Module III

Soil compaction: →

It is compression of soil mass by mechanical means to improve Engineering properties. It is due to escape and compression of air present in the soil mass. Volume reduction occurs due to escape of air under short term loading under constant water content.

→ Due to compaction, permeability, void ratio, compressibility decreases and shear strength increases.

→ Compaction tests are done to determine (i) amount of compaction (ii) the optimum moisture content (OMC) & MDD.

→ Indian Standard light compaction Test (similar to standard proctor Test)

(i) Test results used for, highways, embankments, canal banks.

(ii) Mould volume is 1 litre, soil in 3 layers. Each

593.01 J/m^3 layer is given, 25 hammer blows and height of fall 31 cm.

(iii) Hammer weight 2.6 kg

→ Indian Standard, Heavy compaction Test (similar to modified proctor Test):

(i) Results used for, modern express highways and

2703.88 Runways.

kJ/m^3 (ii) mould capacity 1 litre, soil in 5 layers, each layer 25 hammer blows.

(iii) Hammer weight 4.9 kg and height of fall 45 cm.

→ The compactive effort in the modified proctor test is about 4.56 times, than that in the standard proctor test.

→ If % soil retained on 4.75 mm IS sieve is > 20%, large mould of capacity 2.25 litres is recommended. In such case no. of blows per each layer shall be 56 for both SPT, MPT.

Standard Proctor Test :→

* A compaction curve is, plotted between water content as abscissa and the corresponding dry density as ordinate.

* It is observed that, dry density initially increases with an increase in water content till the maxm density $(\gamma_d)_{max}$ is attained.

* With further increase in water content, the dry density decreases. The water content corresponding to the max^{m} dry density is known as, the optimum water content (OWC) or the optimum moisture content (OMC).

* For a given water content, theoretical max^{m} density, $(\rho_d)_{theomax}$ is obtained corresponding to the condition, when there are no air voids (degree of saturation is equal to 100%). The theoritical max^{m} dry density is also known as, saturated dry density $(\rho_d)_{sat}$

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

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

* The line indicating the theoritical max^{m} dry density can be plotted, along with the compaction curve, It is also known as, zero air void line or 100% saturation line, that doesn't mean that, 10% air void line and 90% saturation line are identical.

Q.5 A Laboratory compaction test, on soil, having specific gravity equal to 2.68 gave a max^m dry density of 1.82 gm/cm^3 and a water content of 17%. Determine the degree of saturation, air content and percentage air-voids at the max^m dry density. What would be, theoretical max^m dry density, corresponding to zero air voids at the optimum water content?

Ans. → $\rho_d = \dfrac{G \rho_w}{1 + \frac{wG}{S}}$ $\Rightarrow S = 0.94 = 94\%$

and $a_c = 1 - S$
$= 1 - 0.94$
$= 0.06 = 6\%$

$\rho_d = \dfrac{(1-\eta_a) G \rho_w}{1 + wG}$ $\Rightarrow \eta_a = 1 - 0.99$
$= 0.01 = 1\%$

when, $\eta_a = 0$, or $(S=1)$
theoretical dry density, at $w' = 17\%$ is given by,

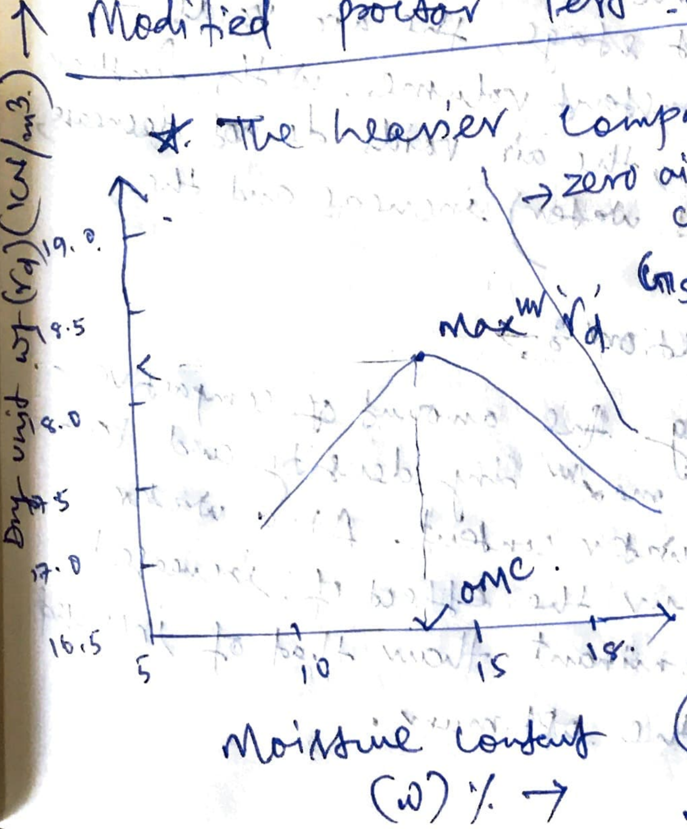
$\rho_d = \dfrac{G\rho_w}{1+wG} = \dfrac{2.68 \times 1}{1 + 0.17 \times 2.68} = 1.84$ gm/cm^3

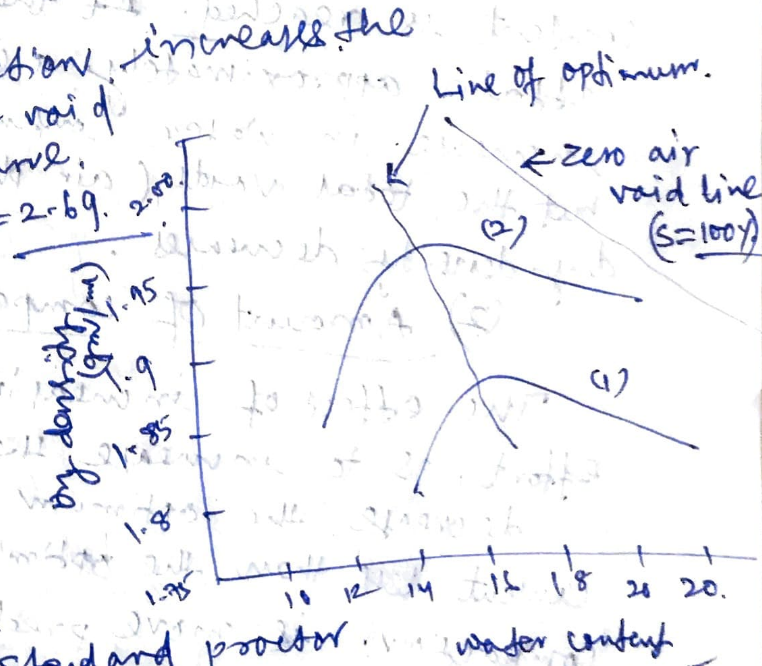
and $\gamma_d = 1.84 \times 9.81 = 18.05$ kN/m^3

Modified proctor Test :→

(1) SPT
(2) MPT

★ The heavier compaction increases the





(Standard proctor compaction test results for a silty clay)

(compaction curves of Standard proctor Test and Modified proctor Test)

max^m dry density but decreases the optimum water content

→ The curve is higher than and to the left of that, obtained from a standard proctor test.

→ The max^m dry density attained, even in the modified proctor test is lower than the theoretical max^m dry density indicated by the zero air-void line.

→ The line of optimum, shown in the fig, joins the points indicating the max^m dry density. It is roughly parallel to the zero air void line.

→ The % age increase in dry density, increase is more for clayey soils than for the sandy soils. (3 to 18%) for most soils.

factors affecting Compaction :→

The increase in dry density depends upon the following factors:

(1) water content :→

The dry density of soil, increases with an increase in the water content, till the optimum water content is 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.

