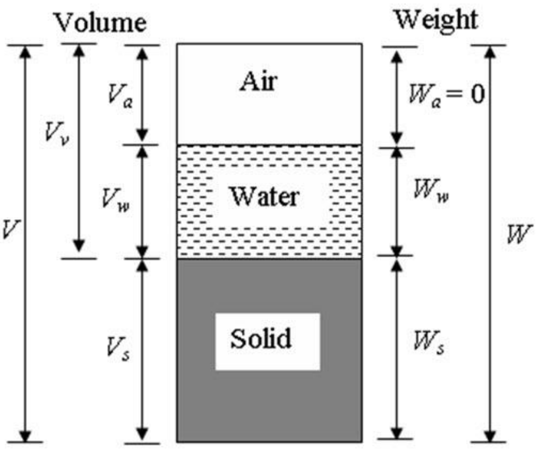
10CV54 – BASIC GEOTECHNICAL ENGINEERING – 1 SOLUTIONS

1 a. Explain three phase system of soil with example. Differentiate between voids ratio and percentage voids.

Soil is particulate material consisting of solid particles (mineral grains, rock fragments) with water and air in the voids between the particles. The water and air contents are readily changed by changes in ambient conditions and location. As the relative proportions of the three phases vary in any soil deposit, it is useful to consider a soil model which will represent these phases distinctly and properly quantify the amount of each phase. A schematic diagram of the three-phase system is shown in terms of weight and volume symbols respectively for soil solids, water, and air. The weight of air can be neglected.

The soil model is given dimensional values for the solid, water and air components.

Total volume, $V = V_s + V_w + V_v$



Void ratio (e) is the ratio of the volume of voids (V_v) to the volume of soil solids (V_s), and is expressed as a decimal.

$$e = \frac{V_v}{V_s}$$

Percentage air voids (n_a) is the ratio of the volume of air to the total volume.

$$n_a = \frac{V_a}{V} x\, 100 = n\ x\ a_c$$

1.b . With usual notation show that wG = e S.

$$w = \frac{W_W}{W_s} = \frac{\gamma_W . V_W}{G_s . \gamma_W . V_s} = \frac{V_W}{G_s . V_s} = \frac{S . V_v}{G_s . V_s} = \frac{S . e}{G_s}$$

1.c A soil in its natural state is fully saturated with a water content of 30% Determine voids ratio, dry unit weight and wet unit weight . Also water content to fully saturate a soil mass of $50m^2$, take G = 2.6.

$$w = \frac{S.e}{G_s}$$

e = .78

$$\gamma_d = \left\lfloor \frac{G\gamma_w}{(1+e)} \right\rfloor$$

14.32kN/m^3

$$\gamma = \frac{(1+w).G_s.\gamma_w}{1+e}$$

25.5 kN/m^3

2a Determine the water content of soil by Pycnometer method. Under what condition it is adopted.

A Pycnometer is a glass jar of about 1 liter capacity, fitted with a brass conical cap by means of a screw type cover. The cap has a small hole of about 6mm diameter at its apex.

The water content (w) of the sample is obtained as

$$w = \left[\frac{M_2 - M_1}{M_3 - M_4} \left(\frac{G-1}{G} \right) - 1 \right] \times 100$$

Where M_1=mass of empty Pycnometer,

M_2= mass of the Pycnometer with wet soil

M_3= mass of the Pycnometer and soil, filled with water,

M_4 = mass of Pycnometer filled with water only.

G= Specific gravity of solids.

Procedure:

1. Wash and clean Pycnometer and dry it.

2. Determine the mass of Pycnometer with brass cap and washer (M_1) accurate to 1.0g.

3. Place about 200 to 400g of wet soil specimen in the Pycnometer wand weigh it with its cap and washer (M_2).

4. Fill water in the Pycnometer containing the wet soil specimen to about half its height.

5. Mix the contents thoroughly with a glass road. Add more water and stir it. Fill the Pycnometer with water, flush with the hole in the conical cap.

6. Dry the Pycnometer from outside and take its mass (M_3).

7. Empty the Pycnometer. Clean it thoroughly. Fill it with water, flush with the hole in the conical cap and weigh (M_4)

This method is adopted when Specific gravity of soil is known.

2b. Discuss the advantages and limitations of sedimentation analysis. Also explain corrections to be applied in Hydrometer.

Sl No	Parameter	Assumptions	Limitations
1	Shape	Shape of the sphere is assumed as spherical	This is not true. All particles are not spherical
2	Medium	Medium is of infinite extent	Its false. Medium is of finite extent
3	Interference	There is no interference between particles	Particle interference will effect settling velocity.
4	Particle size	Is applicable to all particle sizes	Particles greater than 0.2 mm cause turbulence and particles less than 0.002 mm will cause Brownian movement
5	Soil type	Can be used for all soils	Cannot be used for chalky soils because of pre-treatment

Meniscus correction:

Since the soil suspension is opaque, the readings are taken at upper meniscus, though they are to be taken at lower meniscus. Since the hydrometer readings increase downwards, the meniscus correction is to be added to the observed hydrometer reading.

To measure meniscus correction: distilled water is filled in a gas jar and the hydrometer readings are taken at upper and lower meniscus. The difference in this reading gives meniscus correction.

Temperature correction (c_t):

Hydrometer is generally calibrated to a temperature of 27°C. If the temperature is greater than 27°C, the density of suspension decreases. Hence, hydrometer will be going further down. Since the hydrometer reading decreases as we go up, the measured hydrometer reading will be less than the actual hydrometer reading. Hence, correction will be positive.

Similarly, when the temperature is less than 27°C, the density of suspension increases. Hence, hydrometer will not be going down. Since the hydrometer reading increases as we go down, the measured hydrometer reading will be more than the actual hydrometer reading. Hence, correction will be negative.

To measure temperature correction:

At different time periods, along with hydrometer reading, the temperature of the suspension is also measured and its average is determined. If the difference between individual temperature reading and average temperature is not more than 2°C, this correction need not have to be applied. Else using calibration chart provided along with the supply of hydrometer can be used to determine temperature correction.

