Basic Geo Technical Engineering VTU Q Paper Jan/Feb-2023

Question paper and solution

CBCS SCHEME

18CV54

USN

Fifth Semester B.E. Degree Examination, Jan./Feb. 2023

Basic Geotechnical Engineering

Time: 3 hrs. Max. Marks: 100

Note: Answer any FIVE full questions, choosing ONE full question from each module.

Module-1

1 a. With the help of three phase diagram, define the terms, water content, bulk density, dry density, void ratio, air content, relative density. **(08 Marks)**

b. With usual notations prove that $\gamma_d = \frac{(1-\eta_a)G\gamma_\omega}{1+\omega G}$. **(06 Marks)**

c. The total weight (unit weight) of the glacial outwash soil is 16 kN/m^3. The specific gravity of soil particles of the soil is 2.67. The water content of the soil is 17%. Calculate Dry unit weight, porosity, void ratio and degree of saturation. **(06 Marks)**

OR

2 a. What is consistency of soil? Define liquid limit, shrinkage limit, relative consistency and shrinkage ratio. **(08 Marks)**

b. Explain soil classification according to BIS classification system. **(06 Marks)**

c. Draw the grain size distribution curve and determine the uniformity coefficient and coefficient of curvature of the soil for the following data :

Sieve size (mm)	2.4	1.2	0.6	0.3	0.15	0.075	Pan
Mass of Soil retained (g)	0	5	25	215	225	25	0.5

(06 Marks)

Module-2

3 a. What are the different types of clay minerals commonly found in soils? Explain with their structure. **(08 Marks)**

b. Explain soil structure, electrical diffuse double layer and base exchange capacity. **(06 Marks)**

c. Explain factors affecting compaction. **(06 Marks)**

OR

4 a. Differentiate between standard and modified proctor tests. **(04 Marks)**

b. Discuss the effect of compaction on different properties of soil. **(08 Marks)**

c. The observations of a standard proctor test are given below:

Dry density kN/m^3	16.16	17.06	18.61	18.95	18.78	17.13
Water content (%)	9.02	8.81	11.25	13.05	14.40	19.25

(i) Plot the compaction curve and determine OMC.

(ii) Also compute the void ratio and degree of saturation at optimum condition. Take G = 2.77. **(08 Marks)**

Module-3

5 a. Discuss various factor affecting permeability of soils. **(06 Marks)**

b. Explain quick sand and capillary phenomenon. **(06 Marks)**

c. In a falling head permeability test, head causing flow was initially 500 mm and it drops 20 mm in 5 minutes. Calculate the time required for the head to fall to 250 mm. **(08 Marks)**

2. Any revealing of identification, appeal to evaluator and /or equations written eg, 42+8 = 50, will be treated as malpractice.

1 of 2

OR

6 a. What is flow net? Give its characteristics. (06 Marks)
b. Explain the method of locating the phreatic line in a homogeneous earth dam with filter. (08 Marks)
c. Explain effective stress, total stress, neutral stress in soil. What is the significance of effective stress? (06 Marks)

Module-4

7 a. Explain Mohr Columb failure theory of soil. (04 Marks)
b. What are the factors affecting the shear strength of soil? (08 Marks)
c. The stresses on a failure plane in a drained test on a Cohesionless soil are as under :
Normal stress (σ) = 100 kN/m^2
Shear stress (τ) = 40 kN/m^2
Determine the angle of shearing resistance and the angle which the failure plane makes with the major principal stresses. (08 Marks)

OR

8 a. Classify the shear tests based on drainage conditions. How are these drainage condition, realized in the field. (06 Marks)
b. What are the advantages and disadvantages of direct shear test over triaxial test? (06 Marks)
c. A shear vane of 75 mm diameter and 110 mm length was used to measure the shear strength of a soft clay. If a torque of 600 N-m was required to shear the soil. Calculate the shear strength, the vane was then rotated rapidly to cause remoulding of the soil, the torque required in the remoulded state was 200 N-m. Determine the sensitivity of the soil. (08 Marks)

Module-5

9 a. Differentiate compaction from consolidation. (06 Marks)
b. Explain mass spring analogy. (06 Marks)
c. Explain the significance of pre consolidation pressure. Describe the Casagrande method of determining it. (08 Marks)

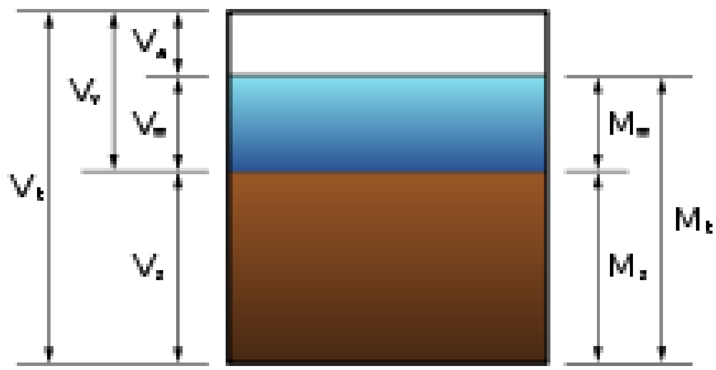
OR

10 a. Explain Pre-consolidated normally consolidated and under consolidated soil. (06 Marks)
b. Explain curve fitting methods used in consolidation test? Explain any one with neat sketches. (08 Marks)
c. A bed of compressible clay 4 m thick has pervious sand on the top and impervious rock at the bottom. In a consolidation test on an undisturbed sample of clay from this deposit, 90% settlement was reached in 4 hours, the sample was 20 mm thick. Estimate the time in years for the building founded over this deposit to reach 90% of its final settlement. (06 Marks)

* * * * *

1. **A. With the help of Phase diagram of soil, define the terms 1. Water content 2. Bulk Density 3. Dry Density 4. Void Ratio. 5. Air content. 6. Relative Density**

Answer: **Soil structure & three phase system**



Void ratio (e) is the ratio of the volume of voids (Vv) to the volume of soil solids (Vs), and it is given by

$$e = \frac{V_V}{V_S}$$

Soil bulk density is the mass of dry soil per unit of bulk volume, including the air space.

The water content (w) of the soil is defined as the ratio of weight of water to the weight of solids present in soil sample.

$$w = \{W_w / W_s\}*100$$

Dry density is the weight of soil solids per unit of total volume of soil mass.

$$\gamma d = Ws / V$$

Air Content is defined as the ratio of volume of air to the volume of voids

$$ac = Va/Vv$$

Density Index (or relative density) of a soil, *ID (Dr)indicates the* relative compactness of the soil mass. This is used in relation to coarse-grained soils or sands.

$$I_D = \frac{(e_{\max} - e_0)}{(e_{\max} - e_{\min})}$$

1.B. With the usual notations prove that $\gamma d = G.(1-na)/1+wG$

Relation between γ_d, [illegible]

2.1.

$$V = V_a + V_w + V_s \quad \text{or} \quad V = V_a + \frac{W_w}{\gamma_w} + \frac{W_d}{\gamma_s}$$

$$1 = \frac{V_a}{V} + \frac{w\,W_d}{\gamma_w\,V} + \frac{W_d}{\gamma_s\,V} = \frac{V_a}{V} + \frac{w\,\gamma_d}{\gamma_w} + \frac{\gamma_d}{\gamma_s}$$

$$\left(1 - \frac{V_a}{V}\right) = \frac{w\,\gamma_d}{\gamma_w} + \frac{\gamma_d}{G\,\gamma_w} \quad \text{or} \quad (1 - n_a) = \frac{\gamma_d}{\gamma_w}\left(w + \frac{1}{G}\right)$$

$$\gamma_d = \frac{(1 - n_a)\,\gamma_w}{w + \frac{1}{G}} \quad \text{or} \quad \gamma_d = \frac{(1 - n_a)\,G\,\gamma_w}{1 + w\,G}$$

1.C.