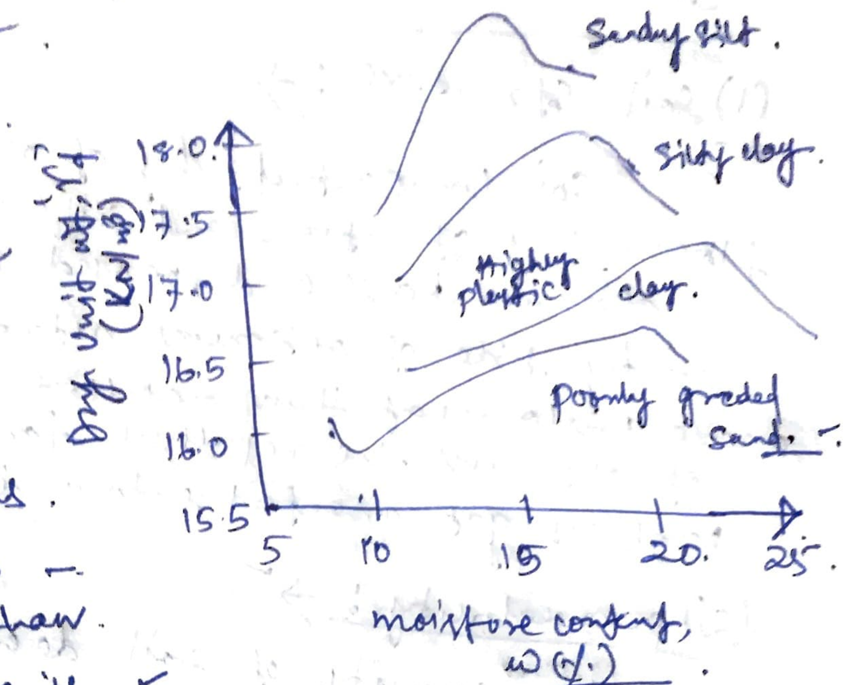
(2) Amount of compaction :→

The effect of increasing the amount of compactive effort, is to increase the max^m dry density and to decrease the optimum water content. At a water content less than the optimum, the effect of increased compaction is more predominant than that of increased compaction more than the optimum.

The line of optimums, corresponds to, air voids of about 5%.

(3) Type of soil :→

The max^m dry density and the optimum water content for different soils are shown.



Coarse grained soils can be, compacted to higher dry density than fine grained soils.

A well graded sand attains attains a much higher dry density than a poorly graded soil.

Heavy clays of very high plasticity have very low dry density and a very high optimum water content.

Addition of even a small quantity of fines to a coarse-grained soil, the soil attains a much higher dry density for the same compactive effort.

(4) Method of compaction :→

For same amount of compactive effort, dry density depends upon method of compaction, such as kneading action, dynamic action or static action.

Soil compacted by the kneading action, the compaction curve obtained is different from, that obtained from, other conventional tests.

(5) Admixture :→ The most commonly used admixtures are, lime, cement and bitumen. which improve the compaction characteristics of the soils.

Effect of compaction on properties of soil :→

(1) Soil structure :→

Soil compacted dry of optimum, have a flocculated structure and soil compacted wet of optimum have a dispersed structure.

On dry side of optimum, attractive forces are more predominant than repulsive forces. On wet side of optimum the repulsive forces increases and particles get oriented into a dispersed structure.

(2) permeability :→

Permeability of soil decreases, with an increase in water content on the dry side of optimum. It reaches min^m value, at approximately the optimum moisture content. Beyond the OMC, the hydraulic conductivity increases slightly. The permeability of dry side of optimum is more than the permeability of wet side of optimum. If the compactive effort is increased, the permeability of soil decreases due to increased dry density and better orientation of particles.

(3) Swelling :→

Soil compacted dry of optimum imbibes more water than the sample compacted wet of the optimum and therefore, more swelling.

(4) pore water pressure :→

The pore water pressure developed for the soil compacted dry of the optimum, is therefore less than that for the same soil compacted wet of optimum.

(5) Shrinkage :→

Soils compacted dry of the optimum shrink less on drying compared with those compacted wet of the optimum. On shrinkage, soils compacted wet of the optimum can pack more efficiently.

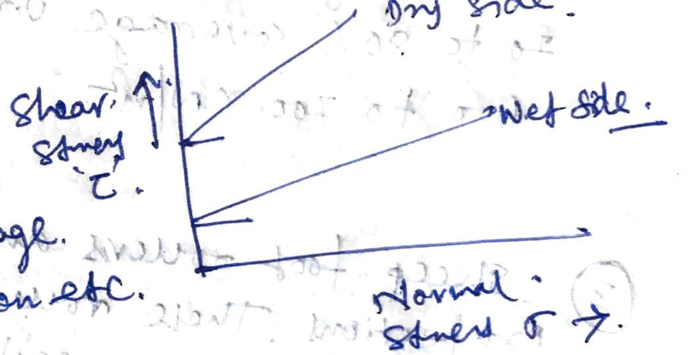
(6) Compressibility :→

The flocculated structure, developed on the dry side of the optimum, offers greater resistance to compression than the dispersed structure on the wet side. Consequently, the soils on the dry side are less compressible.

(7) Shear strength :→

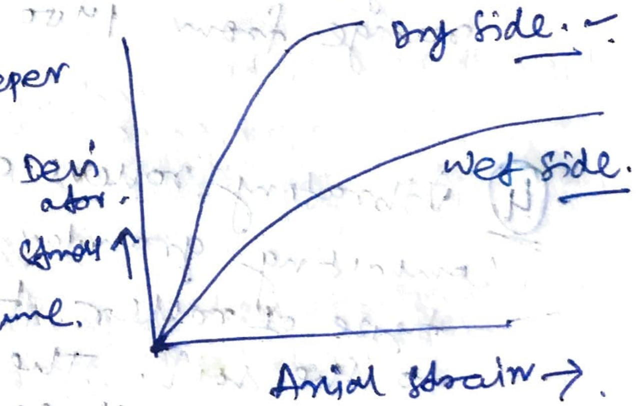
For same values of dry densities, soils compacted, dry of optimum have higher shear strength than that of wet of optimum, at low strains. on the wet side, shear strength further reduced by kneading action than that of static compaction for same amount of compactive effort.

The shear strength, of the compacted soils depends upon the soil type, moulded water content, drainage conditions, method of compaction etc.



(8) Stress - strain relationships :→

The soils compacted, dry of the optimum, have a steeper stress-strain curve than those on the wet side & modulus of elasticity is therefore high. Such soils have brittle failure. Like dense sands or over-consolidated clays.



(Stress-strain curves)

Field compaction.

Most of the compaction, in the field, is done with rollers. The four most common types of rollers are:→

(1) Smooth wheel rollers. (or Smooth drum rollers).
(2) pneumatic rubber-tired roller.

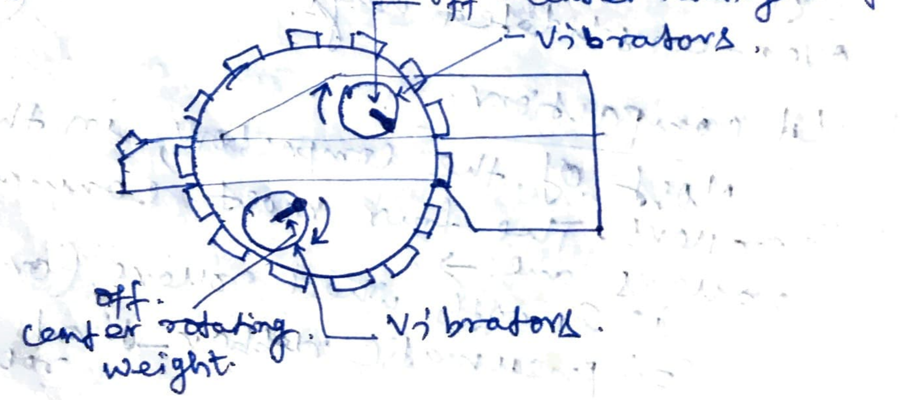
3. Sheep foot rollers. 4. Vibratory rollers.

(1) Smooth-wheel rollers are suitable for proof rolling subgrades and for finishing operation of fills with sandy and clayey soils.
100% coverage under wheels and contact pressure 310 to 380 kN/m^2.