Dispersing agent correction (c_d):

When dispersing agent is added to distilled water, its density increases. Hence, hydrometer will not be going down. Since the hydrometer reading increases as we go down, the measured hydrometer reading will be more than the actual hydrometer reading. Hence, dispersing agent correction will be negative.

To measure dispersing agent correction:

Hydrometer readings are taken in pure distilled water and in distilled water containing dispersing agent. The difference between the two gives this correction

Composite correction (c_c):

The sum of all the three corrections is called as composite correction.

Therefore corrected hydrometer reading, $R'_h = R_h \pm c_m$

2c.

c. A dry sample of weight 50gms is mixed with distilled water to prepare a suspension of 1000 mℓ for hydrometer analysis. The reading of the hydrometer taken after 5 minutes is 25 and the depth of the centre of the bulb below the water surface when the hydrometer was in the jar was 150mm. The volume of the hydrometer 62mℓ and cross section area of Jar 55 cm^2. Assuming G = 2.68 and $\eta = 1.0 \times 10^{-5}$ g-sec/cm^2. Determine the co-ordinates of the point corresponding to above observation. **(06 Marks)**

$$D = \sqrt{18\,\eta\, v / (G_s - G_w)}$$

$$v = L / t$$

$$A = \sqrt{18\,\eta / (G_s - G_w)}$$

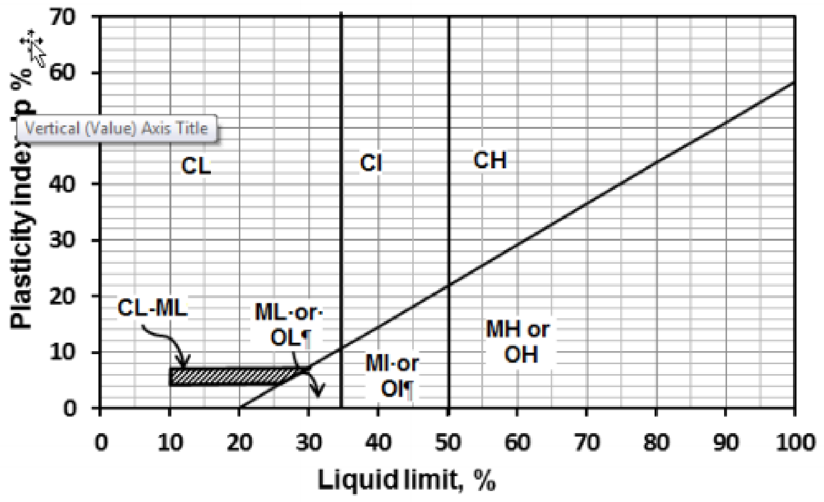
$$\boxed{D = A\sqrt{L\,(cm) / t\,(min)}}$$

D = 0.022mm

3a. Explain with neat sketch plasticity chart and describe its use.

Fine-grained soils are those for which more than 50% of the material has particle size less than 0.075 mm. A plasticity chart is a chart with liquid limit (WL) on X-axis and plasticity index (IP) on Y-axis. According to IS classification, fine grained soils are classified into 9 groups using A-line whose equation is given as IP= 0.73 (WL- 20).

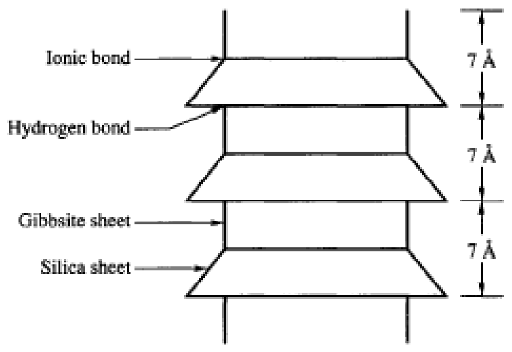
Ip above A-line	Wl<35	CL
Ip above A-line	Wl between 35 and 50	CI
Ip above A-line	Wl>50	CH
Ip below A-line	Wl<35	ML or OL
Ip below A-line	Wl between 35 and 50	MI or OI
Ip below A-line	Wl>50	MH or OH
Ip above A-line	Ip between 4 and 7	CL-ML



3b Explain **with** neat sketches of following minerals.

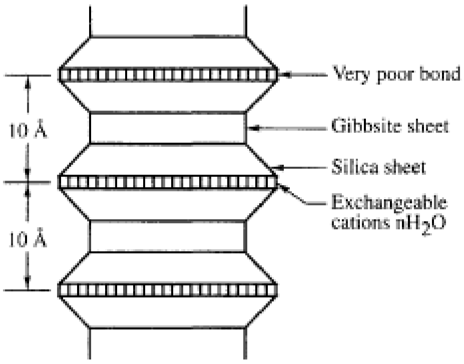
Kaolinite

- The thickness of the layer is about 7 A° (one angstrom = 10~8 cm) thick.
- The kaolinite mineral is formed by stacking silica sheets and gibbsite sheets one above the other.
- The sheets are held to each other by hydrogen bonding.
- The mineral is therefore, stable, and water cannot enter between the sheets to expand the unit cells.
- In the kaolinite mineral there is a very small amount of isomorphous substitution.



Montmorillonite mineral

- The silica and gibbsite sheets are combined in such a way that the tips of the tetrahedrons of each silica sheet and one of the hydroxyl layers of the octahedral sheet form a common layer.
- The thickness of the silica-gibbsite-silica unit is about 10 A°.
- In stacking these combined units one above the other, oxygen layers of each unit are adjacent to oxygen of the neighbouring units with a consequence that there is a very weak bond and an excellent cleavage between them.
- Water can enter between the sheets, causing them to expand significantly and thus the structure can break into 10 A° thick structural units.



