CHAPTER-IV-FOUNDATION SETTLEMENT

INTRODUCTION

- ❖Foundations of all structures have to be placed on soil. The structure may undergo settlement depending upon the characteristics such as compressibility of the strata of soil on which it is founded.
- ❖The term '**settlement**' indicates the sinking of a structure due to the compression and deformation of the underlying soil.
- ❖Clay strata often need a very long time a number of years to get fully consolidated under the loads from the structure. The settlement of any loose strata of cohesionless soil occurs relatively fast.

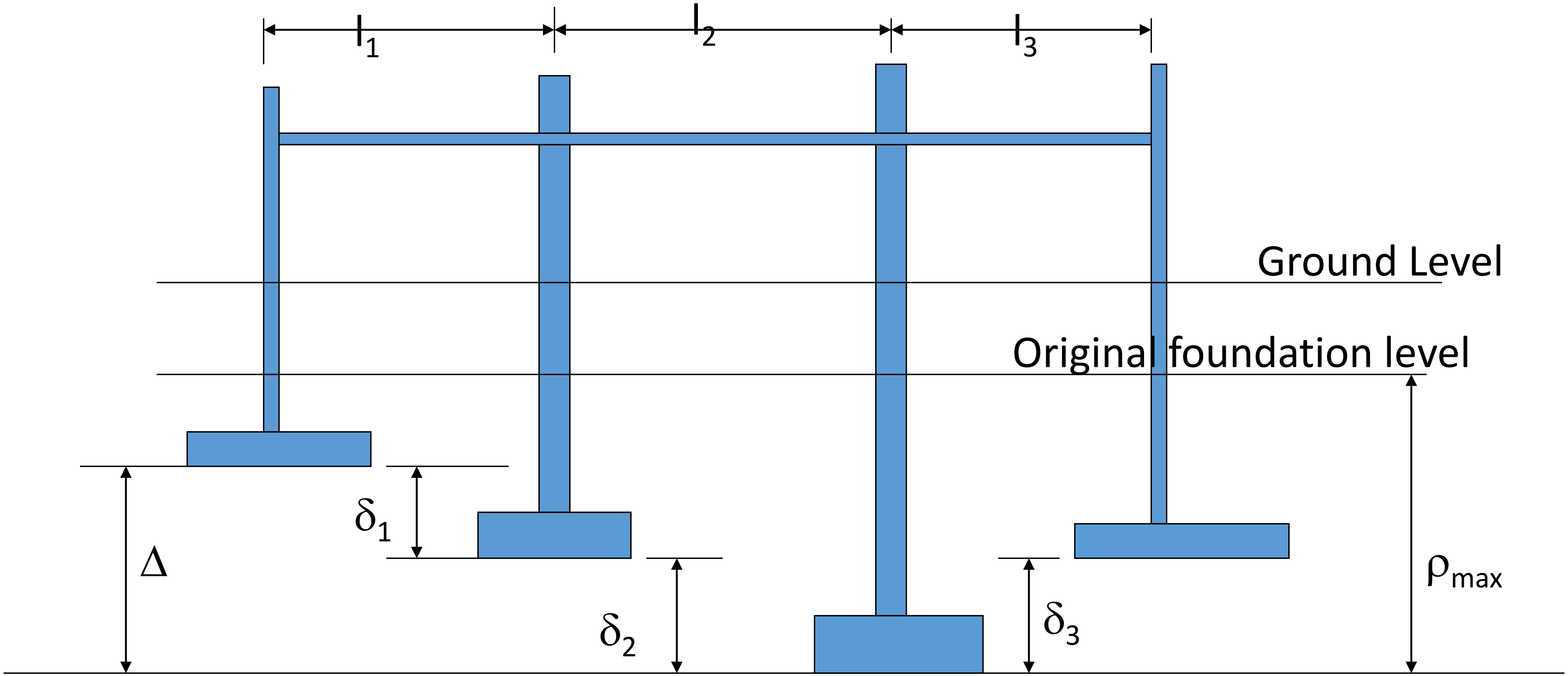
- When a structure is placed on a foundation consisting of soil, the loads from the structure cause the soil to be stressed. The two most important requirements for the stability and safety of the structure are:

(1) **The deformation, especially the vertical deformation**, called ‘settlement’ of the soil, should not be excessive and must be within tolerable or permissible limits.

(2) **The shear strength of the foundation soil** should be adequate to withstand the stresses induced.

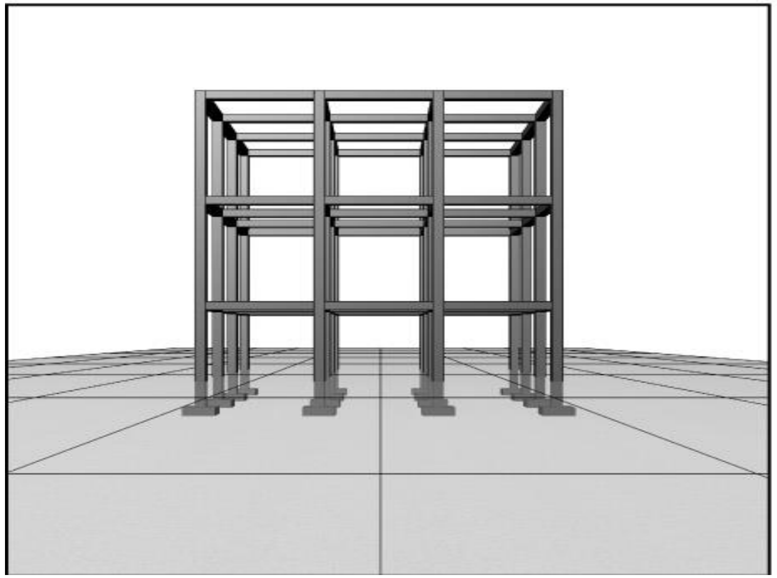
- Compressive deformation occurs when the particles are brought closer together by pressure causing volume changes in the soil.
- The property of a soil by virtue of which volume decrease occurs under applied pressure is termed its ‘Compressibility’.

