CONSOLIDATION

7.1 GENERAL INTRODUCTION

When a load is applied on a material, stresses and strains are produced in the material. In elastic materials such as steel, the stresses and strains are produced simultaneously without any time lag in between. Whereas in non-elastic materials (e.g. soils) stress-strain relationship is time dependent, that is, on application of load, stresses are produced but signs of deformation become visible only after some time. The materials in which the stress-strain relationship is time dependent are called *visco-elastic* materials and soils fall under this category of materials. Furthermore, when soils are subjected to load, they deform, and even when the load is released some permanent deformation remains recorded in its memory. Thus the soils have a system of memory which registers the foot prints of each and every loadings in the form of its geological history. This chapter is devoted for the study of stress-strain-time relationship of a soil mass subjected to external loadings.

7.2 CONSOLIDATION OF SOILS

As stated earlier, when a load is applied to a soil mass, the soil compresses. The compression of soil may be due to any of the following factors or the combined affect of these factors:-

- Distortion (change of shape) of soil grains.
- Compression of air and water in soil voids.
- Reduction of volume due to expulsion of water and/or air from the voids.

Under usual range of loadings applied on a soil mass through the foundations of civil engineering structures, the distortion (that is the deformation caused due to crushing of soil grains) is small and negligible. Fine grained soils in nature are generally saturated and the amount of air is very small and insignificant. Water being incompressible fluid does not cause significant deformation in soils under practical range of loadings. Thus the deformations in saturated soils are mainly due to the reduction in volume brought about by expulsion of water from the voids. This phenomenon is termed as *Consolidation. Thus consolidation is the compression of a soil mass due to expulsion of water when subjected to external compression loads.* The consolidation process is, therefore, essentially a drainage process.

7.3 CONSOLIDATION MODEL (HYDROMECHANICAL ANALOG)

To understand the mechanism of consolidation, consider Fig. 7.1

Fig. 7.1(a) represents a saturated cylinder of soil mass. The porous piston in this figure permits load to be applied to the soil allowing escape of water through the pores of the piston. Fig. 7.1(b) shows a hydromechanical analog in which the spring represents the soil mineral skelaton and water in the cylinder represents the pore fluid in the soil mass. The soil permeability is represented by the valve attached to the otherwise impermeable piston.