- Soils containing a considerable amount of montmorillonite minerals will exhibit high swelling and shrinkage characteristics.
- In montmorillonite, there is isomorphous substitution of magnesium and iron for aluminium.

3c.

c. Following are the results obtained from the tests conducted on two soils A and B. Classify them as per IS classification system. Show the salient steps involved. **(08 Marks)**

Soil	LL	PL	% Retained on IS 75 μm Sieve	% Retained on IS 4.75 mm Sieve	Cu	Cc
A	110	50	40	Zero	-	-
B	-	-	97	05	7	2

Soil A - fine grained soil , PI = 110-50 = 60 , CH

Soil B – 92% . Hence sandy , Cu >7 and well graded soil

4a. State Darcy’s law and derive an expression for coefficient of permeability in falling head permeability test.

Darcy's law states that there is a linear relationship between flow velocity (v) and hydraulic gradient (i) for any given saturated soil under steady laminar flow conditions.

If the rate of flow is q (volume/time) through cross-sectional area (A) of the soil mass, Darcy's Law can be expressed as

v = q/A = k.i where k = permeability of the soil

i = Dh/L

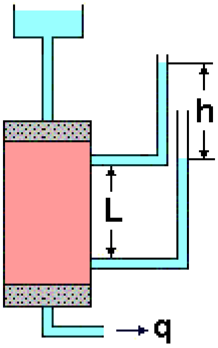
Dh = difference in total heads

L = length of the soil mass

The flow velocity (v) is also called the Darcian velocity or the superficial velocity. It is different from the actual velocity inside the soil pores, which is known as the seepage velocity, vS. At the particulate level, the water follows a tortuous path through the pores. Seepage velocity is always greater than the superficial velocity, and it is expressed as:

where AV = Area of voids on a cross section normal to the direction of flow

n = porosity of the soil

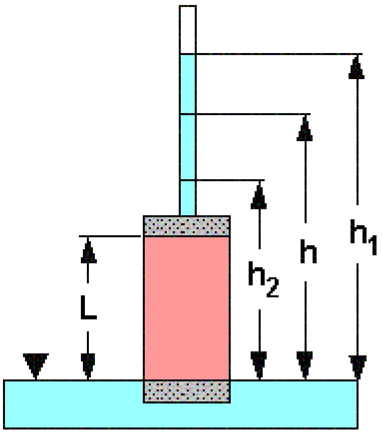


Permeability **k** is obtained from:

$$k = \frac{qL}{Ah}$$

Falling Head Flow

Falling head permeameter is recommended for fine-grained soils.



Total head **h** in standpipe of area **a** is allowed to fall. Hydraulic gradient varies with time.
Heads **h1** and **h2** are measured at times **t1** and **t2**. At any time **t**, flow through the soil sample of cross-sectional area **A** is

$$q = k.h.\frac{A}{L}$$ --------------------- (1)

Flow in unit time through the standpipe of cross-sectional area **a** is

$$= a \times \left(-\frac{dh}{dt}\right)$$ ----------------- (2)

Equating **(1)** and **(2)** ,

$$-a.\frac{dh}{dt} = k.h\frac{A}{L}$$

or $$-\frac{dh}{h} = \left(\frac{kA}{La}\right)dt$$

Integrating between the limits,

$$\log_e\left(\frac{h_1}{h_2}\right) = \frac{k.A}{L.a}(t_2 - t_1)$$

$$k = \frac{L.a.\log_e\left(\frac{h_1}{h_2}\right)}{A(t_2 - t_1)}$$

$$= \frac{2.3L.a\log_{10}\left(\frac{h_1}{h_2}\right)}{A(t_2 - t_1)}$$

4b. Explain the factors affecting permeability of soil.

1. Grain-size

2. Void ratio

3. Composition

4. Fabric or structural arrangement of particles

5. Degree of saturation

6. Presence of entrapped air and other foreign matter.

Permeability varies with the square of particle diameter. It is logical that the smaller the grain-size, the smaller the voids and the lower the permeability. A relationship between permeability and grain-size is more appropriate in case of sands and silts than that in case of other soils since the grains are more equidimensional and its fabric changes are not significant.

Influence of soil composition on permeability is generally of little significance in case of gravels, sands, and silts, unless mica and organic matters are present. However, this is of major importance in the case of clays. Montmorillonite has the least permeability with sodium as the exchangeable ion (less than 10–7 cm/s, even at a very high void ratio of 15.

Fabric or structural arrangement of particles is an important soil characteristic influencing permeability, especially of fine-grained soils. At the same void ratio, it is logical to expect that a soil in the most flocculated state will have the highest permeability and the one in the most dispersed state will have the lowest permeability.

Higher the degree of saturation, higher the permeability. In case of certain sands permeability may increase three-fold when degree of saturation increases from 80% to 100%.

Entrapped air has pronounced effect on permeability. It reduces the permeability of soil. Organic foreign matter also has the tendency to move towards flow channels and choke them, thus decreasing the permeability.

4c

c. A sample of soil for constant head permeability test yielded the following data :
i) Diameter of sample = 7.6cm.
ii) Length of sample = 20cm.
iii) Head causing the flow = 15cm.
iv) Quantity of water collected in 10min = 150 CC.

