| Sub: | Basic Geotechnical Engineering | Sub Code: | 15CV45 | Branch: | CIVIL |
|---|---|---|---|---|---|

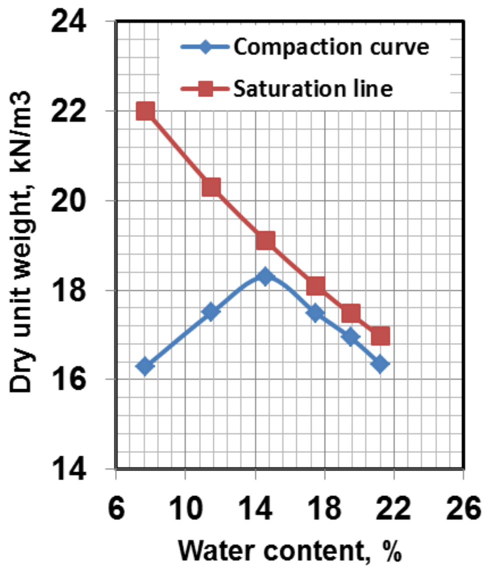
**1 (a) Following are the observations of a compaction test** **[08]**

| Water content (%) | 7.7 | 11,5 | 14.6 | 17.5 | 19.5 | 21.2 |
|---|---|---|---|---|---|---|
| Weight of wet soil (N) | 16.67 | 18.54 | 19.92 | 19.52 | 19.23 | 18.83 |

**If the volume of the compaction mould is 950 cc, assuming G=2.65.**
**Draw the compaction curve**
**Report the maximum dry unit weight and optimum moisture content.**
**Draw 100% saturation line.**
**Estimate the degree of saturation and percentage air voids at OMC and MDD.**

**Curve+OMC+MDD-saturationline -5**
**Estimate the degree of saturation and percentage air voids at OMC and MDD -3**

| Water content (%) | Weight of wet soil (N) | Wet unit weight (kN/m3) | Dry unit weight (kN/m3) | 100% Saturation line |
|---|---|---|---|---|
| 7.70 | 16.67 | 17.55 | 16.29 | 22.01 |
| 11.50 | 18.54 | 19.52 | 17.50 | 20.31 |
| 14.60 | 19.92 | 20.97 | 18.30 | 19.11 |
| 17.50 | 19.52 | 20.55 | 17.49 | 18.10 |
| 19.50 | 19.23 | 20.24 | 16.94 | 17.47 |
| 21.20 | 18.83 | 19.82 | 16.35 | 16.97 |




OMC=14.4 %
Maximum dry unit weight = 18.4 kN/m$^3$

$$\gamma_d = \frac{G\gamma_w}{1+\frac{wG}{S}} = \frac{2.65\times 10}{1+\frac{0.144\times 2.65}{S}} = 18.4$$

S=86.7%

$$18.4 = \frac{G\gamma_w(1-n_a)}{1+wG} = \frac{2.65\times 10\,(1-n_a)}{1+0.144\times 2.65}$$

