LECTURE NOTES FOR WEB LEARNING

Consolidation of Soils

Geotechnical Engg-I
Unit-6
(06CV54)

BY

Dr. S V Dinesh

Professor

Civil Engineering Department

Siddaganga Institute of Technology

Tumkur

6.0 Syllabus

6.1 Introduction and Definition
6.2 Mass-spring analogy
6.3 Terzaghi's one dimensional consolidation theory-assumption and limitation (No derivation)
6.4 Normally Consolidated soils
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6.6 Over consolidated soils
6.7 Pre-consolidation pressure and its determination by Casagrande's method
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6.9 Time rate of consolidation

6.1 Introduction and Definition

Civil Engineers build structures and the soil beneath these structures is loaded. This results in increase of stresses resulting in strain leading to settlement of stratum. The settlement is due to decrease in volume of soil mass. When water in the voids and soil particles are assumed as incompressible in a completely saturated soil system then - reduction in volume takes place due to expulsion of water from the voids. There will be rearrangement of soil particles in air voids created by the outflow of water from the voids. This rearrangement reflects as a volume change leading to compression of saturated fine grained soil resulting in settlement. The rate of volume change is related to the rate at which pore water moves out which in turn depends on the permeability of soil.

Therefore the deformation due to increase of stress depends on the "Compressibility of soils"

As Civil Engineers we need to provide answers for

1. Total settlement (volume change)
2. Time required for the settlement of compressible layer

The total settlement consists of three components

1. Immediate settlement.
2. Primary consolidation settlement
3. Secondary consolidation settlement (Creep settlement)

$$S_t = S_i + S_c + S_{sc}$$

Elastic Settlement or Immediate Settlement

This settlement occurs immediately after the load is applied. This is due to distortion (change in shape) at constant volume. There is negligible flow of water in less pervious soils. In case of pervious soils the flow of water is quick at constant volume. This is determined by elastic theory (E & μ are used).

Primary Consolidation Settlement

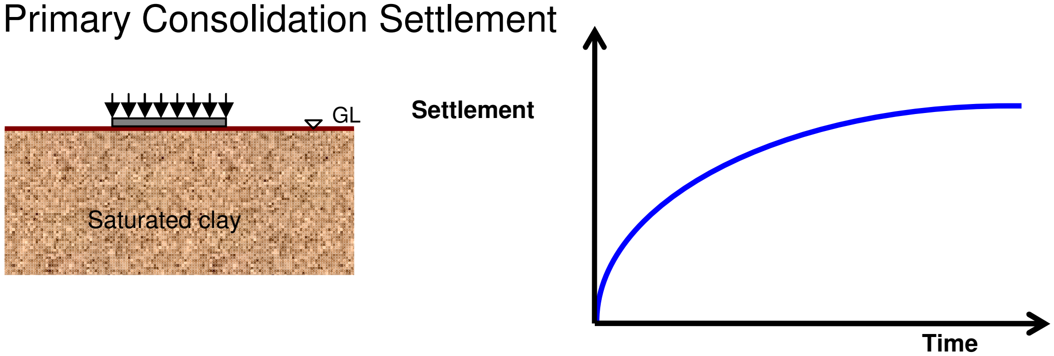


Fig. 1 Settlement versus Time

It occurs due to expulsion of pore water from the voids of a saturated soil. In case of saturated fine grained soils, the deformation is due to squeezing of water from the pores leading to rearrangement of soil particles. The movement of pore water depends on the permeability and dissipation of pore water pressure. With the passage of time the pore water pressure dissipates, the rate of flow decreases and finally the flow of water ceases. During this process there is gradual dissipation of pore water pressure and a simultaneous increase of effective stress as shown in Fig 1. The consolidation settlement occurs from the time water begins move out from the pores to the time at which flow ceases from the voids. This is also the time from which the excess pore water pressure starts reducing (effective stress increase) to the time at which complete dissipation of excess pore water pressure (total stress equal to effective stress). This time dependent compression is called "Consolidation settlement"

Primary consolidation is a major component of settlement of fine grained saturated soils and this can be estimated from the theory of consolidation.

In case of saturated soil mass the applied stress is borne by pore water alone in the initial stages

$\therefore$ At $t = 0$ $\quad \Delta\sigma = \Delta u \quad \Delta\sigma' = 0$

With passage of time water starts flowing out from the voids as a result the excess pore water pressure decreases and simultaneous increase in effective stress will takes place. The volume change is basically due to the change in effective stress $\Delta\sigma'$. After considerable amount of time ($t = \infty$) flow from the voids ceases the effective stress stabilizes and will be is equal to external applied total stress ($\Delta\sigma$) and this stage signifies the end of primary consolidation.

At $t = t_1$ $\quad \Delta\sigma = \Delta\sigma' + \Delta u$

At $t = \infty$ $\quad \Delta\sigma = \Delta\sigma' \quad \Delta u = 0$ (End of primary consolidation)

Secondary Consolidation Settlement:-

This is also called Secondary compression (Creep). "It is the change in volume of a fine grained soil due to rearrangement of soil particles (fabric) at constant effective stress". The rate of secondary consolidation is very slow when compared with primary consolidation.

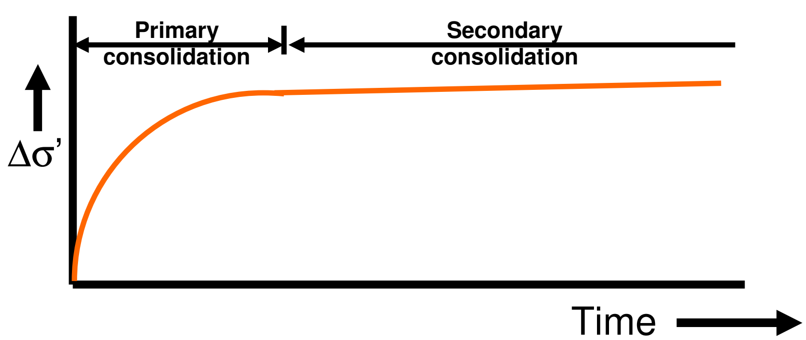


Fig. 2 Effective stress versus Time

Excess Pore water Pressure (Δu)

"It is the pressure in excess of the equilibrium pore water pressure". It is represented as Δu.

$$\Delta u = h\,\gamma_w$$

Where h --- Piezometric head

γ_w --- Unit weight of water

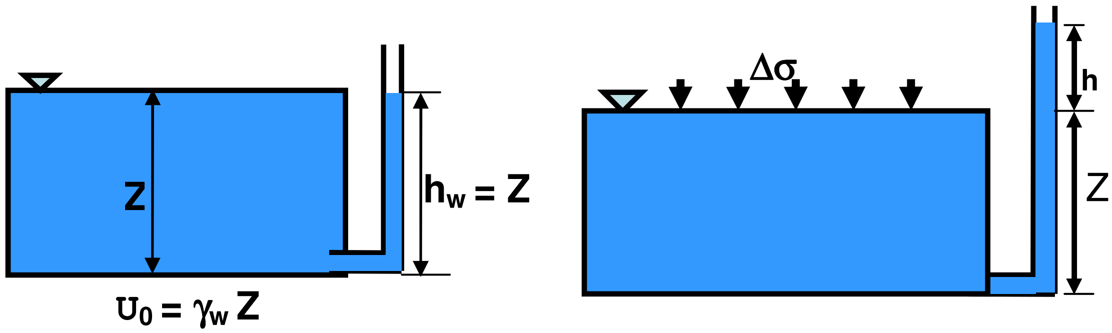


Fig. 3: Excess pore water pressure build up

Compressibility of saturated soil

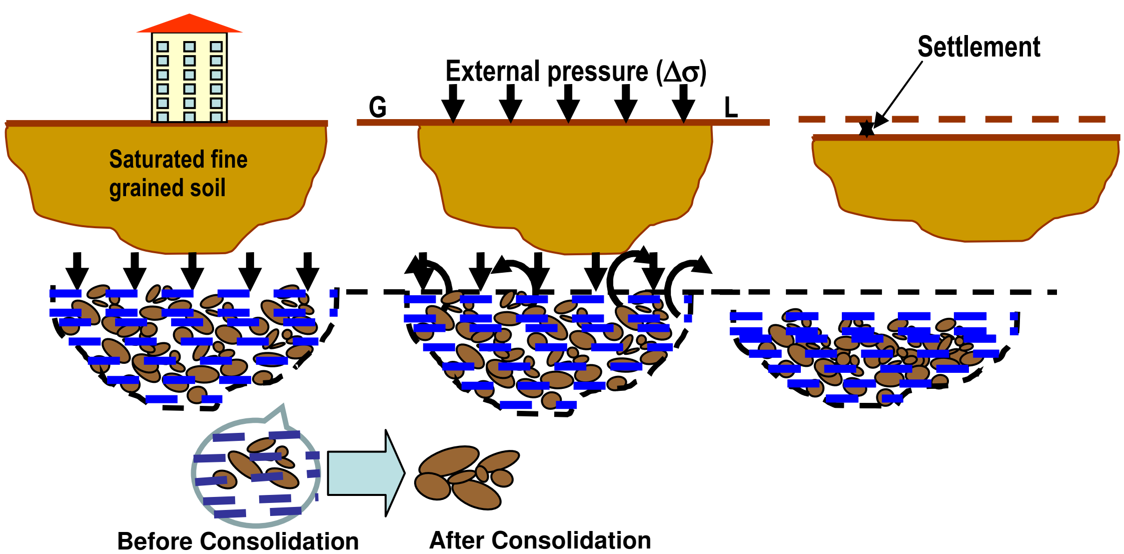


Fig. 4: Mechanism of volume change in saturated fine grained soil under external loading

When saturated soil mass is subjected to external load decrease in volume takes place due to rearrangement of soil particles. Reduction in volume is due to expulsion of water from the voids. The volume change depends on the rate at which water is expelled and it is a function of permeability.