Given data :

$\gamma = 16kN/m^3$

$w = 17\%$

$G = 2.67$

Solution:

$$\gamma_d = \frac{\gamma}{1+w} \Rightarrow \gamma_d = \frac{16}{1+0.17}$$

$$\boxed{\gamma_d = 13.675kN/m^3}$$

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

$$e = \frac{2.67 \times 9.81}{13.675} - 1$$

$$\boxed{e = 0.9154}$$

$$n = \frac{e}{1+e} \Rightarrow n = \frac{0.9154}{1+0.9154}$$

$$\boxed{n = 0.4779} \; n = 47.79\%$$

$$S = \frac{wG}{e} \Rightarrow S = \frac{0.17 \times 2.67}{0.9154}$$

$$= 0.4958$$

$$\boxed{S = 49.58\%}$$

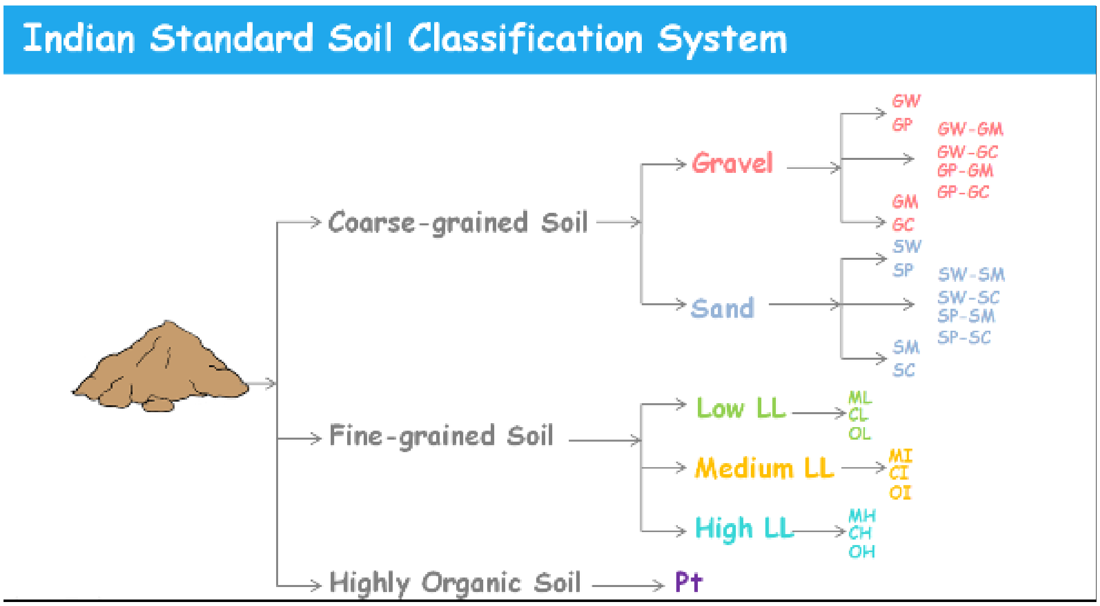
2. **A. The liquid limit is the water content at the transition of liquid state to a plastic state, whereby it gains a certain small shearing strength. It is the minimum water content at which the soil is at the liquid state or the maximum water content at which the soil is in a plastic state.**

Consistency index or relative consistency is the liquid limit of the soil, minus the natural moisture content, divided by the PI. It is related to the LI and is an indicator of the relative shear strength. As CI increases, the firmness, or shear strength of the soil also increases.

Shrinkage Ratio as SR and it is defined as the ratio of the volume change of the soil above shrinkage limit, which we express in the percentage of the soil's minimum volume that is when it was at shrinkage limit, to the corresponding change in water content.

$$SR = \frac{\dfrac{V_1 - V_2}{V_{Shrinkage}} \times 100}{W_1 - W_2}$$

2. B.



Soil particles of size greater than 300 mm are called boulders

particles between 80 mm and 300 mm are called cobbles.

Gravel is defined between 80 mm to 4.75 mm

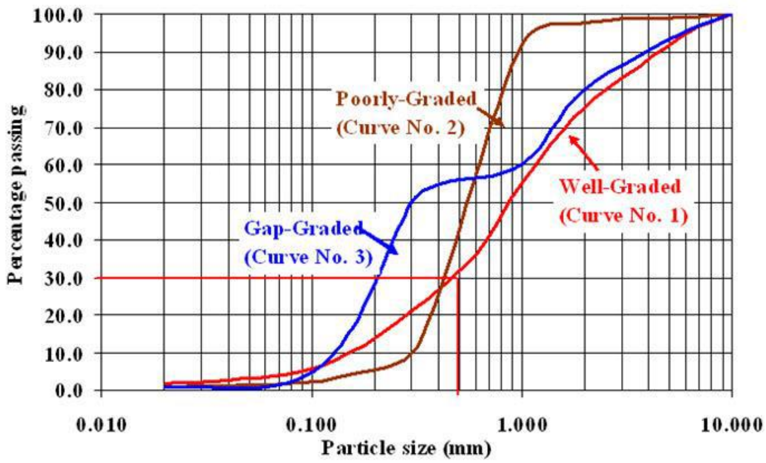
Sand ranges from 0.075 mm to 4.75 mm

Silts particles have size between 0.075 mm and 0.002 mm

And clay particles are of size below 0.002mm or 2 micron.

2.c.

A soil is said to be "well graded", if it contains a good representation of various grain-sizes. This is represented by Curve No 1 in the graphical representation below. If the soil contains grains of mostly one size, it is said to be "uniform" or "poorly graded" and is represented by Curve No 2 in the graphical representation below. A soil is said to be "gap-graded", if it is deficient in a particular range of particle sizes and is represented by Curve no 3 in the graph below. All the different gradations are shown below.



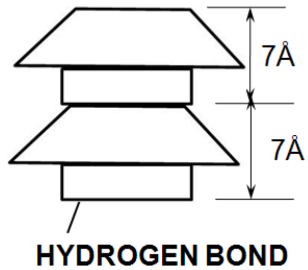
The uniformity of a soil is defined by its "Coefficient of Uniformity"
Cu=D60D10
Similarly coefficient of curvature is given as Cu=[D30]2[D60×D10]
where D60 = Particle size corresponding to 60% finer.
D30 = Particle size corresponding to 30% finer
and D10 = Particle size corresponding to 10% finer or effective size.
Cu will be less than 2 for poorly graded soils, will be greater than 4 for well graded gravels will be greater than 6 for well graded sand. Cc should be between 1 and 3 for well graded soil

3.a.

<u>CLAY MINERALS:</u>

1. **<u>Kaolinite Mineral:</u>**

Kaolinite is the most common mineral of the kaolinite group of minerals. Its basic structural unit consists of alumina sheet (G) combined with a silica sheet (S) . Tips of the silica sheet and one base of the alumina sheet form a common interface. The total thickness of the structural unit is about 7 Angstrom (Å), here one Å = 10^{-10} m or 10^{-7} mm. The kaolinite mineral is formed by stacking, one over the other, several such basic structural units.

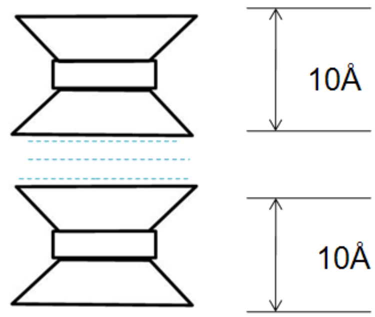


The Structural units join together by hydrogen bond, which develops between the oxygen of silica sheet and the hydroxyles of alumina sheet. As the bond is fairly strong, the mineral is stable. Moreover, water cannot easily enter between the structural units. So, expansion or swelling will not take place.

2. <u>Montmorillonite:</u>

Montmorillonite is the most common mineral of the Montmorillonite group of minerals. The basic structural unit consists of an alumina sheet sandwiched between two silica sheets; Successive structural units are stacked one over another, like leaves of a book. Fig shows two such structural units. The thickness of each structural unit is about 10Å.

The two successive structural units are joined together by a link between oxygen ions of the two silica sheets. The link is due to natural attraction for the cations in the intervening space and due to Vander Waal force. The negatively charged surfaces of the silica sheet attract water in the space between structural units. This results in expansion of the mineral. It may also cause dissociation of the mineral into individual structural units of thickness 10Å. The soil containing a large amount of mineral Montmorillonite exhibits high shrinkage and high swelling characteristics. The water in the intervening space can be removed by heating to 200^0 - 300^0C.



Montmorillonite

Illite Mineral:

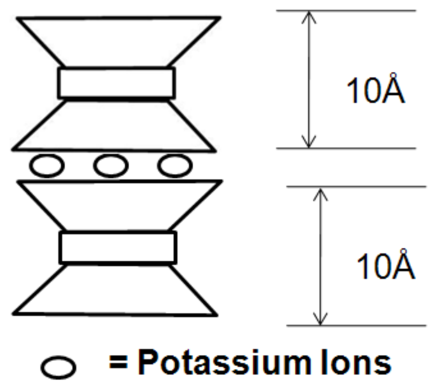
Illite is the main mineral of the illite group. The basic structural unit is similar to that of the mineral montmorillonite. However, the mineral has properties different from montmorillonite due to following reasons.

a. **There is always a substantial amount of isomorphous substitution of silicon by aluminum in silica sheet. Consequently, the mineral has a larger negative charge than that in montmorillonite.**

b. **The link between different structural units is through non-exchangeable potassium (K^+) and not through water. This bonds the units more firmly than in montmorillonite.**

c. **The lattice of illite is stronger than that of montmorillonite and is therefore less susceptible to change.**

d. Illite swells less than montmorillonite. However, selling is more than in kaolinite. e. The space between different structural units is much smaller than in montmorillonite, as the potassium ion just fit between the silica sheet surfaces.



B.