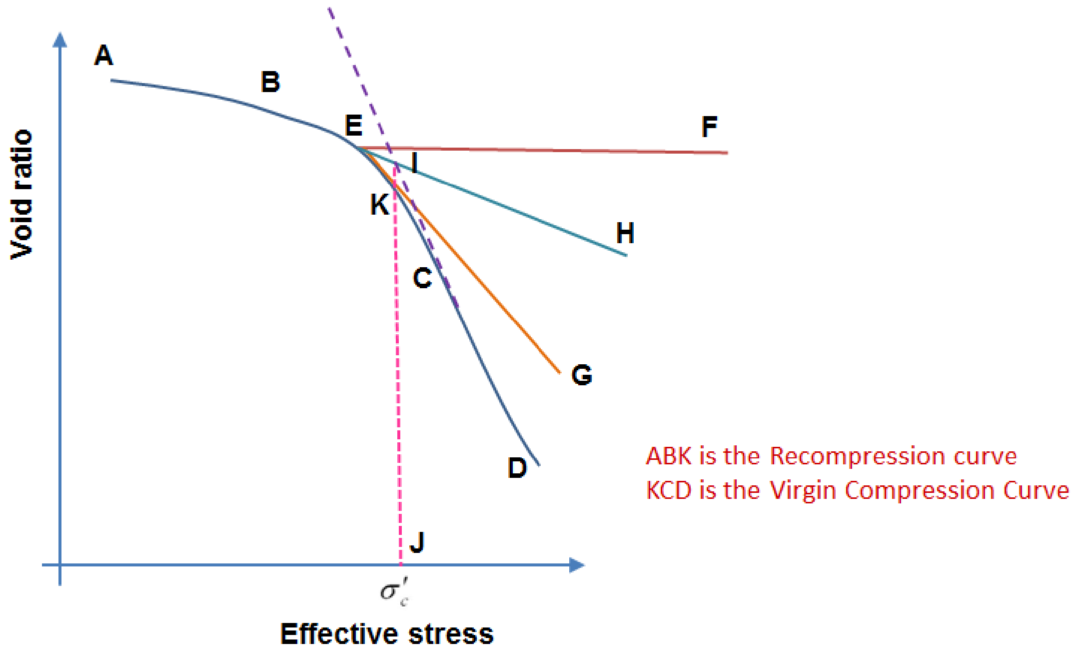
$n_a$= 4.07%

**(b) Define preconsolidation pressure. Explain Casagrande's method of preconsolidation pressure determination.** **[07]**

**Definition-1**
**Figure 3**
**Explanation-3**

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.




| | | |
|---|---|---|
| **(c)** | **Explain Mohr-Coulomb theory of shear strength. Sketch typical strength envelope for a soft clay, clean sand and a silty clay.** | **[05]** |
| | **Mohr Coulomb theory-2**<br>**Typical graphs -3** | |

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. At failure, the Mohr failure envelope will be tangential to the Mohr's circle as shown in Figure 3.a.

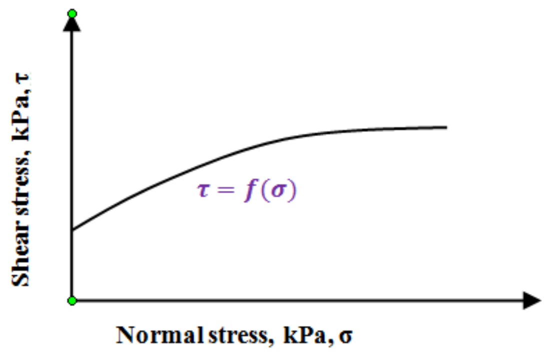
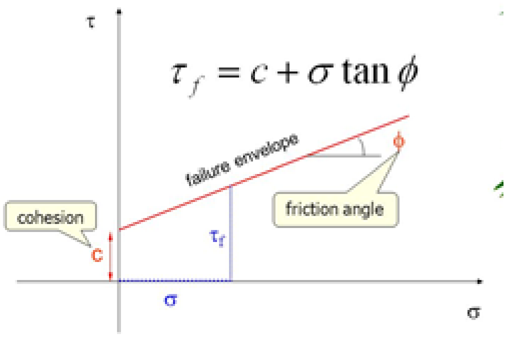



**Figure 1.c.1. Mohr's failure envelope**

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 envelope along with modified failure envelope is given in Figure 3b. Figure 3c shows the Mohr-Coulomb failure envelope for different types of soils.

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



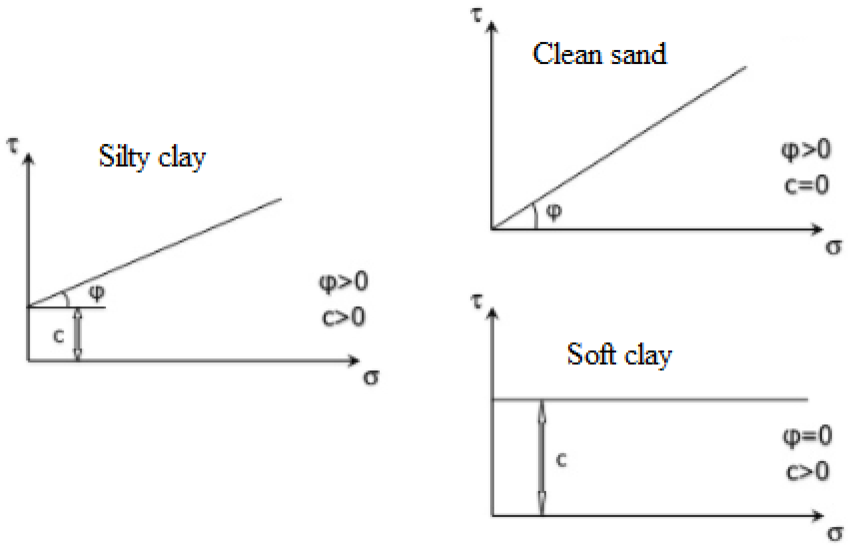
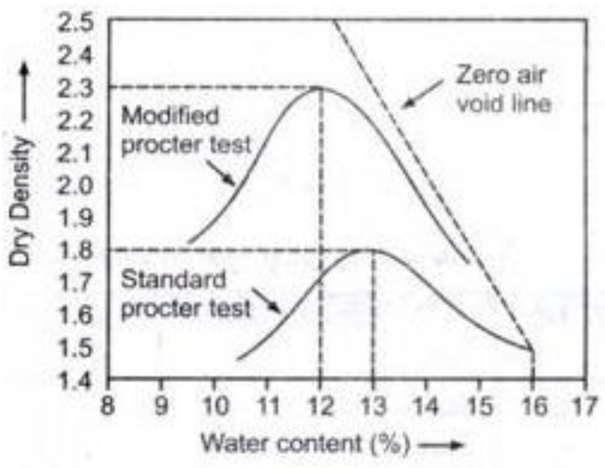