(2) Pneumatic rubber-tired rollers are heavily loaded with several rows of tires. These tires are closely spaced - four to six in a row. These can be used for sandy and clayey soil compaction. Compaction is acheived by combination of pressure and kneading action.
70 to 80% coverage under wheels and contact pressure 600 to 700 kN/m^2

(3) Sheep foot rollers are drum with a large no. of projections. These rollers are most effective in compacting clayey soil. The area of each projection range from 25 to 85 cm^2. The contact pressure range from 1400 to 7000 kN/m^2.

(4) Vibratory rollers are extremely efficient in compacting granular soils. Vibrators can fit any type of roller to provide vibratory effect to the soil. The vibration is produced by rotating off-center weights.



(principles of vibratory rollers)

Module → III

The property of soil, due to which, a decrease in volume occurs, under compressive force is known as the compressibility of soil.

The compression is caused by (i) Elastic deformation and relocations of soil particles.

(ii) Explusion of water or air from the void spaces.

* The compression of a saturated soil, under a steady static pressure is known as Consolidation. It is entirely due to explusion of water from the voids.

Consolidation	Compaction
1. Gradual process of reduction of volume under sustained, static loading.	1. Compaction is a rapid process of reduction of volume by mechanical means such as rolling, tamping and vibration.
2. It causes a reduction in volume of a saturated soil due to squeezing out of water from the soil.	2. Volume of a partially saturated soil decreases because of explusion of air from the voids, at the unaltered water content.
3. Consolidation is a process which occurs in nature when the saturated soil deposits are subjected to static loads caused by the weight of the buildings and other structures.	3. Compaction is an artifical process which is done to increase the density (unit wt) of the soil to improve its properties, before it is put to any use.
4. It is required for the construction of earth dam, canal embankments, highways, runways and in many other engineering applications.	4.

The soil settlement, caused by loads, may be divided into three broad categories:→

1. Elastic settlement (or immediate settlement) which is caused by elastic deformation of dry soil and of moist and saturated soils without any change in the moisture content.
2. primary consolidation settlement, which is the result of a volume change in saturated cohesive soils bcz of explusion of the water that occupies the void-spaces.
3. Secondary consolidation settlement, which is observed in saturated cohesive soils and organic soil and is the result of the plastic adjustment of soil fabrics.

The total settlement of a foundation can then be given as,

$$S_T = S_p + S_s + S_e$$

where, S_T = total settlement.
S_p = primary consolidation settlement.
S_s = secondary consolidation settlement
S_e = elastic settlement.

When foundation are constructed on very compressible clays, the consolidation settlement can be several times greater than the elastic settlement.

Consolidation Test:→

The test is performed in the consolidation test apparatus, known as the consolidometer or an oedometer.

It consist of, loading device and a cylindrical container, called consolidation cell. The consolidation cells are of two types:→

(1) Floating or free ring cell.
(2) Fixed ring cell.

In (1) top porous stone moving downward. and bottom porous stone moving upward.

In (2) the top porous stone moving downward. and bottom porous stone cannot move. It is also used as a variable head permeability test apparatus.

→ The internal diameter of cell varies from 60 mm to 100 mm.

→ Thickness of the sample, for 60 mm dia cell is 20 mm. specimen of dia, 50, 70 may be used in special cases.

→ Before conducting the test, porous stones are boiling in distilled water, about 15 minuts.

→ Bottom porous stone is first placed in the cell and. Ring containing filter paper is fixed on the porous stone. the sample is then placed on the bottom porous stone.

→ Another filter paper, is kept on the top of the sample and then, top porous stone is placed. The loading pad is placed, on the top porous stone. cell is kept under loading unit. The dial gauge is mounted and adjusted.

→ The mould assembly is connected to the water reservoir. to saturate the sample. and level of water in the reservoir. should be approximately same as that of sample.

→ Initial setting pressure of 5.0 kN/m^2 (for soft soil 2.5 kN/m^2) is applied to the sample.

→ It is usual practice to double the previous load in each increment. i.e., usually applied are 20, 40, 80, 160, 320 and 640 kN/m^2 etc.

→ The max^m load intensity is governed by actual loading on the soil in the field, after the construction of the structure.

→ consolidation under final load increment is complete, load is reduced to, one-fourth of the final load and allowed to stand for 24 hrs.

→ For each load increment, dial gauge readings are taken after 0.25, 1.0, 2.25, 4.0, 6.25, 9.0, 12.25, 16.0, 20.25, 25, 36, 49, 64, 81, 100, 121, 144, 169, 196, 225, 289, 324, 400, 500, 600 and 1440 minutes. Sometimes after 49 mints, readings are taken at 1, 2, 4, 8, 10 and 24 hrs.

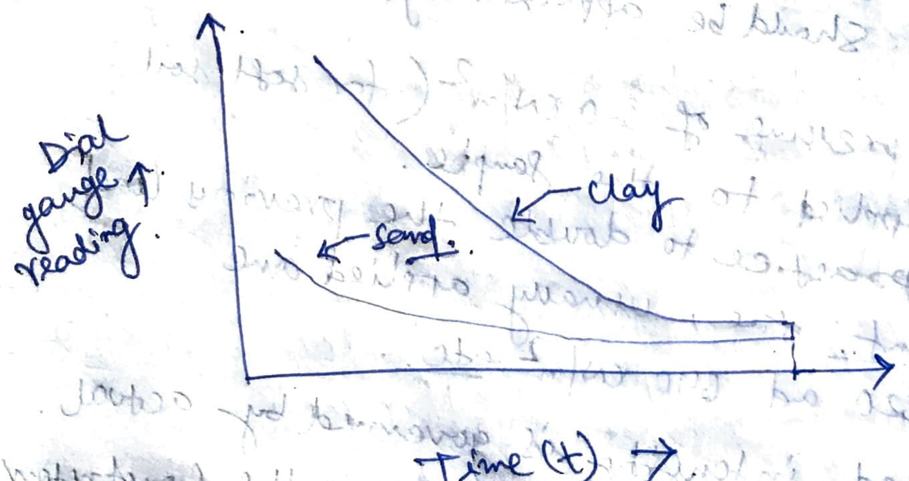
After reduction,

→ The sample takes water and swells. The reading of the dial gauge is taken, when the swelling is complete. The load is further reduced to one-fourth intensity (40 kN/m^2) and swelling recorded after 24 hrs. In this way for each one fourth reduction, swelling is noted after 24 hrs. Throughout the test, the container gutter should be kept filled with water.

→ After completing the test, the sample is taken out and water content is determined.

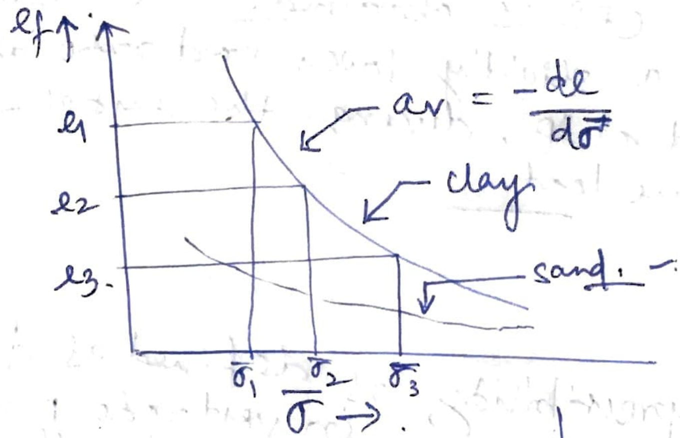
Consolidation Test results :→

(1) Dial gauge reading - time plot :→

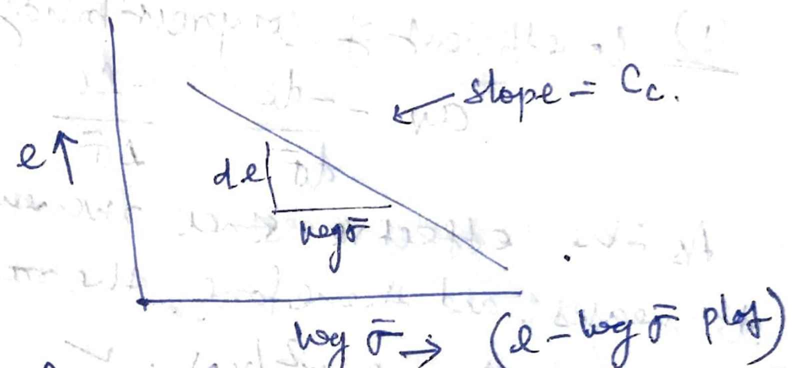


The plot between the dial gauge reading and time is required for determining the co-efficient of consolidation, which is useful for obtaining the rate of consolidation in the field.