Settlement of Foundations

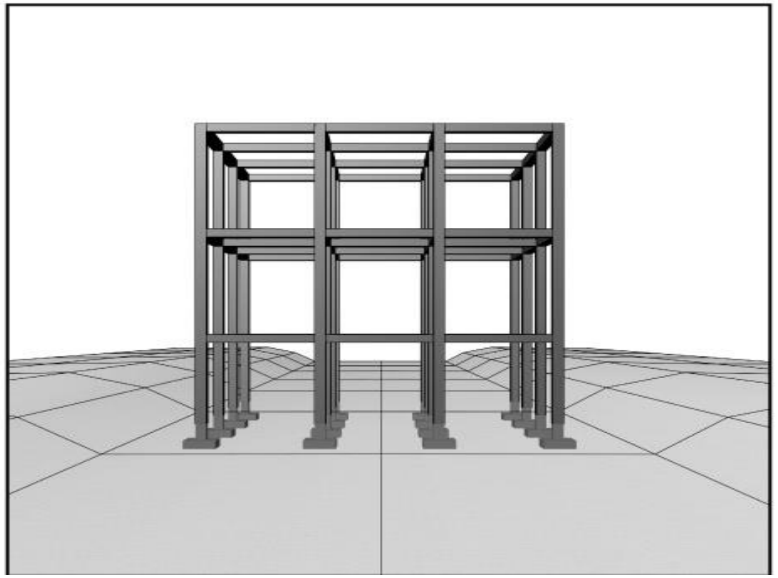


$\delta_1, \delta_2, \delta_3$ = Differential sett., Δ = Greatest differential sett.

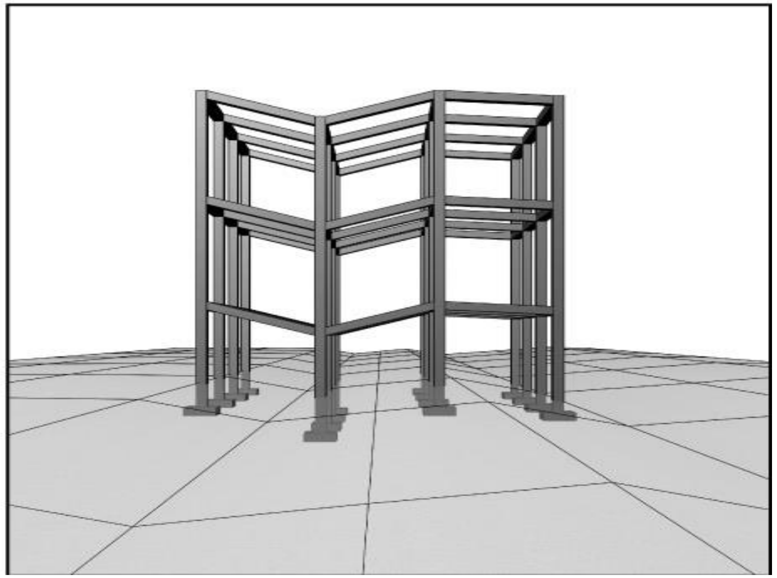
ρ_{max} = maximum total sett., l_1, l_2, l_3 = Bay width, δ / l = angular distortion



NO SETTLEMENT



TOTAL SETTLEMENT



DIFFERENTIAL SETTLEMENT

Uniform settlement is usually of little consequence in a building, but differential settlement can cause severe structural damage

BASIC TERMS RELATED TO CONSOLIDATION SETTLEMENT

Modulus of Volume Change

The ‘modulus of volume change, is defined as the change in volume of a soil per unit initial volume due to a unit increase in effective stress. It is also called the ‘coefficient of volume change’ or ‘coefficient of volume compressibility’ and is denoted by the symbol, *mv*.

$$m_v = -\frac{\Delta e}{(1+e_0)}\cdot\frac{1}{\Delta\bar{\sigma}}$$

Δe represents the change in void ratio and represents the change in volume of the saturated soil occurring through expulsion of pore water, and $(1 + e_0)$ represents initial volume, both for unit volume of solids.

When the soil is confined laterally, the change in volume is proportional to the change in height, ΔH of the sample, and the initial volume is proportional to the initial height H_0 of the sample.

$$m_v = -\frac{\Delta H}{H_0}\cdot\frac{1}{\Delta\bar{\sigma}}$$

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

Consolidation Settlement

The consolidation settlement *Sc*, may also be put in a different, but more common form, as follows:

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

Compression index Cc

The equation for Cc may be approximately written as:

$Cc = 0.007\ (wL - 10)$

w_L being the liquid limit in per cent.

$Cc = 0.009\ (wL - 10)$ for clay soil

INITIAL CONSOLIDATION, PRIMARY AND SECONDARY CONSOLIDATION

INITIAL CONSOLIDATION

The portion of the settlement of a structure which occurs more or less simultaneously with the applied loads is referred to as the *initial* or *immediate settlement*. This settlement is due to the immediate compression of the soil layer under undrained condition and is calculated by assuming the soil mass to behave as an elastic soil.