The total vertical deformation (Consolidation settlement) depends on

1. Magnitude of applied pressure (**σΔ**)
2. Thickness of the saturated deposit

We are concerned with

- Measurement of volume change
- The time duration required for the volume change

6.2 Trezaghi's Spring Mass Analogy

Terzaghi's model consists of a cylindrical vessel with a series of piston separated by springs. The space between springs is filled with water the pistons are perforated to allow for passage of water. Piezometers are inserted at the centers of different compartment to measure the pressure head due to excess pore water pressure.

Terzaghi has correlated the spring mass compression process with the consolidation of saturated clay subjected to external load Δσ.

The springs and the surrounding water represent the saturated soil. The springs represent the soil skeleton networks of soil grains and water in the vessels represents the water in the voids. In this arrangement the compression is one dimensional and flow will be in the vertical direction.

When pressure Δσ is applied this will be borne by water surrounding the spring

$\Delta\sigma = \Delta u$ at time $t = 0$

Δu is called excess hydrostatic pressure due to this water level in all the Piezometer reach the same height 'h' given by $h = \frac{\Delta u}{\gamma_w}$

$\Delta\sigma = \Delta u$ and $\Delta\sigma' = 0$ --------- $t = 0$

There will be no volume change.

After sometime't' there will be flow of water through perforation beginning from upper compartment. In the lower compartment the volume of water remains constant since the flow is in upward direction.

Due to flow of water in the upper segment there will be reduction in volume due to this springs get compressed and they being to carry a portion of the applied load. This signifies a reduction in excess hydrostatic pressure or pore water pressure and increase in effective stress in the upper segments. Whereas there will be no dissipation of excess hydrostatic pressure in lower compartments.

At time t1, t2,--------t=∞ the variation of excess hydrostatic pressure are as indicated by the Isochrones shown in Fig 5. The isochrones indicate that with passage of time there is flow of water from the lower compartments leading to gradual dissipation of excess hydrostatic pressure. At time t = ∞ when no more porewater flows out the excess hydrostatic pressure will be zero in all compartments and the entire load is carried by springs.

At time t = ∞

$$\Delta\sigma = \Delta\sigma' \qquad \Delta u = 0$$

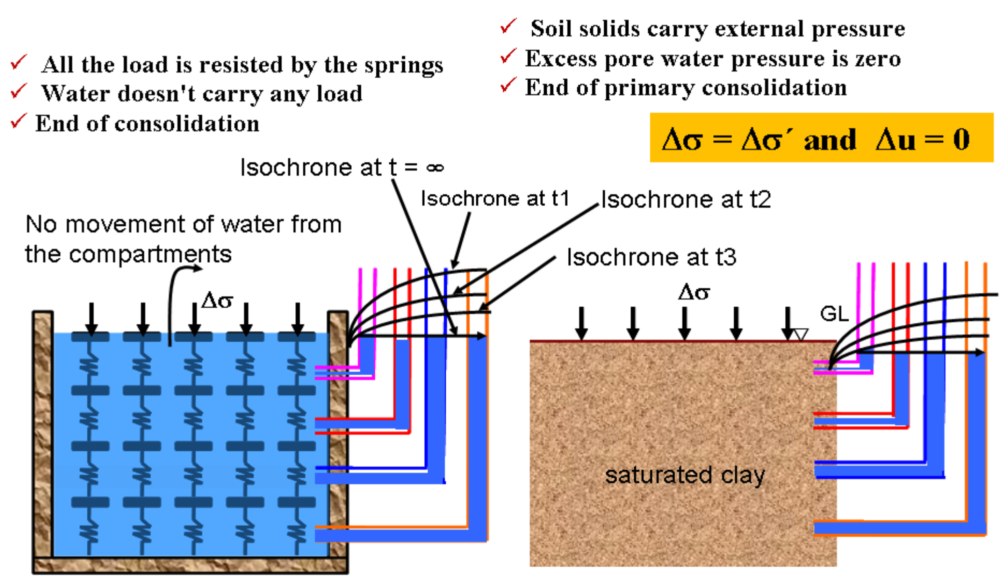
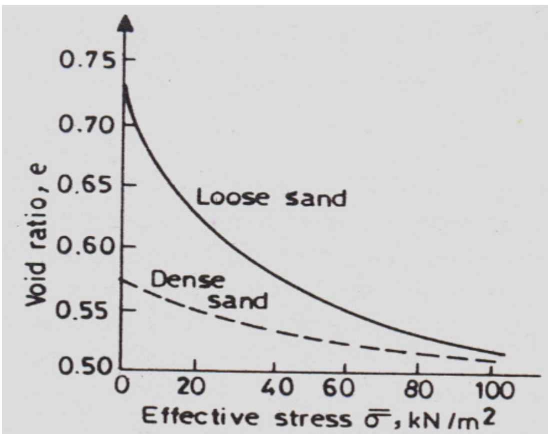


Fig. 5: Compression of spring mass

The compression of a spring mass system is analogous to the consolidation of a saturated fine grained soil deposit subjected to external pressure (Δσ)

Soil Compressibility

Compression of Sand



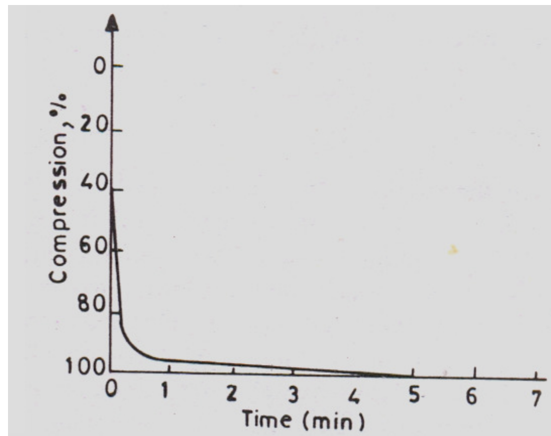
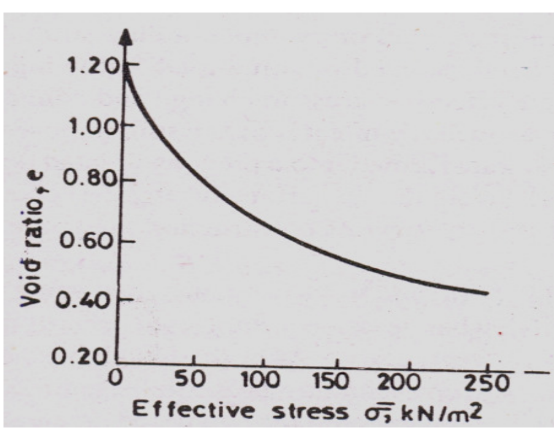


Fig. 6: Void ratio-effective stress and compression-time plots for sand

Sand deposit compresses immediately on load application. Loose sand compresses more than dense sand. Loose and dense sand deposits tend towards the same void ratio

Compression of fine grained soil (Clay)



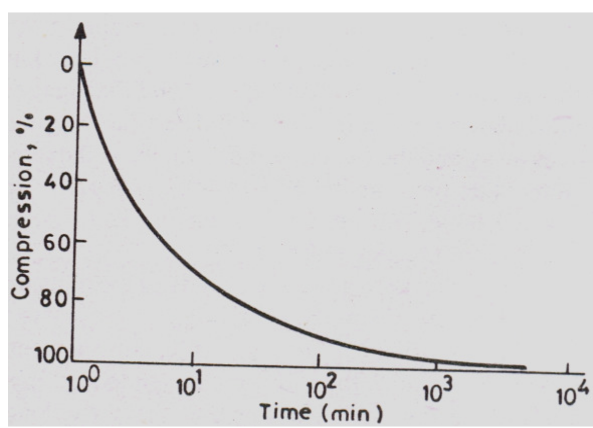


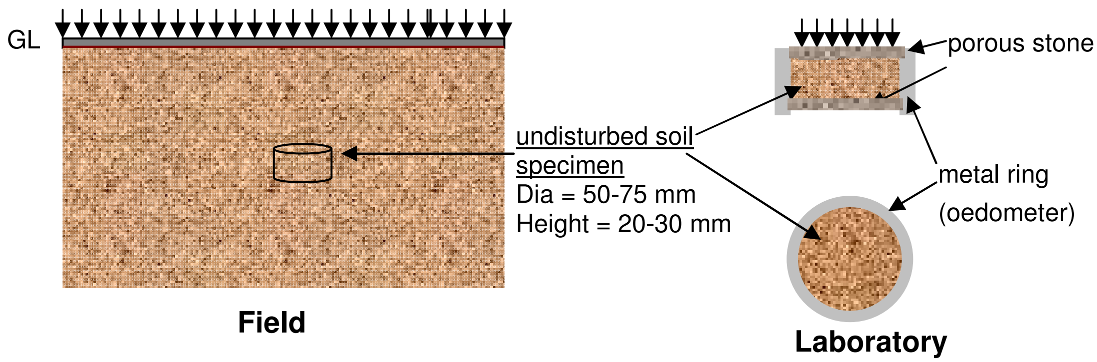
Fig. 6: Void ratio-effective stress and compression-time plots for clay

Time dependent compression takes longer time compared to sand. The magnitude of compression is also large.

Compression of fine grained soil

The compressibility of fine grained soils can be described in terms of voids ratio versus effective stress

~ simulation of 1-D field consolidation in lab.



