UNIT-VII
CONSOLIDATION

Introduction :-

When a soil mass is subjected to a compressive force, like all other materials, its volume decrease. The property of the soil due to which a decrease in volume occurs under compressive forces is known as the compressibility of soil.

The compression of soil can occur due to one or more of the following causes.

1. compression of solid particles and water in the voids.
2. compression and expulsion of air in the voids.
3. Expulsion of water in the voids.

The compression of a saturated soil under a steady static pressure is known as consolidation. It is entairly due to expulsion of water from the voids.

The consolidation of a soil deposit can be divided into 3 stages.

1. Initial consolidation:- when a load applied to a partially saturated soil, a decrease in volume occurs due to expulsion and compression of air

in the voids. A small decrease in volume occurs due to compression of solid particles. The reduction in volume of the soil just after application of load is known as initial consolidation (or) initial compression.

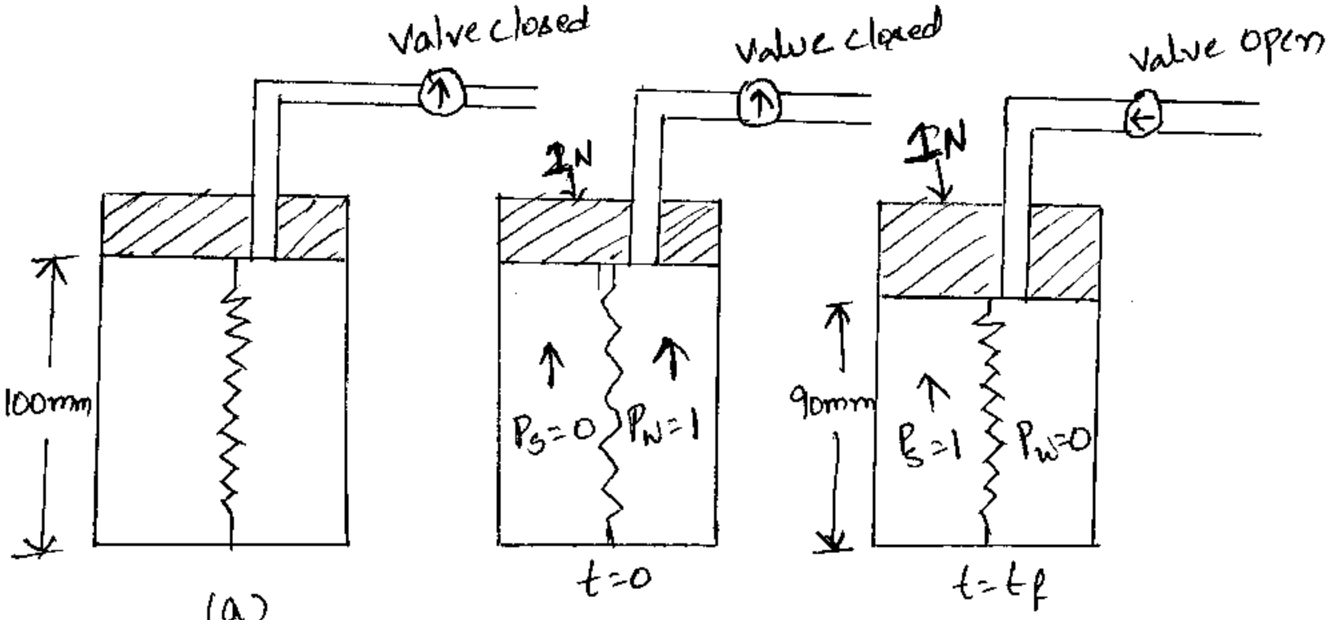
2. Primary consolidation:- After initial consolidation, further reduction in volume occurs due to expulssion of water from voids. When a saturated soil is subjected to a pressure, initially all the applied pressure is taken by water as an excess pore water pressure, as water is almost incompressible as compared with solid particles. A hydraulic gradient develops and the water starts flowing out and a decrease in volume occurs. This reduction in volume is called primary consolidation.

3. Secondary consolidation:- The reduction in volume continuous at a very slow rate even after the excess hydrostatic pressure developed by the applied pressure is fully dissipated and the primary consolidation is complete.

This additional reduction in the volume is called Secondary consolidation.

