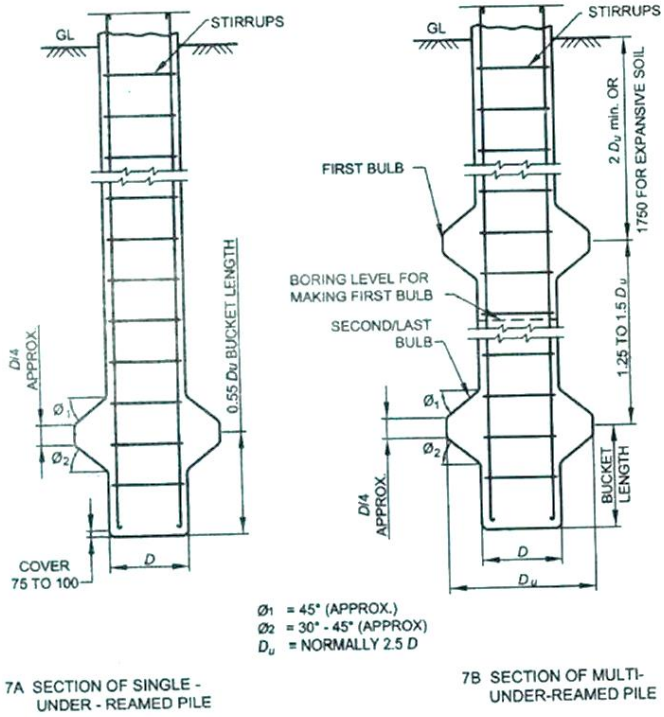


(*i*) for cohesive *soils*. $Q_{nf} = p \cdot c \cdot L_f$

(*ii*) for granular *soils*. $Q_{nf} = \frac{1}{2} Lf^2 p \gamma K \cdot f$

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.
- 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.



Settlement of Pile Groups

Consolidation settlement of Pile group

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

Sc = [Cc * H * log (σo + Δσ)/ σo]/ [1+eo]

Where, Cc = 0.009(wL – 10) for undisturbed soils & = 0.007(wL – 10) for remolded soils eo = initial void ratio, σo = initial overburden pressure at the middle of the clay strata

Δσ = P/ [B + (2*0.5H*tan 30)] 2

Settlement of pile groups in sand:-

Skempton's settlement ratio:- Sg/Si = {[4B + 2.7]/ [B + 3.6]}2

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

Meyerhof's settlement ratio:- Sg/Si = [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.