Soil structure

Geometrical arrangement of soil particles with respect to one another is known as soil structure. The different soil structures include

1. Single grained structure

2. Honey comb structure

3. Flocculated structure

4. Dispersed structure

5. Coarse grained skeleton

6. Clay-matrix structure

Single grained structure

➢ Single-grained structure is characteristic of coarse grained soils, with a particle size greater than 0.02 mm.

➢ Gravitational forces predominate the surface forces and hence grain to grain contact results.

➢ The deposition may occur in a loose state, with large voids or in a dense state, with less of voids.

Honey comb structure

➢ This structure can occur only in fine-grained soils, especially in silt and rock flour.

➢ Due to the relatively smaller size of grains, besides gravitational forces, inter-particle surface forces also play an important role in the process of settling down.

➢ Miniature arches are formed, which bridge over relatively large void spaces.

➢ **This results in the formation of a honey-comb structure, each cell of a honey-comb being made up of numerous individual soil grains.**

➢ **The structure has a large void space and may carry high loads without a significant volume change**

Flocculated structure

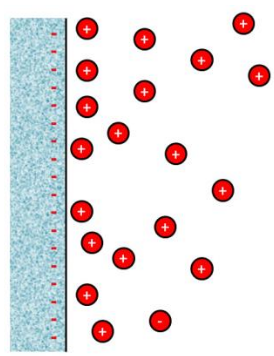
➢ **They have large surface area and electrical forces are important in such soils.**

➢ **They have a negative charge on the surface and a positive charge on the edges.**

Diffuse double layer:

The faces of clay minerals carry a net negative charge because of which the clay particles attract cations in presence of moisture and reach an electrical balanced equilibrium. These cations in turn, attract particles with negative charges and water dipoles. The cations attracted to a clay mineral surface also try to move away from the surface because of their thermal energy.

The net effect of the forces due to attraction and that due to repulsion is that the forces of attraction decrease exponentially with an increase in distance from the clay particles surface. It is believed that immediately surrounding the particle, there is a thin, very tightly held layer about 10Å thick. Beyond this thickness there is a second layer, in which water is mobile. This second layer extends to the limit of attraction. The layer the clay particle surface to the limit of attraction is known as diffuse double layer.

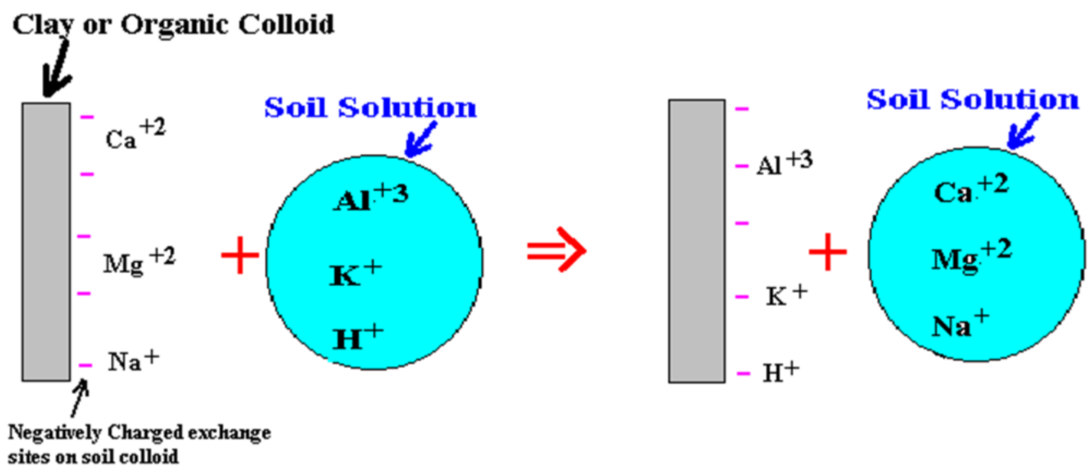


Diffuse Double Layer

In other words, Base Exchange capacity is the capacity of the clay particles to change the cation adsorbed on the surface.

Base exchange capacity/cation exchange capacity

The cations attracted to the negatively charged surface of the soil particles are not strongly attracted. These cations can be replaced by other ions called as exchangeable ions. The soil particles and the exchangeable ions make the system neutral. Base exchange capacity is the capacity of the clay particles to change cation adsorbed on the surface.
Base exchange capacity is expressed in terms of total number of positive charges adsorbed on the surface per 100 g of soil. It is expressed as milliequivalent (meq) which is equal to 6×1020 electronic charges. Thus 1 meq per 100 g means that 100 g of material can exchange 6×1020 electronic charges if the ions are monovalent. If its divalent cation, 100 g of material can exchange 3×1020 electronic charges.
The affinity of attraction is as follows:



Al3+ > Ca2+ > Mg2+ > NH44+ > H+ > Na+ > Li+

C. Factors affecting type of compaction

Following the different factors affecting compaction of soil:

- **Water content**
- **Amount of compaction**
- **Types of soil**
- **Methods of soil compaction**

Effect of Water Content on Compaction of Soil
At low water content, the soil is stiff and offers more resistance to compaction. As the water content is increased, the soil particles get lubricated. The soil mass becomes more workable

and the particles have closer packing. The dry density of the soil increases with an increase in the water content till the optimum water content in reached. At that stage, the air voids attain approximately a constant volume. With further increase in water content, the air voids do not decrease, but the total voids (air plus water) increase and the dry density decreases. Thus the higher dry density is achieved upto the optimum water content due to forcing air voids out from the soil voids. After the optimum water content is reached, it becomes more difficult to force air out and to further reduce the air voids. The effect of water content on the compaction of soil can also be explained with the help of electrical double layer theory. At low water content, the forces of attraction in the adsorbed water layer are large, and there is more resistance to movement of the particles. As the water content is increased, the electrical double layer expands and the inter-particle repulsive forces increase. The particles easily slide over one another and are closely packed. This results in higher dry density.

Amount of Compaction

The compaction of soil increases with the increase in amount of compactive effort. With increase in compactive effort, the optimum water content required for compaction also decreases. At water content less than the optimum, the effect of increased compaction is more predominant. At water content more than the optimum, the volume of air voids become almost constant and the effect of increased compaction on soil is not significant. It may be mentioned that the maximum dry density does not go on increasing with an increase in the compactive effort. For a certain increase in the compactive effort, the increase in the dry density becomes smaller and smaller. Finally a stage is reached beyond which there is no further increase in the dry density with an increase in the compactive effort. The line of optimums which join the peaks of the compaction curves of different compactive efforts follows the general trend of the zero-air void. This line corresponds to air voids of about 5%.

Type of Soil:

The compaction of soil depends upon the type of soil. The maximum dry density and the optimum water content for different soils are shown in figure. In general, coarse grained soils can be compacted to higher dry density than fine-grained soils. With the addition of even a small quantity of fines to a coarse-grained soil, the soils attain a much higher dry density for the same compactive effort. However, if the quantity of the fines in increased to a value more than that required to fill the voids of the coarse-grained soils, the maximum dry density decreases. A well graded sand attains a much higher dry density than a poorly graded soil. Cohesive soils have high air voids. These soils attain a relatively lower maximum dry density as compared with the cohesionless soils. Such soils require more water than cohesionless soils and therefore the optimum water content is high. Heavy clays of very high plasticity have very low dry density and a very high optimum water content.

Method of Soil Compaction:

The dry density achieved depends not only upon the amount of compactive effort but also on the method of compaction. For the same amount of compactive effort, the dry density will depend upon whether the method of compaction utilizes kneading action, dynamic action or static action. For example, in Harvard Miniature compaction test, the soil is compacted by the kneading action, and therefore, the compaction curve obtained is different from that obtained from the other conventional tests in which an equal compactive effort is applied. Different methods of compaction curve give their own compaction curves. Consequently, the lines of optimums are also different.

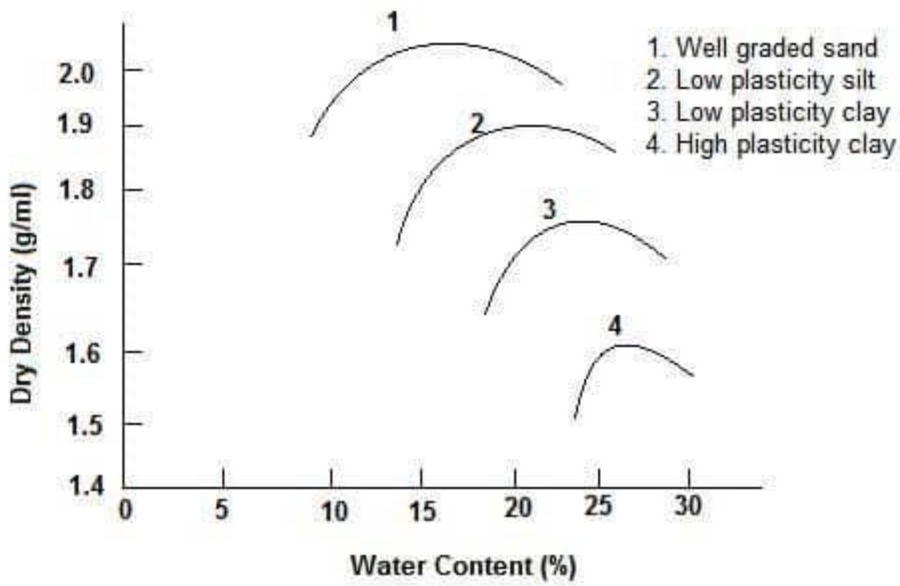


Fig: Compaction curves for different soils

4A.