SPRING ANALOGY FOR PRIMARY CONSOLIDATION:-

The process of primary consolidation can be explained with the help of the Spring analogy given by Terzaghi.



(a) (b) (c)

The above fig shows a cylinder fitted with a tight-fitting piston having a valve. The cylinder is filled with water and contains a spring of specified stiffness. Let the initial length of the spring is weightless and and the spring and water are initially free of stress.

When a load P (say, 1N) is applied to the piston, with its valve closed, the entire load taken by water (fig(b)). The stiffness of spring be 10 mm/N. Let us assume that the piston is weightless and the spring and water are initially free of stress.

~~When a load P (sa~~

the spring is negligible compared with that of the spring is negligible compared with that of water, and consequently, no load is taken by spring.

From equilibrium,

$$P_W + P_s = P \longrightarrow ①$$

where P_W = load taken by water,
P_s = Load taken by spring
P = total load.

for $P = 1N$, The above equation becomes

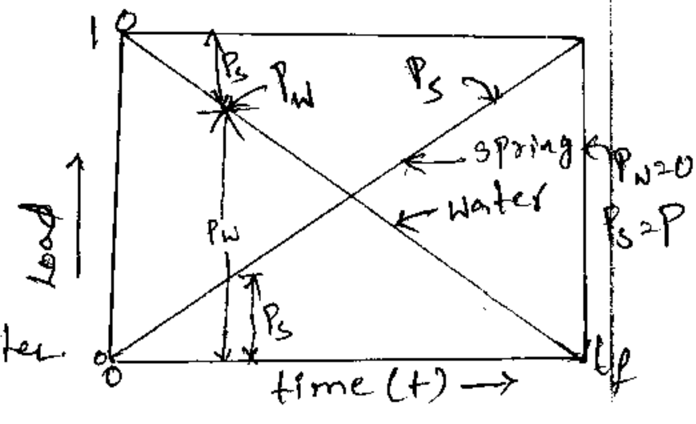
$$P_W + P_s = 1 \longrightarrow ②$$

Initially ($t=0$) when valve is closed $P_s = 0$, Therefore $P_W = 1$

If the valve is now gradually opened, water starts escaping from the cylinder. The spring starts sharing some load and a decrease in its length occurs when a portion (ΔP) of the load is transferred from the water to the spring, The the equation ② becomes

$$\Delta P + (1.0 - \Delta P) = 1.0$$

As more and more water escapes, the load carried by the spring increases. The figure shows the transfer of the load from spring to water.



Pressure - void ratio curves (e-p curves): for clays :-

The compressibility characteristics of clays depen[d] on many factors. The most Impotant factors are:

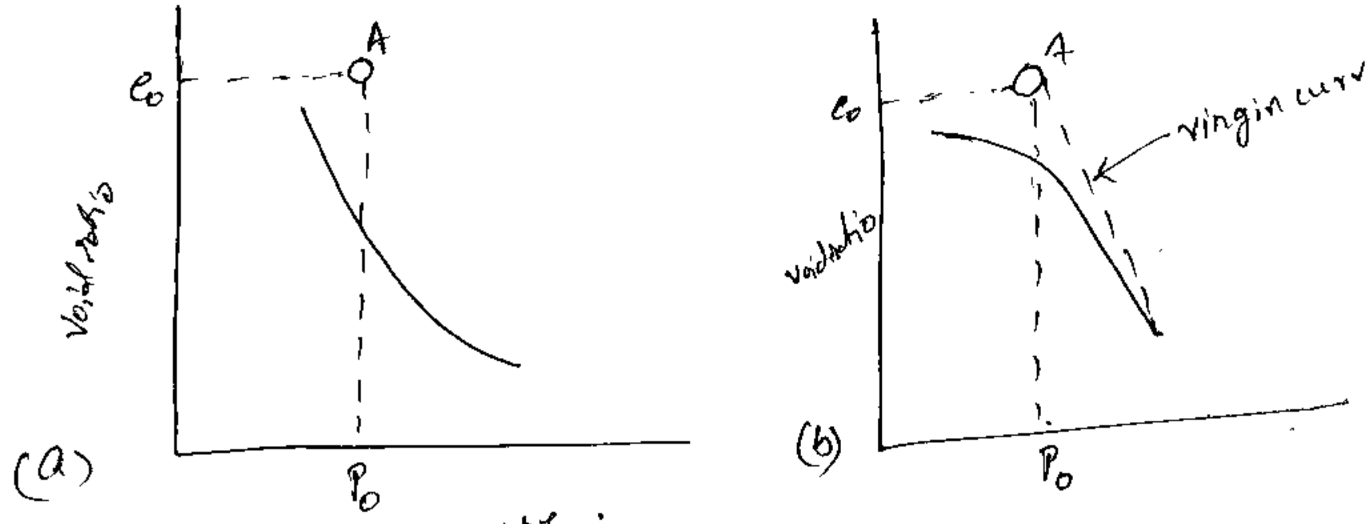
1. whether the clay is normally consolidated or over consolidat[ed]
2. whether the clay is sensitive or insensitive.