(2) Final void ratio – effective stress plot :→



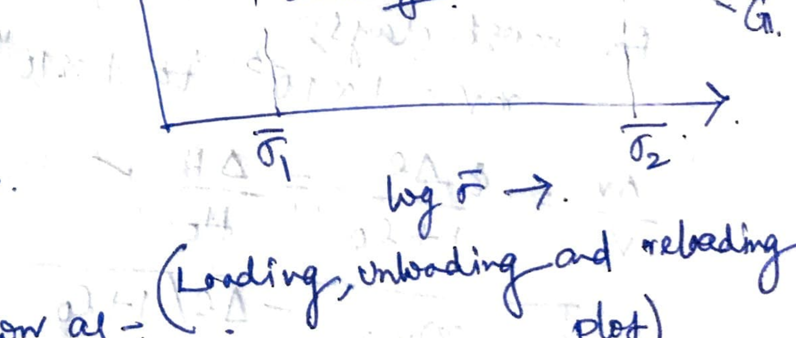
(Final void ratio – $\bar\sigma$ plot)



(e – log $\bar\sigma$ plot)

The plot between the final void ratio, and the effective stress is required for, determination of the magnitude of the consolidation settlement in the field. The slope decreases, with an increase in effective stress. The plot is, practically a st line for a normally consolidated clay, with the range of pressure.

(3) Unloading and Reloading plot :→



(Loading, unloading and reloading plot)

The curve 'AB' indicates, decrease in void ratio, with an increase in the effective stress. It is Loading curve.

After reaching eq^n at effective stress $\bar\sigma_2$, the curve BEC is obtained in Unloading. This is known as Expansion curve or swelling curve. Soil, can not attain the void ratio, existing before the starting of the test, and there is some permanent set or residual deformation.

The specimen, from point 'C' is reloaded. the recompression curve CFD is obtained. The reloaded specimen remains at a slightly lower void ratio at point 'D' than that attained at 'B', during the initial compression for the same load.

Basic definitions :-

(1) Co-efficient of compressibility, is defined as decrease in void ratio per unit increase in effective stress.

$$a_v = \frac{-de}{d\bar{\sigma}} = \frac{-\Delta e}{\Delta\bar{\sigma}}$$

As the effective stress increases, the void ratio decreases, and therefore the ratio $de/d\bar{\sigma}$ is negative.

The unit are m^2/kN.

(2) Co-efficient of volume change (or the co-efficient of volume compressibility) is defined as the volumetric strain, per unit increase in effective stress.

i.e., $m_v = \frac{-\Delta V/V_0}{\Delta\bar{\sigma}}$ — m_v decreases, with an increase in the effective stress.

where,

m_v = co-efficient of volume change.

V_0 = Initial volume.

ΔV = change in volume.

$\Delta\bar{\sigma}$ = change in effective stress.

For most clays,

$m_v = 1\times10^{-3}$ to 1×10^{-4} m^2/kN.

$$\frac{\Delta V}{V_0} = \frac{\Delta e}{1+e_0} = \frac{\Delta H}{H_0}$$

$$\boxed{\therefore m_v = \frac{-\Delta e/1+e_0}{\Delta\bar{\sigma}} = \frac{-\Delta H/H_0}{\Delta\bar{\sigma}}}$$

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

The co-efficient of volume change m_v is more commonly used, in practice than the co-efficient of compressibility a_v.

Compression Index: →

(3) The compression index (C_c) is equal to the slope of the linear portion of the void ratio vs $\log \bar{\sigma}$ plot.

Thus, $$C_c = \frac{-\Delta e}{\log_{10}(\bar{\sigma}/\bar{\sigma}_0)} = \frac{-\Delta e}{\log_{10}\left(\frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_0}\right)}$$

where, $\bar{\sigma}_0$ = initial effective stress.

$\bar{\sigma}$ = final effective stress.

Δe = change in void ratio.

For clays of low to medium sensitivity, i.e. $S_t \le 4$.

(a) for undisturbed soils, $C_c = 0.009(w_L - 10)$

for remoulded soils, $C_c = 0.007(w_L - 10)$.

where, w_L = Liquid limit (%).

The value of C_c normally varies between 0.30 for highly plastic clays and 0.075 for low plastic clays.

$C_c = 0.54(e_0 - 0.35)$ e_0 is initial void ratio.

$C_c = 0.0054(2.6 w_0 - 35)$ w_0 is initial water content.

The co-efficient of compressibility

$$a_v = 0.435 \frac{C_c}{\bar{\sigma}_a}$$ where, $\bar{\sigma}_a$ = average pressure for the increment.

(4) Expansion Index: →

During unloading, BEC, the expansion index or swelling index is,

$$C_e = \frac{-\Delta e}{\log_{10}\left(\frac{\bar{\sigma} + \Delta\bar{\sigma}}{\bar{\sigma}}\right)}$$

Expansion index is, much smaller than the compression index.

(5) Recompression. Index :→

During, reloading, CFD, the recompression index (C_r) is.

$$C_r = \frac{-\Delta e}{\log_{10}\left(\frac{\bar{\sigma} + \Delta\bar{\sigma}}{\bar{\sigma}}\right)}$$

Load during recompression is less than the load, to which the soil has been subjected previously.

The recompression index is appreciably smaller than the compression index C_c.

$\therefore C_r \cong \frac{1}{5}$ to $\frac{1}{10}$ C_c.

(6) Normally Consolidated clays :→

A normally consolidated soil is one, which had, not been subjected to a pressure greater than the present existing pressure.

The portion AB of the curve, represents the soil in, normally consolidated contions and is also called, virgin compression curve.

(7). over consolidated clays :→

A soil is said to be, over-consolidated, if it had been subjected in the past to a pressure in excess of the present pressure.

The soil range CD, when it is recompressed, represents, over consolidated contions.

NB:1) * At point 'C', over consolidation ratio is $\frac{\bar{\sigma}_2}{\bar{\sigma}_1}$

* The max^{m} pressure, to which, an over consolidated soils, had been subjected,

in the past, divided by the present pressure is known as the, over consolidation ratio (OCR).

The NCS & OCS are not diff^nt types of soils but these are conditions, in which a soil exist.

$0.6 \leq I_L \leq 1.0 \rightarrow NCC$

$0.0 \leq I_L \leq 0.6 \rightarrow OCC$.

(8). The settlements, of the structures, built on, over-consolidated clays are small.

Under consolidated clays :→

If the clay deposit has not reached equilibrium, under the applied overburden loads, it is said to be, under consolidated. This normally occurs in areas of recent land fill.

Terzaghi's Theory of consolidation :→

Terzaghi's gave the theory for the determination of rate of consolidation of a saturated soil mass, subjected to static, steady load.

Assumptions :→

(1) The soil is homogeneous & isotropic.
(2) Soil is saturated.
(3) The co-efficient of permeability of the soil, has the same value, at all points, and it remains constant, during the entire period of consolidation.
(4) Darcy's Law is valid, throughout the consolidation process.
(5) Soil is laterally confined, and the consolidation takes place, only in axial direction. Drainage of water also occurs only in the vertical direction.
(6) There is unique relationship between, the void ratio, and the effective stress, and this relationship remains constant, during the load increment.