Standard Proctor Test	**Modified Proctor Test**
305 mm height of drop	450 mm height of drop
25 N hammer	45 N hammer
25 blows/layer	25 blows/layer
3 layers	5 layers
Mould size: 945 ml	Mould size: 945 ml
Energy 605160 N-mm per m^3	Energy 2726000 N-mm per m^3

4.B. Water Content

1. With increase in water content, compacted density increases up to a stage, beyond which compacted density decreases.

2. The maximum density achieved is called MDD and the corresponding water content is called OMC.

3. Particles slide over each other easily increasing lubrication, helping in dense packing.

4. After OMC is reached, air voids remain constant. Further increase in water, increases the void space, thereby decreasing dry density.

Compactive effort

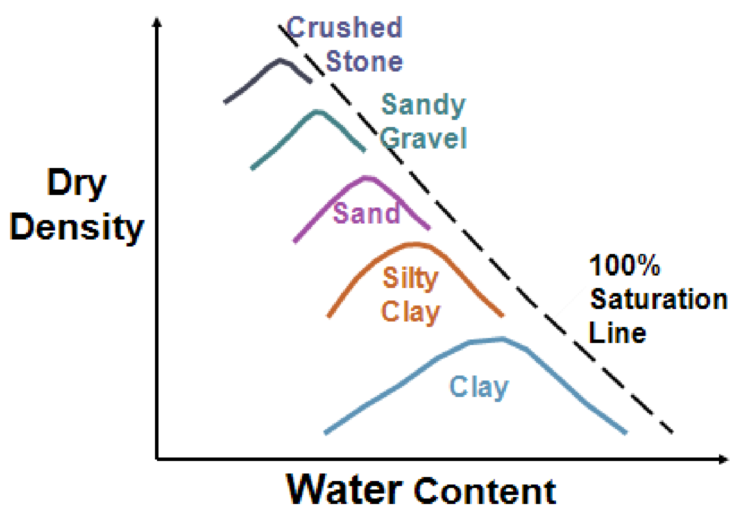
1. Effect of increasing compactive effort is to increase MDD And reduce OMC (Evident from Standard & Modified Proctor's Tests).

2. However, there is no linear relationship between compactive effort and MDD

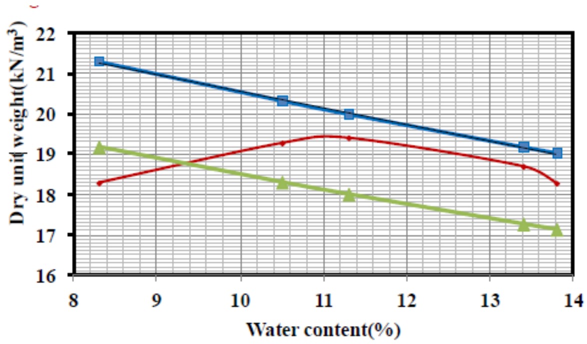
Effect of type of Soil

1. Maximum density achieved depends on type of soil.

2. Coarse grained soil achieves higher density at lower water content and fine grained soil achieves lesser density, but at higher water content.



4.c.



For each reading $\gamma d = \gamma 1 + w$

For eg. γd=19.81+0.083=18.28 kN/m3

For zero air void line $\gamma d = G\gamma(1+wG)$=2.65×9.81(1+0.083×2.65)=21.3 $kN/m3$

For zero air void line $\gamma d = G\gamma(1-na)(1+wG)$=2.65×9.81(1−0.1)(1+0.083×2.65)=19.2 $kN/m3$

MDD = 19.5 kN/m3

OMC = 11.4%

Range of water content- 10% - 12.4%

5.a. Factors Effecting Permeability of Soils

Following are factors effecting permeability of soils.

1. Size of soil particle
2. Specific Surface Area of Soil Particle
3. Shape of soil particle
4. Void ratio
5. Soil structure
6. Degree of saturation
7. Water properties
8. Temperature
9. Adsorbed water
10. Organic Matter

1. Size of Soil Particle

Permeability varies according to size of soil particle. If the soil is coarse grained, permeability is more and if it is fine grained, permeability is low. The relation between coefficient of permeability (k) and particle size (D) can be shown from equation (1) as follows.

$$k \ \alpha \ D^2$$

2. Specific Surface Area of Particles

Specific surface area of soil particles also effects the permeability. Higher the specific surface area lower will be the permeability.

$$k \; \alpha \; \frac{1}{Specific\; Surface\; Area}$$

3. Shape of Soil Particle

Rounded Particles will have more permeability than angular shaped. It is due to specific surface area of angular particles is more compared to rounded particles.

4. Void Ratio

In general, Permeability increases with void ratio. But it is not applicable to all types of soils. For example, Clay has high void ratio than any other types of soil but permeability for clays is very low. This is due to, the flow path through voids in case of clays is extremely small such that water cannot permit through this path easily.

The relation between coefficient of permeability and void ratio can be expressed from equation (1) as

For Clay

$$k \; \alpha \; \frac{C\, e^3}{1+e}$$

Where, C = Shape of the flow path,

e = Void ratio.

For coarse grained soil, "C" can be neglected. Hence

$$k \; \alpha \; \frac{e^3}{1+e}$$

5. Soil Structure

Structure of any two similar soil masses at same void ratio need not be same. It varies according to the level of compaction applied. If a soil contains flocculated structure, the particles are in random orientation and permeability is more in this case.

If the soil contains dispersed structure, the particles are in face to face orientation hence, permeability is very low. The permeability of stratified soil deposits also varies according to the flow direction. If the flow is parallel, permeability is more. If it is perpendicular, permeability is less.

5.b. Quicksand condition occurs when seepage pressure, which acts in the upward direction, overcomes the downward direction pressure due to submerged weight of soil, and the sand grains are forced apart. The result is that the soil has no capability to support a load.

The soil that experiences quicksand condition would lose shear strength and bearing capacity. The shear strength of cohesion-less soil depends on the effective stress. The shear strength is given by:

$$\text{shear strength } (s) = \sigma' \tan \emptyset \qquad \textit{Equation 1}$$

Where:

σ': effective stress.

$\emptyset$: angle of shearing resistance.

The effective stress is given by the following expression:

$$\sigma' = \sigma - u \qquad \textit{Equation 2}$$

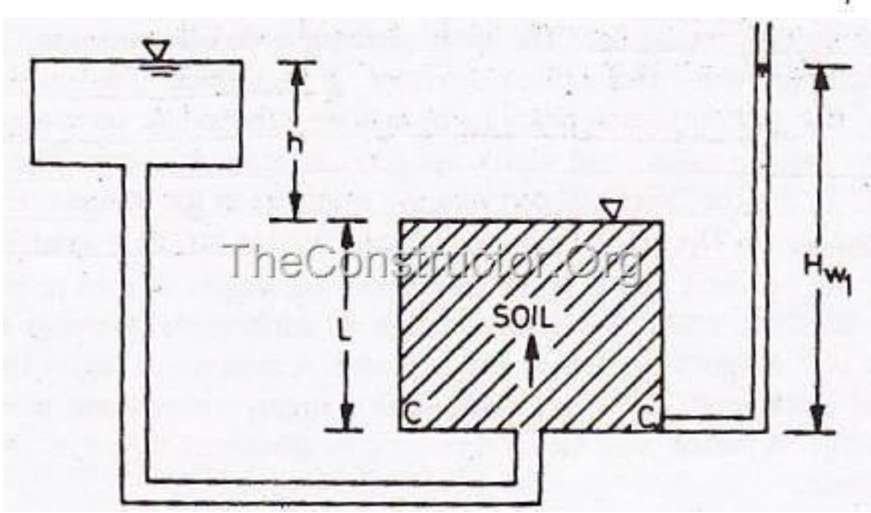


Fig. 1: Quick Sand Condition

The terminologies of equation 2 are explained and illustrated in fig. 2. Plugging components of equation 2 results in the following expressions:

$\sigma' = \gamma_{sat}L - \gamma_w H_{w1}$ *Equation 3*

$\sigma' = (\gamma' + \gamma_w)L - \gamma_w(L + h)$ *Equation 4*

$\sigma' = \gamma' L - \gamma_w h$ *Equation 5*

$\gamma_w h = \gamma_w \cdot \frac{h}{L} L = \gamma_w i L$ *Equation 6*

So, equation 5 may be expressed as follows:

$\sigma' = \gamma' L - \gamma_w i L$ *Equation 7*

In order for the effective stress to become zero, $\gamma' L = \gamma_w i L$ in equation 7, therefore:

$i = \gamma' / \gamma_w$ *Equation 8*

Substituting the value of submerged unit weight in terms of void ratio:

$i_c = \frac{G-1}{1+e}$ *Equation 9*

Taking G=2.67, and e=0.67, the result of equation 9 is equal to one.