**Figure 1.c.2. Mohr coulomb failure envelope for different soils**

| | | |
|---|---|---|
| 2 (a) | **Differentiate between standard and modified compaction.** | **[05]** |
| | **4 points -4**<br>**Graph-1** | |

| 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 $= \frac{MHN_LN_B}{Volume\ of\ mould}$ kNm/m$^3$) | 593 | 2699 |
| 6 | Practical application | Suitable for embankments | Suitable for airfield pavements |




Effect on OMC and MDD
Increases MDD and decreases OMC when compared to standard compaction test

| | | |
|---|---|---|
| **(b)** | **A sample of sand above the water table was found to have a natural moisture content of 15% and a unit weight of 18.84 kN/m$^3$. Laboratory tests on a dried sample indicated values of $e_{min}$ = 0.50 and $e_{max}$ = 0.85 for the densest and loosest states respectively. Compute the degree of saturation and the relative density. Assume $Gs$ = 2.65.** | **[05]** |
| | **Dry density, Degree of saturation and void ratio -3**<br>**Relative density -2** | |

$\gamma = 18.84\ kN/m^3$

w=15%

$$\gamma_d = \frac{18.84}{1+0.15} = 16.38 kN/m^3$$

$$\gamma_{dmax} = 15 kN/m^3$$

$$\gamma_d = \frac{G\gamma_w}{1+\frac{wG}{S}} = \frac{2.65 \times 10}{1+\frac{0.15 \times 2.65}{S}} = 16.38$$

S=64.3 %

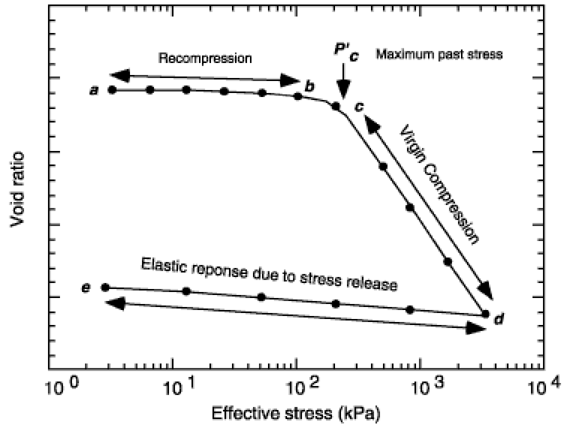
eS=wG

or e=0.618

$$Relative\ density = \frac{e_{max} - e}{e_{max} - e_{min}} = \frac{0.85 - 0.62}{0.85 - 0.50} = 65.7\%$$

| | | |
|---|---|---|
| **(c)** | **Explain NC, OC and UC clay. Also define over consolidation ratio.** | **[05]** |
| | **Figure -1**<br>**Explanation-3**<br>**Definition of OCR -1** | |

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 overconsolidated clay. Such clays exhibit less settlement as indicated by recompression curve.




The preconsolidation 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 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).
Overconsolidation ratio is defined as the ratio of preconsoldiation pressure to the existing pressure.
A soil which has not reached equilibrium under the applied pressure itself is called as underconsoldiated 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

Where $\sigma'_{z0}$ is the existing pressure and $\sigma'_c$ is the preconsolidation pressure.