Assume G = 2.65 , γ_d = 18kN/m^3.
Determine : i) Co-efficient of Permeability ii) Discharge velocity iii) Seepage Velocity. **(06 Marks)**

Permeability **k** is obtained from:

$$k = \frac{qL}{Ah}$$

i) Co efficient of permeability 0.44cm/min

ii) Discharge velocity 5.5 X 10^{-3}cm/sec

iii) Seepage velocity .018cm/sec

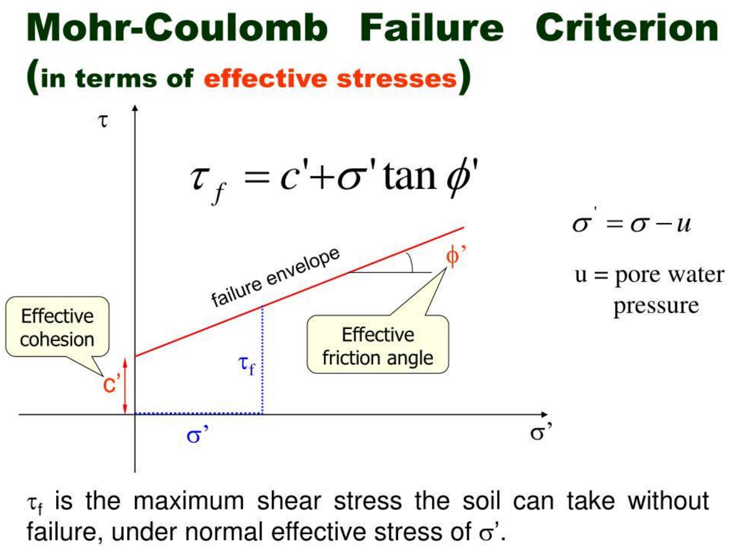
PART B

5a Explain Mohr coulomb failure theory. Sketch Mohr Coulomb envelope for pure clay and sand.

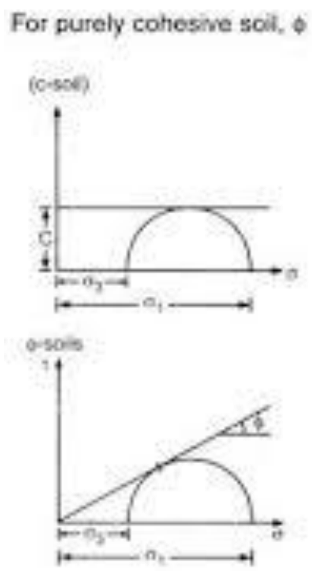
According to Mohr, shear strength is a function of normal stress.

S= f(σ).

The assumed function is curvilinear. This was later modified by Coulomb and was simplified as a linear equation given by Mohr-Coulomb equation. Accordingly, shear stresss is a function of normal stress and angle of internal friction. On including the effective shear parameters, the total stress parameters gets changed to effective shear parameters as shown below.



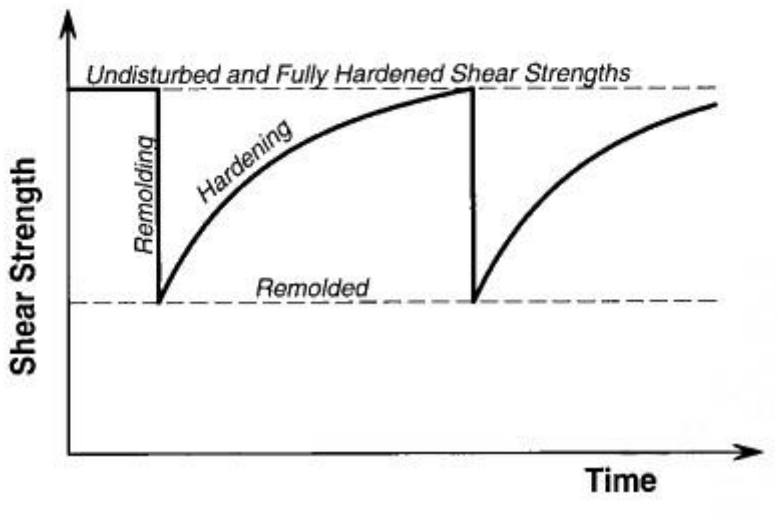
Mohr Coulomb envelope for pure clay and sand



5b. Explain Thixotropy, sensitivity of clay , effective, neutral and total stress in soils.

i) Thixotropy of clay

Clays with flocculent structure loses strength due to disturbance or remoulding. Loss of strength is partly due to permanent destruction of structure and reorientation of molecules in adsorbed layer. Strength loss with destruction of structure can't recovered with time. However, remoulded soil left undisturbed at same water content, regain part of strength due to gradual reorientation of adsorbed molecules of water. This phenomenon of strength loss-strength gain, with no change in volume or water content, is called 'Thixotropy'. This may also be said to be "a process of softening caused by remoulding, followed by a time-dependent return to the original harder state". Higher the sensitivity, larger thixotropic hardening. Extent of strength gain depends on type of the clay mineral. Mineral that absorb large quantity of water in lattice structure, such as Montmorillonite has greater thixotropic gain compared to other stable clay minerals. Figure.1. shows the gain in strength of soil due to thixotropic effect. Thixotropy has important applications in connection with pile-driving operations. The immediate frictional strength of thixotropic clay in driven piles is less compared to frictional strength after one month, because strength gain with passage of time.



ii) Sensitivity of clay

Sensitivity is the measure of loss of strength with remoulding. Sensitivity, S_t is defined as the ratio of unconfined compressive strength of clay in undisturbed state to unconfined compressive strength of a same clay in remoulded state at unaltered water content.

Sensitivity	**Classification**
< 1	Insensitive
1-2	Slightly sensitive
2-4	Medium sensitive
4-8	Very sensitive
8-16	Slightly quick
16-32	Medium quick
32-64	Very quick
>64	Extra quick

(i) Total stress (ii) Neutral stress (iii) Effective stress

Consider a soil column of height h having a cross section area A. let the soil has a unit weight γ. The pressure exerted by the soil increases with depth. There are three types of stresses the total stress, natural stress and effective stress.

At point (A) in Fig.(1), $\boldsymbol{\sigma_T = \gamma h}$

If the soil is saturated, the unit weight of soil is γ_{sat}.