Thus, the effective stress becomes zero for the soil with above values of G and e, when the hydraulic gradient 'i' is unity, i.e. head causing the flow is equal to the length of the specimen.

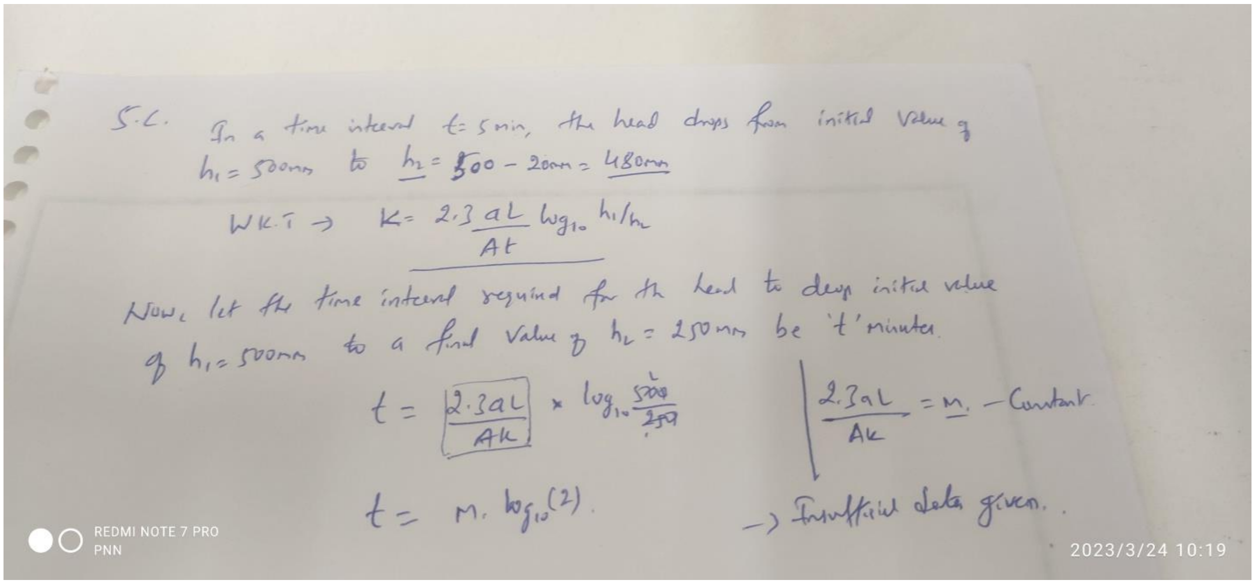
If the critical gradient exceeds, the soil moves upward, and the soil surface appears to be boiling. The quick condition is also known as boiling condition. During this stage, a violent and visible agitation of particles occurs. The discharge suddenly increases due to an increase in the coefficient of permeability occurred in the process. If a weight is placed on the surface of the soil, it sinks down. The soil behaves as a liquid having no shear strength.

Capillary action in soils

Water rises in small diameter, capillary tubes, because of adhesion and cohesion. Adhesion occurs because water sticks to the solid walls of the tube. Cohesion is due to mutual attraction of water molecules. If adhesion is predominant, the liquid will wet the surface. If cohesion is predominant, the liquid level is depressed at the point of contact. Capillary water exists in soils as long as there is an air – water interface. As soon as the soil is submerged under water, the interface is destroyed, and the capillary water becomes normal, free water.

In case of soils, pore size or diameter forms the capillary tube and capillarity height is inversely proportional to the diameter of the pore size. The pore size will be very small for clay when compared to sand. Hence capillary action is predominant in clay.

5.c.


5.c. In a time interval $t = 5$ min, the head drops from initial value of $h_1 = 500mm$ to $h_2 = 500 - 20mm = 480mm$

WKT → $K = \frac{2.3\, aL}{At} \log_{10} h_1/h_2$

Now, let the time interval required for the head to drop initial value of $h_1 = 500mm$ to a final value of $h_2 = 250mm$ be 't' minutes

$t = \boxed{\frac{2.3aL}{Ak}} \times \log_{10} \frac{500}{250}$

$\frac{2.3aL}{Ak} = M$. — Constant

$t = M. \log_{10}(2)$.

→ Insufficient data given.

6.a. Flownets

A flow net for an isometric medium is a network of flow lines and equipotential lines intersecting at right angles to each other. The path which a particle of water follows in its course of seepage through a saturated soil mass is called a flow line. Equipotential lines are lines that intersect the flow lines at right angles. At all points along an equipotential line, the water would rise in piezometric tubes to the same elevation known as the piezometric head.

Properties of flownet

1. Flow and equipotential lines are smooth curves.

2. Flow lines and equipotential lines meet at right angles to each other

3. No two flow lines cross each other.

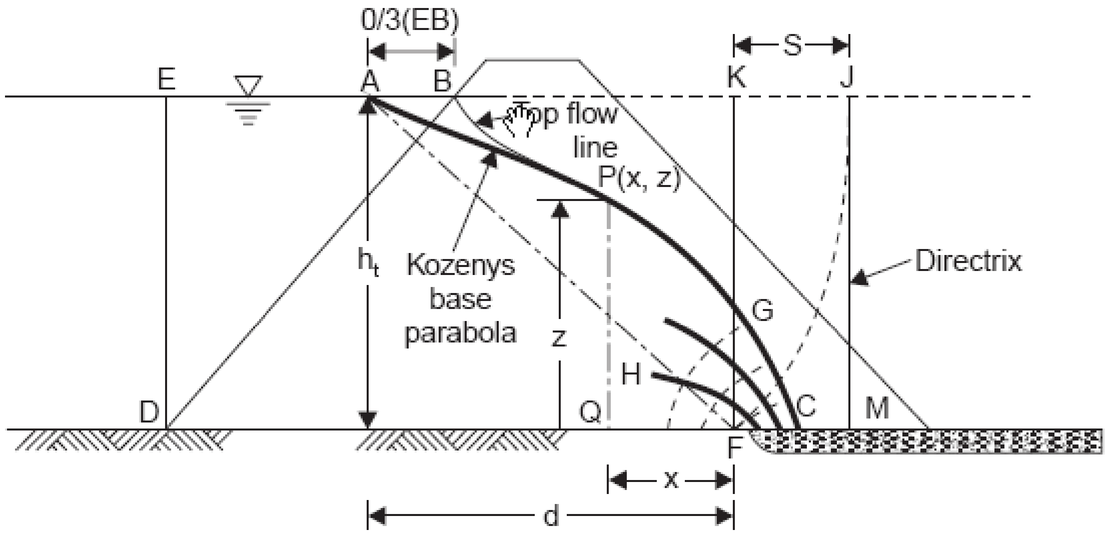
4. No two flow or equipotential lines start from the same point.

5. Discharge Δq between two flow lines remains constant. Similarly drop in head Δh remains constant between two equipotential drops.

6. The ratio of length and width of each field is constant. Its always considered as squares.

6.b. Locating the phreatic line in a homogenous earth dam with horizontal filter

Graphical method:



1. Locate the point A, using BA = 0.3 (BE). A will be the starting point of the Kozeny parabola.

2. With A as centre and AF as radius, draw an arc to cut the water surface (extended) in J. The vertical through J is the directrix. Let this meet the bottom surface of the dam in M.

3. The vertex C of the parabola is located midway between F and M.

4. For locating the intermediate points on the parabola the principle that it must be equidistant from the focus and the directrix will be used. For example, at any distance *x* from F, draw a vertical and measure QM. With F as center and QM as radius, draw an arc to cut the vertical through Q in P, which is the required point on the parabola.

5. Join all such points to get the base parabola. The portion of the top flow line from B is sketched in such that it starts perpendicular to BD, which is the boundary equipotential and meets the remaining part of the parabola tangentially without any kink. The base parabola meets the filter perpendicularly at the vertex C.

6.c. Effective Stress:

Karl Terzaghi was the first to recognize the importance of effective stress. It is the stress transmitted through grain to grain at the point of contact through soil mass. It is also known as inter-granular stress. It is denoted by σ'. When soil mass is loaded. The load is transferred to the soil gains through their point of contact. If at the point of contact, the applied load is greater than the resistance of the grains, then there will be compression in the soil mass.