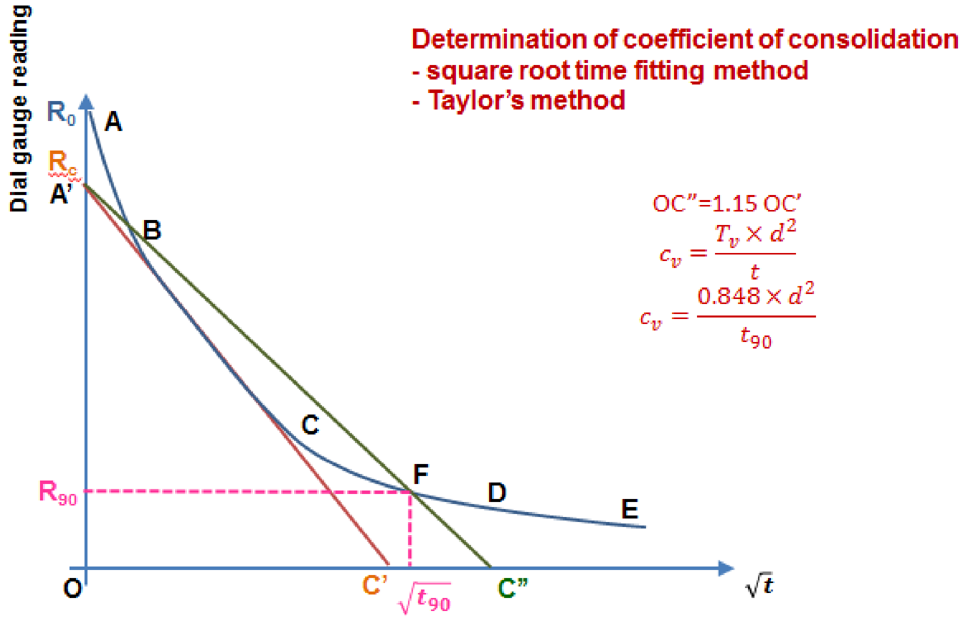
**3 (a)** **How do you determine coefficient of consolidation by Taylor's method.** **[05]**

**Figure 3**
**Explanation – 2**

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 $R_c$.
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 $\sqrt{t_{90}}$. Thus, the time required (t90) for 90% consolidation is calculated.
6. The Coefficient of consolidation (cv) is determined as:

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

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




**(b)** **State assumptions and limitations of Terzaghi's consolidation theory.** **[05]**

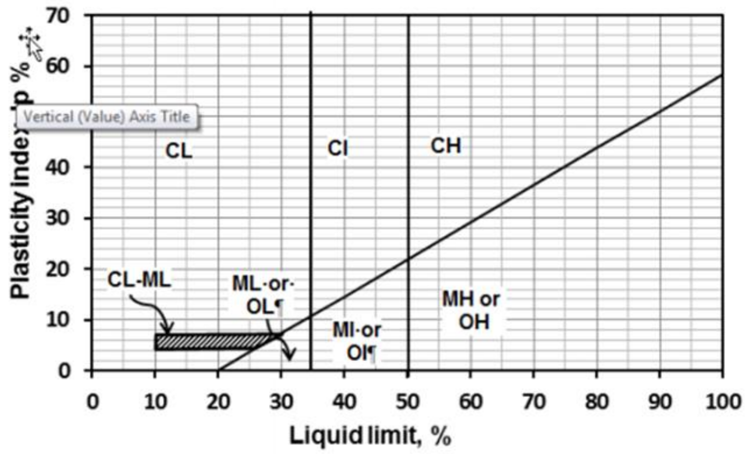
**Assumptions and limitations -3+2**

| Sl No | Assumptions | Limitations |
|---|---|---|
| 1 | Soil is homogenous and isotropic | Coefficient of consolidation never remains constant |
| 2 | Soil is fully saturated | The distance of the drainage path cannot be measured accurately in the field |
| 3 | Consolidation occurs only due to expulsion of excess pore water pressure | Difficulty in locating drainage face |
| 4 | K is constant at all points | In real time consolidation is 3D |
| 5 | Darcy's law is valid throughout the consolidation process | Initial and secondary consolidation are neglected |
| 6 | Consolidation happens only in one direction | Effect of loading period has to be consolidated |
| 7 | Time lag in consolidation is only due to low permeability of soil | Pressure distribution may not be linear or uniform |
| 8 | There exists a unique relationship between void ratio and the effective stress and the relationship remains constant during the load increment | |

**(c)** **Explain plasticity chart with a neat sketch and its use in classification of fine grained soil.** **[05]**

| Soil | Liquid limit, % | Plasticity index | % passing 4.75 mm sieve | % passing 75µ Sieve |
|---|---|---|---|---|
| A | 30 | 10 | 75 | 48 |
| B | 52 | 6 | 98 | 54 |

**Plasticity chart – 3**
**Classification-2**




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).