The basic differential eqⁿ of one-dimensional - Consolidation, is,

$$\boxed{C_v \frac{\partial^2 \bar{u}}{\partial z^2} = \frac{\partial \bar{u}}{\partial t}}$$

where, 'C_v' is the co-efficient of consolidation, $= \frac{K}{m_v \gamma_w} = \frac{K}{m_v g \rho_w}$.

It gives the distribution of hydrostatic excess pressure, $\bar{u}$ with depth 'z' and time 't'.

NB :→ The curve indicating the distribution of excess - hydrostatic pressure are, known as isochrones.

· The hydraulic gradient, at any point is equal to the - slope of the isochrone at that point.

The average degree of consolidation (U) is defined as,

$$U = \frac{U_i - U_t}{U_i}$$

where, U_i = Initial excess hydrostatic pressure, over the entire depth.

U_t = Average excess hydrostatic pressure, after time 't' over the entire depth.

* The solution of the above differential eqⁿ, gives Time factor (T_v) and Degree of consolidation.

where, $T_v = \frac{C_v t}{d^2} = \left(\frac{K}{m_v \gamma_w}\right) \times \frac{t}{d^2}$

* As the consolidation progresses, both co-efficient of permeability (K) and the co-efficient of volume change, (m_v), decreases, but the ratio (K/m_v) remains constant, for a considerable range of pressure.

* For a given soil & for given void ratio 'C_v' increases with increase magnitude of the consolidating pressure.

Co-efficient of consolidation, (C_v) remains almost constant.

$d = H/2$, H = thickness of layer. for open layer.

$d = H$, for half closed layer.

T_v & U also depends upon the distribution of pressure, across the thickness.

$T_v = \frac{\pi}{4} U^2$ (for $U < 0.60$)

$T_v = 1.781 - 0.933 \log_{10}(100 - U\%)$. for, ($U \geq 0.60$)

Determination of co-efficient of consolidation: →

(1) square root of time method.
(2) Logarithim of time method.

(1) square root of time method. : →

R_c = corrected zero reading.

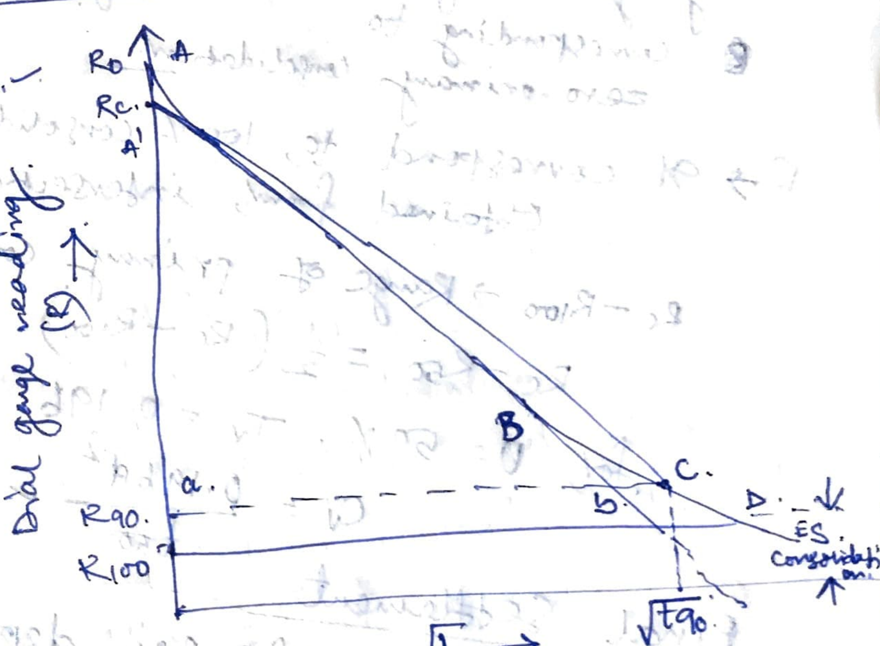
Consolidation between R_0 and R_c is the initial compression.

$C_a = 1.15\, ba$

R_{90} = 90% of U.

point C:

$x = \sqrt{t_{90}}$, $y = R_{90}$



square root of time plot.

DE range is secondary consolidation.

$R_c - R_{100} = \frac{10}{9}(R_c - R_{90})$

from, R_{100} 'D' pt can be evaluated.

For $U = 90\%$. $T_v = 0.848$.

$\therefore C_v = \frac{T_v d^2}{t} = \frac{0.848 d^2}{(\sqrt{t_{90}})^2} = \frac{0.848 d^2}{t_{90}}$.

where, $d = \frac{1}{2}\left(\frac{H_i + H_f}{2}\right)$. Double Drainage

$d = \left(\frac{H_i + H_f}{2}\right)$. Single Drainage.

where, H_i = Initial thickness.

H_f = Final thickness.

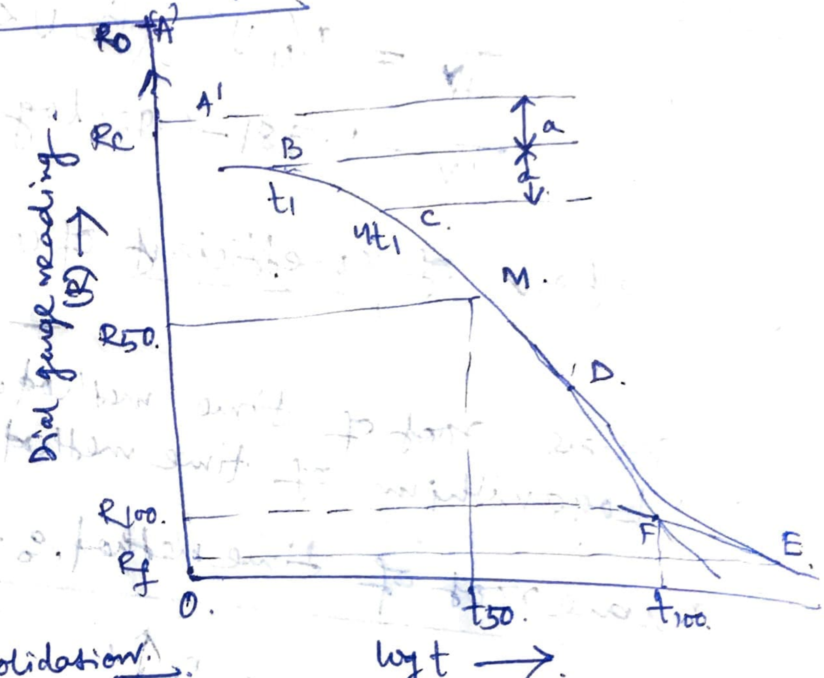
(2) Logarithm of time method :→

curve consists of,

(i) An initial portion, parabolic in shape.

(ii) A middle portion which is almost linear.

(iii) last portion of curve.



R_c → corrected dial gauge reading - corresponding to zero primary consolidation.

F → It correspond to, 100% consolidation, obtained from, intersection of two linear parts.

$R_c - R_{100}$ → Range of primary consolidation.

$R_c - R_{50} = \frac{1}{2}(R_c - R_{100})$

for, $U = 50\%$, $T_v = 0.196$.

$C_v = \frac{0.196 d^2}{t_{50}} = \frac{T_v d^2}{t}$

Final settlement of a soil deposit in the field :→

Settlement of soil deposit consist of two parts :

(i) Computation of magnitude of, final settlement.

(ii) Determination of, time rate of settlement.

* For final settlement, co-efficient of volume change or compression index is required.

* For time rate of, computation co-efficient of consolidation is required.

(1) Final. Settlement using Co-efficient of volume change:→

$S_f = m_v \Delta\bar{\sigma} \, H_o$

or $S_f = (m_v)_m (\Delta\bar{\sigma})_m H_o$.

where, $(m_v)_m$ and $(\Delta\bar{\sigma})_m$ are the values at mid-depth.

(2) Final settlement using void ratio:→

$\Delta H = H_o\left(\dfrac{\Delta e}{1+e_o}\right) \Rightarrow S_f = H_o \cdot \left(\dfrac{\Delta e}{1+e_o}\right)$

where e_o is the initial void ratio.