PRIMARY CONSOLIDATION

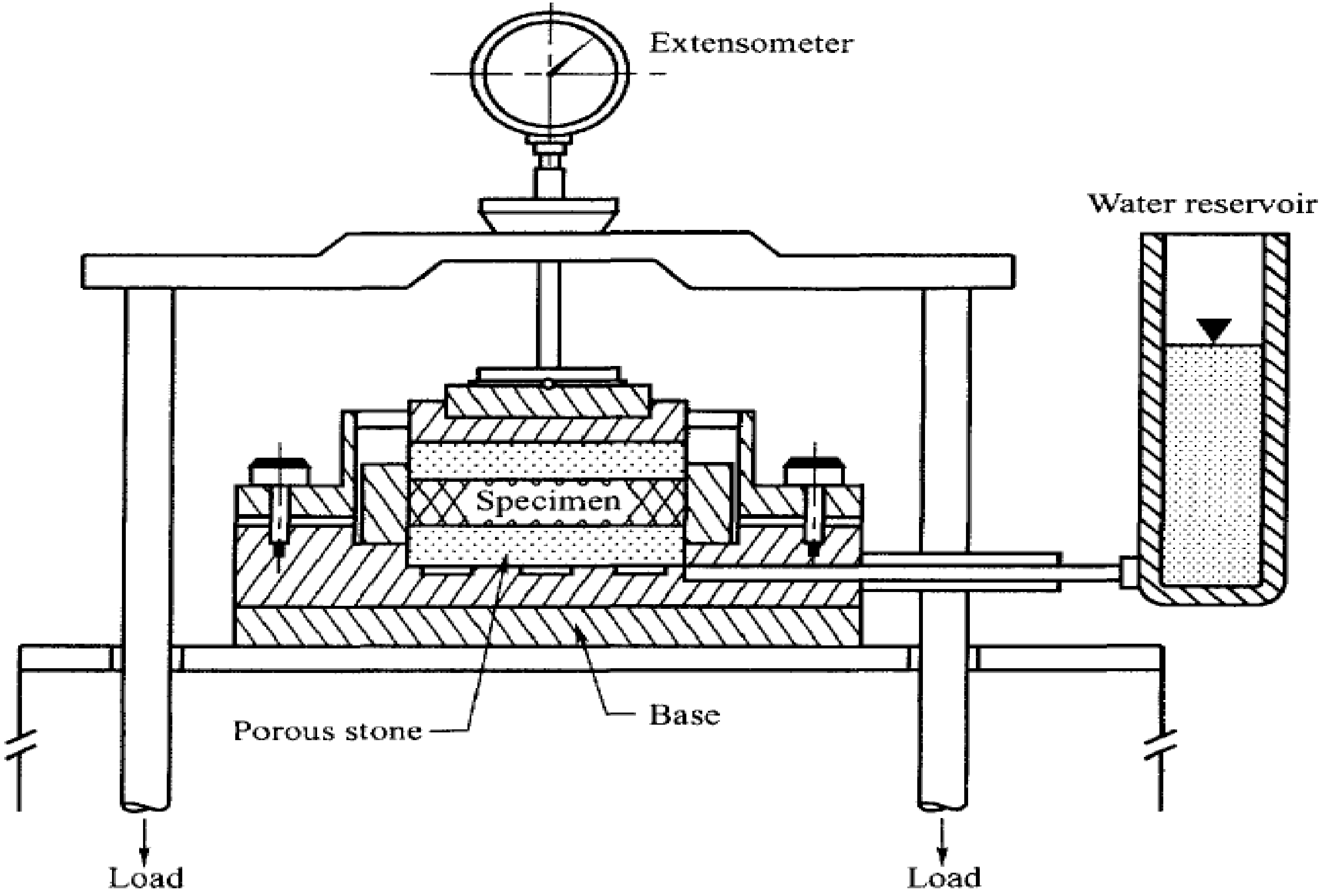
water under the induced hydraulic gradients, the process is referred to as *primary consolidation*. The portion of the settlement that is due to the primary consolidation is called *primary consolidation settlement* or *compression*.

SECONDARY CONSOLIDATION

The secondary consolidation is supposed to start after the primary consolidation ceases, that is after the excess pore water pressure approaches zero. It is often assumed that secondary compression proceeds linearly with the logarithm of time.

CONSOLIDATION TEST

- ❖The consolidation test is usually performed at room temperature, in floating or fixed rings of diameter from 5 to 1 1cm and from 2 to 4cm in height.
- ❖This apparatus is also known as an *oedometer.*



- The soil sample is contained in the brass ring between two porous stones about 1.25cm thick. By means of the porous stones water has free access to and from both surfaces of the specimen.
- The compressive load is applied to the specimen through a piston, either by means of a hanger and dead weights or by a system of levers. The compression is measured on a dial gauge.
- At the bottom of the soil sample the water expelled from the soil flows through the filter stone into the water container.
- At the top, a well-jacket filled with water is placed around the stone in order to prevent excessive evaporation from the sample during the test.
- Water from the sample also flows into the jacket through the upper filter stone.
- The soil sample is kept submerged in a saturated condition during the test.
- The main purpose of the consolidation test on soil samples is to obtain the necessary information about the compressibility properties of a saturated soil for use in determining the magnitude and rate of settlement of structures

The following test procedure is applied to any type of soil in the standard consolidation test.

- ❖Loads are applied in steps in such a way that the successive load intensity, *p*, is twice the preceding one.
- ❖The load intensities commonly used being 1/4, 1/2,1, 2,4, 8, and 16 tons/ft2 (25, 50, 100,200,400, 800 and 1600 kN/m2).
- ❖Each load is allowed to stand until compression has practically ceased (no longer than 24 hours).
- ❖ The dial readings are taken at elapsed times of 1/4, 1/2, *1,2,4*, 8, 15, 30, 60, 120, 240, 480 and 1440 minutes from the time the new increment of load is put on the sample (or at elapsed times as per requirements).

The following data should also be obtained:

1. Moisture content and weight of the soil sample before the commencement of the test. 2. Moisture content and weight of the sample after completion of the test. 3. The specific gravity of the solids. 4. The temperature of the room where the test is conducted.

DETERMINATION OF VOIDS RATIO IN CONSOLIDATION TEST

The pressure-void ratio curve can be obtained if the void ratio of the sample at the end of each increment of load is determined. Accurate determinations of void ratio are essential and may be computed from the following data:

1. The cross-sectional area of the sample *A*, which is the same as that of the brass ring.

2. The specific gravity, Gs, of the solids.

3. The dry weight, *Ws*, of the soil sample.

4. The sample thickness, *h*, at any stage of the test.

Let V_s = volume of the solids in the sample

where

$$V_s = \frac{W}{G_s \gamma_w}$$

where γ_w = unit weight of water

We can also write

$$V_s = h_s A \quad \text{or} \quad h_s = \frac{V_s}{A}$$

where, h_s = thickness of solid matter.