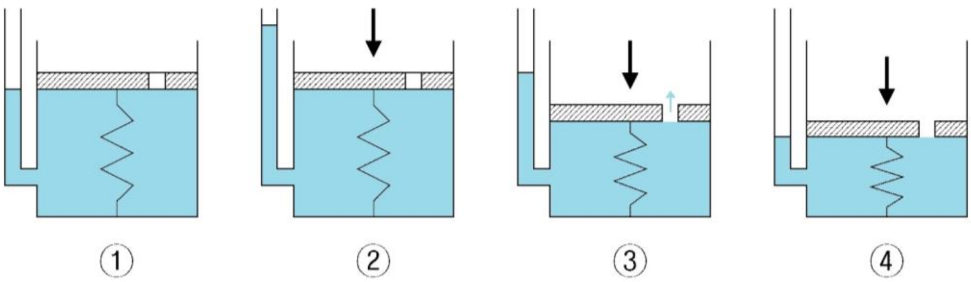
| | | |
|---|---|---|
| Ip above A-line | Wl<35 | CL |
| Ip above A-line | Wl between 35 and 50 | CI |
| Ip above A-line | Wl>50 | CH |
| Ip below A-line | Wl<35 | ML or OL |
| Ip below A-line | Wl between 35 and 50 | MI or OI |
| Ip below A-line | Wl>50 | MH or OH |
| Ip above A-line | Ip between 4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

| Soil type | % of gravel | % of Sand | % of fines | Ip as per soil properties | Ip as per A-line | Soil classification |
|---|---|---|---|---|---|---|
| A | 25 | 27 | 48 | 10 | 7.3 | SC |
| B | 2 | 44 | 54 | 6 | 30.66 | MH or OH |

**4 (a)** **Explain mass spring analogy of consolidation of soils.** **[05]**

**Spring analogy – figure -2**
**Explanation -3**

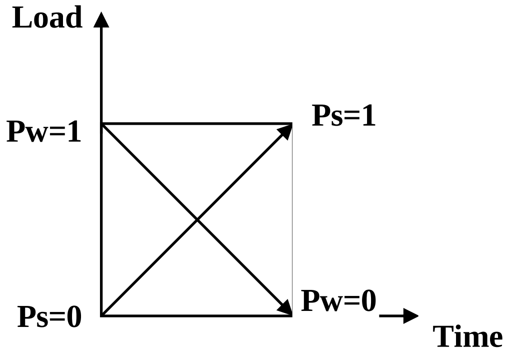
The consolidation process is often explained with an idealized system composed of a spring, a container with a hole in its cover, and water.




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)




| | | |
|---|---|---|
| **(b)** | **Differentiate between compaction and consolidation. The time to reach 30% consolidation of a two way drained saturated clay sample of 12 mm thick in the laboratory is 90 minutes. Determine the time required in years for 75% consolidation of the same soil 11 m thick on an impervious layer subjected to same loading conditions as the laboratory sample.** | **[05]** |
| | **Calculation of Tv -3**<br>**Calculation of time -2** | |