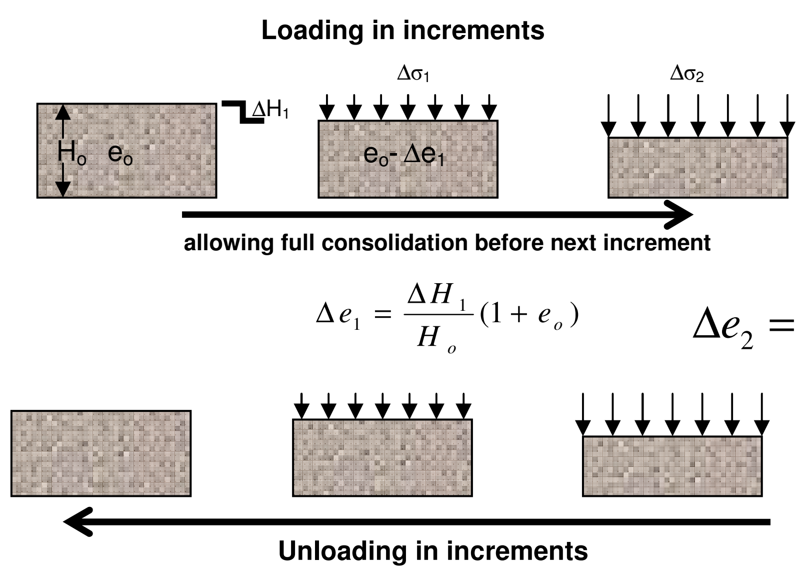


Fig. 7: Compression of soil specimen under laboratory conditions

A laboratory soil specimen of dia 60mm and height 20mm is extracted from the undisturbed soil sample obtained from the field. This sample is subjected to 1D consolidation in the lad under various pressure increments. Each pressure increment is maintained for 24 hrs and equilibrium void ratio is recorded before the application of the next pressure increment. Then a plot of void ratio versus effective stress is made as shown in Fig 7 and 8.

When the sample is recompressed from point D it follows DE and beyond C it merges along BCF and it compresses as it moves along BCF

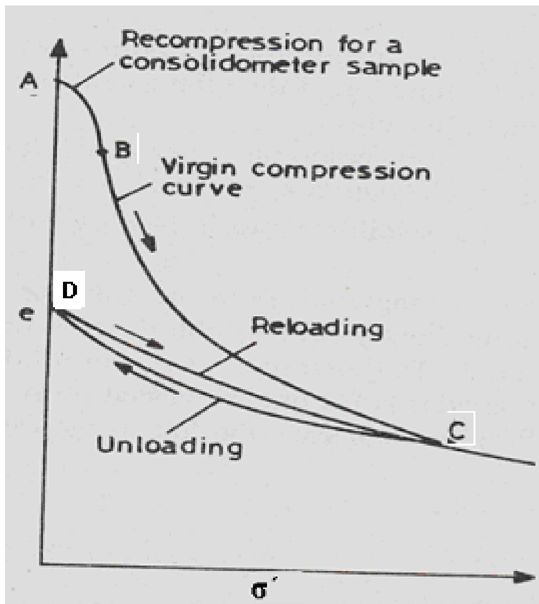


Fig. 8: Void ratio versus effective stress (on arithmetic plot)

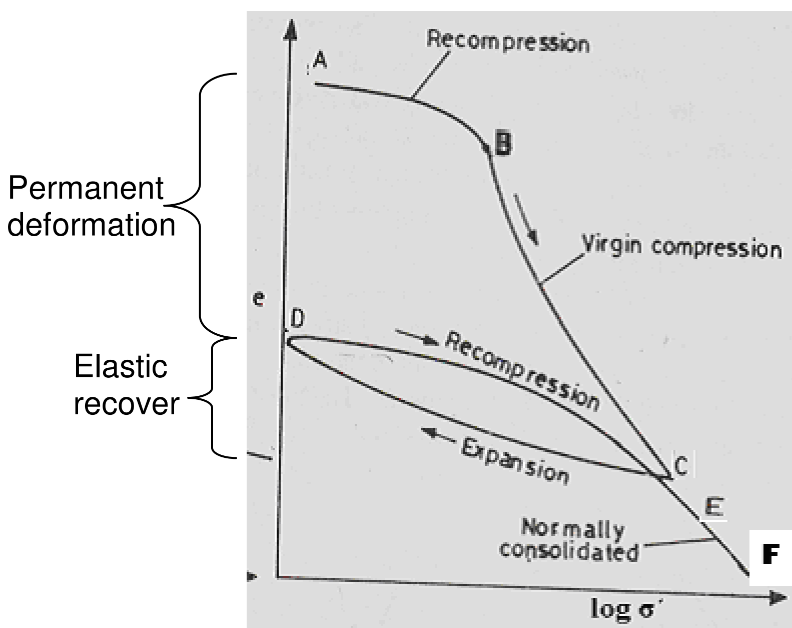


Fig. 9: Void ratio versus logarithm of effective stress (Semi-log plot)

During the initial stages (at low effective stress) sample follows recompression path (portion AB) and undergoes less compression. Beyond this is the virgin compression line (portion BC) also called the normal compression line and the sample undergoes large compression.

1. BC – Virgin compression curve also called normal consolidation line
2. From ‘C’ when the sample is unloaded, sample expands and traces path CD (expansion curve unloading)
3. Sample undergoes Permanent strain due to irreversible soil structure and there is a small elastic recovery.
4. The deformation recovered is due to elastic rebound
5. When the sample is reloaded-reloading curve lies above the rebound curve and makes an hysteresis loop between expansion and reloading curves.
6. The reloaded soils shows less compression.
7. Loading beyond ‘C’ makes the curve to merge smoothly into portion EF as if the soil is not unloaded.

Terzaghi’s 1D Consolidation Equation

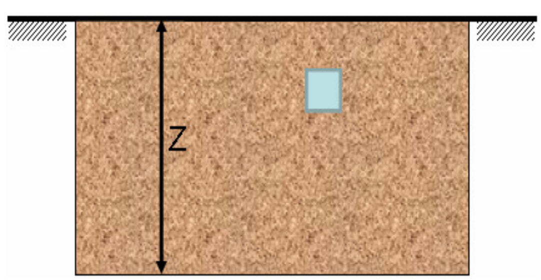


Fig. 10: Saturated soil strata

Assumptions:

- The soil medium is completely saturated
- The soil medium is isotropic and homogeneous
- Darcy's law is valid for flow of water
- Flow is one dimensional in the vertical direction
- The coefficient of permeability is constant
- The coefficient of volume compressibility is constant
- The increase in stress on the compressible soil deposit is constant (**$\Delta\sigma'$** = constant)
- Soil particles and water are incompressible

One dimensional theory is based on the following hypothesis

1. The change in volume of soil is equal to volume of pore water expelled.
2. The volume of pore water expelled is equal to change in volume of voids.
3. Since compression is in one direction the change in volume is equal to change in height.

The increase in vertical stress at any depth is equal to the decrease in excess pore water pressure at the depth

$$\Delta\sigma' = \Delta u$$

This is Terzaghi's one dimensional consolidation equation

$$\frac{\partial u}{\partial t} = c_v \frac{\partial^2 u}{\partial z^2}$$

This equation describes the variation of excess pore water pressure with time and depth

Limitation of 1D consolidation

1. In the deviation of 1D equation the permeability (Kz) and coefficient of volume compressibility (m_v) are assumed constant, but as consolidation

progresses void spaces decrease and this results in decrease of permeability and therefore permeability is not constsnt

The coefficient of volume compressibility also changes with stress level. Therefore C_v is not constant

2. The flow is assumed to be 1D but in reality flow is three dimensional
3. The application of external load is assumed to produce excess pore water pressure over the entire soil stratum but in some cases the excess pore water pressure does not develop over the entire clay stratum.

Solution of 1D consolidation

$$\frac{\partial u}{\partial t} = c_v \frac{\partial^2 u}{\partial z^2} \quad \text{----------------} \; 1$$

The solution of variation of excess pore water pressure with depth and time can be obtained for various initial conditions.

Uniform excess pore water pressure with depth

1. Single Drainage (Drainage at top and bottom impervious)
2. Double Drainage (Drainage at top and bottom)

Single Drainage (drainage at top and bottom impervious)

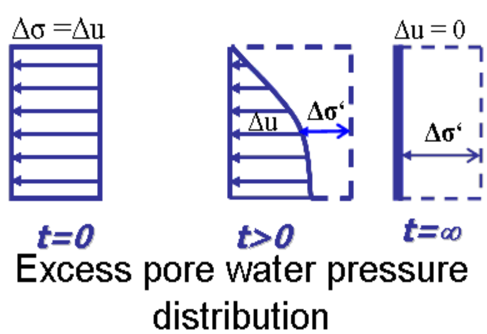


Fig. 11: Excess porewater pressure distribution of single drainage

Double Drainage

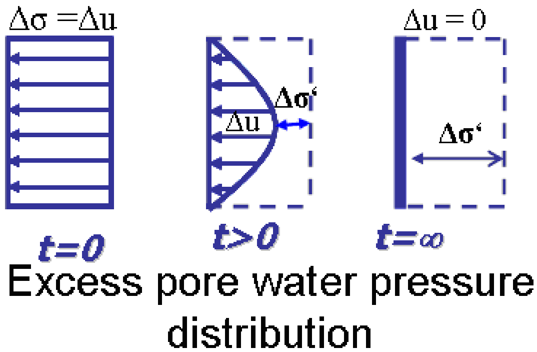


Fig. 12: Excess porewater pressure distribution of double drainage

Boundary Conditions are

i) At t = 0 $\Delta u = \Delta\sigma$ and $\Delta\sigma' = 0$

ii) At the top z = 0 $\Delta u = 0$ $\Delta\sigma = \Delta\sigma'$

iii) At the bottom z = 2Hdr $\Delta u = 0$ $\Delta\sigma = \Delta\sigma'$

A solution of equation (1) for the above boundary conditions using Fourier series is given by

$$\Delta u_{(z,t)} = \sum_{m=0}^{\infty} \frac{2\Delta u_0}{M} \sin\left(\frac{MZ}{H_{dr}}\right) e^{-M^2 T_v}$$

$$M = \frac{\pi}{2}(2m+1)$$ Where m = +ve integer with values from 0 to ∞

$$Tv = \frac{c_v t}{H_{dr}^{\ 2}}$$ Where Tv =Time factor (dimensionless)