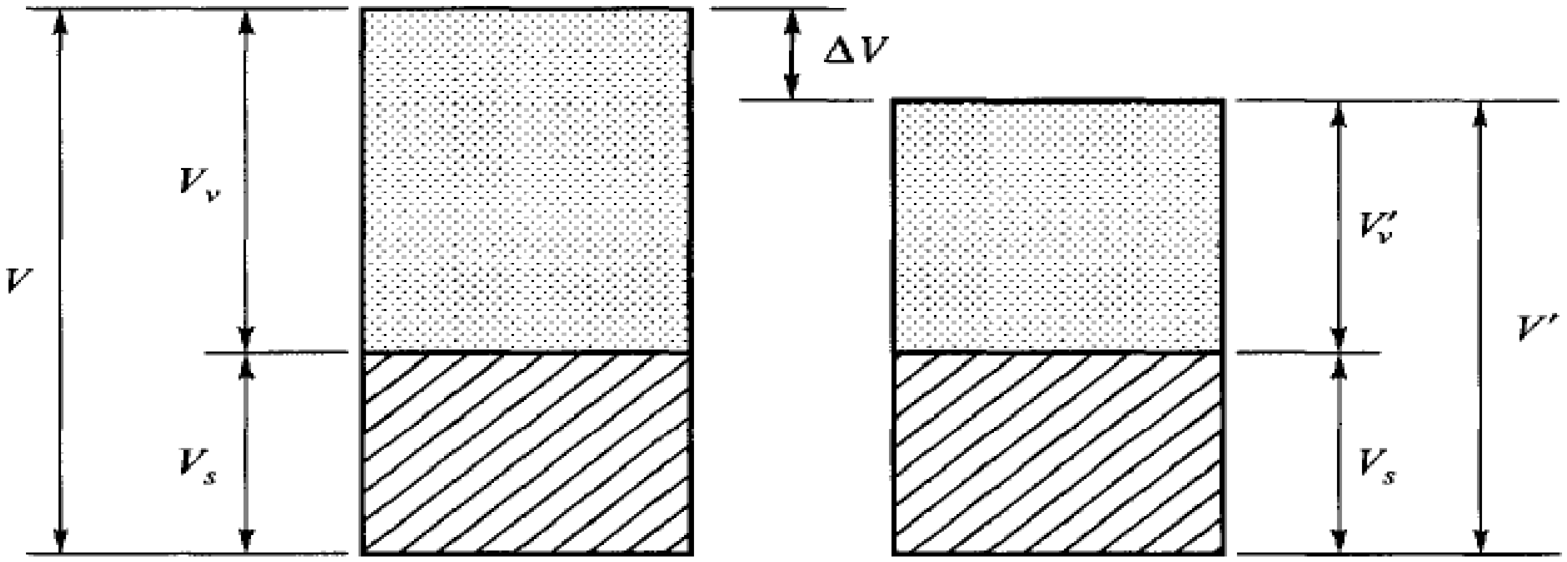
If e is the void ratio of the sample, then

$$e = \frac{Ah - Ah_s}{Ah_s} = \frac{h - h_s}{h_s}$$

In one-dimensional compression the change in height Δh per unit of original height h equals the change in volume ΔV per unit of original volume V.

$$\frac{\Delta h}{h} = \frac{\Delta V}{V}$$

V may now be expressed in terms of void ratio e.



(a) Initial condition (b) Compressed condition

We may write

$$V_v = eV_s, \quad V = V_s(1+e), \quad V_v' = e'V_s$$

$$V' = V_s(1+e')$$

$$\frac{\Delta V}{V} = \frac{V - V'}{V} = \frac{V_s(1+e) - V_s(1+e')}{V} = \frac{e-e'}{1+e} = \frac{\Delta e}{1+e}$$

Therefore,

$$\frac{\Delta h}{h} = \frac{\Delta e}{1+e}$$

or

$$\Delta e = \frac{1+e}{h}\Delta h$$

wherein, Δe = change in void ratio under a load, h = initial height of sample, e = initial void ratio of sample, e' = void ratio after compression under a load, Δh = compression of sample under the load which may be obtained from dial gauge readings.

TERZAGHI'S THEORY OF CONSOLIDATION

Terzaghi (1925) advanced his theory of one-dimensional consolidation based upon the following assumptions.

1. The soil is homogeneous (*kz* is independent of *z*).

2. The soil is completely saturated (S = 100%).

3. The soil grains and water are virtually incompressible (γw is constant and volume change of soil is only due to change in void ratio).

4. The behaviour of infinitesimal masses in regard to expulsion of pore water and consequent consolidation is no different from that of larger representative masses (Principles of calculus may be applied).

5. The compression is one-dimensional (u varies with z only).

6. The flow of water in the soil voids is one-dimensional, Darcy's law being valid.

$$\left(\frac{\partial v_x}{\partial x} = \frac{\partial v_y}{\partial y} = 0 \text{ and } v_z = k_z \cdot \frac{\partial h}{\partial z}\right).$$

Also, flow occurs on account of hydrostatic excess pressure ($h = u/\gamma_w$).

DETERMINATION OF COEFFICIENT OF CONSOLIDATION

The more generally used fitting methods are the following:

(*a*) The square root of time fitting method

(*b*) The logarithm of time fitting method

DETERMINATION OF COEFFICIENT OF CONSOLIDATION BY THE SQUARE ROOT OF TIME FITTING METHOD

- ❖This method has been devised by D.W. Taylor (1948).
- ❖The coefficient of consolidation is the soil property that controls the time-rate or speed of consolidation under a load-increment.
- ❖The relation between the sample thickness and elapsed time since the application of the load increment is obtainable from an oedometer test and is somewhat as shown in Fig. for atypical load-increment.