A clay is said to be normally consolidated if th[e] present effective overburden pressure P_o is the maxim[um] Pressure to which the layer has ever been subjected at any time in its history, whereas a clay layer is sai[d] to be over consolidated if the layer was subjected at one time in its history to a greater effective over-burden pressure, P_c, than the present pressure, P_o. The ratio $\frac{P_c}{P_o}$ is called the over consolidation ratio (O_{CR}).

The over consolidation of a clay stratum may hav[e] been caused due to some of the following factors.

1. Due to weight of an over burden of soil which has eroded.
2. Due to weight of continental ice sheet that melted.
3. Due to desiccation of layer close to the surface.

Experience indicates that the natural moisture content W_n, is commonly close to the liquid limit, W_L, for normally consolidated clay soil whereas for the over-consolidated clay, W_n is close to plastic limit W_p.



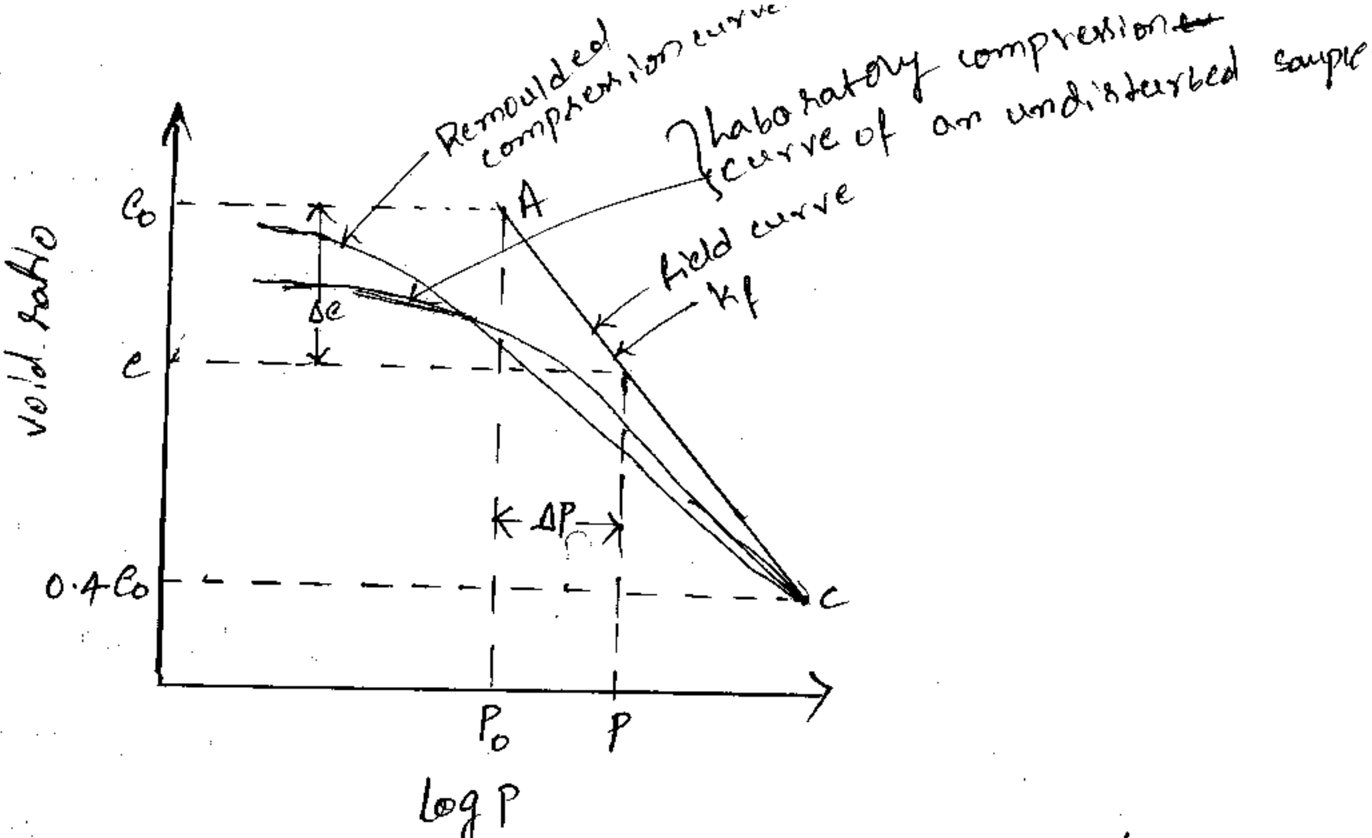
figs ① e-P curve

e log - P curve

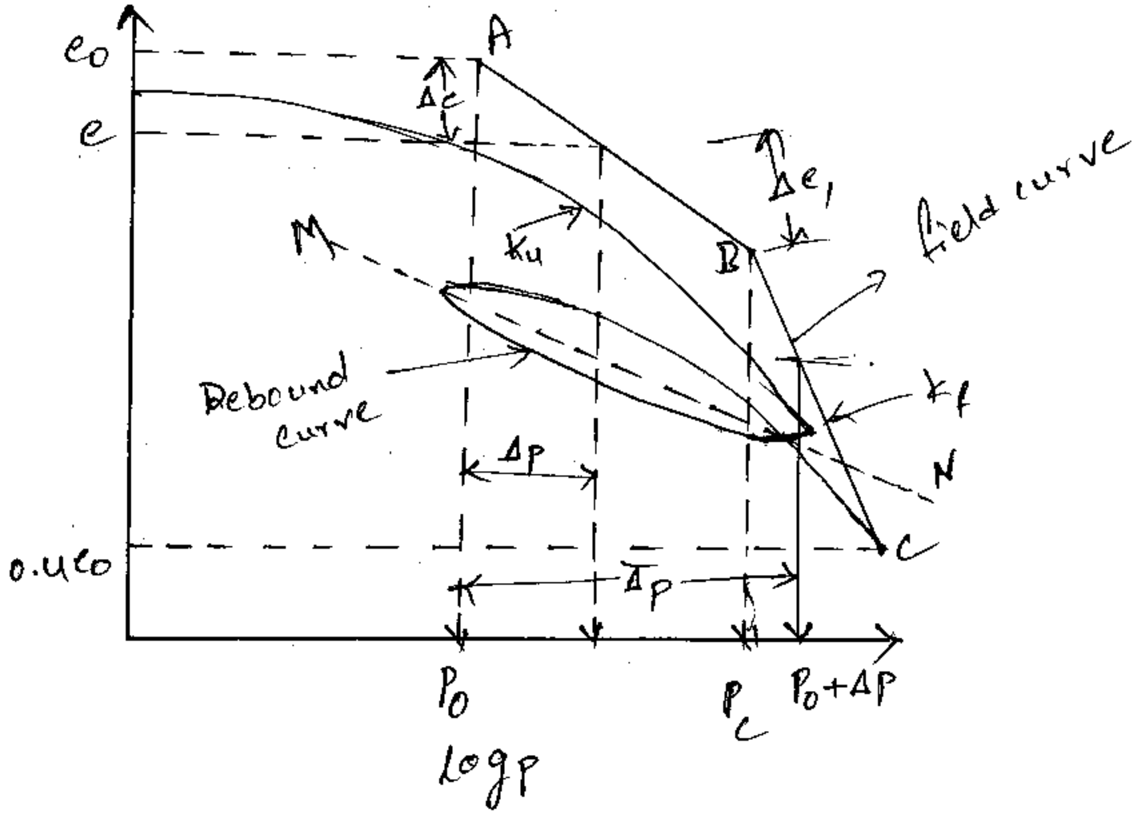
It has been explained earlier with reference to the above figures ① that the laboratory e-log P curves of an undisturbed sample does not pass through Point A and always passes below the point. It has been found from investigation that the inclined straight portion of e-logP curves of undisturbed or remoulded samples of clay soil intersect at one point at a low void ratio and corresponds to $0.4e_0$ shown as point c in the fig ②. It is therefore, logical to assume the field curve labelled as K_f should also pass through this point