Given: $H_{lab}$=0.012 m ; $d_{lab}$=0.006 m

$H_{field}$=11 m ; $d_{field}$=11 m

$$T_v = \frac{\pi}{4} \times U^2$$

$$T_v = \frac{\pi}{4} \times 0.3^2 = 0.071$$

$$T_v = -0.9331.\log(1-u) - 0.0851 = -0.9331.\log(1-0.75) - 0.0851 = 0.477$$

$$c_v = \frac{0.071 \times 0.006^2}{90} = \frac{0.477 \times 11^2}{t_{75}}$$

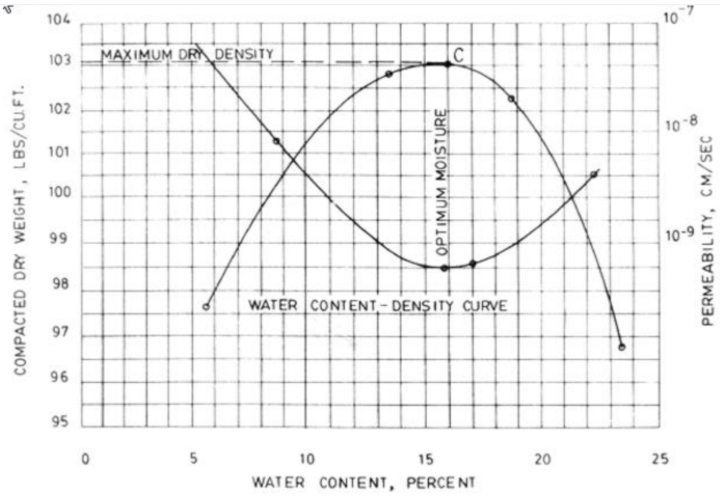
$t_{60} = 2032288732\ min$= 1411311.62 days = 3866.6 years

| | | |
|---|---|---|
| **(c)** | **Discuss the effect of compaction on engineering properties of soils.** | **[05]** |
| | **5 properties with sketches – 5 marks** | |

When a soil is compacted, it changes its engineering properties and thereby behaves differently. Some of the engineering properties which changes on application of compactive effort is briefly described below.

1. Permeability:

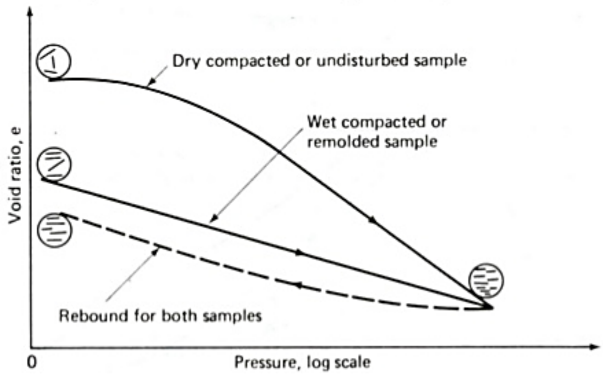
The effect of compaction is to decrease the permeability. In the case of fine grained soils it has been found that for the same dry density soil compacted wet of optimum will be less permeable than that of compacted dry of optimum.




2. Compressibility:

In case of soil samples initially saturated and having same void ratio, it has been found that in low pressure range a wet side compacted soil is more compressible than a dry side compacted soil, and vice versa in high pressure range.

At the high applied stresses the sample compacted on the dry side is more compressible than the sample compacted on the wet side.

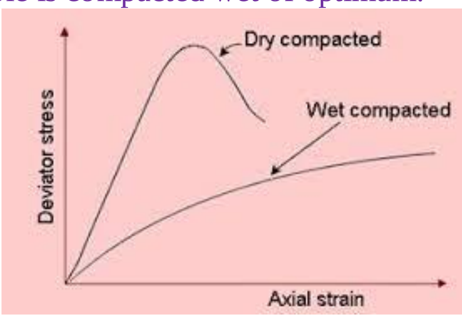



3. Pore pressure

In undrained shear test conducted on saturated samples of clay it has been found that lower pore pressures develop at low strains when the sample is compacted dry of optimum, content.

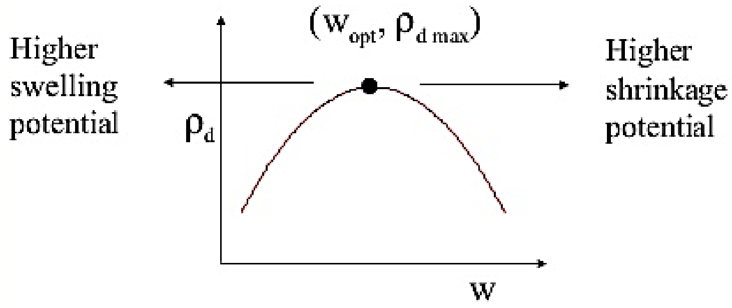
4. Stress-strain relation

Samples compacted dry of optimum produce much steeper stress-strain curves with peaks at low strains, whereas samples compacted wet of optimum, having the same density, produce much flatter stress-strain curves with increase in stress even at high strains compared to the case when the sample is compacted wet of optimum.




5. Shrinkage and swelling:

At same density a soil compacted dry of optimum shrinks appreciably less than that of compacted wet of optimum. Also the sol compacted dry of optimum exhibits greater swelling characteristics than samples of the same density compacted wet of optimum.




6. Soil Structure

Soil compacted at dry of OMC has a flocculated structure. With increase in water the soil gets oriented in a dispersed manner.