In that case, total stress is $\sigma_T = \gamma_{sat}.h$

The upward pressure exerted by water is given by

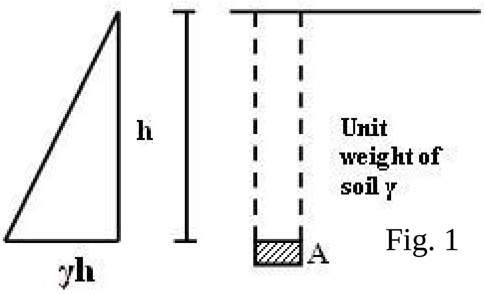
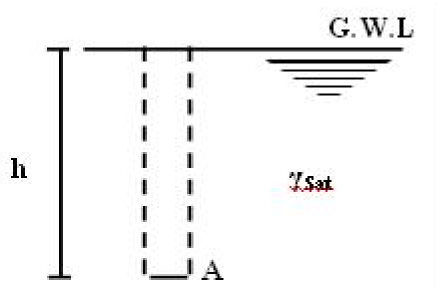


Fig. 1

neutral pressure.

Neutral pressure , $u = \gamma_{w}.h$



Effective stress is the net pressure exerted by the soil solids at any depth.

$\sigma' = \sigma_T - u$

$\sigma' = \gamma_{sat.}h - \gamma_{w.}h = (\gamma_{sat.} - \gamma_{w.})\,h = \gamma_{sub}.h$

5c.

ii) Total, neutral and effective stresses in soils.

c. Two identical specimens 4cm diameter and 8cm height of partly saturated compacted soil are tested in a triaxial cell under undrained conditions. The first specimen failed at deviator load of 720N under a cell pressure of 100kN/m². Second specimen failed at deviator load of 915N under a cell pressure of 200kN/m². The increase in the volume of first specimen at failure is 1.2mℓ and shortens by 0.6cm. The increase in the volume of second specimen at failure is 1.6mℓ and shortens by 0.8cm. Determine apparent cohesion and angle of shearing resistance by analytical method. **(08 Marks)**

Initial area $A_1 = 12.57 cm^2$

Initial Volume $V_1 = 100.56\ cm^3$

First specimen $\Delta L = 0.6$ cm, $\Delta V = 1.2\ cm^3$, Second specimen $\Delta L = 0.8$ cm, $\Delta V = 1.6\ cm^3$

Area at failure = $\frac{V + \Delta 1}{L - \Delta L}{}_1$ $13.75\ cm^2$, $14.2\ cm^2$

$\sigma_{D\,=}$ $524 kN/m^2$, $644\ kN/m^2$, $\sigma_{1\,=}$ $624 kN/m^2$, $844\ kN/m^2$

$624 = 100N_{\varphi} + 2c_u\sqrt{\ } N_{\varphi}$, $844 = 200N_{\varphi} + 2c_u\sqrt{\ } N_{\varphi}$

$c_u = 13.6\ kN/m^2$

$\varphi = 22^0$

6a Determine compaction energy imported to soil during Light and Heavy compaction.

Sl No	**Description**	**Standard compaction**	**Modified compaction**
1	No of layers (N_L)	3	5
2	No of blows(N_B)	25	25
3	Mass of rammer, kg (M)	2.6	4.89
4	Height of rammer fall, mm (H)	310	450
5	Compaction energy =	593	2699

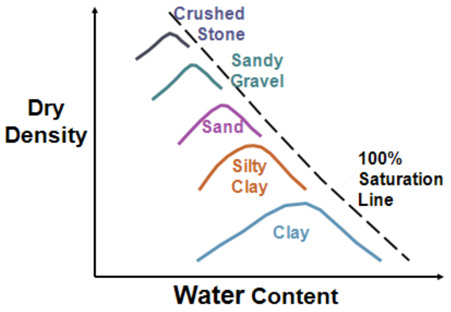
	$\dfrac{MHN_L N_B}{Volume\ \ of\ \ mould}$ (kNm/m^3)		

6b. What are objectives of compaction and also discuss the factors affecting compaction?

The factors which affect the degree of compaction are given below.

1. Type of soil

Normally, heavy clays, clays & silts offer higher resistance to compaction where as sandy soils and coarse grained or gravelly soils are amenable for easy compaction. The coarse grained soils yield higher densities in comparison to clays. A well graded soil can be compacted to higher density.



2. Compactive effort / compactive energy

The term compactive effort or compactive energy means type of equipment or machinery used for compaction. Greater the compactive effort, greater will be the compaction energy, greater will be the extent of compaction. The equipment's used for compaction of soil can be broadly classified into the following categories

3. Layer thickness / thickness of lift

Degree of compaction is inversely proportional to the layer thickness, i.e. for a given compactive energy, thicker layer will be less compacted as compared to thin layer. Generally 200 to 300 mm layer thickness is used in the field to achieve homogeneous compaction.

4. Number of roller passes

It is obvious that density increases as the no of roller passes increases. But after certain number of roller passes, there is no further increase in density. So it is very crucial to determine the number of roller passes for a given type of equipment, for a given type of soil at optimum moisture content.

5. Moisture content

Proper control of moisture content in soil is necessary for achieving desired density. Maximum density with minimum compacting effort can be achieved by compaction of soil near its OMC (optimum moisture content). Relative compaction is the ratio of field dry density to laboratory dry density.

6. Contact pressure

Contact pressure depends on the weight of the roller wheel and the contact area. In case of pneumatic roller, the tyre inflation pressure also determines the contact pressure in addition to wheel load. A higher contact pressure increases the dry density and lowers the optimum moisture content.

7. Speed of rolling

The greater the speed of rolling, the more length of embankment can be compacted in one day. But at greater speed there is likely to be insufficient time for the desired deformations to take place and more passes may be required to achieve the required compaction.

7. Admixtures

Addition of admixtures like flyash, granulated blast furnace slag, cement, lime, gypsum when added to problematic soils, improve its compaction characteristics.