This compression is partly due to the elastic compression of the grains at the points of contact and partly due to relative sliding between particles. This load per unit area of soil mass responsible for deformation of the soil mass is termed as effective stress.

Neutral Stress:

It is the stress or pressure transmitted through the pore fluid. It is also termed as pore pressure and is denoted by u. In saturated soil, pores of the soil mass are filled with water. When the saturated soil mass is loaded, the load is not transmitted through the grains. The load is transferred to the pore water. As water is incompressible, a pressure is developed in the pore water.

This pressure is called pore pressure or pore water pressure. Pore pressure does not have any measurable influence on the mechanical property of the soil like void ratio, shear strength etc. This pressure or stress is called neutral stress.

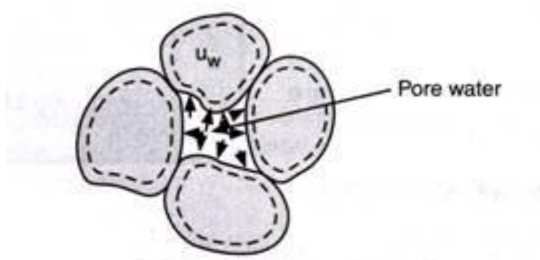


FIG. 5.2 Neutral stress

Total Stress:

Total stress is equal to the sum of the effective stress and the neutral stress. It is denoted by σ.

σ = σ + u

Effective stress cannot be measured in the field by any instrument. It can only be calculated after measuring total stress and pore pressure. Thus effective stress is not a physical parameter, but is only very useful mathematical concept for determination of engineering behaviour of soil.

Importance of Effective Stress in Engineering Problems:

The effective stress plays an important role in:

(i) Settlement of soil

(ii) Shear strength of soil

Settlement of Soil:

The phenomenon of gradual reduction in volume of soil due to expulsion of water from soil pores is called consolidation or compression or settlement of soil. Figure 5.3 shows a compression curve of clay. It is a curve between effective stress σ and void ratio e. It is clear

from the graph that when σ increases e decreases i.e., due to increase in the effective stress the compression of soil will increase.

The final consolidation settlement may be calculated by using the formula

$S = m_V H$
Where m_V is the coefficient of volume compressibility
H is the thickness of compressible layer

$\Delta\sigma$ is the average increase in effective pressure.

From the above equation it is clear that settlement of soil is directly proportional to the effective pressure. So the settlement of soil depends upon the effective stress or effective pressure. As the effective stress increases, the settlement of the soil also increases.

Shear Strength of Soil:
Many geotechnical engineering problems require an assessment of shear strength including:
(a) Structural foundations:
Load from a structure is transferred to ground through foundation. This produces shear stress and compressive stress. If shear stress produced is more than the shear strength of soil, shear failure occurs which cause the structure to collapse.

(b) Earth slopes:
On a sloping ground, gravity produces shear stresses in the soil. If these stresses exceed the shear strength, a landside occurs.

7.a.

Shear strength of a soil represents the resistance to shear stresses. According to Mohr, failure is caused by a critical combination of normal and shear stresses as represented by equation (1).

Or $\tau = f(\sigma)$ (1)

Graphically equation (1) will be curved in shape as seen in Fig. 5.1. At failure, the Mohr failure envelope will be tangential to the Mohr's circle.

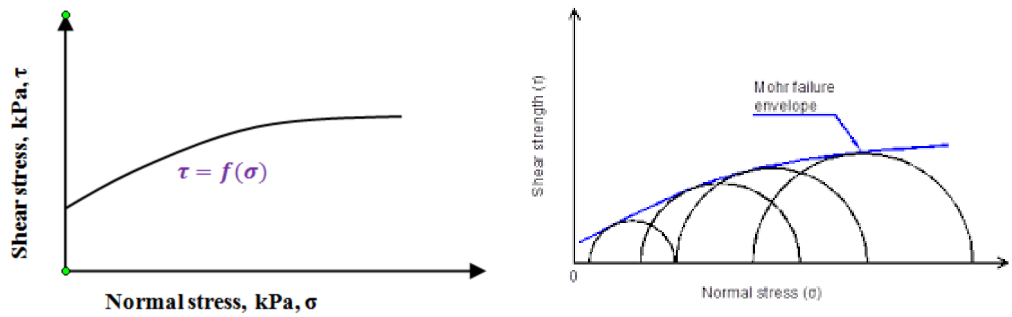


Figure 5.1. Mohr's failure envelope

Coulomb modified Mohr's theory by stating that shear strength of soil is dependent on two parameters: cohesion between the soil particles and the friction between the particles. Accordingly Equation (1) was modified and the equation for modified failure envelope is given by Equation (2). Mohr's modified failure envelope is given in Fig.5.2

$\tau = c + \sigma \tan\varphi$ (2)

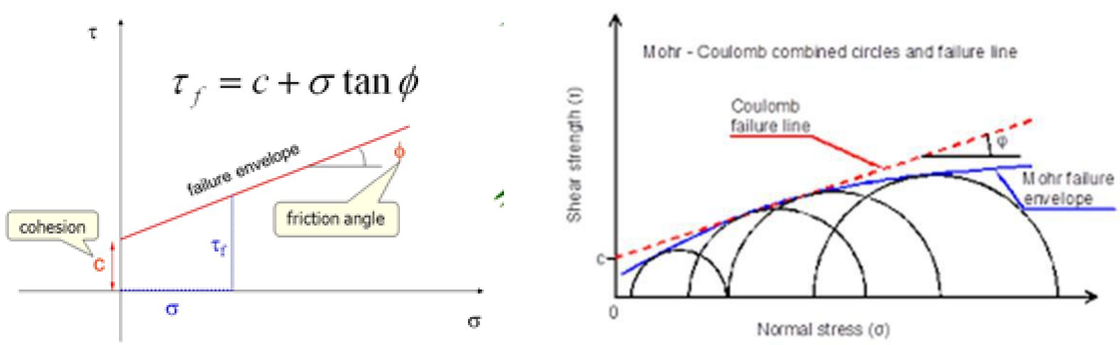


Figure 5.2. Mohr's failure envelope

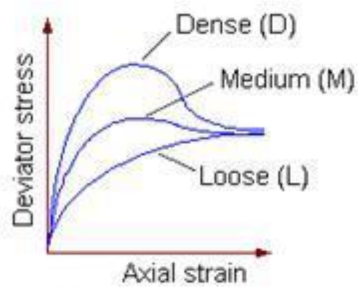
7.b.

The principal factors effecting the shearing strength of cohesion less soil are

1) Shape of particles: Angular particles will always develop higher shear resistance when compared to rounded particles.

2) Gradation: Well graded soil particles will develop higher shear resistance than poorly or uniformly graded soils.

3) Denseness: Typical plot of deviator stress versus axial strain for dense and loose soils are as shown below in Fig. 5.3:



As the denseness of soil increases, the soils strain softening behavior whereas for loose soils, it exhibits a strain hardening behavior.

4) Confining pressure: Higher the confining pressure higher will be the deviator stress as seen in figure below in Fig. 5.4

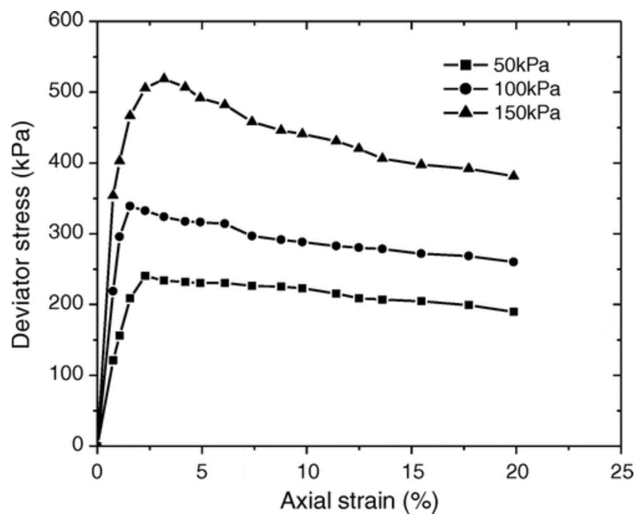


Fig. 5.4. Deviator stress versus axial strain for different confining pressures

The graph presents the variation of deviator stress for 3 different confining pressures: 50, 100 and 150 kPa.

5) Deviator stress: As the deviator stress increases, particles will be subjected to crushing. This will reduce the shear resistance of soils.

6) Vibration and repeated loading: vibrations and repeated loading intend to reduce the shear strength of soil.

7) Type of minerals: The shear resistance of soil is dependent upon the clay mineral.

8) Capillary moisture: Capillary moisture provides apparent cohesion and increases effective shear parameters. This in fact increases shear resistance of the soil.

9) Strain rate. The faster a soil specimen is sheared (i.e., a fast strain rate), the higher will be the value of the un-drained shear strength

9) Clay content: Higher the clay content higher will be the cohesion of the sample.