The field curve can be drawn from point A, having co-ordinates (e_0, P_0) which corresponds to the in-situ condition of the soil. The straight line AC in fig (2a) gives the field curves K_f for normally consolidated clay soil of low sensitivity.

The field curve for over consolided clay soil cons of two straight lines, represented by AB and BC in fig (2b). schmertmann (1955) has shown that th initial section AB of the field curve is parallel to the mean slope slope MN of the Rebound labora curve. Point B is the intersection point of the vertical line passing through the perconsolidation pressure Pc on the abscissa and the sloping lin AB. since point c is the intersection of the laborat compression curve and the horizontal line at voidrati $0.4e_0$, line BC can be drawn. The slope of lin MN which is the slope of the rebound curv is called the swell index C_s.



(a) Normally consolidated clay soil



(b) Preconsolidated clay soil.

Fig. 2 :- e-log P curve

Magnitude and rate of consolidation:-

It has been explain that the ultimate settlement S_t of a clay layer due to consolidation may be computed by using the following two equations

$$S_t = \frac{C_c}{1+e_0} H \log_{10} \frac{P_0 + \Delta P}{P_0} \longrightarrow (1)$$

where S_t is ultimate settlement

C_c is compression index.

$$S_t = \Sigma H_i \frac{C_c}{1+e_0} \log_{10} \frac{P_0 + \Delta P}{P_0}$$

If S is the settlement at any time t after the imposition of load on the clay layer, the degree of consolidation of the layer in time t may be expressed as

$$U\% = \frac{S}{S_t} \times 100\ \%.$$

Since U is a function of the time factor T, we may write

$$U\% = 100 f(T) = \frac{S}{S_t} \times 100$$

The rate of settlement curve of a structure built on a clay layer may be obtained by the following procedure

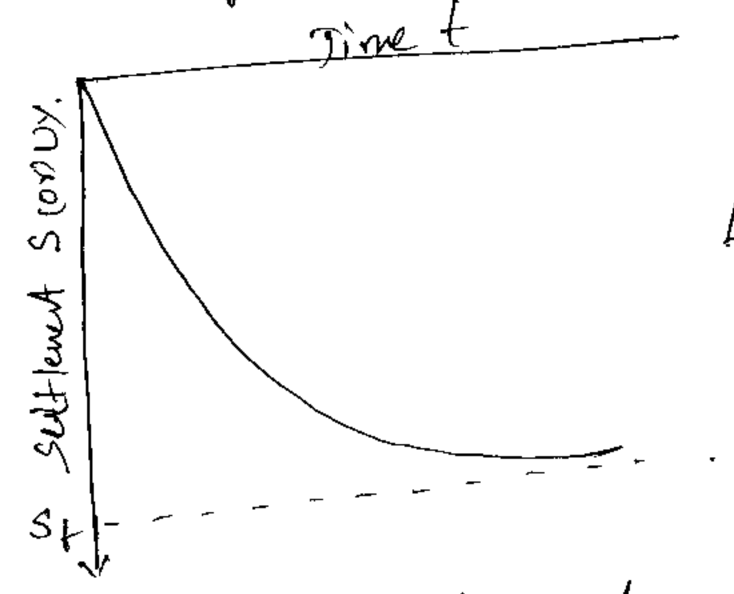
1. From consolidation test data, compute m_v and c_v
2. compute the total settlement S_f that the clay stratum would experience with the increment of load ΔP.

