UNIT - III

STRESS DISTRIBUTION AND SETTLEMENT

Stress Distribution - Soil media - Boussinesq theory - Use of Newmarks influence chart - Components of settlement - immediate and consolidation settlement - Terzaghi's one dimensional consolidation theory - Computation of rate of settlement - $\sqrt{t}$ and log t methods - e-log p relationship - Factors influencing compression behaviour of soils.

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STRESS DISTRIBUTION :

Consider stresses within a soil mass due to its own weight. Stresses due to self weight are sometimes known as geostatic stresses.

Let us take soil mass to be bounded by horizontal plane (ground surface) xy and the z-axis be directed downwards. Under this condition, soil mass is said to be semi-infinite.

When there is no external loading, the ground plane becomes a principal plane since it is devoid of any shear loading.

Within soil mass,

$$\tau_{xy} = \tau_{xz} = \tau_{yz} = 0.$$

$$\boxed{\sigma_z = \gamma z}$$ ← From equilibrium equation

γ - Unit Weight of soil

σ_z - Vertical stress at a point within and soil mass, situated at a depth z below ground surface.

From compatibility equations,

$$\sigma_x = \sigma_y = \frac{\mu}{1-\mu}\,\gamma z.$$

$$\boxed{\sigma_x = \sigma_y = K_0\,\gamma z} \qquad K_0 = \frac{\mu}{1-\mu}$$

μ - Poisson ratio

K_0 - Coefficient of lateral pressure @ rest.

At certain point within soil mass, the stresses are caused due to both surface loadings as well as due to self weight of soil above it.

Total stress = Stress due to Self weight + Stress due to surface loads.

Stress tensor:

$$\begin{bmatrix} \sigma_x & \tau_{yx} & \tau_{zx} \\ \tau_{xy} & \sigma_y & \tau_{zy} \\ \tau_{xz} & \tau_{yz} & \sigma_z \end{bmatrix}$$

σ - Normal stress

τ - Shear stress.

Strain Tensor:

$$\begin{bmatrix} \varepsilon_x & \gamma_{yx}/2 & \gamma_{zx}/2 \\ \gamma_{xy}/2 & \varepsilon_y & \gamma_{zy}/2 \\ \gamma_{xz}/2 & \gamma_{yz}/2 & \varepsilon_z \end{bmatrix}$$

ε - direct strain

γ - Shearing strain

BOUSSINESQ EQUATIONS.

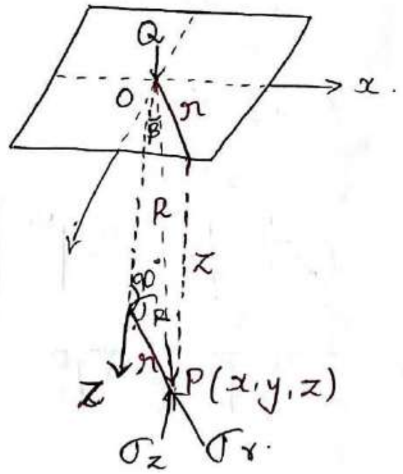
CONCENTRATED FORCE:

Assumptions:

* Soil mass is an elastic medium, for which modulus of elasticity E is constant.
* Soil mass is homogeneous, it has identical properties at every point in it in identical directions.
* Soil mass is isotropic, it has identical elastic properties in all directions through any point of it.
* Soil mass is semi-infinite, it extends infinitely in all directions below level surface.

Let a point load Q act at the ground surface, at a point O which may be taken as origin of x, y & z axis.

Let us find stress components at a point P in soil mass, having coordinates x, y, z, having radial distance r and vertical distance z from point O.



σ_R - Polar Radial Stress.

Using logarithmic stress function, Boussinesq showed that polar radial stress

$$\sigma_R = \frac{3}{2}\frac{Q}{\pi}\frac{\cos\beta}{R^2}$$

R - Polar radial coordinate of point P

$$R = \sqrt{r^2 + z^2} = \sqrt{x^2+y^2+z^2} \qquad r = \sqrt{x^2+y^2}$$

$$\cos\beta = \frac{z}{R}$$

σ_z - Vertical stress.

τ_{rz} - Tangential stress.

$$\sigma_z = \sigma_R \cos^2\beta$$

$$= \frac{3}{2}\frac{Q}{\pi}\frac{\cos^3\beta}{R^2} \qquad \because \cos\beta = \frac{z}{R}$$

$$\sigma_z = \frac{3}{2}\frac{Q}{\pi}\frac{z^3}{R^5}$$

$$\sigma_z = \frac{3Q}{2\pi}\frac{z^3}{(r^2+z^2)^{5/2}}$$

Multiply & divide by z^2.

$$\sigma_z = \frac{3Q}{2\pi}\frac{z^2}{z^2}\frac{z^3}{(r^2+z^2)^{5/2}}.$$

$$= \frac{3Q}{2\pi z^2}\frac{z^5}{(r^2+z^2)^{5/2}}$$

$$= \frac{3Q}{2\pi z^2}\frac{z^5}{(z^2)^{5/2}\left[1+\frac{r^2}{z^2}\right]^{5/2}}.$$

$$= \frac{3Q}{2\pi z^2}\frac{\cancel{z^5}}{\cancel{z^5}\left[1+\left(\frac{r}{z}\right)^2\right]^{5/2}}.$$

$$\boxed{\sigma_z = \frac{3Q}{2\pi z^2}\left[\frac{1}{1+\left(\frac{r}{z}\right)^2}\right]^{5/2}}$$

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$$\tau_{rz} = \frac{1}{2}\sigma_R \sin 2\beta \qquad \because \sin 2\beta = 2\sin\beta\cos\beta$$

$$= \frac{1}{\cancel{2}}\sigma_R(\cancel{2}\sin\beta\cos\beta)$$

$$= \frac{3}{2}\frac{Q}{\pi}\frac{\cos\beta}{R^2}\sin\beta\cos\beta \qquad \sin\beta = \frac{r}{R}, \quad \cos\beta = \frac{z}{R}.$$

$$= \frac{3Q}{2\pi R^2}\sin\beta\cos^2\beta.$$

$$= \frac{3Q}{2\pi R^2}\frac{r}{R}\cdot\frac{z^2}{R^2}$$

$$= \frac{3Q\,r\,z^2}{2\pi R^5}$$

$$= \frac{3Q}{2\pi}\frac{r z^2}{(r^2+z^2)^{5/2}}$$

$$= \frac{3Q r}{2\pi}\frac{\cancel{z^2}}{(z^{\cancel{2}})^{\cancel{5/2}\,3}\left[1+\frac{r^2}{z^2}\right]^{5/2}}.$$

$$\boxed{\tau_{rz} = \frac{3Qr}{2\pi z^3}\left[\frac{1}{1+\left(\frac{r}{z}\right)^2}\right]^{5/2}}.$$

Bousinesq influence factor (K_B)

$$\sigma_z = \frac{3Q}{2\pi z^2}\left[\frac{1}{1+\left(\frac{r}{z}\right)^2}\right]^{5/2}$$

$$= K_B\frac{Q}{z^2}$$

$$\boxed{K_B = \frac{3}{2\pi}\left[\frac{1}{1+\left(\frac{r}{z}\right)^2}\right]^{5/2}}.$$

K_B - Boussinesq influence factor.

Influence factor - dimensionless.

Intensity of vertical pressure, directly below point load ($r=0$),

$$\sigma_z = \frac{3Q}{2\pi z^2}\left[\frac{1}{1+\left(\frac{r}{z}\right)^2}\right]^{5/2} \quad \because r=0.$$

$$= \frac{3Q}{2\pi z^2}$$

$$\boxed{\sigma_z = 0.4775\frac{Q}{z^2}}$$ ← At point directly below point load.

PRESSURE DISTRIBUTION DIAGRAMS:

By means of Boussinesq's stress distribution theory, the following vertical pressure distribution diagrams can be prepared

* Stress isobar or isobar diagram
* Vertical pressure distribution on a horizontal plane.
* Vertical pressure distribution on a vertical line.

Isobars:

It is a curve or contour connecting all points below ground surface of equal vertical pressure.

It is a spatial, curved surface of the shape of a bulb, because vertical pressure on a given horizontal plane is same in all directions at points located at equal radial distances around axis of loading. (A)

The zone in a loaded soil mass bounded by an isobar of given vertical pressure intensity is called a pressure bulb.

The vertical pressure at every point on surface of pressure bulb is same.

Suppose an isobar of value $\sigma_z = 0.25\,Q$. per unit area is to be plotted.

$\sigma_z = K_B \frac{Q}{Z^2}$.