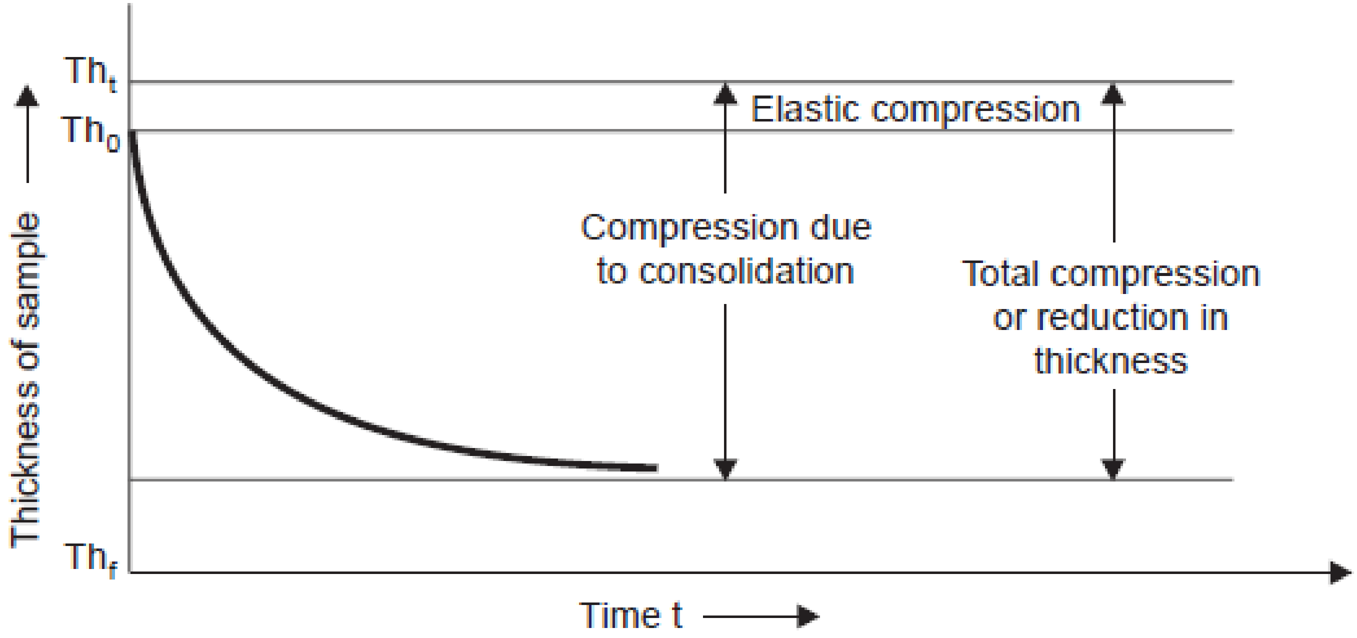
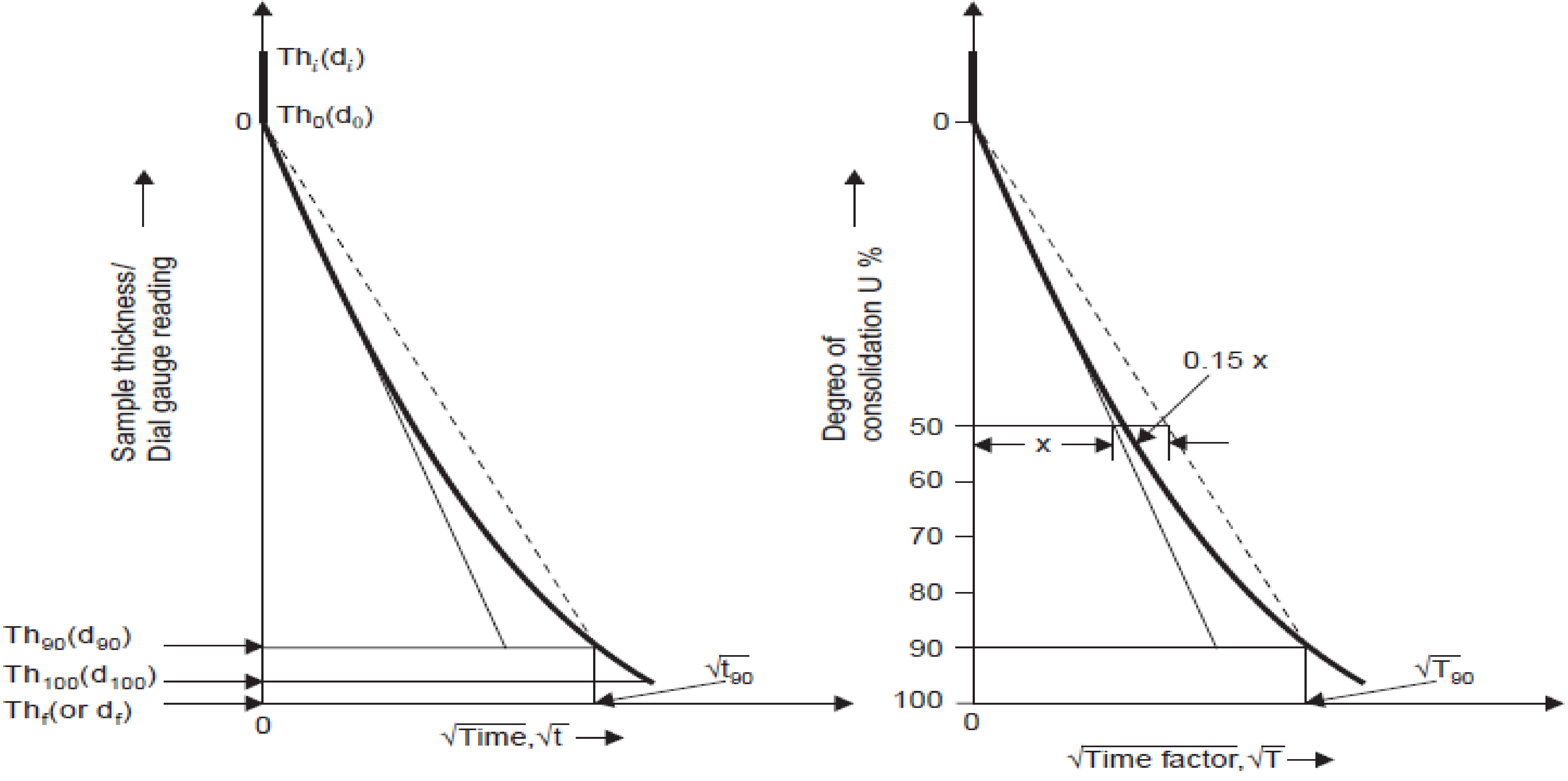


Fig. Time versus reduction in sample thickness for a load-increment

- ❖This figure depicts change in sample thickness with time essentially due to consolidation; only the elastic compression which occurs almost instantaneously on application of load increment is shown.
- ❖The effect of prolonged compression that occurs after 100% dissipation of excess pore pressure is not shown or is ignored; this effect is known as 'Secondary consolidation', which is briefly presented in the following section.