10) Stress history: the behavior of Over-consolidated clay is similar to dense sand and that of normally consolidated clay, it will be similar to loose sand.

7.c.

7.c.

Resultant stress $= \sqrt{\sigma^2+\tau^2}$

$= \sqrt{100^2+40^2}$

$= 107.70\ \text{kN/m}^2$

$\tan\phi = \tau/\sigma = 40/100$

$\phi = 21.80$

$\phi = \underline{21^\circ 48'}$

$\theta = 45^\circ + \phi/2$

$\theta = 45 + 21^\circ 48'$

$\theta = \underline{66^\circ 48'}$

8.a. The shear strength parameters in the case of saturated soils depend very much upon the drainage conditions and therefore in the laboratory shear test, the drainage condition expected in the field for a particular problem should be simulated. Based on drainage condition the shear tests are classified as following 3 types.

1. **Unconsolidated Undrained Test (UU test)**
2. **Consolidated Undrained Test (CU test)**
3. **Consolidated Drained Test (CD test)**

1. UNCONSOLIDATED UNDRAINED TEST (UU)

Drainage is not permitted throughout the test. In the case of direct shear test drainage is not permitted during the application of both normal stress and shear stress. In the case of triaxial compression test drainage is not permitted during the application of both cell pressure and deviator stress. Since the test is conducted fast allowing no time for either consolidation of sample initially or dissipation of pore pressure in later stage, the test is also called quick test.

2. CONSOLIDATED UNDRAINED TEST (CU)

In this type of shear test the soil specimen is allowed to consolidate fully under initially applied stress and then sheared quickly without allowing dissipation of pore pressure. In the

case of direct shear test the specimen is allowed to consolidate fully under applied normal stress and then sheared at high rate of strain to prevent dissipation of pore pressure during shearing. In the case of triaxial compression test the specimen is allowed to consolidate fully under the applied cell pressure and then the pore water outlet is closed and the specimen is subjected to increasing deviator stress at higher rate of strain.

3. CONSOLIDATED DRAINED TEST (CD)

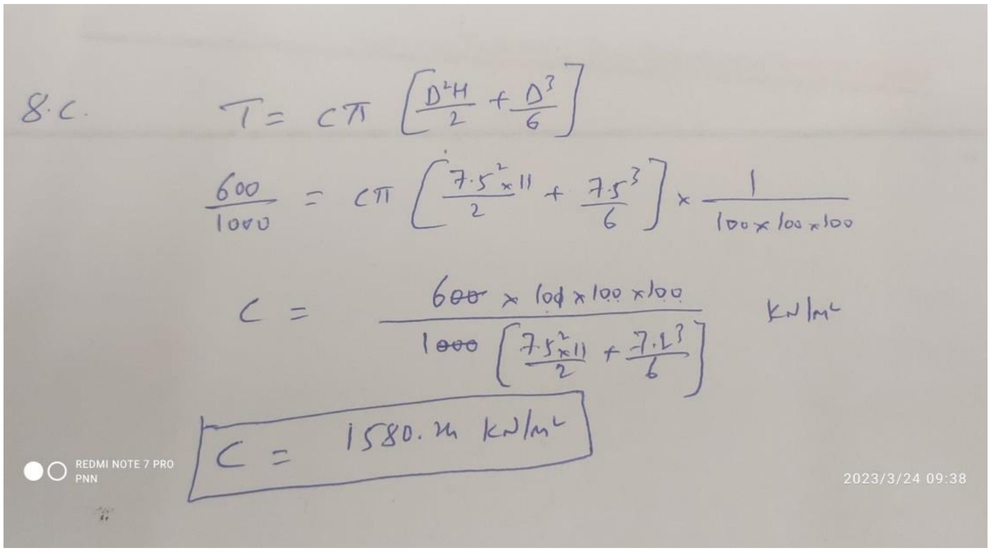
In this type of shear test drainage is allowed throughout the test. The specimen is allowed to consolidate fully under the applied initial stress and then sheared at low rate of strain giving sufficient time for the pore water to drain out at all stages. The test may continue for several hours to several days.

8.b. Advantages of Direct shear test

- **The sample preparation is easy.**
- **The test is simple and convenient.**
- **As the thickness of the sample is relatively small, the drainage is quick and the pore pressure dissipates very rapidly.**
- **Direct shear test is ideally suited for conducting drained tests on cohesion less soils.**
- **The apparatus is relatively cheap.**

- **Disadvantages of Direct shear test**
- **The stress condition is known only at failure. The conditions prior to failure are indeterminate and, therefore, the Mohr circle cannot be drawn.**
- **In direct shear test ,the stress distribution on the failure plane (horizontal plane) is not uniform.**
- **The area under shear gradually decreases as the test progresses. But the corrected area cannot be determined and therefore, the original area is taken for the computation of stresses.**
- **The orientation of failure plane is fixed. This plane may not be the weakest plane.**
- **Control on the drainage conditions is very difficult. So, only drained tests can be conducted on highly permeable soils.**
- **The measurement of pore water pressure is not possible in direct shear test.**
- **The side walls of the shear box cause lateral restraint on the specimen and do not allow it to deform laterally.**

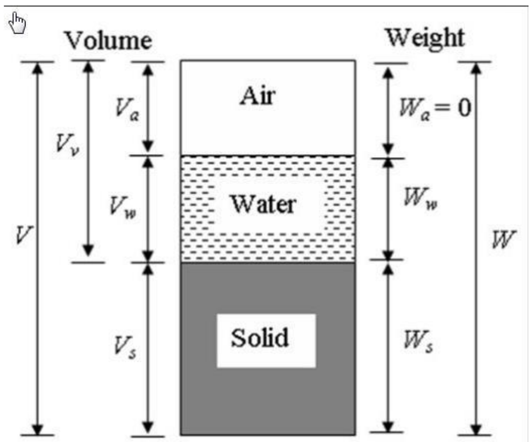
8.c.

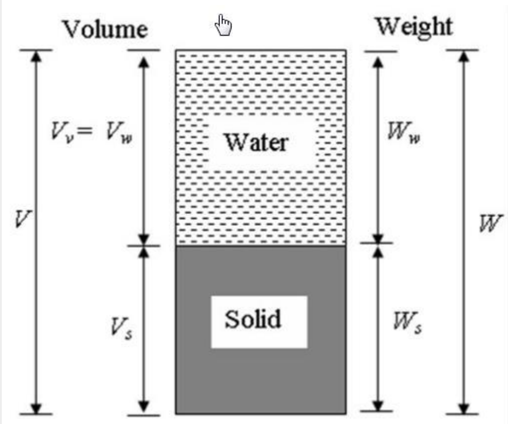


9.a.

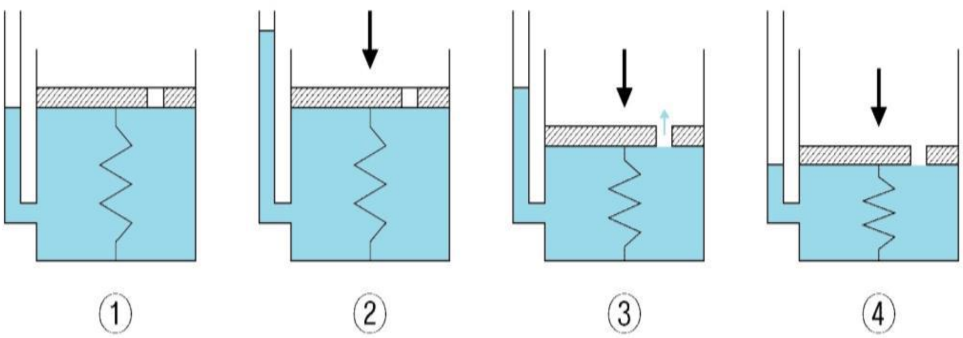
Sl No	Compaction	Consolidation
1	Compaction is the process by which solid soil particles are packed more closely together by mechanical means.	Consolidation is the process by which soil particles are packed more closely together under the application of static loading
2	It is achieved through reduction of air voids.	It is achieved through gradual drainage of water from soil pores.
3	It is a rapid and artificial process. Applicable to cohesive and cohesionless soils.	It is a gradual and natural process. In some soils it takes many years. Applicable to cohesive soils
4	Proper compaction of soil is achieved at optimum moisture content.	Consolidation is strictly applicable for saturated or nearly saturated clays or soils with low permeability.

5	Soil exists in three phases before and after compaction	Soil exists in two phases before and after consolidation





9.b. The consolidation process is often explained with an idealized system composed of a spring, a container with a hole in its cover, and water.



In this system, the spring represents the compressibility or the structure itself of the soil, and the water which fills the container represents the pore water in the soil.