(a). Normally consolidated soils:→

Compression index of normally consolidated soil is constant.

$\therefore C_c = \dfrac{-\Delta e}{\log_{10}(\bar{\sigma}_o + \Delta\bar{\sigma} / \bar{\sigma}_o)}$.

$\Rightarrow -\Delta e = C_c \log_{10}(\bar{\sigma}_o + \Delta\bar{\sigma} / \bar{\sigma}_o)$.

$$\boxed{\Rightarrow S_f = \frac{C_c}{1+e_o} \cdot H_o \cdot \log_{10}\left(\frac{\bar{\sigma}_o + \Delta\bar{\sigma}}{\bar{\sigma}_o}\right)}$$

(b) preconsolidated soils:→

As 'C_r' is considerably smaller than the compression-index 'C_c', final settlements are small in case of preconsolidated soils as compared to normally consolidated soils.

i.e., $-\Delta e = C_r \log\left(\dfrac{\bar{\sigma}_o + \Delta\bar{\sigma}}{\bar{\sigma}_o}\right)$

$\Rightarrow S_f = \dfrac{C_r}{1+e_o} \cdot H_o \cdot \log\left(\dfrac{\bar{\sigma}_o + \Delta\bar{\sigma}}{\bar{\sigma}_o}\right)$.

This is valid, where $\bar{\sigma}_o + \Delta\bar{\sigma} < \bar{\sigma}_c$ (pre consolidation pressure)

when, $\underbrace{\bar{\sigma}_o < \bar{\sigma}_c}_{\text{Recompression}} \underbrace{< \bar{\sigma}_o + \Delta\bar{\sigma}}_{\text{Compression}}$.

$$\therefore S_f = \frac{C_r}{1+e_0} \cdot H_0 \cdot \log_{10}\left(\frac{\bar{\sigma}_c'}{\bar{\sigma}_0'}\right) + \frac{C_c}{1+e_0} \cdot H_0 \cdot \log\left(\frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_c}\right)$$

Secondary consolidation.

The magnitude of secondary consolidation is given by,

$$S_s = C_\alpha \times H \times \log_{10}\left(\frac{t_2}{t_1}\right)$$

$$= \frac{C_t}{1+e_p} \times H \times \log_{10}\left(\frac{t_2}{t_1}\right)$$

$$= \frac{\Delta e}{1+e_p} \times \frac{1}{\log_{10}\left(\frac{t_2}{t_1}\right)} \times$$

Problems :-

BMDas 383

Q1 The following are the results of a laboratory consolidation test:

pressure, $\bar{\sigma}'$ (kN/m²)	void ratio, 'e'	Remarks	pressure $\bar{\sigma}'$ (kN/m²)	void ratio, 'e'	Remarks
25	0.93	Loading	800	0.61	Loading
50	0.92		1600	0.52	
100	0.88		800	0.535	unloading
200	0.81		400	0.555	
400	0.69		200	0.57	

(a) Draw an, $e-\log\bar{\sigma}_0'$ graph and determine the preconsolidation pressure $\bar{\sigma}_c'$

(b) Calculate C_c and ratio of $\frac{C_{se}}{C_{ac}}$

(c) On the basis of the average, $e-\log\bar{\sigma}$ plot, Calculate the void ratio at $\bar{\sigma}_0' = 1000$ kN/m².

Ans :→ (a). From the plot of $e - \log \bar{\sigma}$, casagrande's graphic procedure is,

$\sigma_c' = 120\ kN/m^2$.

(b). for loading branch :→

$$C_c = \frac{-\Delta e}{\log_{10}\left[\bar{\sigma}/\bar{\sigma}_0\right]}$$

$\sigma_1 = 800,\ e_1 = 0.61$
$\sigma_2 = 1600,\ e_2 = 0.52$

$$= \frac{-(e_2 - e_1)}{\log \frac{\sigma_2'}{\sigma_1'}} = \frac{e_1 - e_2}{\log \frac{\sigma_2'}{\sigma_1'}}$$

$$= \frac{0.81 - 0.69}{\log_{10}\left(\frac{400}{200}\right)} = 0.4 \quad \text{or,} \quad \frac{0.61 - 0.52}{\log_{10}\left(\frac{1600}{800}\right)} = 0.3$$

for unloading branch :→

$$C_e = \frac{-\Delta e}{\log_{10}\left(\frac{\bar{\sigma}}{\bar{\sigma}_0}\right)}$$

$\sigma_1 = 400,\ e_1 = 0.555$
$\sigma_2 = 200,\ e_2 = 0.57$

$$= \frac{-(e_2 - e_1)}{\log\left(\frac{\sigma_2'}{\sigma_1'}\right)}$$

$$= \frac{0.555 - 0.57}{\log_{10}\left(\frac{200}{400}\right)} = 0.05$$

$$\frac{C_e}{C_c} = \frac{0.05}{0.4} = 0.125 \quad \text{or,} \quad \frac{C_e}{C_c} = \frac{0.05}{0.3} = 0.17$$

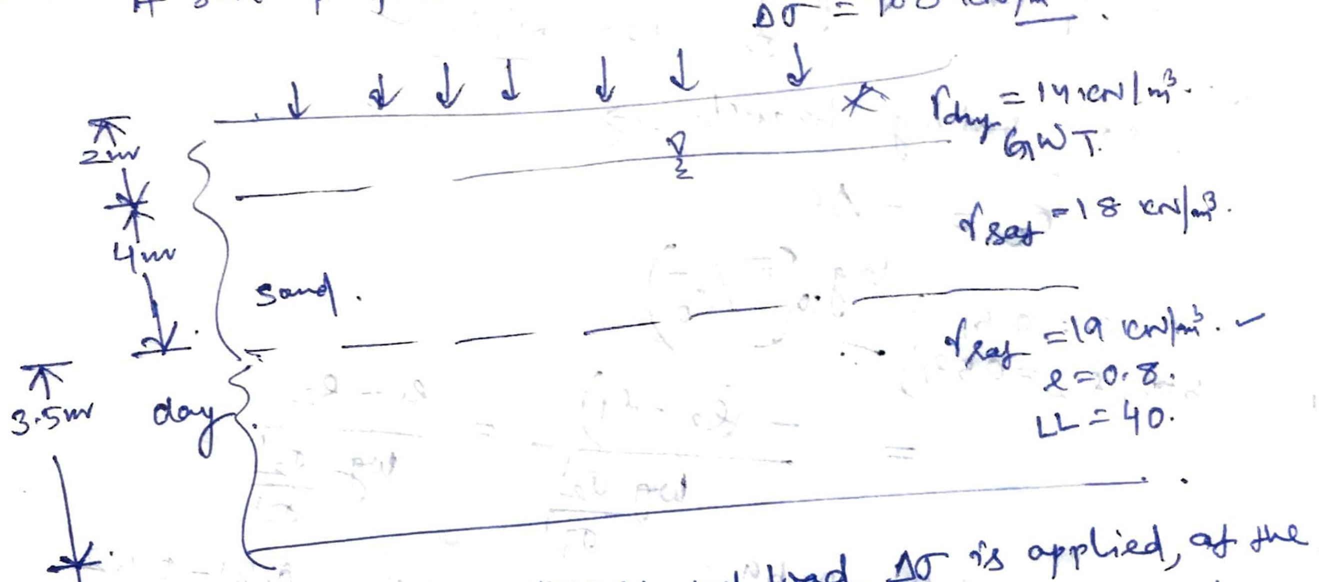
(c). $$C_c = \frac{e_1 - e_2}{\log \frac{\sigma_2'}{\sigma_1'}} \Rightarrow 0.3 = \frac{0.61 - e_2}{\log\left(\frac{1000}{800}\right)}$$

$$\Rightarrow e_2 = 0.61 - 0.3 \times \log(1.25)$$

$= 0.58$ Ans

BMDas
385 Q.2

A soil profile is shown in fig,



If the uniformly distributed load, $\Delta\sigma$ is applied, at the ground surface, what is the settlement of the clay-layer caused by primary consolidation, if,

a. The clay is normally consolidated. use $C_r \simeq \frac{1}{5} C_c$.

b. The preconsolidation pressure $\sigma_c' = 200\ kN/m^2$

use, $C_s \simeq \frac{1}{5} C_c$. $\sigma_c' = 150\ kN/m^2$.