(a) Sample thickness/Dial gauge reading versus square root of time (Laboratory curve)

(b) Degree of consolidation versus square root of time factor (Theoretical curve from Terzaghi's theory)

- ❖The laboratory curve shows a sudden initial compression, called ‘elastic compression’ which may be partly due to compression of gas in the pores.
- ❖ The corrected zero point at zero time is obtained by extending the straight line portion of the laboratory plot backward to meet the axis showing the sample thickness/dial gauge reading.
- ❖The so-called ‘primary compression’ or ‘primary consolidation’ is reckoned from this corrected zero.
- ❖ A dashed line is constructed from the corrected zero such that its abscissae are 1.15 times those of the straight line portion of the laboratory plot.
- ❖The intersection of the dashed line with the laboratory plot identifies the point representing 90% consolidation in the sample.
- ❖The time corresponding to this can be read off from the laboratory plot.
- ❖The point corresponding to 100% primary consolidation may be easily extrapolated on this plot.

The coefficient of consolidation, c_v, may be obtained from

$$c_v = \frac{T_{90} H^2}{t_{90}}$$

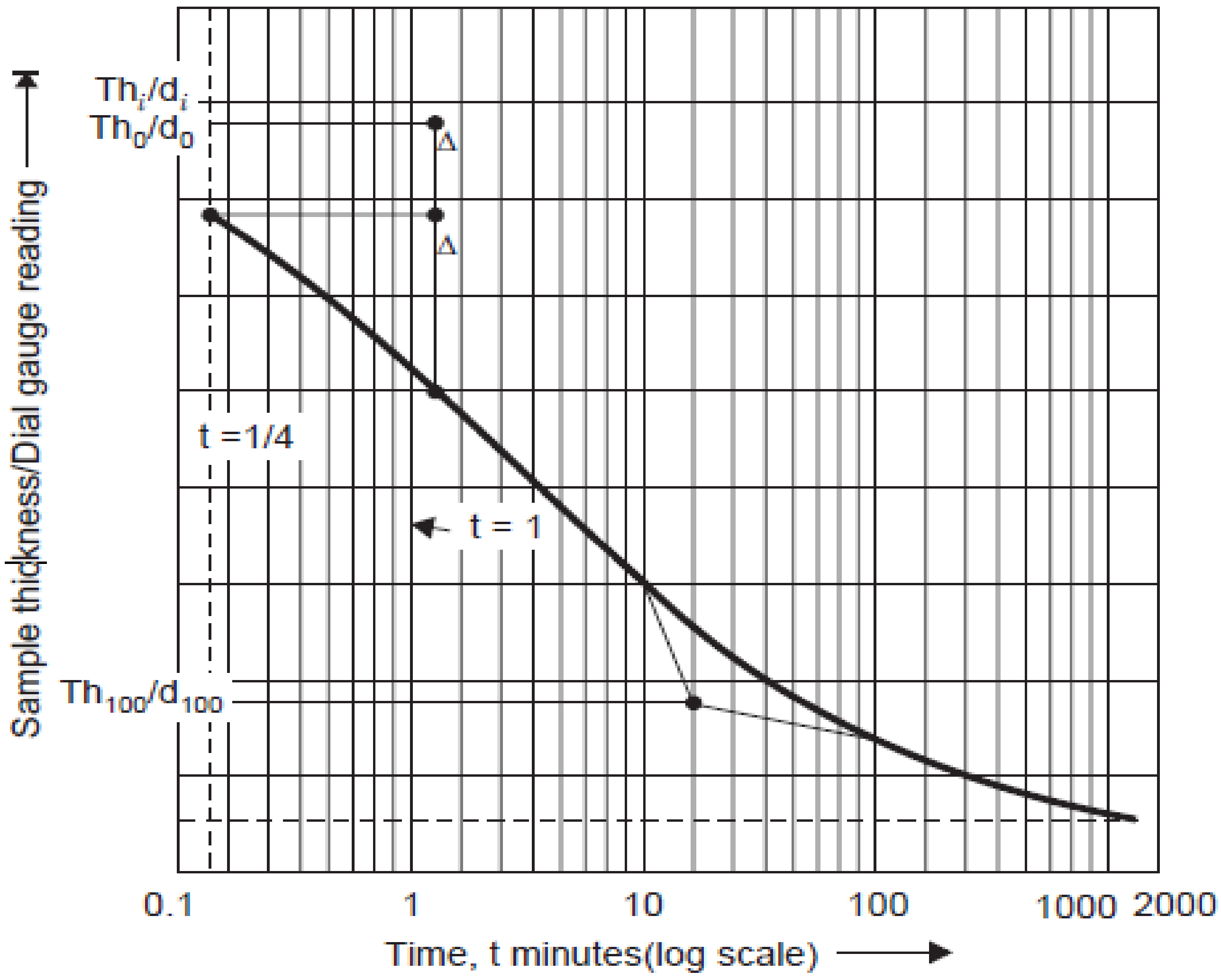
where t_{90} is read off from Fig. 7.27(*a*)