Graphical solution of 1D consolidation equation

$$\Delta u_{(z,t)} = \sum_{m=0}^{\infty} \frac{2\Delta u_0}{M} \sin\left(\frac{MZ}{H}\right) e^{-M^2 T_v}$$

The solution of consolidation equation consists of the following three variables

1. The depth z
2. The excess pore water pressure Δu
3. The time (t) after application of loading

The above variables are expressed in the form of the following non-dimensional terms as

Sl. No	Variables	Non-dimensional terms
1	Depth (z)	Z = z /H (Drainage path ratio)
2	Excess pore pressure (Δu)	Uz --- consolidation ratio This represents the dissipated pore water pressure to initial excess pore water pressure
3	Time (t)	T_v (Time factor)

The graphical solution of the above equation is as shown below

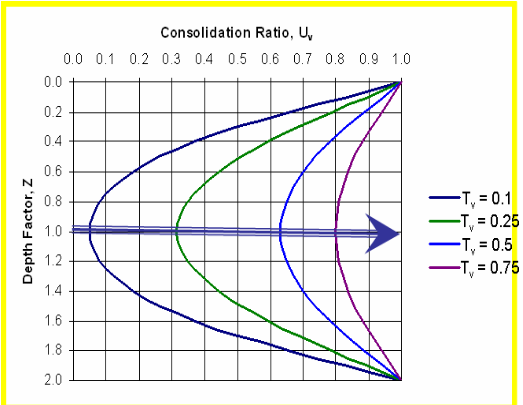


Fig. 13: Terzaghi's solution for one-dimensional consolidation

This indicates the progress of consolidation with time and depth for a given set of boundary conditions.

Degree of Consolidation (U_z)

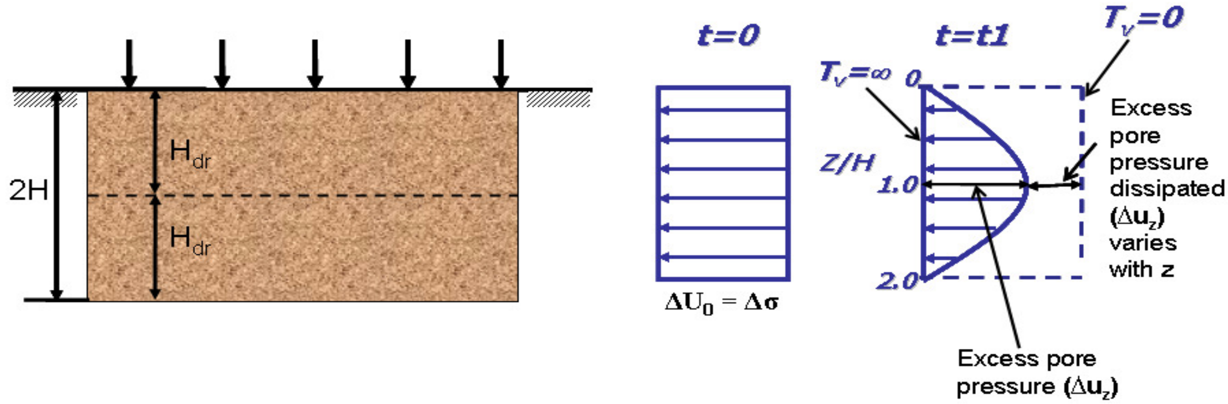


Fig. 21: (a) Section of clay layer, (b) Excess pore pressure distribution

The degree of consolidation at any depth is given by

$$U_z = \frac{\Delta u_0 - \Delta u_z}{\Delta u_0}$$

$$1 - \frac{\Delta u_z}{\Delta u_0} = \frac{\Delta \sigma'_z}{\Delta u_0}$$

Δu_o = Initial excess pore water pressure at that depth

Δu_z = Excess pore water pressure at that depth

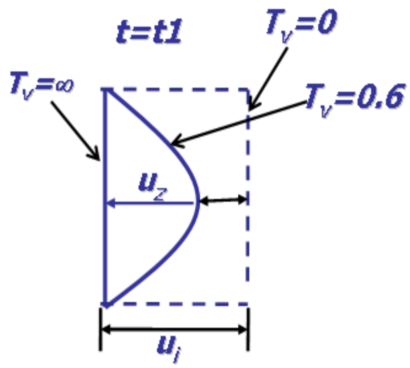
$$u_z = 1 - \frac{\Delta u_z}{\Delta u_0}$$

$$u_z = 1 - \sum_{m=0}^{\infty} \frac{2}{M} \sin\left(\frac{MZ}{H}\right) e^{-M^2 T_v}$$

u_z= Degree of consolidation at a particular depth at any given time

From practical point of view, the average degree of consolidation over the entire depth at any given time is desirable.

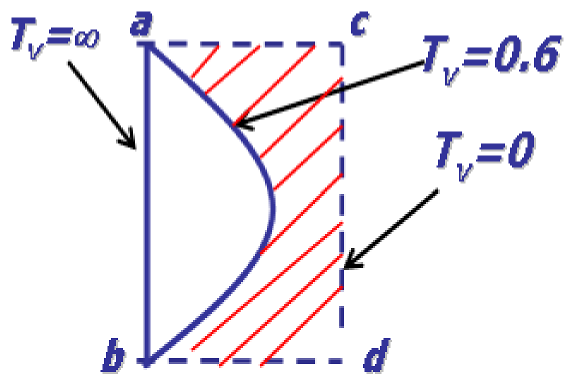
At any given time $\boldsymbol{u_z}$ *varies* with location and hence the degree of consolidation u_z also varies



u_z = Degree of consolidation at a certain level

This term u_z is not useful instead the average degree of consolidation U for the entire soil deposit is necessary

Average Degree of Consolidation (U)



The average degree of consolidation for the whole soil deposit at any time is given by

$$U = \frac{\text{Area of the diagram of excess pore water pressure dissipated at any time}}{\text{Area of the diagram of initial excess pore water pressure}}$$

$$U = \frac{\text{Area Shaded}}{\text{Area of abcd}}$$

Mathematically $U = f(T_v)$

$$u = 1 - \sum_{m=o}^{\infty} \frac{2}{M} e^{-M^2 T_v}$$

U depends on T_v

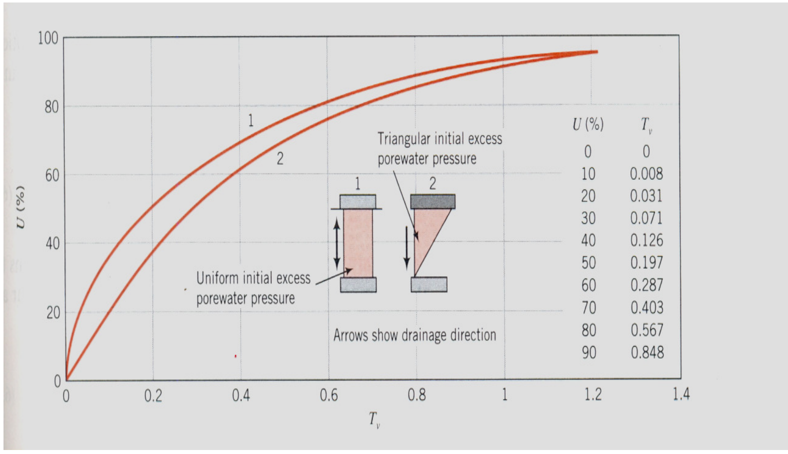


Fig. 22: Degree of consolidation versus time factor (T_v)

As per Taylor (1948) solution, the following approximation is possible

$$when \qquad U \leq 60\,\% \qquad Tv = \frac{\pi}{4} U^2$$

$$For \quad U > 60\% \qquad Tv = 1.781 - 0.933 \log(100 - U\%)$$

Typical values of T_v

U = 50%	Tv = 0.197
U = 60%	Tv = 0.287
U = 90%	Tv = 0.848

Compressibility Properties

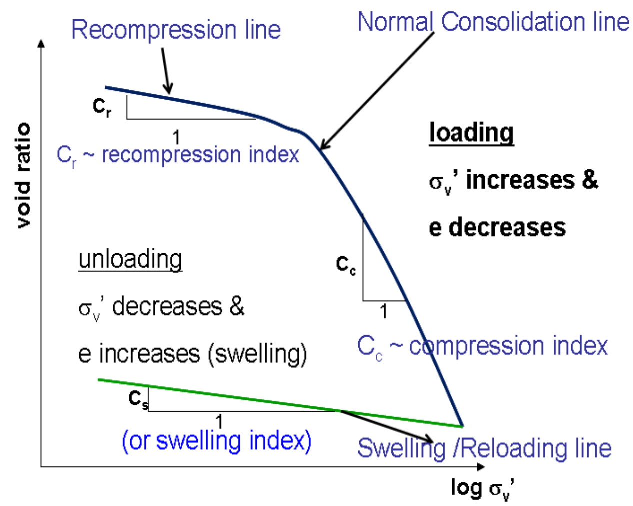


Fig. 23: e – log σ_v' plot

Coefficient of compression/compression index (C_c)

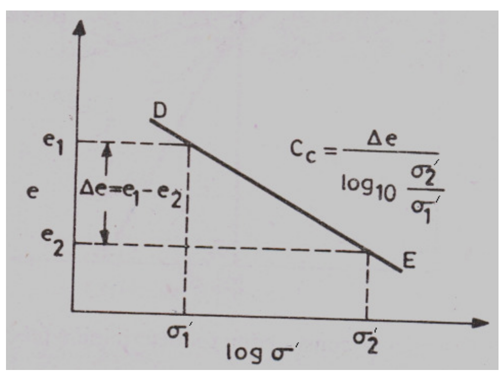


Fig. 24: e – log σ' plot

It is the slope of the normal consolidation line in a plot of void ratio-logarithm of effective stress (e - logσ').