$K_B \frac{Q}{Z^2} = 0.25\,Q$.

$\boxed{K_B = 0.25\,Z^2}$

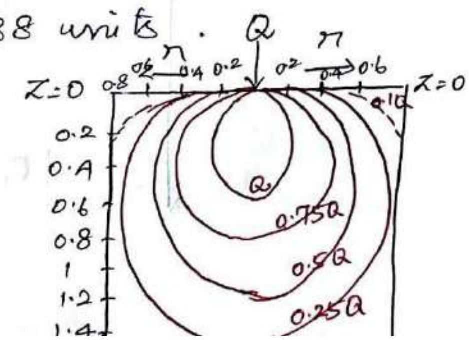
r/z values can be calculate from Table 13.1 in Soil Mechanics by B.C.Punmia Book.

ISOBAR DATA : $(\sigma_z = 0.25\,Q)$.

Z (units)	K_B	r/z	r (units)
0.2	0.01	1.92	0.38
0.4	0.04	1.3	0.52
0.6	0.09	0.97	0.58
0.8	0.16	0.74	0.59
1	0.25	0.54	0.54

When $r = 0$, $K_B = 0.4775$.

$$z = \sqrt{\frac{0.4775}{0.25}} = 1.38 \text{ units}.$$



Vertical pressure distribution on a horizontal Plane :

The vertical pressure distribution on any horizontal plane @ a depth z below ground surface, due to concentrated load is given by

$$\sigma_z = K_B \frac{Q}{z^2}$$

z is known.

r - horizontal distance

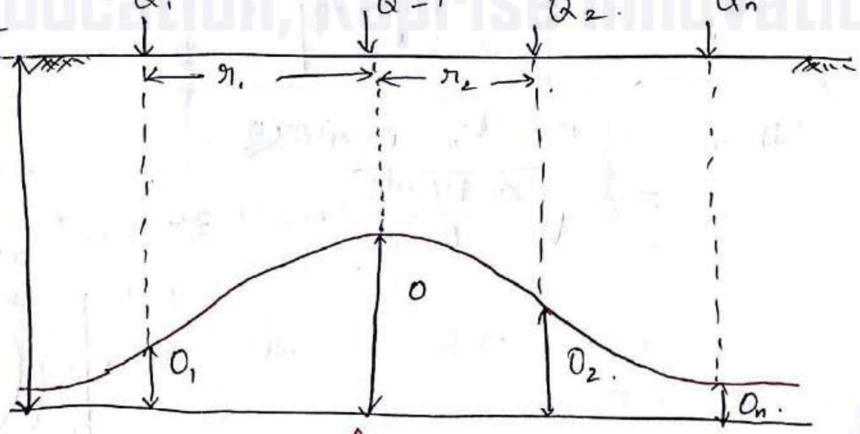
Below the load, i.e $r = 0$,

$$\sigma_z = 0.4775 \frac{Q}{z^2}$$

r/z	K_B	σ_z
0	0.4775	Maximum.
0.5	0.2733	57% of maximum.
1	0.0844.	17.7% of maximum
2	0.0085	1.8% of maximum

When $r = 2z$, the vertical pressure is negligible.

Influence Diagram



Such a diagram is helpful in (5) computing the vertical stress σ_z at A due to number of concentrated loads $Q_1, Q_2, \ldots Q_n$ situated at radial distances $r_1, r_2 \ldots r_n$ from vertical axis through point A.

The vertical stress is then given by.

$$\sigma_z = \Sigma Q \cdot O.$$

$$= Q_1 O_1 + Q_2 O_2 + \ldots \ldots Q_n O_n$$

where $O, O_1, O_2 \ldots O_n$ are ordinates of influence diagram plotted for σ_z at A.

Vertical pressure distribution on vertical line

σ_z decreases with increase in depth z.

$$\sigma_z = K_B \frac{Q}{z^2}$$

If $r = 1$ unit.

z (units)	$\frac{r}{z}$	K_B	$\frac{K_B}{z^2}$	σ_z
0	∞	—	—	Intermediate
0.2	5	0.0001	0.0025	0.0025Q.
0.5	2	0.0085	0.0340	0.0340Q.
1	1	0.0844	0.0844	0.0844Q.
2	0.5	0.2733	0.0683	0.0683Q.
4	0.25	0.4103	0.0256	0.0256Q.

The vertical stress increases first, attains a maximum value and then decreases. It can be shown that maximum value of σ_z on a vertical line is obtained at the point of intersection of the vertical plane with a radial line at $\beta = 39°15'$ through point load.

Corresponding value of $r/z = 0.817$.

$$z = \frac{r}{0.817}$$

If $r = 1$, $z = \frac{1}{0.817}$

$z = 1.225$

$K_B = 0.1332$.

$$(\sigma_z)_{max} = \frac{0.1332\,Q}{(1.225)^2}$$

$$\boxed{(\sigma_z)_{max} = 0.0888\,Q}$$

VERTICAL PRESSURE UNDER A UNIFORMLY LOADED CIRCULAR AREA

Boussinesq equation for vertical stress due to single concentrated load can now be extended to find vertical pressure on any point on vertical axis passing through the centre of uniformly loaded circular area.

The figure shows a uniformly loaded circular area of radius a and load intensity q per unit area.

Consider an elementary ring of radius r and width δr on the loaded area. If elementary ring is further divided into small parts, each of area δA, the load on each elementary area will be $q\delta A$.

This load may be considered as point load. Vertical pressure at point P, situated at ⑥ depth z on vertical axis through centre of area, is given by.

$$\delta\sigma_z = \frac{3}{2\pi}(q\,\delta A)\frac{z^3}{(r^2+z^2)^{5/2}}.$$

Integrating over entire ring of radius r, vertical stress $\Delta\sigma_z$ is given by

$$\Delta\sigma_z = \frac{3q}{2\pi}(\Sigma\,\delta A)\frac{z^3}{(r^2+z^2)^{5/2}}.$$

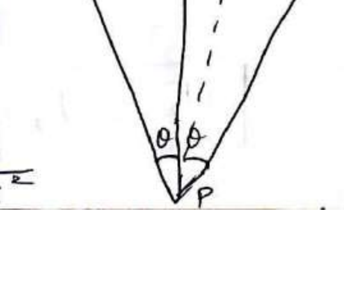
$$= \frac{3q}{2\pi}\,\Sigma\,\delta(\pi r^2)\,\frac{z^3}{(r^2+z^2)^{5/2}}.$$

$$= \frac{3q}{2\pi}\,\frac{2\pi r\,\delta r}{(r^2+z^2)^{5/2}}\,z^3$$

$$= 3qr\,\delta r\,\frac{z^3}{(r^2+z^2)^{5/2}}.$$

The total vertical pressure σ_z due to entire loaded area is given by integrating above expression b/w limits $r=0$ to $r=a$.

$$\sigma_z = 3qz^3\int_0^a \frac{r\,dr}{(r^2+z^2)^{5/2}}.$$



Put $r^2+z^2 = n^2$.

Differentiate w.r.to r.

$2r\,dr + 0 = 2n\,dn$.

$r\,dr = n\,dn$.

when $r=0$, $n=z$. when $r=a$, $n=\sqrt{a^2+z^2}$

$$\sigma_z = 3q\, z^3 \int_z^{\sqrt{a^2+z^2}} \frac{n\,dn}{(n^2)^{5/2}}$$

$$= 3q\, z^3 \int_z^{\sqrt{a^2+z^2}} \frac{\cancel{n}\,dn}{n^5}$$

$$= 3q\, z^3 \int_z^{\sqrt{a^2+z^2}} \frac{dn}{n^4}$$

$$= -\frac{\cancel{3}}{\cancel{3}} q\, z^3 \left[\frac{1}{n^3}\right]_z^{\sqrt{a^2+z^2}}$$

$$= -q\, z^3 \left[+\frac{1}{n^3}\right]_z^{\sqrt{a^2+z^2}}$$

$$= -q\, z^3 \left[\frac{1}{(a^2+z^2)^{3/2}} - \frac{1}{z^3}\right]$$

$$= q\, z^3 \left[\frac{1}{z^3} - \frac{1}{(a^2+z^2)^{3/2}}\right]$$

$$= q\left[1 - \frac{z^3}{(a^2+z^2)^{3/2}}\right]$$

$$= q\left[1 - \frac{\cancel{z^3}}{\cancel{z^3}\left[1+\frac{a^2}{z^2}\right]^{3/2}}\right]$$

$$\sigma_z = q\left[1 - \left\{\frac{1}{1+\frac{a^2}{z^2}}\right\}^{3/2}\right]$$

$$\boxed{\sigma_z = K_B\, q}$$

Differentiation

x^n

$n x^{n-1}$

n^{-4}

$-4\, n^{-4-1}$

Integration.