T_{90} is 0.848 from Terzaghi's theory

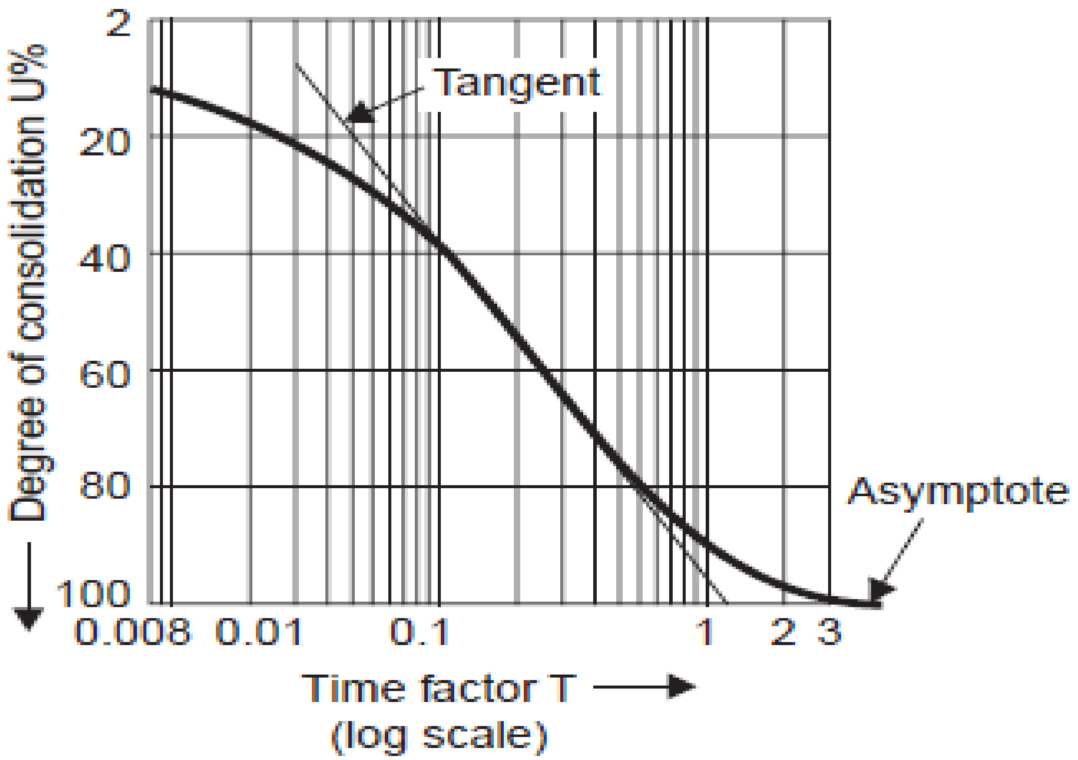
H is the drainage path, which may be taken as half the thickness of the sample for double drainage conditions,

DETERMINATION OF COEFFICIENT OF CONSOLIDATION BY THE LOGARITHM OF TIME FITTING METHOD

- This method was devised by A. Casagrande and R.E. Fadum (1939).
- The point corresponding to 100 per cent consolidation curve is plotted on a semi-logarithmic scale, with time factor on a logarithmic scale and degree of consolidation on arithmetic scale, the intersection of the tangent and asymptote is at the ordinate of 100% consolidation.
- A comparison of the theoretical and laboratory plots in this regard is shown in Figs.



(a) Sample thickness/Dial gauge reading versus logarithm of time (Laboratory curve)



(b) Degree of consolidation versus logarithm of time factor (Theoretical curve from Terzaghi's theory)

- Since the early portion of the curve is known to approximate a parabola, the corrected zero point may be located as follows:
- The difference in ordinates between two points with times in the ratio of 4 to 1 is marked off; then a distance equal to this difference may be stepped off above the upper points to obtain the corrected zero point.
- This point may be checked by more trials, with different pairs of points on the curve. After the zero and 100% primary compression points are located, the point corresponding to 50% consolidation and its time may easily be obtained and the coefficient of consolidation computed from:

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

where t_{50} is read off from Fig. 7.28(a)

$T_{50} = 0.197$ from Terzaghi's theory, and

H is the drainage path as stated in the previous subsection.

The primary compression ratio may be obtained as given in the previous subsection.

PRE-CONSOLIDATION

- The pre-consolidation pressure for an over-consolidated soil should not be exceeded in construction, if possible.
- Consolidation settlements will small if the effective vertical stress in the soil layer remains below its pre-consolidation pressure.
- If effective vertical stress in the soil layer exceeds its pre-consolidation pressure, the consolidation settlements will be large due to further yielding of the soil layer.
- The estimation of pre consolidation pressure is greatly affected by the amount of disturbance experienced by the soil sample.
- The steps in the geometrical construction for determination of pre-consolidation.

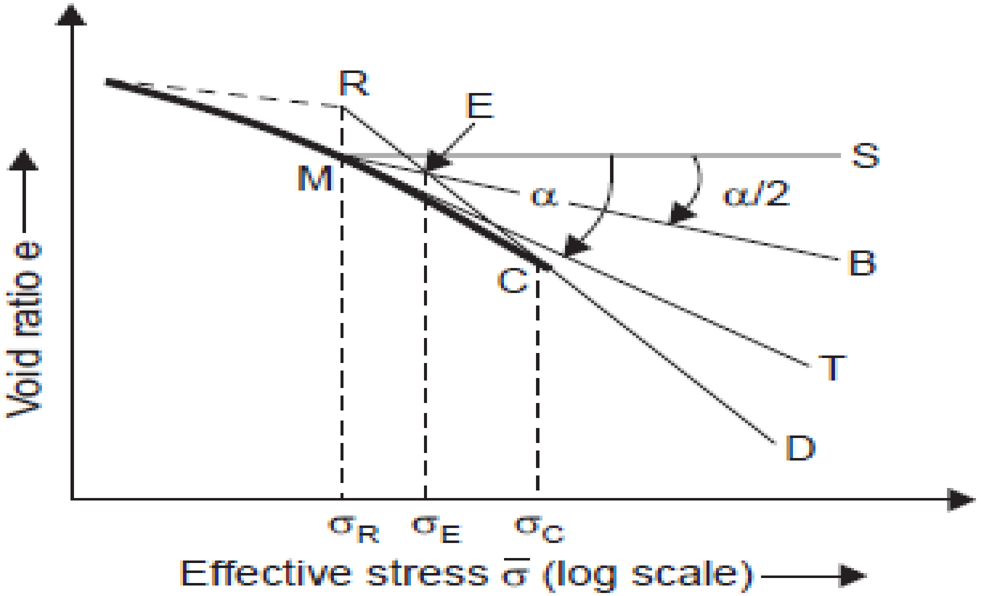


Fig. A. Casagrande's procedure for determining preconsolidation pressure

1. The point of maximum curvature **M** on the curved portion of the *e* vs. log $\overline{\sigma}$ plot is located.

2. A horizontal line **MS** is drawn through **M**.

3. A tangent **MT** to the curved portion is drawn through **M**.

4. The angle **SMT** is bisected, **MB** being the bisector.

5. The straight portion **DC** of the plot is extended backward to meet **MB** in **E**.

6. The pressure corresponding to the point **E**, $\overline{\sigma}_E$, is the most probable past maximum effective stress or the preconsolidation pressure.