It is given by

$$C_c = \frac{e_1 - e_2}{\log_{10} \frac{\sigma_2'}{\sigma_1'}}$$

Empirical correlations

$C_c = 0.009$ (LL-10)	Undisturbed clays
$C_c = 0.007$ (LL-10)	Remoulded soil sample
$C_c = 1.15$ (e0-0.30)	Upper bound values
$C_c = 0.30$ (e0-0.27)	Lower bound values

The value of Cc is constant for a given soil. The compression index is used to determine primary consolidation settlement of normally consolidated soils. A high value of C_c indicates high compressibility and higher consolidation settlement.

Swelling Index (Cs)

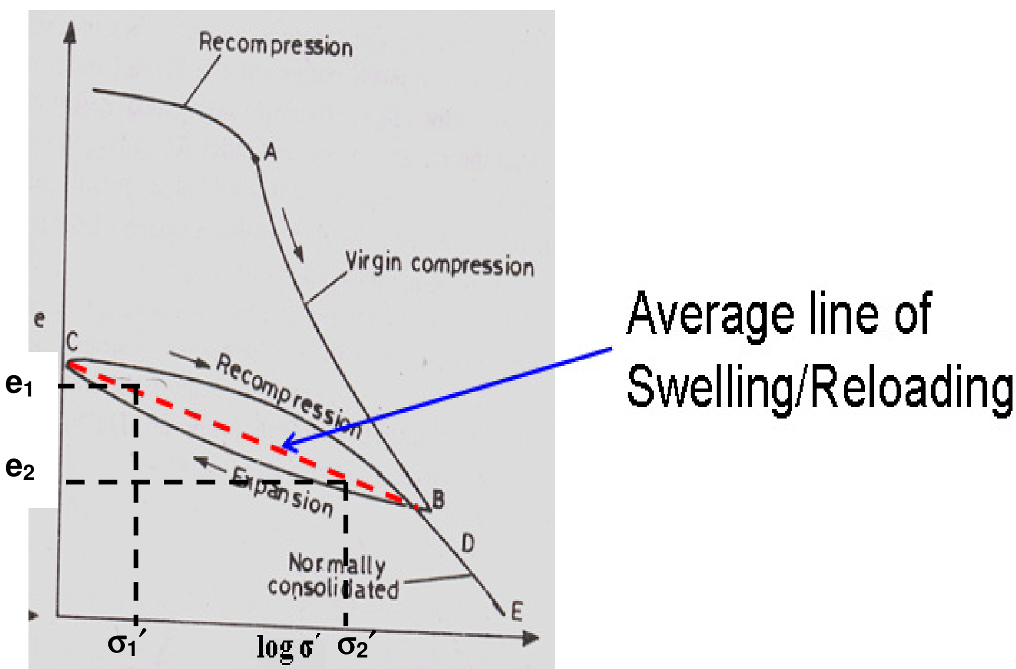


Fig. 25: e – log σ_v' plot

It is the average slope of the unloading/reloading curves in e – logσ´ plot given by

$$C_s = \frac{e_1 - e_2}{\log 10 \dfrac{\sigma_2'}{\sigma_1'}}$$

Co-efficient of compressibility (a_v)

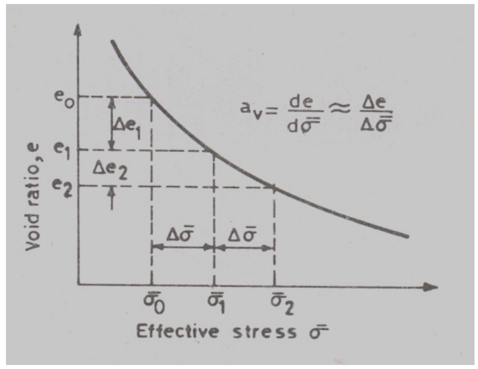


Fig. 26: Void ratio versus effective stress plot

It is the slope of the void ratio versus effective stress for a given stress increase Δσ´ in void ratio versus effective stress plot as shown

$$a_v = \frac{\Delta e}{\Delta \sigma'} = \frac{e_1 - e_2}{\sigma_2' - \sigma_0'}$$

a_v decrease with increase in effective stress

Co-efficient of volume compressibility (m_v)

It is the ratio of change in volume of a soil per unit initial volume due to unit increase in effective stress and is given by

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

Δe = Change in void ratio
e_0 = Initial void ratio
Δσ´= increase in effective stress

Preconsolidation Pressure σ_{pc}'

It is the maximum effective stress experienced by a soil in its stress history (past existence)

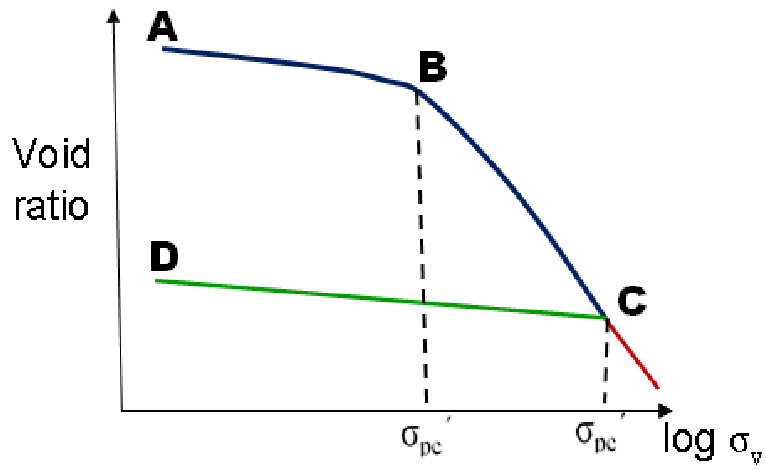


Fig. 27: Void ratio versus effective stress (log scale)

- For the soil loaded along the recompression curve AB the effective stress close to point B will be the preconsolidation pressure.
- If the soil is compressed along BC and unloaded along CD and then reloaded along DC the effective stress close to point C will be the new preconsolidation pressure.

Effect of Stress History

Based on the stress history (preconsolidation pressure) soils are classified as

1. Normally Consolidated Soils

2. Over Consolidated Soils

3. Under Consolidated Soils

6.4 Normally Consolidated Soils

It is a soil deposit that has never subjected to a vertical effective stress greater than the present vertical stress.

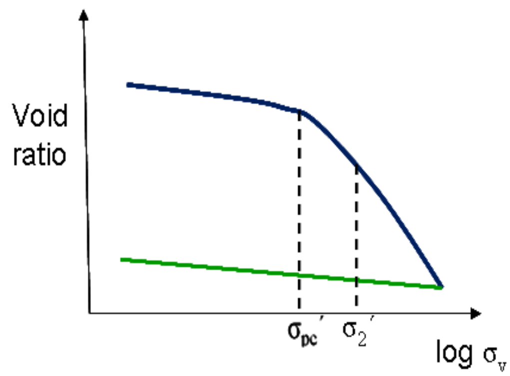


Fig. 28: Void ratio versus effective stress (log scale)

The stress state σ_2' represent normally consolidated soil.

6.5 Under Consolidated Soils

A soil deposit that has not consolidated under the present overburden pressure (effective stress) is called Under Consolidated Soil. These soils are susceptible to larger deformation and cause distress in buildings built on these deposits.

6.6 Over Consolidated Soils

It is a soil deposit that has been subjected to vertical effective stress greater than the present vertical effective stress.

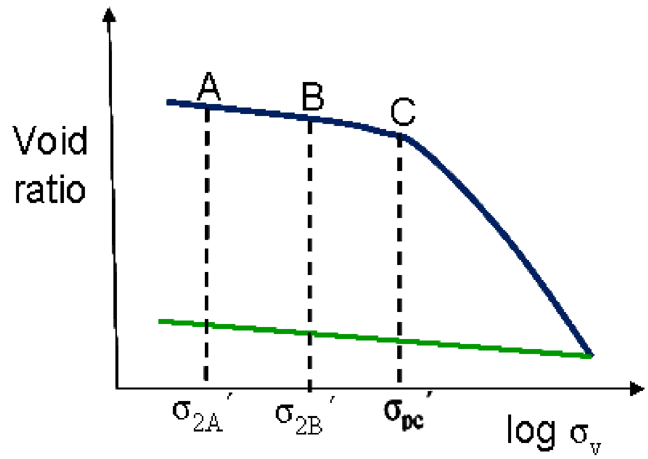


Fig. 29: Void ratio versus effective stress (log scale)

The stress state σ_{2A}' and σ_{2B}' represent over consolidated soil (well with in preconsolidation pressure) Over consolidated soil deposits are less compressible and therefore structures built on these soils undergo less settlement.

Over Consolidation Ratio (OCR)

It is the defined as the ratio of preconsoliadtion pressure to the present vertical effective stress

$$OCR = \frac{\sigma_{pc}'}{\sigma_z'}$$

This is indicative of the position of soil away from the normal consolidated line

OCR =1 Normally consolidated Soils

Note: -- Soils having higher OCR are less compressible

-- They show elastic behavior to certain extent

6.7 Determination of Preconsolidation Pressure (Yield Stress)

Step 1. Conduct an oedometer test on the undisturbed soil sample obtained from the field.

Step 2. Plot e - log σ´ plot as shown. The equilibrium void ratio at the end of each of the pressure increments are used in obtaining e - log σ´ plot.