{ c_v = co-eff of volume compressibility }
{ $m_v = \frac{a_v}{1+e_o}$ " " " }

3. From the theoretical curve giving the relation between U and T, find T for different degrees of consolidation, say 5, 10, 20, 30 per cent etc.
4. compute from equation $t = \frac{TH^2}{C_v}$ the values of t for different values of T. It may be noted here that for drainage on both sides H is equal to half the thickness of the clay layer
5. Now a curve can be plotted giving the relation between t and U% or t and S as shown in the below fig.

a_v = co-eff of compressibility

ΔP = net change in pressure

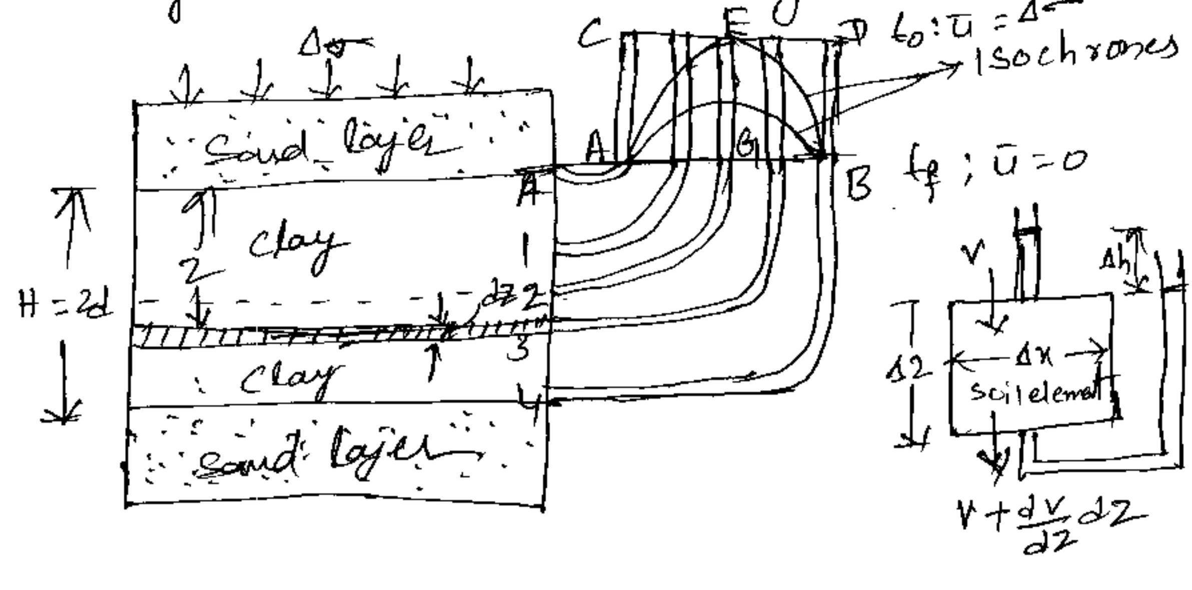


Time - settlement curve.

TERZAGHI'S THEORY OF CONSOLIDATION:-

Assumptions:- Terzaghi (1925) gave the theory for the determination of the rate of consolidation of a saturated soil mass subjected to a static, steady load. The theory is based on the following assumption.

1. The soil is homogeneous and isotropic
2. The soil is fully saturated.
3. Soil particles and water are incompressible.
4. Darcy's law is valid throughout the consolidation process
5. coefficient of permeability is constant during consolidation.
6. Excess pore water drains out only in the vertical direction.
7. The time lag in consolidation is due to entirely to the low permeability of the soil
8. The change in thickness of the layer during consolidation is insignificant



The above figure shows a clay layer of thickness H, sandwhich between two layers of sand which serves as drainage faces. When the layer is subjected to a pressure increment $\Delta\sigma$, excess hydrostatic pressure is set up in the clay layer. At the time t_0, the instant of pressure application, whole of the consolidati[ng] Pressure $\Delta\sigma$ is carried by the Pore water so that the Initial excess hydrostatic pressure $\bar{U}_0$ is equal to $\Delta\sigma$, and is represented by a straight line $\bar{u}=\Delta\sigma$ on the pressure distribution diagram. As water starts escaping into the sand, the excess hydrostatic pressure at the pervious boundaries drops to zero and remains so at all times. After a very great time t_f, the whole of the excess hydrostatic pressure is dissipated so that $\bar{u}=0$, represented by line AGB. At an intermediate time t, the consolidating pressure $\Delta\sigma$ is partly carried by water and partly by soil, and the following relationship is obtained : $\Delta\sigma = \Delta\sigma' + \bar{u}$. The distribution of excess hydrostatic pressure $\bar{u}$ at any time 't' is Indicated by the curve AFB, joining water levels in the Piezometric tubes; ~~this curve~~ is this curve is known as isochrone, and number of such isochrones can be drawn at various time intervals t_1, t_2, t_3 etc. The slope of isochrones at any point at a given time indicates the rate of change of $\bar{u}$ with depth.