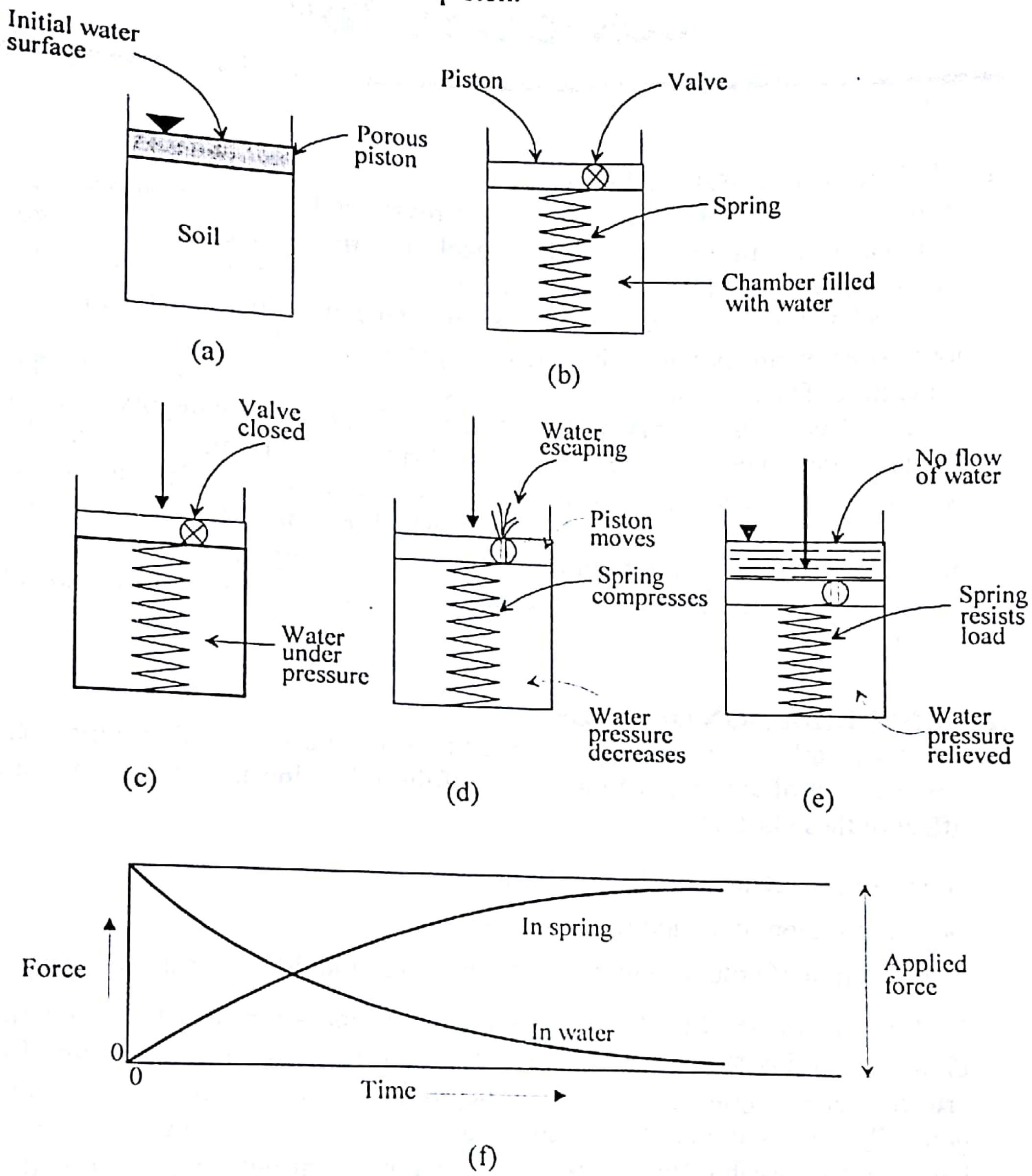


Figure 7.1 Hydromechanical analogy for load-sharing and consolidation. (a) Physical example. (b) Hydromechanical analog; initial condition. (c) Load applied with valve closed. (d) Piston moves as water escapes. (e) Equilibrium with no further flow. (f) Gradual transfer of load.

In Fig. 7.1 (c) an external stress is applied on the piston with the valve closed. Essentially all the applied stress is resisted by an increase in the pore pressure known as hydrostatic excess pore water pressure (neutral pressure). At this stage, following relationship exists:

$\Delta\sigma_v = \Delta u$ (that is the total stress is equal to the pore pressure.)

Next open the valve. The fluid pressure within the cylinder will cause the water to flow through this valve (Fig. 7.1d). As the water escapes the spring starts shortening due to the transfer of load from the fluid to the spring (i.e. at this instant, load sharing between the water and the spring starts). At any instant of time the following relation holds good:

$$\Delta\sigma_v = \Delta\bar{\sigma}_v + \Delta u \qquad 7.1$$

Where,

$\Delta\sigma_v$ = total stress (i.e. applied external pressure)

$\Delta\bar{\sigma}_v$ = the stress carried by the spring that is by soil skelaton or particles, in general, called as effective stress or inter-granular pressure.

Δu = pore water pressure also known as neutral pressure.

If for a given applied external stress the valve is kept, open for a time large enough, eventually a condition is reached [Fig. 7.1 (e)] when the entire applied stress is carried by the spring and the pore pressure is dissipated to original hydrostatic condition. At this stage threre will be no further flow of water through the valve.

Fig. 7.1 (f) represents load sharing graph between the spring and the water.