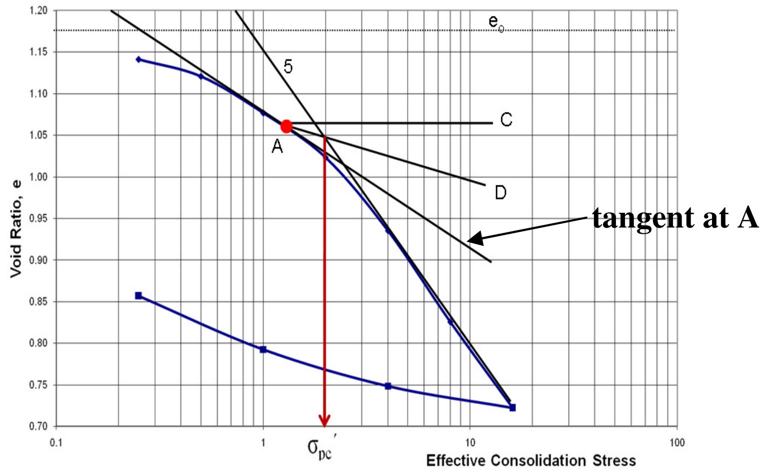


Fig. 30: Casagrande construction for determining Preconsolidation stress

Step 3. Select the point of maximum curvature (Point A) on the e - log σ´ curve

Step 4. Draw a tangent at the point of maximum curvature (Point A)

Step 5. Draw a horizontal line AC

Step 6. Draw the bisector line AD between the tangent and horizontal line

Step 7. Extend the normally consolidated line to intersect the bisector line at ‘O’

Step 8. The vertical effective stress corresponding to point of intersection (O) is the preconsolidation pressure (σ'_{pc})

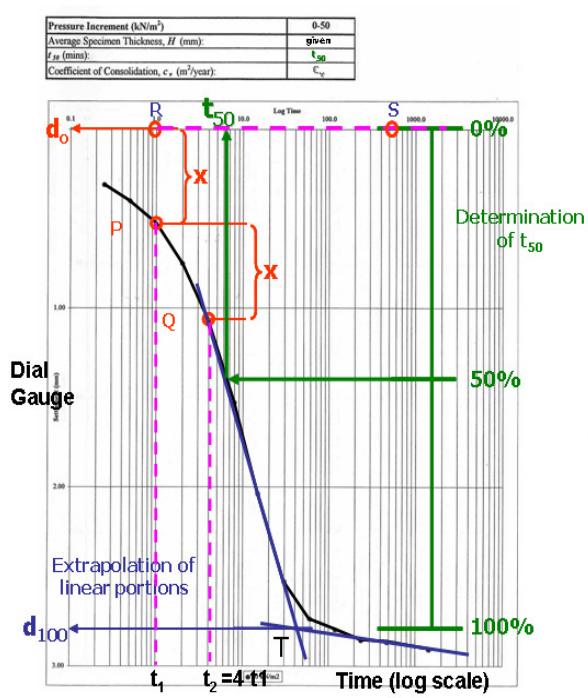
6.8 Determination of coefficient of consolidation (C_v) from laboratory data

The coefficient of three graphical procedure are used

1. Logarithm of time method
2. Square root of time method
3. Hyperbola method

Log – time curve fitting method

The basis for this method is the theoretical (U_z) versus log T_v curve and experimental dial gauge reading and log t curves are similar.



$$C_v = \frac{0.197 \times H^2}{t_{50}}$$

Fig. 31: Log-time curve fitting method

Steps

1. Plot the dial reading of compression for a given pressure increment versus time to log scale as shown in fig. 31.

2. Plot two points P and Q on the upper portion of the consolidation curve (say compression line) corresponding to time t_1 and t_2 such that $t_2=4t_1$

3. Let x be the difference in dial reading between P and Q. locate R at a vertical distance x above point P

4. Draw a horizontal line RS the dial reading corresponding to this line is d_0 which corresponds with 0% consolidation.

5. Project the straight line portion of primary and secondary consolidation to intersect at point T. The dial reading corresponding to T is d_{100} and this corresponds to 100% consolidation.

6. Determine the point V on the consolidation curve which corresponds to the dial reading of $\frac{d_0 + d_{100}}{2} = d_{50}$. The time corresponding to point V is t_{50} i.e time for 50% consolidation.

7. Determine C_v from $$C_v = \frac{T_v H^2}{t}$$

For 50% U_z $\quad T_v = 0.197 \left(Tv = \frac{\pi}{4}\left(\frac{U_z}{100}\right)^2 \right)$

$$C_v = \frac{0.197 H^2}{t_{50}}$$

Square-root – time curve fitting method

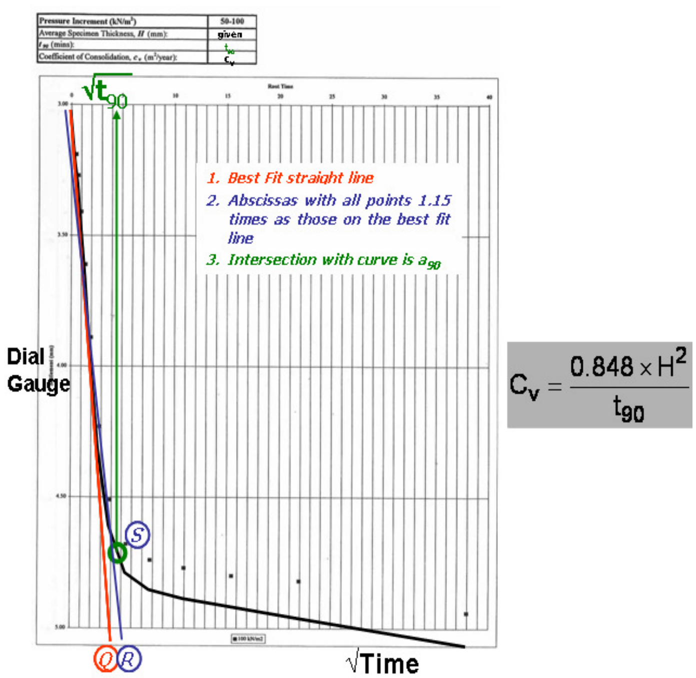


Fig. 32: Log-time curve fitting method

Steps

1. Plot the dial reading and square root of time i.e √T for a pressure increment as shown in fig. 32.

2. Draw a tangent PQ to the initial portion of the plot as shown in fig.

3. Draw a line PR such that OR=1.15OQ.

4. The intersection of the line PR with the second portion of the curve i.e point S is marked.

5. The time corresponding to point S represent √t90 (Square root of time for 90% consolidation)

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

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

For $U_z > 60\%$ $T_v = 1.781 - 0.933 \log10\,(100 - U\,\%)$

$$T_v = 0.848$$

$$C_v = \frac{0.848 H^2}{t_{90}}$$

Hyperbola method for determining C_v

Sridharan and Prakash (1985) have proposed this method

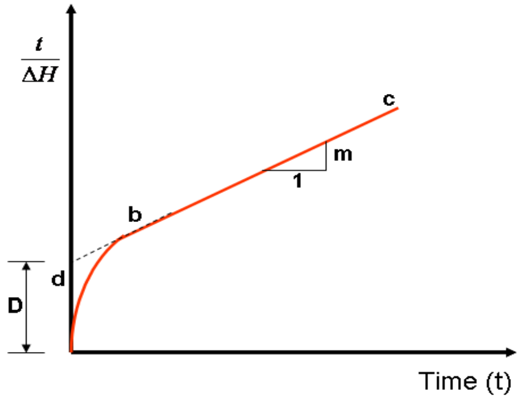


Fig. 33: $\frac{t}{\Delta H}$ versus time

1. Conduct an Odeometer test and record time (t) and compression (ΔH) data for the required pressure increment
2. Plot the graph of t/ΔH (y-axis) versus time t (x-axis) as shown in figure
3. Draw a tangent line along the straight line portion bc and extend it to intersect the y-axis at point d.
4. Determine the slope 'm' of the line bc
5. Obtain Cv from $C_v = 0.3\left(\frac{m H^2{}_{dr}}{D}\right)$

This method is very simple and provides good results for u=60 to 90%

6.9 Time Rate of consolidation

We know that

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

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

For a given degree of consolidation (U) ---T_v is Constant

$$t \propto \frac{H^2}{C_v}$$

Therefore the time required for a given degree of consolidation is proportional to the length of the drainage path

- If the time required to reach a certain degree of consolidation is measured in the laboratory on a sample obtained from the field
- The time taken by the field deposit of known thickness can be predicted by using

$$t_f = \frac{H_f^{\,2}}{H_L^{\,2}} \times t_L$$

t_f = Time required for field consolidation
t_L = Time required for laboratory consolidation
H_F = Thickness of soil in the site
H_L = Thickness of laboratory sample

Settlement Calculations

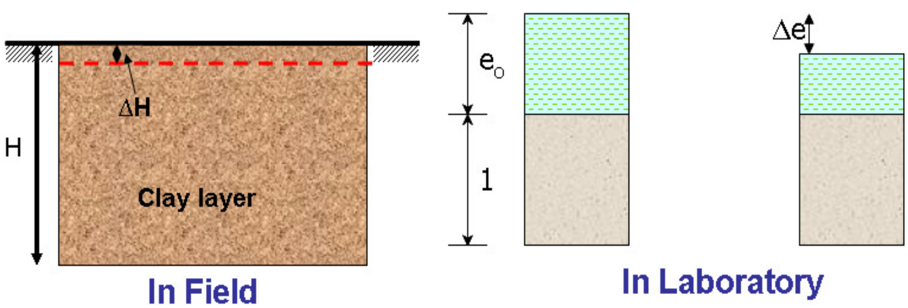


Fig. 34: Compression of field deposit

If the clay layer of thickness H when subjected to an increase in average effective overburden pressure from σ'_0 to σ'_1 ($\sigma'_0+\Delta\sigma'$) there will be consolidation settlement of ΔH.