At any time t, the hydraulic head h corresponding to the excess hydrostatic pressure is given by

$$h = \frac{\bar{u}}{\gamma_w} \longrightarrow (1) \quad \left\{ \because \frac{\partial h}{\partial z} = \frac{1}{\gamma_w} \frac{\partial \bar{u}}{\partial z} \right\}$$

Hence the hydraulic gradient i is given by

$$i = \frac{\partial h}{\partial z} = \frac{1}{\gamma_w} \frac{\partial u}{\partial z} \longrightarrow (2)$$

Thus, the rate of change of $\bar{u}$ along the depth of the layer represents the hydraulic gradient. The velocity with which the excess pore water flow at the depth z is given by Darcy's law.

$$v = ki = \frac{k}{\gamma_w} \frac{\partial u}{\partial z} \longrightarrow (3) \quad \{\because (2)\}$$

The rate of chang of velocity along the depth of the layer is then given by

$$\frac{\partial v}{\partial z} = \frac{k}{\gamma_w} \frac{\partial^2 \bar{u}}{\partial z^2} \longrightarrow (4)$$

consider a small soil element of size dx, dz, and of width dy perpendicular to the xz plane. If v is the velocity of water at the entry into the elements, the velocity at the exit will be equal

to $v + \frac{\partial v}{\partial z} dz \longrightarrow$ (a)

The quantity of water entering the soil element = v dx dy

The quantity of water leaving the soil element = $\left(v + \frac{\partial v}{\partial z} dz\right) dx\, dy$

$$= \left(v + \frac{\partial v}{\partial z} dz\right) dx\, dy \quad \text{—— (b)}$$

Hence the net quantity of water dq squeezed out of the soil element per unit time is give by

$$\Delta q = \frac{\partial v}{\partial z} dx\, dy\, dz \longrightarrow (5) \quad \{\because (b) - (a)\}$$

The decrease in the volume of soil is equal to the volume of water squeezed out.

Hower from the eq $\Delta V = -m_v V_0 \Delta \sigma' \longrightarrow (6)$

where V_0 = volume of soil element at time t_0 = dx dy dz

$\therefore$ change of volume per unit time is given by

$$\frac{\partial(\Delta V)}{\partial t} = -m_v\, dx\, dy\, dz \frac{\partial(\Delta \sigma')}{\partial t} \longrightarrow (7)$$

Equating (5) and (7), we get

$$\frac{\partial v}{\partial z} = -m_v \frac{\partial(\Delta\sigma')}{\partial t} \longrightarrow (8)$$

Now $\Delta\sigma = \Delta\sigma' + \bar{u}$, where $\Delta\sigma$ is constant

$$\therefore \quad \frac{\partial(\Delta\sigma')}{\partial t} = -\frac{\partial\bar{u}}{\partial t} \longrightarrow (9)$$

Hence, from (8) and (9) $\frac{\partial v}{\partial z} = m_v \frac{\partial\bar{u}}{\partial t} \longrightarrow (10)$

combining equations (4) and (10), we get

$$\frac{\partial\bar{u}}{\partial t} = \frac{K}{m_v \gamma_w} \frac{\partial^2 \bar{u}}{\partial z^2} \longrightarrow (11)$$

$$\frac{\partial\bar{u}}{\partial t} = C_v \frac{\partial^2 \bar{u}}{\partial z^2} \longrightarrow (12)$$

Where C_v = coefficient of consolidation $= \frac{K}{m_v \gamma_w}$

$$= \frac{K(1+e_0)}{a_v \gamma_w}$$

The eqt^n (12) is the basic differential equation consolidation which relates the rate of change of excess hydrostatic pressure to the rate of expulsion of excess pore water from a unit volume of soil during the same time interval.

The term coefficient of consolidation C_v used in the equation is adopted to indicate the combined effects of permeability and compressibility of soil on the rate of volume change. The units of C_v are cm^2/sec.