From this hydromechanical analog following conditions are evidents:

(1) The magnitude of deformation (settlement) in a consolidation is dependent only on the compressibility of the soil (i.e. the stiffness of the spring). The compressibility is expressed in term of a coefficient known as compression index (C_c).

(2) The rate of consolidation is a function of both permeability and compressibility of the soil. The combined effect of permeability and the compressibility is represented by a co-efficient termed as the co-efficient of consolidation (C_v)

(3) The time required for the consolidatin process is related to the following two factors:

(a) The time should be directly proportional to the volume of water which must be squeezed out of the soil mass. This volume of water in turn be related to the product of stress change, the compressibility of the soil mineral skelaton, and volume of the soil.

(b) The time should be inversely proportional to how fast the water can flow through the soil mass. On the other hand, velocity of flow = ki (i.e. the product of permeability and hydraulic gradient) and the hydraulic gradient i is the head lost per unit length through which the fluid must flow.

Mathematically these two considerations (i.e. a and b) can be expressed by the relation:

$$t \approx \frac{(\Delta\sigma)(m_v)(H)}{(k)(\frac{\Delta\sigma}{\gamma_w})/H} \qquad 7.2$$

Where,

t = the time required to complete some percentage of consolidation process.

$\Delta\sigma$ = the change in the applied stress causing consolidation.

m_v = co-efficient of volume change per drainage face

H = the thickness of the soil mass per drainage face (i.e. drainage path)

k = co-efficient of permeability of the soil mass.

i = hydraulic gradient = head lost per unit length

$= (\frac{\Delta\sigma}{\gamma_w})/H$

Equation 7.2 is reduced to $$t = \frac{m_v H^2}{k}\gamma_w$$

or $$t = \frac{T m_v \gamma_w H^2}{k}$$

Let $\frac{k}{m_v \gamma_w} = C_v$ = Co-efficient of consolidaton

Then,

$$t = \frac{T H^2}{C_v} \quad \text{or}$$

$$T = \frac{t\, C_v}{H^2} \qquad 7.3$$

Where T = dimensionless constant known as time factor.

Table 7.1 presents the values of T for a linear distribution of excess pore water pressure.

These relations tell us that the consolidation time:

(1) Increases with increasing co-efficient of consolidation (C_v) of the soil mass or co-efficient of volume change (m_v).

(2) Decreases with increasing permeability co-efficient (k)

(3) Increases rapidly with increasing thickness of soil mass (H)

(4) Is independent of the magnitude of the stress changes ($\Delta\sigma$).

Table 7.1 Time factor values at different degrees of consolidation

U_{avg}	T
0.1	0.008
0.2	0.031
0.3	0.071
0.4	0.126
0.5	0.197
0.6	0.287
0.7	0.403
0.8	0.567
0.9	0.848
0.95	1.163
1.0	∞

U_{avg} = average degree of consolidation.

Approximately:

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

For $U > 60\%$, $T = 1.781 - 0.933 \log(100 - U\%)$

7.4 OEDOMETER (CONSOLIDOMETER) TEST

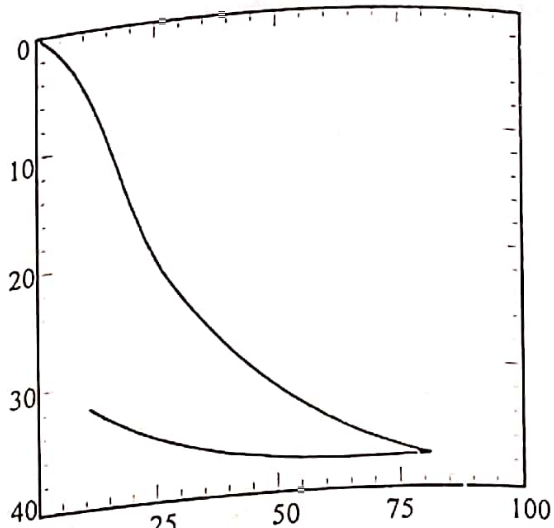
When a soil mass under the foundation of a structure is loaded vertically, the compression of the soil can be assumed to be one dimensional. To simulate the one dimensional compression in the laboratory, the soil sample is compressed in a special device called an oedometer or consolidometer. Schematic sketches of two commonly used oedometer are shown in Fig. 7.2