6c

c. Following are the results obtained from a standard compaction test :

Water content, W(%)	13.5	20.2	25	35	45
Bulk unit weight, γ_b kN/m³	16.3	19.4	18.8	18	17.2

Plot compaction curve and obtain maximum dry unit weight and OMC. Also plot 100% saturation line. Show specimen calculation. G = 2.65. **(10 Marks)**

10

Water content, %	13.5	20.2	25	35	45
Density of wet soil	16.3	19.4	18.8	18	17.2
Dry density	14.3	16.13	15.04	13.33	11.86

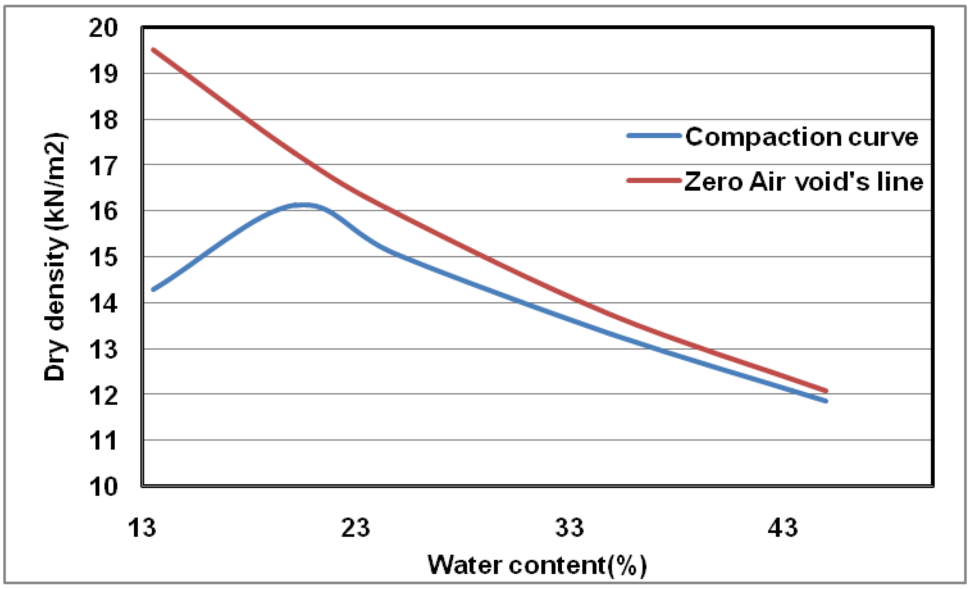
$$\gamma_d = \frac{\gamma_w G}{1+e}$$

$$14.3 = \frac{10\times 2.65}{1+e}$$

e=0.85

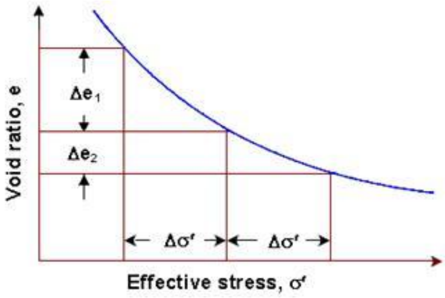
eS=wG; $0.85 \times S = 0.163 \times 2.65$

Or S=50.8%



7a Define compression Index, coefficient of compressibility and coefficient of volume compressibility

The compressibility of soils under one-dimensional compression can be described from the decrease in the volume of voids with the increase of effective stress. This relation of void ratio and effective stress can be depicted either as an **arithmetic plot** or a **semi-log plot.**



In the arithmetic plot as shown, as the soil compresses, for the same increase of effective stress **Ds',** the void ratio reduces by a smaller magnitude, from **De$_1$** to **De$_2$**. This is on account of an increasingly denser packing of the soil particles as the pore water is forced out. In fine soils, a much longer time is required for the pore water to escape, as compared to coarse soils.

It can be said that the compressibility of a soil decreases as the effective stress increases. This can be represented by the slope of the void ratio – effective stress relation, which is called the **coefficient of compressibility, a_v**.

$$a_v = -\frac{de}{d\sigma'}$$

For a small range of effective stress, $a_v = -\dfrac{\Delta e}{\Delta \sigma'}$

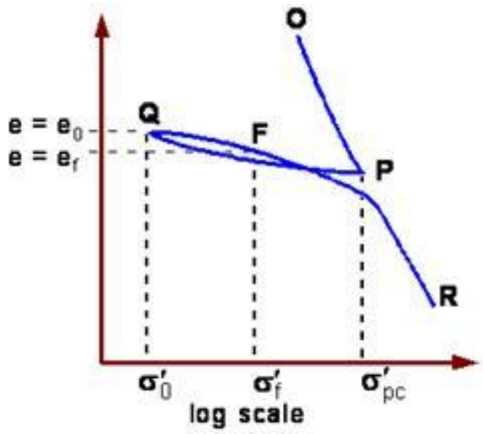
The -ve sign is introduced to make a_v a positive parameter.

If **e$_0$** is the initial void ratio of the consolidating layer, another useful parameter is the **coefficient of volume compressibility**, m_v, which is expressed as

$$m_v = \frac{a_v}{1+e_0}$$

It represents the compression of the soil, per unit original thickness, due to a unit increase of pressure.

The figure shows the relation of void ratio and effective stress of a clay soil as a semi-log plot.



OP corresponds to initial loading of the soil. PQ corresponds to unloading of the soil. QFR corresponds to a reloading of the soil. Upon reloading beyond P, the soil continues along the path that it would have followed if loaded from O to R continuously.

The preconsolidation stress, s'pc, is defined to be the maximum effective stress experienced by the soil. This stress is identified in comparison with the effective stress in its present state. For soil at state Q or F, this would correspond to the effective stress at point P.

If the current effective stress, s', is equal (note that it cannot be greater than) to the preconsolidation stress, then the deposit is said to be normally consolidated (NC). If the current effective stress is less than the preconsolidation stress, then the soil is said to be over-consolidated (OC).

It may be seen that for the same increase in effective stress, the change in void ratio is much less for an overconsolidated soil (from e0 to ef), than it would have been for a normally consolidated soil as in path OP. In unloading, the soil swells but the increase in volume is much less than the initial decrease in volume for the same stress difference.