The strain $$\varepsilon = \frac{\Delta H}{H} \quad in\ field$$

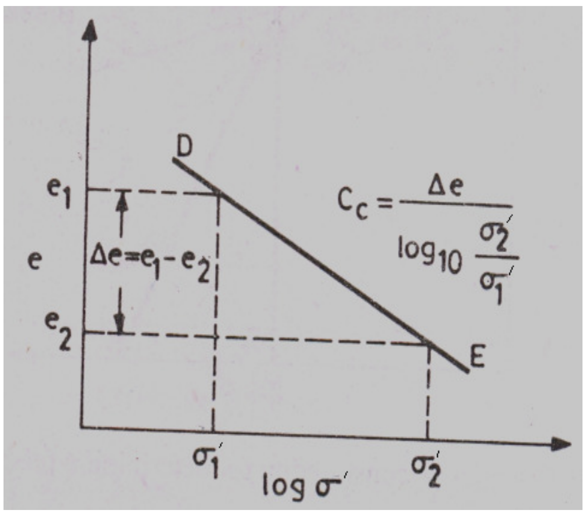
$$\varepsilon = \frac{\Delta e}{1+e_0} \quad in\ lab$$

Equating

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

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

Case-1 For normally consolidated soils



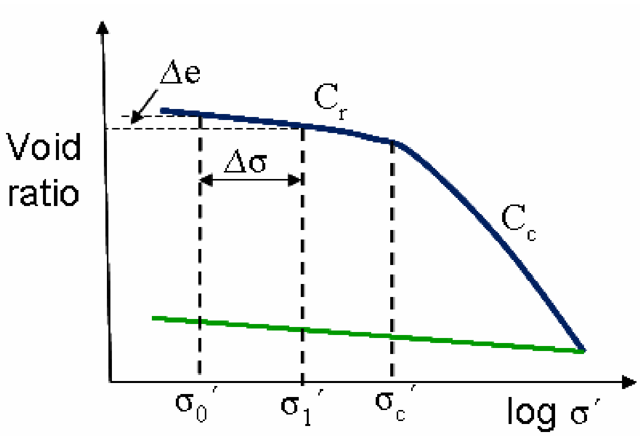
$$C_c = \frac{\Delta e}{\log_{10} \dfrac{\sigma_1'}{\sigma_0'}} = \frac{\Delta e}{\log_{10}\left(\dfrac{\sigma_0' + \Delta\sigma}{\sigma_0'}\right)}$$

$$\Delta e = C_c \log_{10}\left(\frac{\sigma_0' + \Delta\sigma}{\sigma_0'}\right)$$

$$\Delta H = \frac{C_c}{1+e_0} H \log_{10}\left(\frac{\sigma_0' + \Delta\sigma}{\sigma_0'}\right)$$

Case-2 Over consolidated soils

Case 1: $\sigma_1' < \sigma_c'$

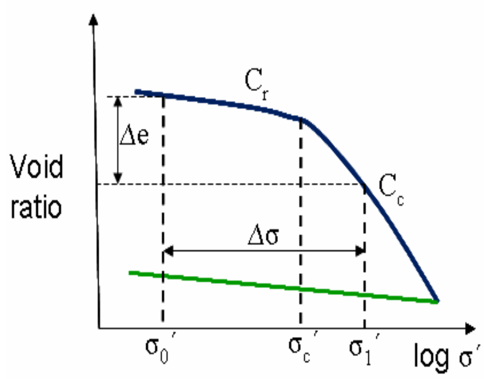


$$\Delta e = C_r \log_{10}\left(\frac{\sigma_1'}{\sigma_0'}\right)$$

$$\Delta H = \frac{C_r}{1+e_0} H \log_{10}\left(\frac{\sigma_0' + \Delta\sigma}{\sigma_0'}\right)$$

$$\sigma_0' < \sigma_c' < \sigma_1'$$

Case 2:



$$\Delta e = \Delta e_1 + \Delta e_2$$

$$\Delta e = C_r \log_{10}\left(\frac{\sigma_c'}{\sigma_0'}\right) + C_c \log_{10}\left(\frac{\sigma_1'}{\sigma_c'}\right)$$

$$\Delta H = C_r \log_{10}\left(\frac{\sigma_c'}{\sigma_0'}\right) + C_c \log_{10}\left(\frac{\sigma_0' + \Delta\sigma}{\sigma_c'}\right)$$

Secondary Consolidation Settlement

It is the settlement due to plastic compression arising out of readjustment of soil particles at constant effective stress. It occurs after primary consolidation settlement.

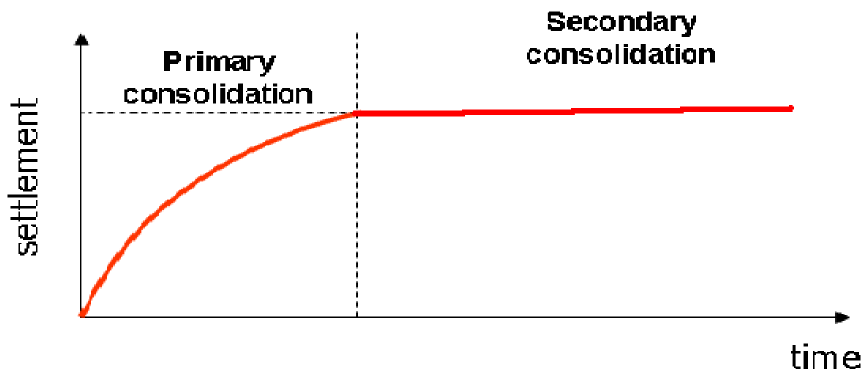


Fig. 35: Settlement versus time

The plot of void ratio (deformation) versus log time is as shown

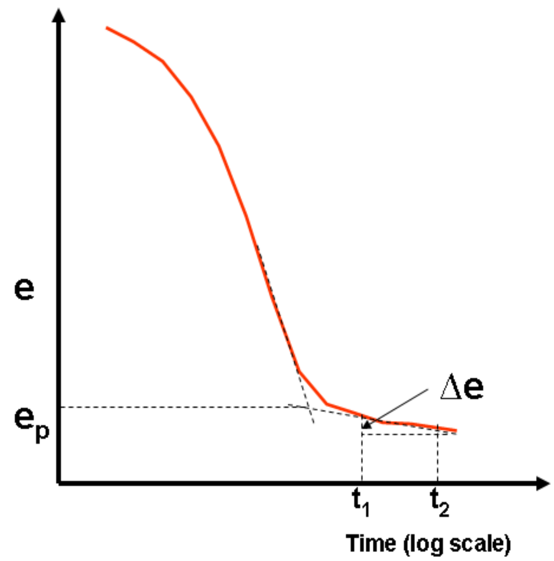


Fig. 36: Void ratio versus time (log scale)

The Secondary Compression index (Cα) is

$$C_\alpha = \frac{\Delta e}{\log t_2 - \log t_1} = \frac{\Delta e}{\log \frac{t_2}{t_1}}$$

Cα = Secondary compression index

Δe = Change in void ratio

Obtain

$$C'_\alpha = \frac{C_\alpha}{1 + e_p}$$

e_p= Void ratio at the end of primary consolidation

Secondary Compression index (Cα) is

$$S_{sc} = C'_\alpha \; H \; \log\left(\frac{t_1}{t_2}\right)$$

Dec.06/Jan.07, Q.No-6(c), Jan/Feb 2006, Q.No-6(c)

20 mm thick undisturbed sample of saturated clay is tested in laboratory with drainage allowed through top and bottom. Sample reaches 50% consolidation in 35 minutes. If clay layer from which sample was obtained is 3.0 m thick and is free to drain through top and bottom surfaces, calculate the time required for same degree of consolidation in the field. What is the time required if the drainage in the field is only through the top?

Solution:

For the same degree of consolidation, Tv is the same. Hence

$$t \propto \frac{d^2}{c_v}$$

Also, since both the soils are the same,

$$t \propto d^2$$

1. For the same case of double drainage:

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

Where d2= drainage path in the field = 3/2 = 1.5m = 150 cm

d1= drainage path in the laboratory = 2.0/2 = 1.0cm

t1 = time for 50% consolidation in the laboratory = 35 min

$$\therefore \quad t_2 = t_1\left(\frac{d_2}{d_1}\right)^2 = 35\left(\frac{150}{1.0}\right)^2 \quad \min utes$$

$$= 547 \ days$$

2. For the same case of single drainage:

d2 = 3m = 300cm

$$\therefore \quad t_2 = t_1\left(\frac{d_2}{d_1}\right)^2 = 35\left(\frac{300}{1.0}\right)^2 \quad \min utes$$

$$= 2188 \ days$$

Feb.02, Q.No-4(c)

Following data were obtained from a consolidation test on a clay sample with double drainage conditions:

Void ratio at 100 kPa = 1.37

Void ratio at 200 kPa = 1.25

Thickness of the soil sample at 100kPa =20mm

Coefficient of permeability = 5x10-7 mm/sec

Calculate (i) Compression index

(ii) Coefficient of volume change

(iii) Coefficient to consolidation in mm2/year

Solution:

$$\boldsymbol{\sigma'_0 = 100\,\text{KPa} = 100\ \text{KN/m}^2; \qquad e_0 = 1.37}$$

$$\boldsymbol{\sigma' = 200\,\text{KPa} = 200\ \text{KN/m}^2; \qquad e = 1.25}$$

$$\boldsymbol{\therefore \Delta\sigma' = 100\ \text{KPa} = 100\ \text{KN/m}^2; \quad \Delta e = -0.12}$$

$$Compression\ index = C_c = \frac{e_0 - e}{\log_{10}\dfrac{\sigma'}{\sigma'_0}} = \frac{\Delta e}{\Delta \log_{10}\sigma'}$$

$$C_c = \frac{0.12}{\log_{10} 100}$$

$$C_c = 0.06$$

$$Coefficien\,t\ of\ \ volume\ \ change\,(m_v)$$

$$m_v = -\frac{\Delta e}{1+e_0}\frac{1}{\Delta\sigma'} = \quad \frac{0.12}{1+1.37}\frac{1}{100}$$

$$= 5.06\times 10^{-4}\ m^2 / KN$$

$$\mathbf{Coefficien\ t\ of\ Consolidat\ ion\ (c_v)}$$

$$\mathbf{c_v = \frac{k}{m_v \times \gamma_w} = \frac{5.0\times 10^{-10}}{5.06\times 10^{-4}\ X\ 9.81} = 1.01\ X\ 10^{-7}\ m^2/sec}$$

$$\mathbf{= 1.16\times 10^{-6}\ mm^2/sec}$$

July/Aug 2005, Q.No-7(c), Dec-08/Jan-09, Q.No-8(d)

In a consolidation test voids ratio decreased from 0.70 to 0.60 when the load was changed from 50 kN/m2 to 100 kN/m2. Compute coefficient of compressibility and coefficient of volume change.