$x^n = \frac{x^{n+1}}{n+1}$

$n^{-4} = \frac{n^{-4+1}}{-4+1}$

$= \frac{n^{-3}}{-3}$

K_B - Boussinesq influence factor for uniformly distributed circular load

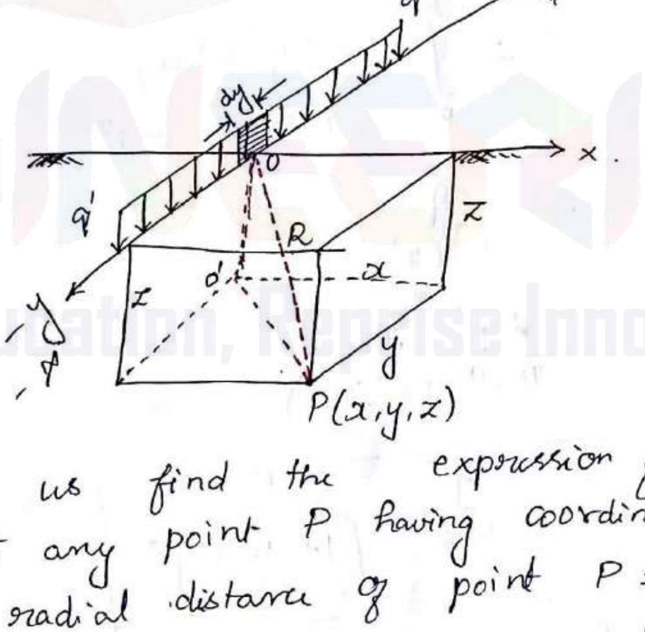
$$\boxed{K_B = 1 - \left[\frac{1}{1+\left(\frac{a}{z}\right)^2}\right]^{3/2}}$$

If θ is the angle which the line joining the point P makes with outer edge of loading,

$$\boxed{\sigma_z = q\left[1 - \cos^3\theta\right]}$$

VERTICAL PRESSURE DUE TO A LINE LOAD

Let us consider an infinitely long line load of intensity q' per unit length, acting on the surface of a semi-infinite elastic medium.



Let us find the expression for vertical stress at any point P having coordinates (x, y, z)

The radial distance of point $P = r$

$$r = \sqrt{x^2 + y^2}$$

The polar distance of point $P = R$

$$R = \sqrt{r^2 + z^2} = \sqrt{x^2 + y^2 + z^2}$$

Consider a small length dy along line load. The elementary load in this length will be equal to $q' dy$, which may be considered as concentrated load.

Vertical stress

$$\Delta \sigma_z = \frac{3(q' dy) z^3}{2\pi R^5}$$

$$= \frac{3 q' dy \; z^3}{2\pi (x^2+y^2+z^2)^{5/2}}.$$

$$\sigma_z = \int_{-\infty}^{+\infty} \frac{3q' z^3 dy}{2\pi (x^2+y^2+z^2)^{5/2}}.$$

$$= 2\int_{0}^{\infty} \frac{3q' z^3 dy}{2\pi (x^2+y^2+z^2)^{5/2}}.$$

$$= \frac{2 q' z^3}{\pi (x^2+z^2)^2}$$

$$= \frac{2q' \cancel{z^3}}{\pi \cancel{z^4}\left(1+\frac{x^2}{z^2}\right)^2}$$

$$\boxed{\sigma_z = \frac{2q'}{\pi z} \frac{1}{\left[1+\left(\frac{x}{z}\right)^2\right]^2}}$$

When P is situated vertically below line load, at depth z, we have $x = 0$.

$$\boxed{\sigma_z = \frac{2q'}{\pi z}}$$

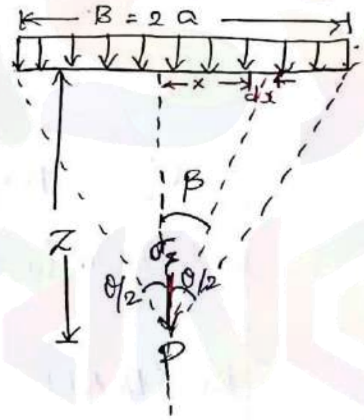
VERTICAL PRESSURE UNDER STRIP LOAD

Consider a strip load of width dx, at distance x from centre. The elementary line load intensity along this elementary strip of width dx will be $q.dx$.

The vertical pressure P due to this elementary line load is given by

$$\Delta\sigma_z = \frac{2(q\,dx)}{\pi z}\left[\frac{1}{1+\left(\frac{x^2}{z}\right)}\right]^2$$

Total vertical pressure due to whole strip is given by



$$\sigma_z = \frac{2q}{\pi z}\int_{-B/2}^{B/2}\frac{dx}{\left[1+(x/z)^2\right]^2}$$

$$= \frac{2q}{\pi z}\cdot 2\int_{0}^{B/2}\frac{dx}{\left[1+(x/z)^2\right]^2}$$

$x/z = \tan\beta$.

$x = z\tan\beta$.

$dx = z\sec^2\beta\,d\beta$.

$$\sigma_z = \frac{4q}{\pi \cancel{z}}\int_{0}^{\theta/2}\frac{\cancel{z}\sec^2\beta\,d\beta}{(1+\tan^2\beta)^2}$$

$$= \frac{4q}{\pi}\int_{0}^{\theta/2}\cos^2\beta\,d\beta$$

$$\boxed{\sigma_z = \frac{q}{\pi}(\theta+\sin\theta)}$$

VERTICAL PRESSURE UNDER A UNIFORMLY LOADED RECTANGULAR AREA

Let us take the case of rectangular load area of length 2a & width 2b

A more common form of vertical stress under corner of rectangular area of size a.b is as follows.

$m = a/z \quad , \quad n = b/z$

$$\sigma_z = \frac{q}{4\pi}\left[\frac{2mn\sqrt{(m^2+n^2+1)}}{m^2+n^2+m^2n^2+1} \cdot \frac{m^2+n^2+2}{m^2+n^2+1} + \tan^{-1}\frac{2mn\sqrt{(m^2+n^2+1)}}{m^2+n^2-m^2n^2+1}\right]$$

$\sigma_z = Kq$.

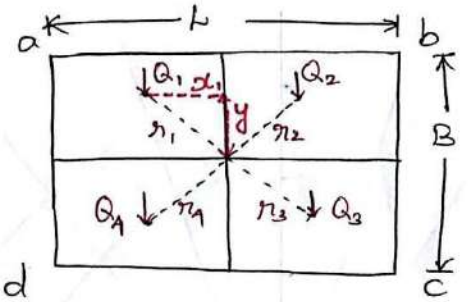
K - Influence factor.

EQUIVALENT POINT LOAD METHOD:

(Approximate method)

- To find vertical stress at any point due to any loaded area.

The entire area is subdivided into a number of small area units & total distributed load over a unit area is replaced by a point load of same magnitude acting at centroid of area unit.



$$\sigma_z = \frac{1}{z^2}\left[Q_1 K_{B_1} + Q_2 K_{B_2} + \dots Q_n K_{B_n}\right].$$

If all point are of equal magnitude Q'