Ans :→ The avg. eff stress, at the middle of clay layer is,

ⓐ $\sigma_0 = 2 \times \gamma_{dry} + 4 \left[\gamma_{sat(sand)} - \gamma_w\right] + \frac{3.5}{2}\left[\gamma_{sat(clay)} - \gamma_w\right]$

$\Rightarrow \sigma_0 = 2 \times 14 + 4(18 - 9.81) + 1.75(19 - 9.81)$

$= 76.08\ kN/m^2$

For Normally Consolidated clay,

$$S_f = \frac{C_c}{1+e_0} \cdot H_0 \cdot \log_{10}\left(\frac{\sigma_0 + \Delta\sigma}{\sigma_0}\right)$$

where

$C_c = 0.009(LL - 10)$

$= 0.009(40 - 10)$

$= 0.27$

$$= \frac{0.27 \times 3.5}{1 + 0.8} \times \log\left(\frac{76.08 + 100}{76.08}\right)$$

$= 0.191\ m = 191\ mm$

(b) $\bar{\sigma}_0 + \Delta\bar{\sigma} = 76.08 + 100$

$= 176.08 \text{ kN/m}^2$

$\sigma_c' = 200 \text{ kN/m}^2$

$C_r = \frac{C_c}{5} = \frac{0.27}{5} = 0.054$

$\bar{\sigma}_0 + \Delta\bar{\sigma} < \sigma_c'$

Hence, for pre-consolidated soil,

$$S_f = \frac{C_r}{1+e_0} \cdot H_0 \cdot \log\left(\frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_0}\right)$$

$$= \frac{0.054}{1+0.8} \times 3.5 \times \log\left(\frac{76.08+100}{76.08}\right)$$

$$= 0.038 \text{ m} = 38 \text{ mm}$$

(c) $\bar{\sigma}_0 + \Delta\bar{\sigma} = 176.08 \text{ kN/m}^2 > \bar{\sigma}_c = 150 \text{ kN/m}^2$

$\therefore \sigma_0' < \bar{\sigma}_c' < \bar{\sigma}_0 + \Delta\bar{\sigma}$

i.e., $76.08 < 150 < 176.08$

$$\therefore S_f = \frac{C_r}{1+e_0} H_0 \log\left(\frac{\sigma_c'}{\bar{\sigma}_0}\right) + \frac{C_c}{1+e_0} H_0 \log\left(\frac{\sigma_0' + \Delta\bar{\sigma}}{\bar{\sigma}_c}\right)$$

$$= \frac{0.054}{1+0.8} \times 3.5 \log\left(\frac{150}{76.08}\right) + \frac{0.27}{1.8} \times 3.5 \times \log\left(\frac{176.08}{150}\right)$$

$$= 0.0675 \text{ m} = 67.5 \text{ mm}$$

BM Das 399 Q.3

For a normally consolidated, laboratory clay specimen, drained on both sides, the following are given:

$\sigma_0' = 150 \text{ kN/m}^2$, $e = e_0 = 1.1$

$\sigma_0' + \Delta\sigma' = 300 \text{ kN/m}^2$, $e = 0.9$

Thickness of clay specimen = 25 mm.

Time for 50% consolidation = 2 min.

a. Determine the hydraulic conductivity (m/min) of the clay, for the loading range.

b. How long (in days) will it take for a 1.8 m clay layer in the field (drained on one side) to reach 60% consolidation?

Ans:→ ⓐ

The co-efficient of volume change,

$$m_v = \frac{a_v}{1+e_0} = \frac{\frac{-\Delta e}{\Delta \sigma'}}{1+e_0}$$

$\Delta e = e - e_0$
$= 0.9 - 1.1$
$= -0.2$.

$$= \frac{\frac{0.2}{150}}{1+1.81} = 6.35 \times 10^{-4}\ m^2/kN.$$

$\Delta\sigma' = 300 - 150$
$= 150\ kN/m^2$.

$$e_{av} = \frac{1.1 + 0.9}{2} = 1.0.$$

For, $U = 50\%$, $T_v = 0.197$

$$C_v = \frac{T_v d^2}{t} = \frac{0.197 \times \left(\frac{0.025}{2}\right)^2}{2}$$

$$= 1.53 \times 10^{-4}\ m^2/min.$$

$$\therefore K = C_v m_v \gamma_w = (1.53 \times 10^{-5})(6.35 \times 10^{-4})(9.81)$$

$$= 95.3 \times 10^{-9}\ m/min.$$

$$= 95.3 \times 10^{-7}\ cm/min.$$

ⓑ

$$T_v = \frac{C_v t}{d^2}$$

$$\Rightarrow T_{60} = \frac{C_v t_{60}}{d^2}$$

$$\Rightarrow t_{60} = \frac{T_{60} d^2}{C_v}.$$

For, $U = 60\%$. $T_v = T_{60} = 0.286$

$$\therefore \Rightarrow t_{60} = \frac{0.286 \times 1.8^2}{1.53 \times 10^{-5}} = 60,565\ min$$

$$= 42.06\ days.$$

Q.4 405

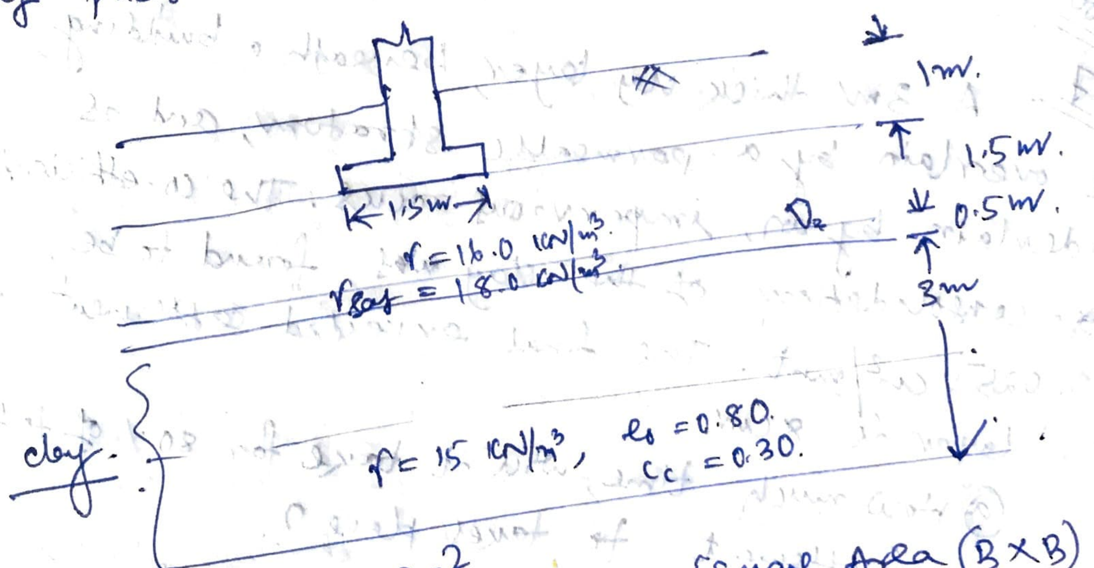
During a laboratory consolidation test, the time and dial gauge readings obtained, from an increase of pressure on the specimen, from 50 kN/m² to 100 kN/m², are given here.

Time (min)	Dial gauge reading (cm × 10^{-4})	Time (min)	Dial gauge reading (cm × 10^{-4})
0.	3975	16.0	4572
0.1	4082	30.0	4737
0.25	4102	60.0	4923
0.5	4128	120.0	5080
1.0	4166	240.0	5207
2.0	4224	480.0	5283
4.0	4298	960.0	5334
8.0	4420	1440.0	5364

Using the logarithm of time method, determine Cv. The avg height of the specimen, during consolidation was 2.24 cm, and it was drained at the top and bottom.