The distance from the normal consolidation line has an important influence on soil behaviour. This is described numerically by the overconsolidation ratio (OCR), which is defined as the ratio of the preconsolidation stress to the current effective stress.

$$OCR = \frac{\sigma'_{pc}}{\sigma'}$$

Note that when the soil is normally consolidated, OCR = 1. Settlements will generally be much smaller for structures built on overconsolidated soils. Most soils are overconsolidated to some degree. This can be due to shrinking and swelling of the soil on drying and rewetting, changes in ground water levels, and unloading due to erosion of overlying strata.

For NC clays, the plot of void ratio versus log of effective stress can be approximated to a straight line, and the slope of this line is indicated by a parameter termed as compression index, Cc.

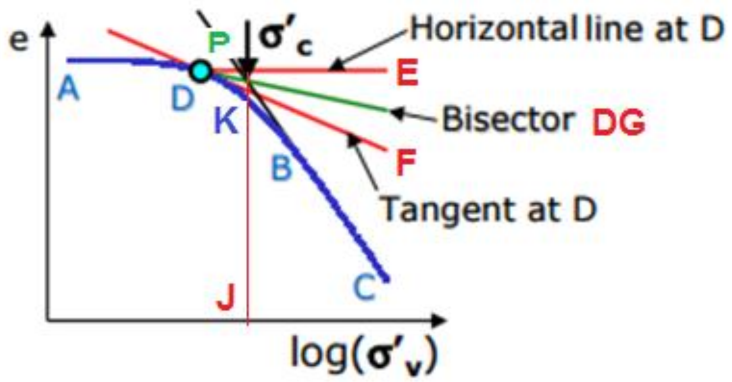
$$C_c = \frac{\Delta e}{\log_{10}\left(\frac{\sigma'_2}{\sigma'_1}\right)}$$

7b Explain casagrande’s method of obtaining preconsolidation pressure.

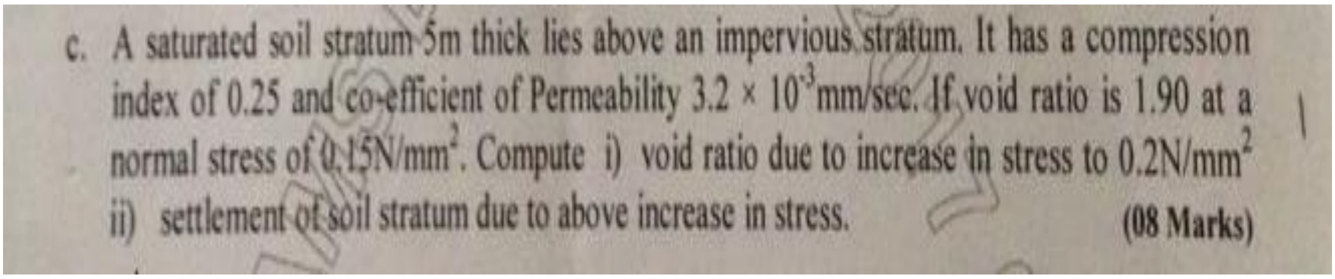
The maximum pressure to which the soil has been subjected to it, in the past is called as preconsolidation pressure. Casagrande’s method for estimating preconsoldiation pressure is as explained below:

1. Choose by eye the point of maximum curvature on the consolidation curve Say D.
2. Draw a horizontal line from this point, line DE.
3. Draw a line tangent to the curve at the point D, line DF.
4. Bisect the angle made from the horizontal line DE and the tangent line DF. Name the bisector as DG.
5. Extend the "straight portion" of the virgin compression curve (high effective stress, low void ratio: almost vertical on the right of the graph) up to the bisector line DG so as to intersect at P.
6. Drop vertical PJ and the abscissa of PJ indicate pre consolidation pressure.

7. Vertical PJ intersect e-log $\bar{\sigma}$ curve at K, Curve ADK indicates recompression curve and curve KBC indicate virgin compression curve.



7c


c. A saturated soil stratum 5m thick lies above an impervious stratum. It has a compression index of 0.25 and co-efficient of Permeability 3.2×10^{-3}mm/sec. If void ratio is 1.90 at a normal stress of 0.15N/mm^2. Compute i) void ratio due to increase in stress to 0.2N/mm^2 ii) settlement of soil stratum due to above increase in stress. (08 Marks)

Change in void's ratio

$$\Delta e = Cc \log_{10}\frac{\sigma_1}{\sigma_2} = 0.28 \log_{10}\frac{0.2_1}{0.15} = 0.035$$

Settlement of soil stratum due to above increase in stress

$$\Delta H = 5 \text{ X } \frac{0.035}{1+1.9} = 6.03\text{cm}$$

8a List merits and demerits of Triaxial over direct shear test.

The first stage simulates in the laboratory the in-situ condition that soil at different depths is subjected to different effective stresses. Consolidation will occur if the pore water pressure which develops upon application of confining pressure is allowed to dissipate. Otherwise the effective stress on the soil is the confining pressure (or total stress) minus the pore water pressure which exists in the soil.

During the shearing process, the soil sample experiences axial strain, and either volume change or development of pore water pressure occurs. The magnitude of shear stress acting on different planes in the soil sample is different. When at some strain the sample fails, this limiting shear stress on the failure plane is called the shear strength.

The triaxial test has many **advantages** over the direct shear test:

• The soil samples are subjected to uniform stresses and strains.

• Different combinations of confining and axial stresses can be applied.

• Drained and undrained tests can be carried out.

• Pore water pressures can be measured in undrained tests.