$$\boxed{\sigma_z = \frac{Q'}{z^2}\,\Sigma K_B}$$

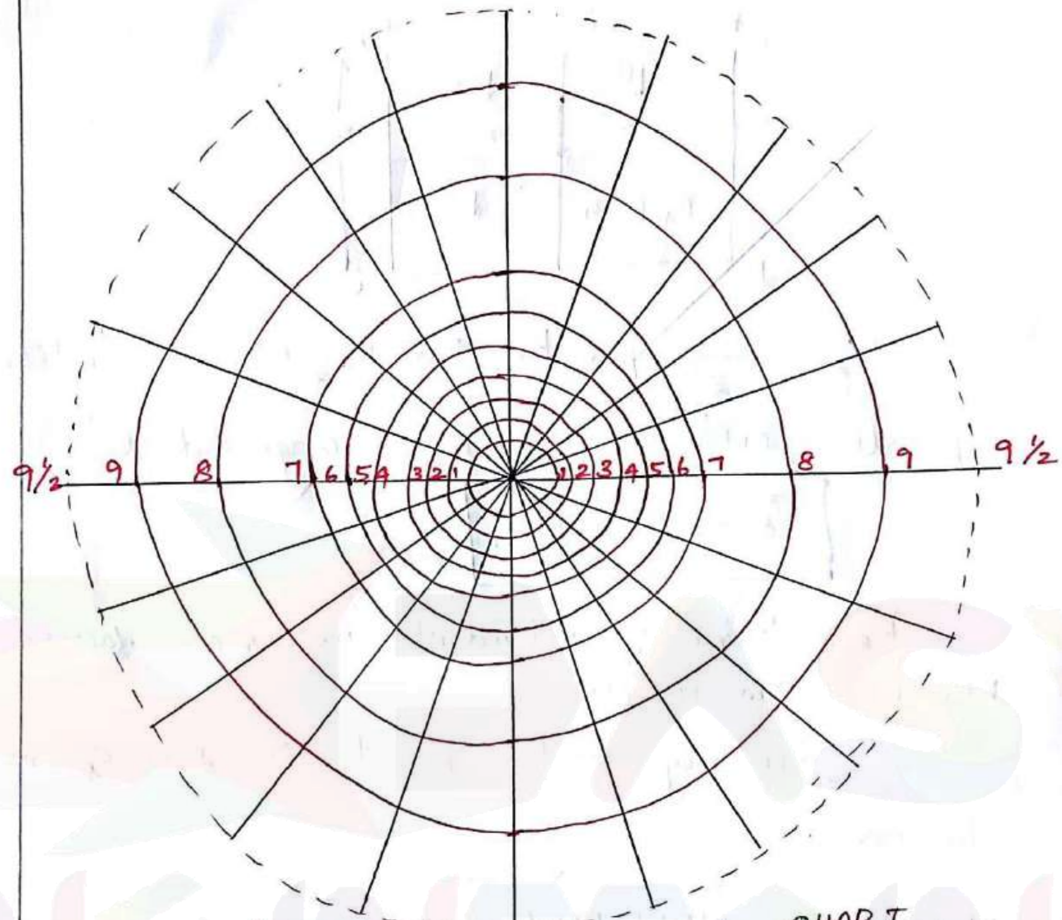
ΣK_B = Sum of individual influence factors for various area units.

Accuracy will depend on size of area unit choosen.

NEWMARK'S INFLUENCE CHART: [Accurate Method]

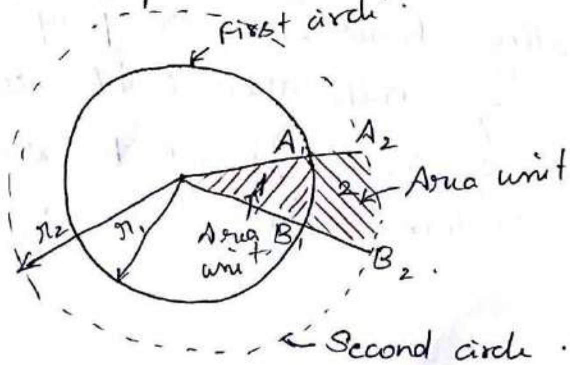
A more accurate method of determining the vertical stress at any point under a uniformly loaded area of any shape is with the help of influence chart or influence diagram suggested by Newmark (1942).

A chart, consisting of number of circles & radiating lines, is so prepared that the influence of each area unit is the same at centre of circles, i.e. each area unit causes the equal vertical stress at the centre of diagram.



NEWMARK'S INFLUENCE CHART

Let a uniformly loaded circular area of radius r_1 cm be divided into 20 sectors (area units). If q is the intensity of loading and σ_z is vertical pressure at depth z below centre of area, each unit such as OA_1B_1 exerts a pressure equal to $\sigma_z/20$ at centre.



From formula of σ_z from uniformly loaded circular area, (10)

$$\sigma_z = q\left[1-\left[\frac{1}{1+(a/z)^2}\right]^{3/2}\right].$$

Here $a = r_1$

$$\frac{\sigma_z}{20} = \frac{q}{20}\left[1-\left\{\frac{1}{1+\left(\frac{r_1}{z}\right)^2}\right\}^{3/2}\right]$$

$$\boxed{\frac{\sigma_z}{20} = i_f\, q}$$

i_f - influence value.

$$i_f = \frac{1}{20}\left[1-\left\{\frac{1}{1+(r_1/z)^2}\right\}^{3/2}\right].$$

If i_f be made equal to an arbitrary fixed value, say 0.005, we have.

$$\frac{\sigma_z}{20} = i_f\, q$$

$$= 0.005\, q.$$

$$i_f = \frac{1}{\text{No. of radial lines} \times \text{No. of concentric circles}}$$

$$= \frac{1}{20 \times 10}$$

$$\boxed{i_f = 0.005}$$

$$\frac{q}{20}\left[1-\left\{\frac{1}{1+(r_1/z)^2}\right\}^{3/2}\right] = 0.005\, q.$$

r_1 can be found from this equation, when z is known.

The pressure due to first concentric circle OA_1B_1 is $0.005q$.

The pressure due to second concentric circle $A_1B_1\, A_2B_2$ is $0.005q$.

Total pressure $OA_2B_2 = 2 \times 0.005 q$.

Illrly, the radii of 3rd, 4th, 5th, 6th, 7th, 8th, 9th circles can be calculated.

The radius of 10th circle is given by equation,

$$\frac{q}{20}\left[1-\left\{\frac{1}{1+\left(\frac{r_{10}}{z}\right)^2}\right\}^{3/2}\right] = 10\times 0.005q = q/20.$$

which means r_{10} = infinity.

Vertical pressure $\boxed{\sigma_A = 0.005q \times N_A}$

N_A - Number of area units under loaded area.

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ONE DIMENSIONAL CONSOLIDATION

When a compressive load is applied to soil mass, a decrease in its volume takes place. The decrease in volume of soil mass under stress is known as compression.

The property of soil mass pertaining to its susceptibility to decrease in volume under pressure is known as compressibility.

Voids - air - compressible - escape easily from void.

Saturated soil mass -

Voids - incompressible water - compression - water expelled out of voids.

Such a compression results from long term static load & consequent escape of pore water is termed as consolidation

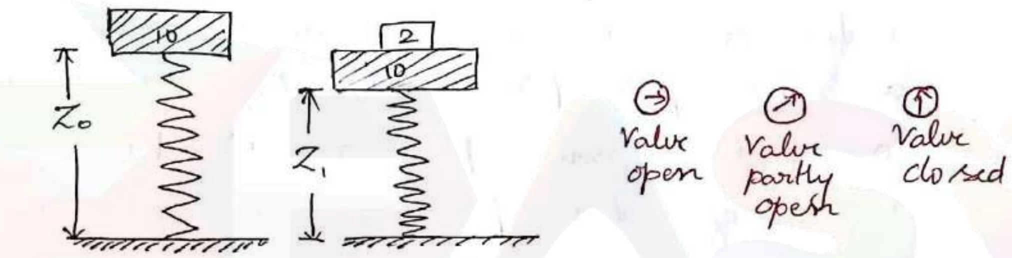
According to Terzaghi : "Every process involving a decrease in water content of a saturated soil without replacement of water by air is called a process of consolidation".

Consolidation - decrease in water content

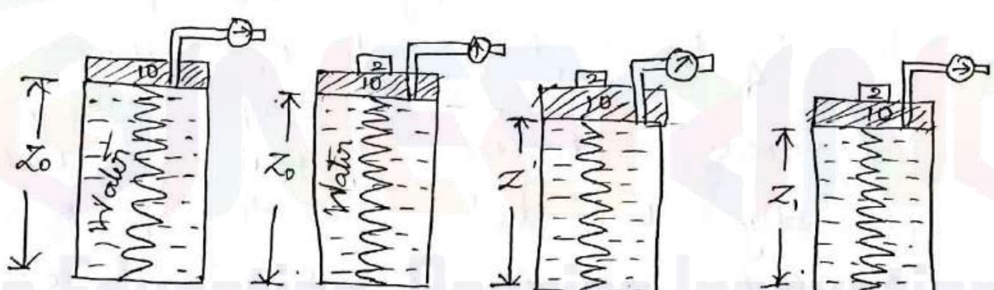
Swelling - increase in water content.

Compaction - Expulsion of air

SPRING ANALOGY.



a) $\sigma = \sigma'$ b) $\sigma = \sigma'$



c) $\sigma = \sigma' = 10$ d) $\sigma = \sigma' + \bar{u}$ e) $12 = (10 + \Delta\sigma') + (2 - \Delta\sigma')$ f) $\sigma = \sigma' + \bar{u}$

Total pressure = Pressure in spring + Pressure in water.

$$\sigma = \sigma' + u.$$

When there is a pressure increment, the whole of pressure is first taken by water. As water escapes out of system, the load transfer takes place from water to spring till spring is deformed by full amount corresponding to applied stress increment.