➢ The container is completely filled with water, and the hole is closed. (Fully saturated soil)

➢ A load of 1 kN is applied onto the cover, while the hole is still unopened. At this stage, only the water resists the applied load. (Development of excessive pore water pressure)

➢ As soon as the hole is opened, water starts to drain out through the hole and the spring shortens. (Drainage of excessive pore water)

➢ After some time, the drainage of water no longer occurs. Now, the spring alone resists the applied load. (Full dissipation of excessive pore water pressure. End of consolidation)

9.c. 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. Plot void ratio vs effective stress variation and mark it as ABCD.

2. Choose by eye the point of maximum curvature on the consolidation curve Say E.

3. Draw a horizontal line from this point, line EF.

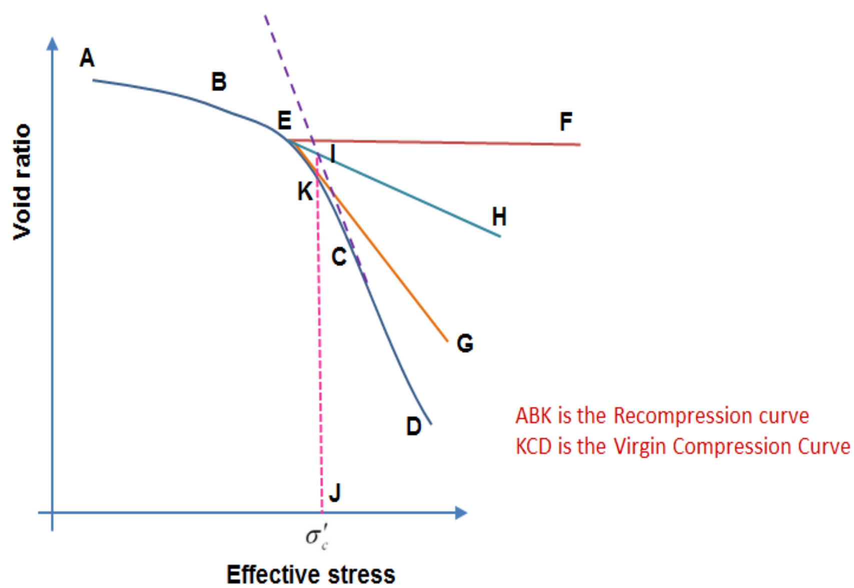
4. Draw a line tangent to the curve at the point E, line EG.

5. Bisect the angle made from the horizontal line EF and the tangent line EG. Name the bisector as EH.

6. 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 I.

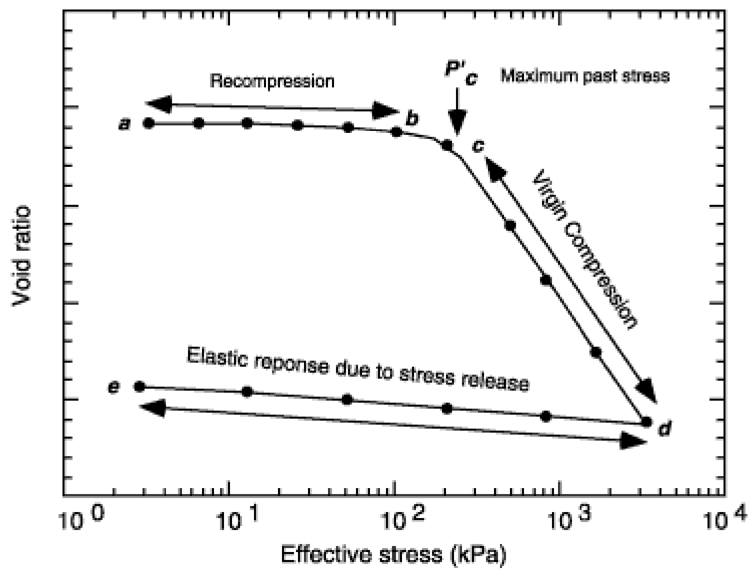
7. Drop vertical IJ and the abscissa of IJ indicate pre consolidation pressure.

8. Vertical IJ intersect e-log $\bar{\sigma}$ curve at K, Curve ABK indicates recompression curve and curve KCD indicate virgin compression curve.



10.a. A soil which is subjected to a pressure for the first time in its life time is called as normally consolidated clay. Such clays exhibit high compression which is indicated by virgin compression curve.

A soil which is subjected to a pressure greater than the existing pressure is called as over-consolidated clay. Such clays exhibit less settlement as indicated by recompression curve



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

Over-consolidation ratio is defined as the ratio of pre-consolidation pressure to the existing pressure.

A soil which has not reached equilibrium under the applied pressure itself is called as under-consolidated clay. Eg. Landfills.

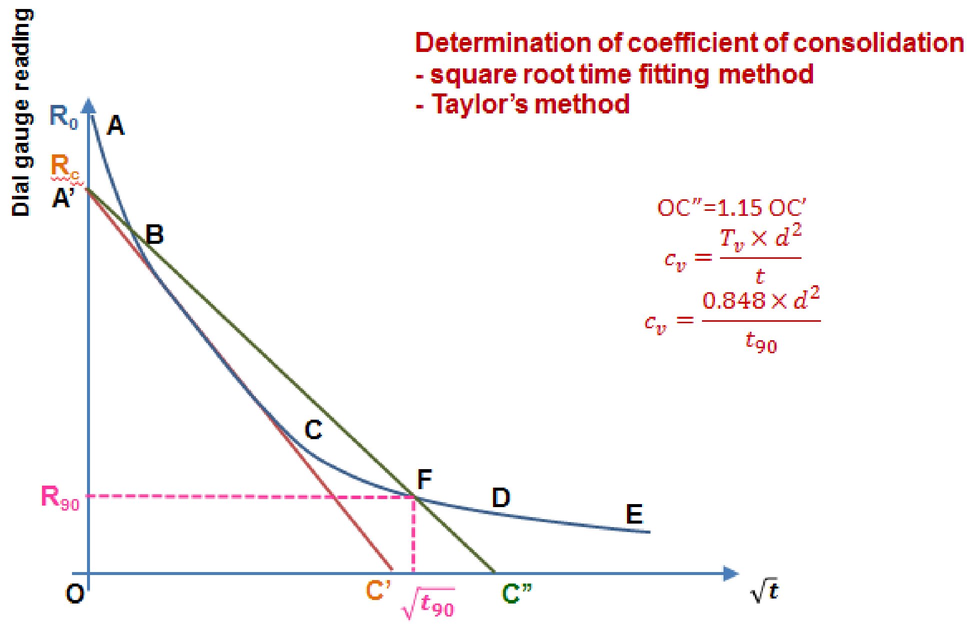
Normally and
Over-Consolidated Soils

$\sigma'_{zo} = \sigma'_c$ ..... Normally consolidated

$\sigma'_{zo} < \sigma'_c$ ..... Over consolidated

$\sigma'_{zo} > \sigma'_c$ ..... Under consolidated

10.b.



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 ie., curve ABCDE.

2. The initial straight part can be extended backwards to meet at A'. The dial gauge reading at A' corresponds to Rc.

3. Extend the straight part to meet X axis at C'.

4. Starting from Rc, draw another straight line such that its abscissa is 1.15 times the abscissa of first straight line.

5. The intersection point between the second straight line and experimental curve represents the R90 and corresponding time is determined and noted as √t90. Thus, the time required (t90) for 90% consolidation is calculated.

6. The Coefficient of consolidation (cv) is determined as:

cv=**0.848**d**2**t**90** *where d is the drainage path=d for single face drainage and=d*2*for two face drainage*

10.c.

Example 7.6: There is a bed of compressible clay of 4 m thickness with pervious sand on top and impervious rock at the bottom. In a consolidation test on an undisturbed specimen of clay from this deposit 90% settlement was reached in 4 hours. The specimen was 20 mm thick. Estimate the time in years for the building founded over this deposit to reach 90% of its final settlement. (S.V.U.—B.E., (R.R.)—Sept., 1978)

This is a case of one-way drainage in the field.

∴ Drainage path for the field deposit, $H_f = 4$ m = 4000 mm. In the laboratory consolidation test, commonly it is a case of two-way drainage.

∴ Drainage path for the laboratory sample, $H_1 = 20/2 = 10$ mm

Time for 90% settlement of laboratory sample = 4 hrs.

Time factor for 90% settlement, $T_{90} = 0.848$

∴ $$T_{90} = \frac{C_v t_{90_f}}{H_f^2} = \frac{C_v t_{90_l}}{H_l^2}$$

or $$\frac{t_{90_f}}{H_f^2} = \frac{t_{90_l}}{H_l^2}$$

∴ $$t_{90f} = \frac{t_{90_l}}{H_l} \times H_f = \frac{4 \times (4000)^2}{10^2} \text{ hrs}$$

$$= \frac{4 \times 400}{24 \times 365} \text{ years}$$

$$\approx \mathbf{73 \text{ years.}}$$