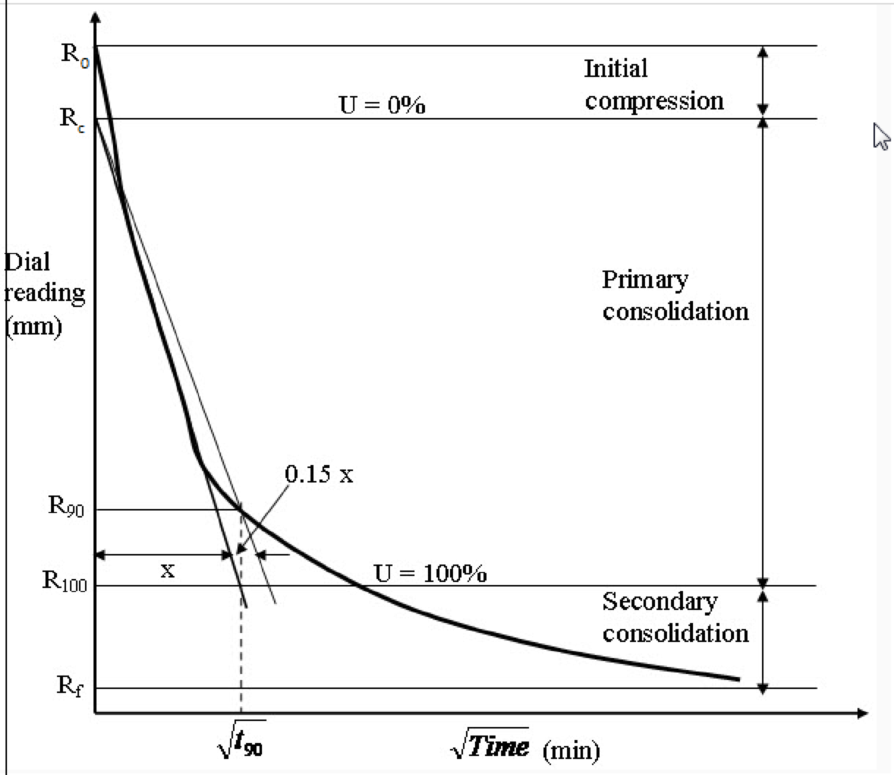
• The complete stress-strain behaviour can be determined.

8b Explain the coefficient of consolidation by square time fitting method.

1. From the oedometer test the dial reading (settlement) corresponding to a particular time is measured. From the measured data, dial reading vs square root (time) graph is drawn.
2. A straight line can be drawn passing through the points on initial straight portion of the curve (as shown in Figure). The intersection point between the straight line and the dial reading axis is denoted as Rc which is corrected zero reading i.e U = 0%. Starting from Rc, draw another straight line such that its abscissa is 1.15 times the abscissa of first straight line.
3. The intersection point between the second straight line and experimental curve represents the R90 and corresponding time is determined and noted as $\sqrt{t_{90}}$. Thus, the time required (t90) for 90% consolidation is calculated.
4. The Coefficient of consolidation (cv) is determined as:

$$c_v = \frac{0.848d^2}{t_{90}}$$

$$where\ d\ is\ the\ drainage\ path = d\ for\ single\ face\ drainage\ and$$
$$= \frac{d}{2} for\ two\ face\ drainage$$



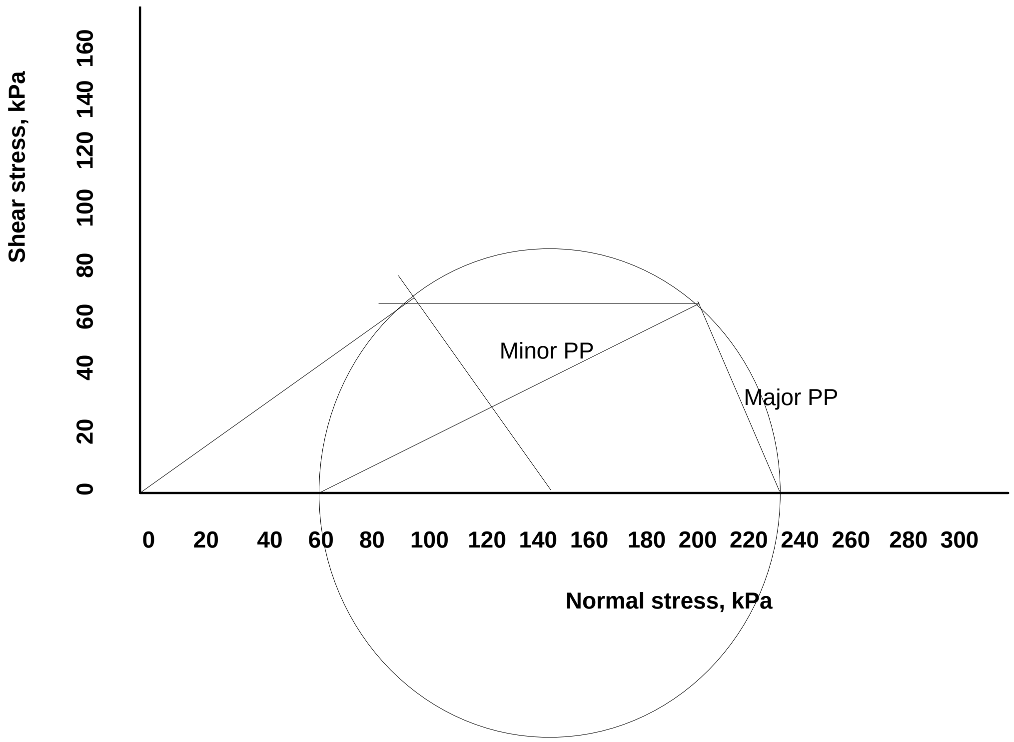
8c

c. In a direct shear test on a specimen of clean dry sand a normal stress of $200kN/m^2$ was applied and failure occurred at a shear stress of $140kN/m^2$. Determine i) Angle of shearing resistance ii) Principal stresses during failure iii) Direction of principal planes with respect to plane to shearing.
Draw a neat sketch of Mohr circle showing the directions of Major and Minor principal planes with reference to shearing.
(08 Marks)

Phi= 35.0°

σ_3 = $50kN/m^2$

σ_1 = $100kN/m^2$


Shear stress, kPa
0
20
40
60
80
100
120
140
160
Minor PP
Major PP
0
20
40
60
80
100
120
140
160
180
200
220
240
260
280
300
Normal stress, kPa