This analogy can be applied to consolidation process of soil mass consisting of soil water system.

Grain Structure - Spring.

Voids filled with water - cylinder.

Valve opening - Permeability of soil mass.

<u>Pore</u> <u>Pressure</u> ($\bar{u}$)

The pressure that builds up in pore water due to load increment on soil is terme as excess pore pressure / excess hydrostatic pressu or hydrodynamic pressure ($\bar{u}$). because it is in excess of initial pressure in water under stati condition.

The excess hydrostatic pressure forces the water to drain out of voids.

As water starts escaping from voids, the excess hydrostatic pressure in water gets gradually dissipated and the pressure incremen is shifted as an increase in effective pressure on soil solids and soil mass decrease in volume.

When whole of pressure increment or consolidation pressure is carried as an increase in effective pressure on solids, no more water escapes from voids and a condition of equilibrium is attained.

Hydrodynamic lag:

The delay caused in consolidation by the slow drainage of water out of saturated soil mass is called hydrodynamic lag.

Primary Consolidation:

The reduction in volume of soil which is due principally due to squeezing out of water from voids is termed primary consolidation primary compression or primary time effect.

Secondary Consolidation:

Even after reduction of all excess hydrostatic pressure to zero, some compression of soil takes place at very slow rate. This is known as secondary consolidation, secondary compression or secondary time effect.

During secondary compression, some of highly viscous water between the points of contact is forced out from b/w particles.

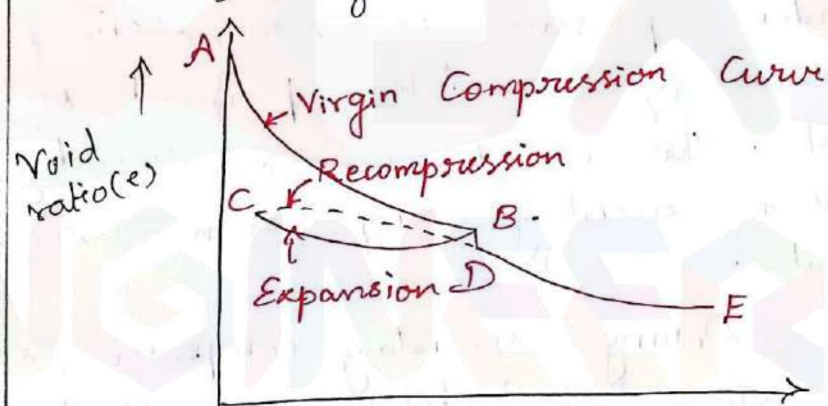
CONSOLIDATION OF LATERALLY CONFINED SOIL

If a remoulded soil is laterally confined in a consolidometer, consisting of a metal ring and porous stones are placed both at its top & bottom faces, the compression or consolidation of soil sample takes place under a vertical pressure applied on the top of porous stones. The porous stones provide free drainage of water and air from or into soil sample.

Under a given applied pressure, a final settlement & equilibrium voids ratio is attained after certain time.

At the equilibrium stage, the applied pressure naturally becomes effective pressure σ' on soil. The pressure can then be increased and a new equilibrium voids ratio is attained.

Thus, a relation can be obtained b/w effective pressure σ' & equilibrium voids ratio e in the form of curve.



Effective Pressure σ' (KN/m²) →

At any intermediate stage at B, the pressure is completely removed, the sample expands as represented by the expansion curve BC.

During expansion, the sample never attains the original void ratio, because of some permanent compression mainly due to some irreversible orientation undergone by soil particles under compression. (13)

If soil is again put under compression a recompression curve such as CD is obtained, the void ratio at D being always less than that at B. at the same pressure.

On further pressure increments, the curve DE is obtained.

The portion AB of the curve represents the compression of soil which has not been subjected in past to pressures greater than those which are being applied for the present compression. Such a curve is called virgin compression curve & curve DE is virgin curve.

σ' - abscissa } Semi-log plot.
e - ordinate } Virgin compression curve - Straight line.

It can be expressed by

$$\boxed{e = e_0 - C_c \log_{10} \frac{\sigma'}{\sigma_0'}}$$

e_0 - initial voids ratio corresponding to initial pressure σ_0'

e - void ratio at increased pressure σ'

C_c - Compression index (dimensionless).

The compression index represents slope of linear portion of the pressure - voids ratio curve & remains constant within a fairly large range of pressure.

$$\boxed{C_c = \frac{e_0 - e}{\log_{10} \frac{\sigma'}{\sigma_0'}}} = \frac{\Delta e}{\Delta \log_{10} \sigma'}$$

$$\boxed{\Delta e = C_c \log_{10} \frac{\sigma_0' + \Delta \sigma'}{\sigma_0'}}.$$

The expansion curve is also a fairly straight line on semilog plot & is expressed as

$$e_0 = e + C_s \log_{10} \frac{\sigma'}{\sigma_0'}$$

C_s - Expansion/ Swelling index. It is a measure of volume increase due to removal of pressure -

Skempton conducted consolidation tests on number of clays, & gave following equation.

$$\boxed{C_c = 0.007 (w_L - 10\%)}.$$

C_c of remoulded sample,

$$\boxed{C_c = 0.009 (w_L - 10\%)}$$

Hough gave equation for precompressed soils.

$$\boxed{C_c = 0.3 (e_0 - 0.27)}$$

e_0 - insitu void ratio.

Coefficient of Compressibility (a_v) :

It is defined as the decrease in void ratio per unit increase of pressure :

$$\boxed{a_v = \frac{-\Delta e}{\Delta \sigma'} = \frac{e_0 - e}{\sigma' - \sigma_0'}}.$$

For a given difference in pressure, value of coefficient of compressibility decreases as the pressure increases.

Coefficient of volume Change (m_v)

It is defined as the change in volume of a soil per unit of initial volume due to given unit increase in the pressure:

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

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

When soil is laterally confined, change in volume is proportional to change in thickness ΔH and initial volume is proportional to initial thickness H_0

$$\boxed{m_v = -\frac{\Delta H}{H_0}\,\frac{1}{\Delta\sigma'}}$$

SETTLEMENT:

The two types of settlement are.

i) Initial settlement

ii) Consolidation settlement

Initial Settlement :

The change in thickness, ΔH due to pressure increment is given by

$$\boxed{\Delta H = -m_v H_0 \,\Delta\sigma'}$$

(-) Sign - decrease of void ratio/thickness with increase in pressure.

Consolidation Settlement :

a) Using coefficient of volume change (m_v)

b) Using void ratio.

1) Final settlement using coefficient of volume change (m_v).

ρ_f - Consolidation settlement.

$\rho_f = m_v H \Delta \sigma'$

This is on assumption that pressure increment is transmitted uniformly over thickness H.

In practical cases, under a finite surface loading, intensity of $\Delta\sigma'$ decreases with depth of layer in a non-linear manner. In such circumstances, consolidation settlement $\Delta\rho_f$ of an element of thickness dz is calculated under an average effective pressure increment $\Delta\sigma'$.

$\Delta\rho_f = m_v \Delta\sigma' dz$

Integrating for total thickness H of layer.

$$\rho_f = \int_0^H m_v \Delta\sigma' dz.$$

m_v & $\Delta\sigma'$ are variables.

The numerical integration can be performed by dividing the total thickness H into no. of thin layers & $\Delta\sigma'$ at mid-height of each layer may be considered to represent a constant average pressure increment for the layer. Settlement of each layer can then be calculated.

Total Settlement = Sum of individual settlements of various thin layers.

$$\frac{\Delta H}{H} = \frac{e_0 - e}{1+e_0}$$

$$\rho_f = \Delta H = \frac{e_0 - e}{1+e_0} H.$$

i) Normally consolidated soils :

Compression index for normally consolidated soil is constant.

$$\rho_f = H \frac{C_c}{1+e_0} \log_{10} \frac{\sigma'}{\sigma_0'}$$

$$\sigma' = \sigma_0' + \Delta\sigma'$$

$$\boxed{\rho_f = H \frac{C_c}{1+e_0} \log_{10} \frac{\sigma_0' + \Delta\sigma'}{\sigma_0'}}$$

ii) Preconsolidated soils :

Swelling index $C_s < C_c$.

$$\boxed{\rho_f = H \frac{C_s}{1+e_0} \log_{10} \frac{\sigma_0' + \Delta\sigma'}{\sigma_0'}}$$

CONSOLIDATION OF UNDISTURBED SPECIMEN :

OCR - Over Consolidation Ratio

Soil deposits may be divided into 3 classes.