Arora 5.5 296

A footing has a size of 3.0m by 1.50m and it causes a pressure increment of 200 kN/m², at its base. Determine the consolidation settlement, at the middle of the clay layer. Assume 2:1 pressure distribution, and consider the variation of pressure across, the depth of clay layer.



Hint $\Delta\sigma_z = \dfrac{qB^2}{(B+z)^2}$ Square Area (B×B)

$\Delta\sigma_z = \dfrac{q(B\times L)}{(B+z)(L+z)}$ Rectangular area (B×L)

$$\Delta q_z = \frac{q \times (B \times 1)}{(B+Z) \times 1}$$ Strip area, (with B, Unit length)

$$\Delta \sigma_z = \frac{qD^2}{(D+Z)^2}$$ Circular Area, (Diameter, D)

$$\therefore \Delta \bar{\sigma} = \frac{1}{6}\left[(\Delta \bar{\sigma})_t + 4(\Delta \bar{\sigma})_m + (\Delta \bar{\sigma})_b\right]$$

$$S_f = \frac{C_c}{1+e_0} \cdot H_0 \cdot \log_{10}\left(\frac{\bar{\sigma}_0 + \Delta \bar{\sigma}}{\bar{\sigma}_0}\right)$$

Q.6 Arora 298.

A clay stratum, 5m thick, has the initial void ratio of 1.50 and the effective overburden pressure, of 120 kN/m². When the sample is subjected to an increase of pressure of 120 kN/m², the void ratio reduced to, 1.44. Determine the co-efficient of volume compressibility and the final settlement of the stratum.

Ans:→ $m_v = \frac{-\Delta e/1+e_0}{\Delta \bar{\sigma}}$

$$= \frac{(1.5 - 1.44)/(1+1.5)}{120} = 2 \times 10^{-4}\ m^2/kN.$$

$$S_f = m_v \Delta \bar{\sigma} H_0.$$

$$\Rightarrow S_f = 2 \times 10^{-4} \times 120 \times 5 \times 10^3\ mm.$$

$$= 120\ mm.$$

Arora 300.

Q.7. A 3m thick clay layer, beneath a building, is overlain by a permeable stratum, and is underlain by an, impervious rocks. The co-efficient of consolidation of the clay was found to be, 0.025 cm²/mnt. The final expected settlement for the layer is 8 cm.

(a) How much time, will it take for 80% of total settlement to takes place?

(b) Determine the time required, for a settlement of 2.5 cm to occur.

(c) Compute the settlement that would occur in 1 year.

Ans ↳ $d = 3m = 300$ cm.

(a). $C_v = T_v d^2/t \Rightarrow 0.025 = \dfrac{T_v (300)^2}{t}$

$\Rightarrow t = 3.6 \times 10^6 T_v \rightarrow$ (1).

when, $U = 80\%$, $T_v = 0.567$.

$\therefore \Rightarrow t = 3.6 \times 10^6 \times 0.567 = 2.041 \times 10^6$ mints

$= 3.883$ years.

(b). When the settlement is 2.5 cm.

$U = \dfrac{2.5}{8.0} \times 100 = 31.25\%$

when, $U = 31.25\%$, $T_v = 0.078$.

from eqn (1) $\Rightarrow t = 3.6 \times 10^6 \times 0.078 = 0.2808 \times 10^6$ mints

$= 195$ days.

(c) from eqn (1) $\rightarrow 1 \times 365 \times 24 \times 60 = 3.6 \times 10^6 T_v$

$\Rightarrow T_v = 0.146$

when, $T_v = 0.146$ & $U = 0.429$.

$\therefore$ Settlement $= 0.429 \times 8 = 3.432$ cm.

Arora 300.

Q.8. A clay layer, 4 m thick, has a final settlement of 6.0 cm. The layer has double drainage. If the coefficient of consolidation is 0.02 cm²/minute, determine the time required for different %age of consolidation, from 10% upto 90%. and hence plot the time-settlement curve.

Ans! $\rightarrow T_v = C_v t/d^2$.

$T_v = 0.02 \times t/(200)^2 \Rightarrow t = 2 \times 10^6 T_v$.

when time is in yrs,

$t = \dfrac{2 \times 10^6 T_v}{60 \times 24 \times 365} = 3.805 T_v$.

The settlement are calculated, from the values of 'U', and the final settlement.

is, $S = U \times S_f$ or $S = 6U$.

U (%)	10	20	30	40	50	60	70	80	90
Tv	0.008	0.031	0.071	0.126	0.196	0.287	0.403	0.567	0.848
t (years)	0.030	0.118	0.270	0.479	0.746	0.092	1.533	2.157	3.227
s (cm)	0.6	1.2	1.8	2.4	3.0	3.6	4.2	4.8	5.4

Aroora 21, 09 30)

An area is underlain by a stratum of clay layer, 6m thick. The layer is doubly drained, and has the co-efficient of consolidation of 0.3 m^2/month.

(a) How long would it take for a surcharge load, to cause a settlement of 40 cm if the same load causes a final settlement of 60 cm?

(b) If the sand drains, ($s = 3m$ and $r_w = 0.30$) are used, determine the time required for 90% consolidation. Take $C_{vr} = 2.0\ m^2$/month. Assume the triangular layout of drains. Neglect vertical consolidation in this case.

Ans: (a) $U = 40/60 = 0.6667 = 66.67\%$

when, $U = 66.67\%$, $T_v = 0.364$

$\Rightarrow C_v = T_v d^2/t$

$\Rightarrow t = 0.364 \times 3^2/0.30 = 10.92$ months.

(b) For triangular layout of drains,

$R = 0.525 S$
$n = R/r_w$
$T_r = \dfrac{C_{vr} t}{4R^2}$

$R = 0.525 \times S$

$n = R/r_w = 1.575/0.30 = 5.25$

For, $U_r = 90\%$ & $n = 5.25$, $T_r = 0.270$

$T_r = C_{vr} t/4R^2$

$\Rightarrow 0.270 = \dfrac{2.0 \times t}{4 \times (1.575)^2}$

$\Rightarrow t = 1.34$ months.

A Clay layer, 4m thick is subjected to a pressure of 55 kN/m². If the layer has a double drainage and undergoes 50% consolidation in one year, determine the co-efficient of consolidation. Take $T_v = 0.196$. If the co-efficient of permeability is 0.020 m/yr, determine the settlement in one year, (determine the co-efficient of consolidation.) Take, $T_v = 0.196$, and rate of flow of water, per unit area, in one year.

Ans. → $C_v = T_v d^2/t = 0.196 \times (2.0)^2/(1)$

$\Rightarrow C_v = 0.784 \text{ m}^2/\text{yr}$.

$m_v = \dfrac{k}{C_v \gamma_w} = \dfrac{0.020 \times 1000}{0.784 \times 1000 \times 9.81}$

$= 2.6 \times 10^{-3} \text{ m}^2/\text{kN}$.

$S_f = m_v \cdot H_0 \cdot \Delta\sigma = 2.6 \times 10^{-3} \times 4 \times 55$

$= 0.572 \text{ m}$.

Settlement after 1 year, $= 0.5 \times 0.572$

$= 0.286 \text{ m}$.

Since,

$U \propto \sqrt{t}$ for, $U < 0.60$.

$s \propto \sqrt{t}$

$\Rightarrow s^2 \propto t \quad \Rightarrow t = Cs^2$, when $t = 1$ yr, $s = 0.286$ m.

Therefore, $C = \dfrac{1}{(0.286)^2} = 12.226$.

$\Rightarrow t = 12.226\, s^2$

or, $\dfrac{ds}{dt} = \dfrac{1}{2 \times 12.226\, s} = \dfrac{1}{24.452\, s} = 0.143 \text{ m/yr}$.

Discharge per unit area,

per surface $= 0.143/2 = 0.072 \text{ m}^3/\text{yr}/\text{m}^2$.