EXAMPLES

Example 1: In a consolidation test the following results have been obtained. When the load was changed from 50kN/m2 to 100kN/m2, the void ratio changed from 0.70 to 0.65. Determine the coefficient of volume decrease, *mv* and the compression index, *Cc*.

Solution

$e_0 = 0.70 \qquad \bar{\sigma}_0 = 50 \text{ kN/m}^2$

$e_1 = 0.65 \qquad \bar{\sigma} = 100 \text{ kN/m}^2$

Coefficient of compressibility, $a_v = \dfrac{\Delta e}{\Delta \bar{\sigma}}$, ignoring sign.

$$= \frac{(0.70 - 0.65)}{(100 - 50)} \text{ m}^2/\text{kN} = 0.05/50 \text{ m}^2/\text{kN} = 0.001 \text{ m}^2/\text{kN}.$$

Modulus of volume change, or coefficient of volume decrease,

$$m_v = \frac{a_v}{(1+e_0)} = \frac{0.001}{(1+0.70)} = \frac{0.001}{1.7} \text{ m}^2/\text{kN}.$$

$$= 5.88 \times 10^{-4} \text{ m}^2/\text{kN}$$

Compression index, $C_c = \dfrac{\Delta e}{\Delta(\log \bar{\sigma})} = \dfrac{(0.70 - 0.65)}{(\log_{10} 100 - \log_{10} 50)}$

$$= \frac{0.05}{\log_{10} \frac{100}{50}} = \frac{0.05}{\log_{10} 2} = \frac{0.050}{0.301} = \mathbf{0.166.}$$

Example .2: A sand fill compacted to a bulk density of 18.84 kN/m^3 is to be placed on a compressible saturated marsh deposit 3.5 m thick. The height of the sand fill is to be 3 m. If the volume compressibility m_v of the deposit is 7×10^{-4} m^2/kN, estimate the final settlement of the fill.

Ht. of sand fill = 3 m

Bulk unit weight of fill = 18.84 kN/m^3

Increment of the pressure on top of marsh deposit $\Delta\bar{\sigma} = 3 \times 18.84$

$= 56.52$ kN/m^2

Thickness of marsh deposit, $H_0 = 3.5$ m

Volume compressibility $m_v = 7 \times 10^{-4}$ m^2/kN

Final settlement of the marsh deposit, ΔH

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

$$= 7 \times 10^{-4} \times 3500 \times 56.52 \text{ mm}$$

$$= \mathbf{138.5 \text{ mm}}.$$

Example 3: A layer of soft clay is 6 m thick and lies under a newly constructed building. The weight of sand overlying the clayey layer produces a pressure of 260 kN/m2 and the new construction increases the pressure by 100kN/m2. If the compression index is 0.5, compute the settlement. Water content is 40% and specific gravity of grains is 2.65.

Solution

Initial pressure, $\overline{\sigma}_0 = 260 \text{ kN/m}^2$

Increment of pressure, $\Delta\overline{\sigma} = 100 \text{ kN/m}^2$

Thickness of clay layer, $H = 6 \text{ m} = 600 \text{ cm}$.

Compression index, $C_c = 0.5$

Water content, $w = 40\%$

Specific gravity of grains, $G = 2.65$

Void ratio, $e = wG$, (since the soil is saturated) $= 0.40 \times 2.65 = 1.06$

This is taken as the initial void ratio, e_0.

Consolidation settlement,

$$S = \frac{H.C_c.}{(1+e_0)} \log_{10}\left(\frac{\overline{\sigma}_0 + \Delta\overline{\sigma}}{\overline{\sigma}_0}\right)$$

$$= \frac{600 \times 0.5}{(1+1.06)} \log_{10}\left(\frac{260+100}{260}\right) \text{ cm}$$

$$= \frac{300}{2.06} \log_{10}\left(\frac{360}{260}\right) \text{ cm}$$

$$= 21.3 \text{ cm}.$$

Example 4: There is a bed of compressible clay of 4m thickness with pervious sand on top and impervious rock at the bottom. In a consolidation test on an undisturbed specimen of clay from this deposit 90% settlement was reached in 4 hours. The specimen was 20 mm thick. Estimate the time in years for the building founded over this deposit to reach 90% of its final settlement.

<u>Solution</u>

This is a case of one-way drainage in the field.

∴ Drainage path for the field deposit, H_f = 4 m = 4000 mm. In the laboratory consolidation test, commonly it is a case of two-way drainage.

∴ Drainage path for the laboratory sample, H_1 = 20/2 = 10 mm

Time for 90% settlement of laboratory sample = 4 hrs.

Time factor for 90% settlement, T_{90} = 0.848

$$\therefore \quad T_{90} = \frac{C_v t_{90_f}}{H_f^2} = \frac{C_v t_{90_l}}{H_l^2}$$

or

$$\frac{t_{90_f}}{H_f^2} = \frac{t_{90_l}}{H_l^2}$$

$$\therefore \quad t_{90f} = \frac{t_{90_l}}{H_l} \times H_f = \frac{4 \times (4000)^2}{10^2} \text{ hrs}$$

$$= \frac{4 \times 400}{24 \times 365} \text{ years}$$

$$\approx \textbf{73 years.}$$