* Pre-consolidated or overconsolidated - OCR > 1, U > 100
* Normally consolidated - OCR = 1, U = 100
* Under consolidated - OCR < 1, U < 100

Clay - Precompressed / Preconsolidated / Overconsolidated if it has been subjected to pressure in excess of its present overburden pressure. The temporary overburden pressure is known as preconsolidation pressure.

Normally Consolidated soil: is one which has never been subjected to an effective pressure greater than existing overburden pressure & which is completely consolidated by existing overburden.

Under. Consolidated soil:

A soil which is not fully consolidated under the existing overburden pressure is called an under-consolidated soil.

TERZAGHI'S THEORY OF ONE DIMENSIONAL CONSOLIDATION:

Assumptions:

* The soil is homogeneous and fully saturated.
* Soil mass & water are incompressible
* Deformation of soil is entirely due to change in volume
* Darcy's law for velocity of flow of water through soil is perfectly valid.
* Coefficient of permeability is constant during consolidation.
* Load is applied in one direction only & deformation occurs only in direction of load application. i.e Soil is restrained against lateral deformation

* Excess pore water drains out only in vertical direction. (16)

* Boundary is free surface offering no resistance to flow of water from soil.

* Change in thickness during consolidation is insignificant.

* Time lag in consolidation is entirely due to permeability of soil & thus secondary consolidation is disregarded.

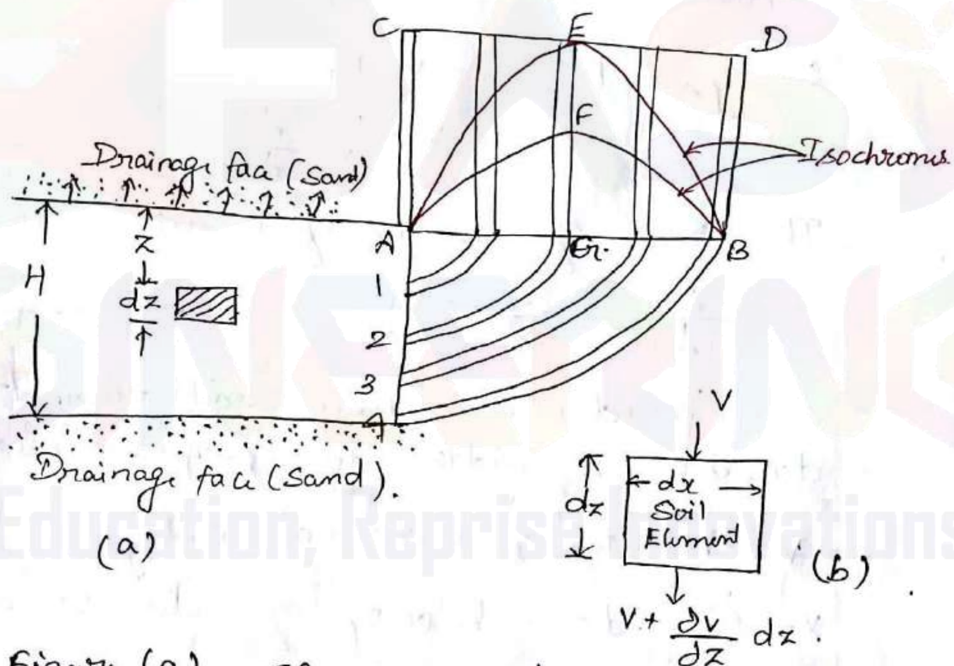


Figure (a) Shows a clay layer of thickness H sandwitched b/w two layers of sand which serves as drainage faces.

When layer is subjected to pressure increment $\Delta\sigma$, excess hydrostatic pressure is set up in clay layer.

At time t_0 – $\Delta\sigma = \bar{u}$ – CED

At time t_f: $\bar{u} = 0$ · - AFB

At any time t, $\Delta\sigma = \Delta\sigma' + \bar{u}$ - AFB.

$\bar{u} = h\,\gamma_w$.

Hydraulic head, $h = \dfrac{\bar{u}}{\gamma_w}$ —— ①

Hydraulic gradient $i = \dfrac{dh}{dz} = \dfrac{d}{dz}\left(\dfrac{\bar{u}}{\gamma_w}\right)$.

$$i = \frac{1}{\gamma_w}\frac{d\bar{u}}{dz} \quad \text{—— ②}.$$

Thus, rate of change of $\bar{u}$ along depth of layer represents hydraulic gradient.

Darcy's law $v = ki$

$$v = \frac{k}{\gamma_w}\frac{d\bar{u}}{dz}. \quad \text{—— ③}$$

The rate of change of velocity is

$$\frac{dv}{dz} = \frac{k}{\gamma_w}\frac{d^2\bar{u}}{dz^2} \quad \text{—— ④.}$$

Consider a small soil element of size dx, dz & width dy perpendicular to xz plane.

v - Velocity of water @ entry

$v + \dfrac{dv}{dz}\cdot dz$ - Velocity of water @ exit.

Quantity = Velocity × Area.

Quantity of water entering soil = $v\,dx\,dy$

Quantity of water leaving soil = $\left[v + \dfrac{dv}{dz}dz\right]dx\,dy$.

Net quantity of water squeezed out of soil

= Quantity leaving soil (−) Quantity entering soil.

$$\Delta q = -v\, \cancel{dx}\, dy + v\, dx\, dy + \frac{\partial v}{\partial z}\, dx\, dy\, dz$$ (19)

$$\Delta q = \frac{\partial v}{\partial z}\, dx\, dy\, dz \qquad \text{--- (5)}$$

Decrease in volume of soil is equal to volume of water squeezed out.

$$\Delta V = -m_v V_0 \Delta\sigma' \qquad \text{--- (6)}$$

$$V_0 = dx\, dy\, dz$$

Change of volume per unit time

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

Equating (5) & (7)

$$\Delta q = \frac{\partial(\Delta V)}{\partial t}$$

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

Combining (A) & (8)

$$\frac{K}{\gamma_w}\frac{\partial^2 \bar{u}}{\partial z^2} = -m_v \frac{\partial(\Delta\sigma')}{\partial t}$$

$$\Delta\sigma = \Delta\sigma' + \bar{u} \implies \Delta\sigma \text{ is constant}$$

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

$$\frac{K}{\gamma_w}\frac{\partial^2 \bar{u}}{\partial z^2} = -m_v \frac{\partial \bar{u}}{\partial t}$$

$$\boxed{\frac{\partial \bar{u}}{\partial t} = \frac{-K}{m_v \gamma_w} \cdot \frac{\partial^2 \bar{u}}{\partial z^2}}$$

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

C_v - Coefficient of consolidation

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

C_v - cm^2/sec

This is the basic differential equation of consolidation which relates rate of change of excess hydrostatic pressure to rate of expulsion of excess pore water from unit volume of soil during same time interval.

The solution of consolidation equation is

$$\boxed{T_v = \frac{K}{m_v \gamma_w} \frac{t}{d^2}} \Longrightarrow \boxed{T_v = \frac{C_v t}{d^2}}$$

$$\boxed{T_v = \frac{K(1+e_0)}{a_v \gamma_w} \frac{t}{d^2}}$$

$$\frac{T_v}{C_v} = \text{constant}$$

$$t \propto d^2$$

T_v - Time factor

t - Thickness of clay layer.

Degree of consolidation

$$U = \frac{\text{Excess pore pressure dissipated}}{\text{Initial excess}} \times 100$$

$$= \frac{\text{Initial excess} - \text{Present excess}}{\text{Initial excess}} \times 100.$$

If initial excess & Present excess is same, then $U = 0$ - No consolidation.

If Present excess is 0, $U = 100\%$. - Full consolidation.

$$\boxed{U = \frac{\text{Present Settlement}}{\text{Ultimate settlement}} \times 100}$$

If $U < 60\%$, $T_v = \frac{\pi}{4}\left(\frac{U\%}{100}\right)^2$

If $U > 60\%$, $T_v = 1.7813 - 0.9332 \log_{10}(100 - U\%)$

LABORATORY CONSOLIDATION TEST

Apparatus - Consolidometer.

* Consists of loading frame & Consolidation cell.
* Porous stones are put on top & bottom ends of specimen.
  - Floating ring cell
  - Fixed ring cell.

	Fixed ring cell	Floating ring cell
1.	Only top porous stone is permitted to move downwards as specimen compresses.	Both top & bottom porous stones are free to compress the specimen towards middle.
2.	Permeability of specimen at any stage can be directly measured.	Permeability of specimen at any stage cannot be directly measured. Smaller effects of friction b/w specimen ring & soil specimen

Vertical compression of specimen is measured by means of dial gauge.