Solution

$$\Delta e = e_0 - e = 0.70 - 0.60 = 0.10$$

$$\Delta\overline{\sigma} = 100 - 50 = 50\ KN/m^2$$

$$Coefficient\ of\ Compressibility(a_v)$$

$$a_v = \frac{\Delta e}{\Delta\overline{\sigma}} = \frac{0.10}{50} = 0.002\ m^2/KN$$

$$Coefficien\ t\ of\ Compressib\ ility(a_v)$$

$$a_v = \frac{\Delta e}{\Delta\overline{\sigma}} = \frac{0.10}{50} = 0.002\ m^2/KN$$

$$Coefficien\ t\ of\ Volume\ Compressib\ ility(m_v)$$

$$m_v = \frac{a_v}{1+e_0} = \frac{0.002}{1+0.70} = 0.0012\ m^2/KN$$

Jan/Feb 2005, Q.No-6(c), Dec-08/Jan-09, Q.No-8(d)

In a consolidation test the void ratio of soil sample decreases from 1.20 to 1.10 when the pressure is increased from 160 to 320 kN/m2. Calculate the coefficient of consolidation if the coefficient of permeability is 8.0X10-7mm/sec.

Solution

$$\Delta e = e_0 - e = 1.20 - 1.10 = 0.10$$
$$\Delta\bar{\sigma} = 320 - 160 = 160\ KN/m^2$$

$$Coefficient\ of\ Volume\ Compressibility (m_v)$$
$$m_v = \frac{a_v}{1+e_0} = \frac{0.625\ X\ 10^{-3}}{1+1.20} = 2.84\ X\ 10^{-4}\ m^2/KN$$

$$Coefficient\ of\ Consolidation\ (c_v)$$
$$c_v = \frac{k}{m_v \times \gamma_w} = \frac{8.0\times 10^{-10}}{2.84\times 10^{-4}\ X\ 9.81} = 2.87\ X\ 10^{-7}\ m^2/\sec$$

July/Aug 2003, Q.No-7(b)

Find the time required for 50% consolidation in a soil stratum, 9.0 m thick with a pervious strata on top and bottom. Also determine the co-efficient of consolidation given that k =10-9m/sec, e0 =1.5, av =0.003 m2/KN, Time factor = 0.2.

Solution

$$\textbf{For}\quad \mathbf{U < 60\%, T_v = \frac{\pi}{4}\left(\frac{U}{100}\right)^2}$$
$$\textbf{Hence}\quad \mathbf{T_v}\ \textbf{for}\ \mathbf{U = 50\%}\ \textbf{is given by}$$
$$\mathbf{\therefore (T_v)_{50} = \frac{\pi}{4}\left(\frac{50}{100}\right)^2 = 0.1956 \approx 0.197}$$

$$t_{50} = \frac{(Tv)_{50}\ d^2}{c_v}; (d = \frac{1}{2}\times 9 = 4.5)$$
$$= \frac{0.197\ (4.5)^2}{8.49\times 10^{-8}} = 46.98\times 10^6\ \sec$$

$$t_{50} = 544\ days$$

$$Coefficient\ of\ Volume\ Compressibility (m_v)$$

$$m_v = \frac{a_v}{1+e_0} = \frac{0.003}{1+1.50} = 1.2\ X\ 10^{-3}\ m^2 / KN$$

$$Coefficient\ of\ Consolidation\ (c_v)$$

$$c_v = \frac{k}{m_v \times \gamma_w} = \frac{1.0\times 10^{-9}}{1.2\times 10^{-3}\ X\ 9.81} = 8.49\ X\ 10^{-8}\ m^2 / \sec$$

Feb.02, Q.No-4(b)

Differentiate between normally consolidated and under consolidated soils

Normally Consolidated Soils: It is defined as the soil which has never been subjected to an vertical effective pressure greater than the existing (present) overburden pressure and which is completely consolidated under the existing overburden.

Under Consolidated Soils: It is defined as the soil which is not fully consolidated under the existing overburden pressure
Ex: Fills, debris dumped without compaction

Dec-08/Jan-09, Q.No-8(a)

What are the assumptions made in Terzaghi's one dimensional consolidation theory.

Assumption:

1. The soil medium is completely saturated
2. The soil medium is isotropic and homogeneous
3. Darcy's law is valid for flow of water
4. Flow is one dimensional in the vertical direction
5. The coefficient of permeability is constant

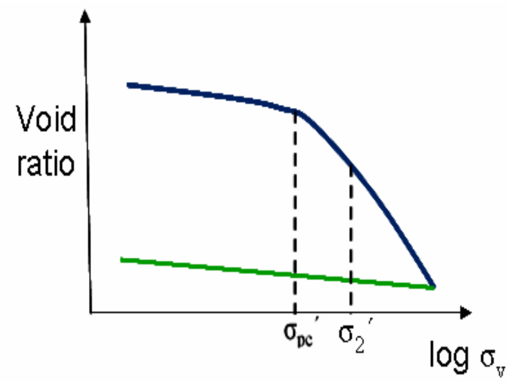
6. The coefficient of volume compressibility is constant
7. The increase in stress on the compressible soil deposit is constant
8. Soil particles and water are incompressible

Dec-08/Jan-09, Q.No-8(a)

Distinguish between Normally and over consolidated soils.

Normally Consolidated Soils

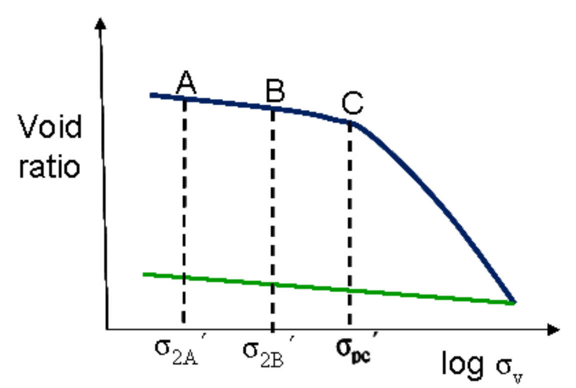
It is a soil deposit that has never subjected to a vertical effective stress greater than the present vertical stress



The stress state σ2´ represent normally consolidated soil.

Over Consolidated Soils

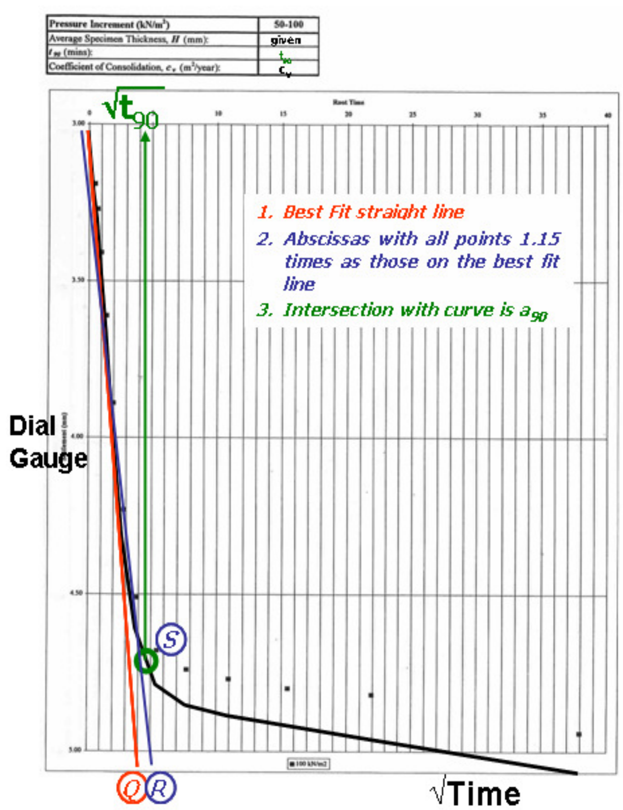
It is a soil deposit that has been subjected to vertical effective stress greater than the present vertical effective stress.



The stress state σ2A´ and σ2B´ represent over consolidated soil.

Jan/Feb 2006, Q.No-6(b), Jan/Feb 2005, Q.No-6(b)

Explain in detail, how to determine the coefficient of consolidation by square of time fitting method?



$$C_v = \frac{0.848 \times H^2}{t_{90}}$$

Steps

1. Plot the dial reading and square root of time i.e√T for a pressure increment as shown in fig.

2. Draw a tangent PQ to the initial portion of the plot as shown in fig.

3. Draw a line PR such that OR=1.15OQ.

4. The intersection of the line PR with the second portion of the curve i.e point S is marked.

5. The time corresponding to point S represent √t90 (Square root of time for 90% consolidation)

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

For Uz > 60% Tv = 1.781-0.933 $C_v = \frac{T_v H^2}{t}$

Tv = 0.848

$$C_v = \frac{0.848 H^2}{t_{90}}$$