After completion of consolidation under desired maximum vertical pressure, specimen is unloaded & allowed to swell. Final dial reading is recorded & specimen is taken out. Test data are used to determine

1. Voids ratio & coefficient of volume change.
2. Coefficient of consolidation
3. Coefficient of permeability.

1. Determination of voids ratio & coefficient of volume change:

Two methods : i) Height of solids method.
ii) Change in voids ratio method

Change in voids ratio - Only for fully saturated specimens.

Height of solids - Both saturated as well as unsaturated specimens.

i) Height of solids method:

Height of solids $H_s = \frac{M_d}{G A \rho_w} = \frac{W_d}{G A}$

H_s - Height of solids (cm)
M_d - Mass of dried specimen (g)
W_d - Weight of dried specimen (g)
A - C/S area of specimen (cm^2)
G - Specific gravity of soil.

void ratio $\boxed{e = \frac{H - H_s}{H_s}}$

H = Specimen height @ equilibrium

$H = H_0 + \Sigma \Delta H = H_1 + \Delta H$

H_0 - Initial height of specimen

ΔH = change in specimen thickness under any pressure increment

H_1 - Height of specimen at beginning of load increment.

ii) Change in voids ratio method :

$$e_f = w_f G$$

$eS_r = wG$.
If $S_r = 1$
$e = wG$

e_f - Final void ratio
w_f - Final water content.

$$\boxed{\frac{\Delta e}{1+e} = \frac{\Delta H}{H}}$$

$$\boxed{\Delta e = \frac{1+e_f}{H_f} \Delta H}$$

H_f - Final height of specimen.

Coefficient of volume change :

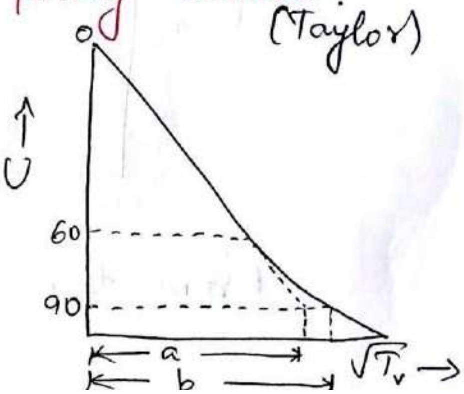
$$\boxed{m_v = -\frac{\Delta e}{1+e_0} \frac{1}{\Delta \sigma'}}$$

$$\boxed{m_v = -\frac{\Delta H}{H_0} \frac{1}{\Delta \sigma'}}$$

2. Determination of coefficient of consolidation :
Two methods :
i) Square root of time fitting method.
ii) Logarithm of time fitting method.

i) Square root of time fitting method : (Taylor)

Figure shows theoretical characteristic curve b/w degree of consolidation & $\sqrt{T_v}$



Upto $U = 60\%$, curve is straight.

Abscissa @ $U = 90\% = 1.15$ (Abscissa @ $U = 60\%$).

Abscissa $= \sqrt{t}$
Ordinate $= R$

R - dial reading
t - Time.

R_0 - Initial dial reading
R_c - Corrected zero reading.

Consolidation b/w R_0 & R_c - Initial consolidation.

From R_c, B is drawn,

$B = 1.15A$.

Intersection of B with consolidation curve $= U$-90%.

$$\boxed{(T_v)_{90} = 0.848}$$

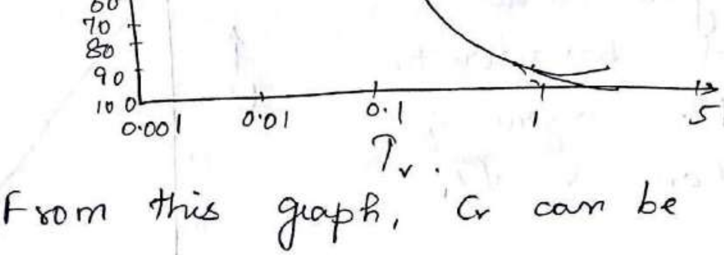
$$\boxed{C_v = \frac{0.848\, d^2}{t_{90}}}$$

From this C_v can be calculated.

ii) Logarithm of time fitting method: (Casagrande).

Abscissa - $\log_{10} T_v$
Ordinate - $U\%$.

Semi log plot of lab time-consolidation curve.



From this graph, C_v can be calculated.

3. Determination of coefficient of permeability

A falling head permeability test can be performed on consolidation specimen attaching a stand pipe to fixed ring consolidometer, when the consolidation of specimen is complete under a particular pressure increment.

Coefficient of permeability

$K = C_v m_v \gamma_\omega$

$$\boxed{K = \frac{C_v a_v \gamma_w}{1+e_0}}$$

Knowing C_v & m_v, K can be calculated.

PROBLEMS:

1. An undisturbed sample of clay 24mm thick, consolidated 50% in 20 minutes, when tested in laboratory with drainage allowed at top & bottom. The clay layer, from which sample was obtained, is 4m thick in field. How much time will it take to consolidate 50%, with double drainage? If clay stratum has only single drainage, calculate the time to consolidate 50%. Assume uniform distribution of consolidation pressure.

Same degree of consolidation, T_v - same.

Both soils are same, C_v - Same.

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

$$t \propto d^2$$

a) Double drainage :

$$t \propto d^2$$

Field - 2
Lab - 1

$$\frac{t_2}{t_1} \propto \left(\frac{d_2}{d_1}\right)^2$$

$U = 50\%$

$t_1 = 20$ min

$t_2 = ?$

Double drainage $= d/2$.

$d_1 = 24\text{ mm}/2$

$= 12\text{mm}$

$= 0.012\text{ m}$.

$d_2 = 4\text{ m}/2 = 2\text{m}$.

$$t_2 = t_1\left(\frac{d_2}{d_1}\right)^2 = 20\left[\frac{2}{0.012}\right]^2 = 555,566.67 \text{ min.}$$

$$= \frac{555566.67}{24 \times 60} \text{ days}$$

$$\boxed{t_2 = 386 \text{ days}}.$$

b) Single drainage :

$$t \propto d^2$$

$$\boxed{\begin{array}{l} d = H \text{ - Single} \\ d = H/2 \text{ - Double} \end{array}}$$

$$t_2 = t_1\left(\frac{d_2}{d_1}\right)^2.$$

$t_1 = 20$ mins.

$d_1 = 24\text{ mm}/2 = 12\text{ mm} = 0.012\text{m}$. [Double]

$d_2 = 4$ m [Single].

$$t_2 = 20\left[\frac{4}{0.012}\right]^2 = 2,222,220 \text{ min}$$

$$\boxed{t_2 = 1544 \text{ days}}$$

$T_{\text{single drainage}} = \cancel{4}\,4 \times T_{\text{double drainage}}$.

$$\boxed{T_S = 4\,T_D}$$

2. An undisturbed sample of clay stratum 2m thick, was tested in laboratory & average value of coefficient of consolidation was found to be 2×10^{-4} cm²/sec. If a structure is built on clay stratum, how long it will take to attain half the ultimate settlement under load of structure? Assume double drainage:

$C_v = 2 \times 10^{-4}$ cm²/sec.

$H = 2$ m.

$U = 50\%$ Double drainage

$t = ?$

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

$d = H/2 = 2/2 = 1\text{m} = 100\text{ cm}$

$$U < 60\%, \quad T_v = \frac{\pi}{4}\left(\frac{U}{100}\right)^2$$

$$T_v = \frac{\pi}{4}\left(\frac{50}{100}\right)^2$$

$$\boxed{T_v = 0.197}$$

$$t = \frac{0.197 \times (100)^2}{2 \times 10^{-4}}$$

$t = 9850000$ secs $/ 24 \times 60 \times 60$

$$\boxed{t = 114 \text{ days}}$$

Time to attain half the ultimate settlement $= 114$ days.

3. Two clay specimens A & B of thickness 2cm & 3cm, have equilibrium void ratio 0.68 & 0.72 respectively under a pressure of 200 KN/m². If the equilibrium void ratios of the two soils reduced to 0.5 & 0.62 respectively when pressure was increased to 400 KN/m², find ratio of coefficient of permeability of two specimens. The time required by specimen A to reach 40% degree of consolidation is 1/4 of that required by specimen B for reaching 40% degree of consolidation.

$U = 40\%, \quad t_A = \frac{1}{4} t_B.$

$H_A = 2cm = d_A \quad H_B = 3cm = d_B$

$\sigma = 200 \ KN/m^2, \quad e_A = 0.68, \quad e_B = 0.72.$

$\sigma = 400 \ KN/m^2, \quad e_A = 0.5, \quad e_B = 0.62.$

$$C_v = \frac{K}{m_v \gamma_\omega}. \qquad \gamma_{\omega A} = \gamma_{\omega B}.$$

$$\frac{C_{vA}}{C_{vB}} = \frac{\frac{K_A}{m_{vA} \cancel{\gamma_{\omega A}}}}{\frac{K_B}{m_{vB} \cancel{\gamma_{\omega B}}}} = \frac{K_A}{K_B} \cdot \frac{m_{vB}}{m_{vA}}.$$

Coefficient of permeability - K $\quad \frac{K_A}{K_B} = ?$

$$\frac{K_A}{K_B} = \frac{C_{vA}}{C_{vB}} \cdot \frac{m_{vA}}{m_{vB}}.$$

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

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

$$\frac{T_{vA}}{T_{vB}} = \frac{C_{vA} t_A}{C_{vB} t_B} \cdot \left(\frac{d_B}{d_A}\right)^2$$

$U = 40\%$. $-$ $T_{vA} = T_{vB}$.

$t_B = 4 t_A$

$$\frac{C_{vA}}{C_{vB}} = \frac{t_B}{t_A}\left(\frac{d_B}{d_A}\right)^2$$

$$= \frac{4\cancel{t_A}}{\cancel{t_A}}\left(\frac{3}{2}\right)^2$$

$$\boxed{\frac{C_{vA}}{C_{vB}} = 9}$$

$$\cancel{m_{vA} = \frac{0.72 - 0.68}{1+0.}}$$

$$m_{vA} = \frac{0.68 - 0.5}{1+0.68} / (400 - 200)$$

$$= 5.36 \times 10^{-4} \ m^2/KN.$$

$$m_{vB} = \frac{0.72 - 0.62}{1+0.72} / 200$$

$$= 2.91 \times 10^{-4} \ m^2/KN.$$

$$\boxed{\frac{m_{vA}}{m_{vB}} = 1.842}$$

$$\frac{m_{vA}}{m_{vB}} = \frac{5.36 \times 10^{-4}}{2.91 \times 10^{-4}} = 1.842$$

$$\frac{K_A}{K_B} = \frac{C_{vA}}{C_{vB}} \cdot \frac{m_{vA}}{m_{vB}} = 9 \times 1.842$$

$$\boxed{\frac{K_A}{K_B} = 16.58}$$

4. The loading period for a new building extended from July 1980 to July 1982. In July 1985, the average measured settlement was found to be 6.78 cm. If it is known that ultimate settlement will be about 25cm, Estimate settlement in July 1991. Assume double drainage to occur.

July 1981 - July 1985 - 4 years.

$t_1 = 4$ yrs $= 6.78$ cm.

Degree of consolidation

Ultimate settlement = 25cm

$$U = \frac{6.78}{25}$$

4 yrs, $U = 27.12\%$.

July 1981 - July 1991 - 10 yrs.

$$U_{10} = \frac{\rho_{10}}{25} \qquad \rho \text{ - Settlement}.$$

$$U_{10} = 0.04\,\rho_{10}$$

$$T_v = \frac{C_v t}{d^2}, \quad T_v = \frac{\pi}{4}\left(\frac{U}{100}\right)^2$$

$$\frac{\dfrac{\cancel{C_{v_1}} t_1}{\cancel{d_1^2}}}{\dfrac{\cancel{C_{v_2}} t_2}{\cancel{d_2^2}}} = \frac{\cancel{\frac{\pi}{4}}\left(\frac{U_1}{\cancel{100}}\right)^2}{\cancel{\frac{\pi}{4}}\left(\frac{U_2}{\cancel{100}}\right)^2}$$

$$\frac{t_1}{t_2} = \left(\frac{U_1}{U_2}\right)^2$$

$$\frac{4}{10} = \left(\frac{0.2712}{0.04\,\rho_{10}}\right)^2$$

Settlement in July 1991 = 10.72 cm

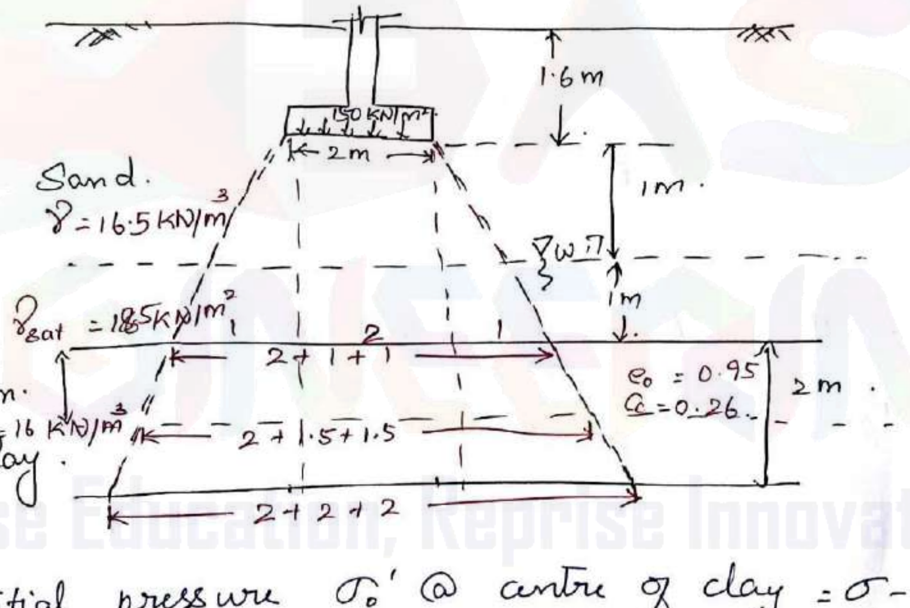
$$\rho_{10}^2 = \left(\frac{0.2712}{0.04}\right)^2 \times \frac{10}{4}$$

$$\boxed{\rho_{10} = 10.72\text{ cm}}$$

5. A building column has a footing area of 2m × 3m & transmits a pressure increment of 150 KN/m² at its base embedded 1.6m below ground level. Assuming pressure distribution of 2 vertical to 1 horizontal, Determine consolidation settlement @ middle of clay layer. Consider pressure variation across thickness of clay layer also. Given the following.

i) For sand, $\gamma = 16.5 KN/m^3$ & $\gamma_{sat} = 18.5 KN/m^3$.

ii) For clay, $\gamma_{sat} = 16 KN/m^3$, $e_0 = 0.95$, $C_c = 0.26$.



Initial pressure σ_0' @ centre of clay $= \sigma - u$

$= (2.6 \times 16.5) + (18.5 \times 1) + (16 \times 1) - (1 \times 9.81) - (1 \times 9.81)$

$= (2.6 \times 16.5) + (18.5 - 9.81) + (16 - 9.81)$

$= 57.78 \ KN/m^2$.

$$\boxed{\sigma_0' = 57.78 \ KN/m^2}$$

Pressure increase @ top, middle & bottom of clay layer.

$$Area = 2 \times 3\ m.$$

$$(\Delta\sigma)_t = \frac{150 \times 2 \times 3}{(2+2)\ (3+2)} = 45\ KN/m^2.$$

$$(\Delta\sigma)_m = \frac{150 \times 2 \times 3}{(2+3)(3+3)} = 30\ KN/m^2.$$

$$(\Delta\sigma)_b = \frac{150 \times 2 \times 3}{(2+4)(3+4)} = 21.43\ KN/m^2.$$

Average pressure is found by Sympson's rule:

$$\Delta\sigma = \frac{1}{6}\left[\Delta\sigma_t + 4\Delta\sigma_m + \Delta\sigma_b\right]$$

$$= \frac{1}{6}\left[45 + (4 \times 30) + 21.43\right]$$

$$= 31.07\ KN/m^2.$$

Settlement @ middle of clay layer:

$$e_f = \frac{C_c}{1+e_0} H \log_{10} \frac{\sigma_0' + \Delta\sigma}{\sigma_0'}$$

$$= \frac{0.26}{1+0.95} \times 2 \times \log_{10} \frac{57.78 + 31.07}{57.78}$$

$$= 0.0498\ m$$

$$\boxed{e_f = 49.8\ mm}.$$

———— x ————

Secondary Consolidation:

When excess pore pressure due to consolidation has been dissipated, the change in void ratio continues, but at reduced rate. This phenomenon is secondary consolidation. It is very small, so it is neglected.

—— x —— x ——