The details of this test can be found in any soil testing manual and a brief description is presented below:

In this test a soil sample is carefully trimmed and placed into a consolidation ring. The ring is relatively rigid and does not allow any lateral deformation. On both ends of the sample porous stones are placed to facilitate drainage from either ends druing consolidation process. Usually the ratio of the diameter to the height of the sample is between 2.5 and 5, depending upon the diameter of the sampler.

To study the relationship between load and deformation, compression load on the test sample is applied in several increments and each increment is allowed to remain on the sample until the further consolidation is negligible (usually for 24 hours). For each increment of load deformation versus time are recorded and time consolidation curve is drawn. Usually the load is applied in increments of 0.1, 0.2, 0.4, 0.8, 1.6, 3.2, 6.4, 12.8, and 25.6 kg/cm² and for each load increment deformation is recorded at time intervals of 0.25, 0.5, 1.0, 2, 4, 8, 15, 30, 60, 120, 240, 480 and 1440 minutes.

After completion of loading sequence, unloading is done in decrements to provide data for expansion during load release.



(a)

e
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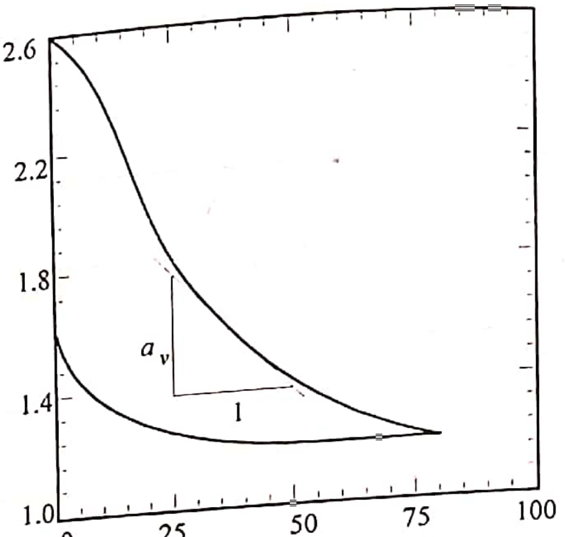
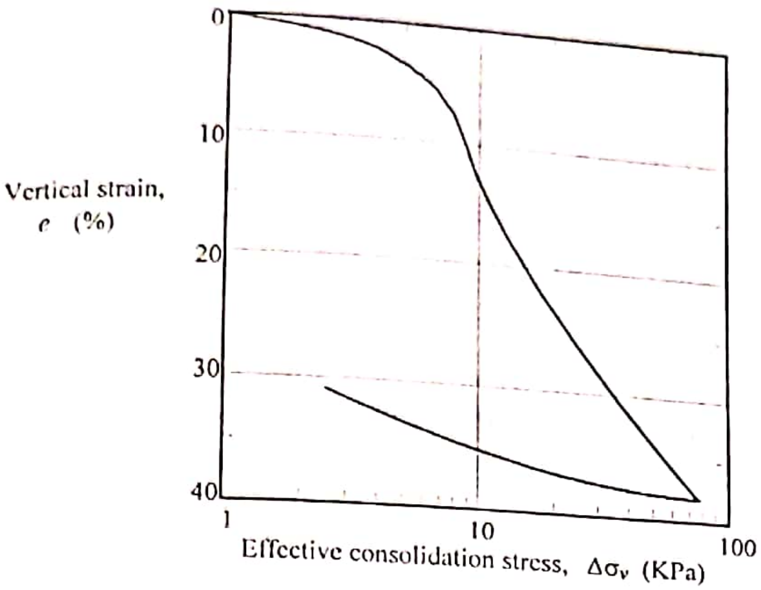
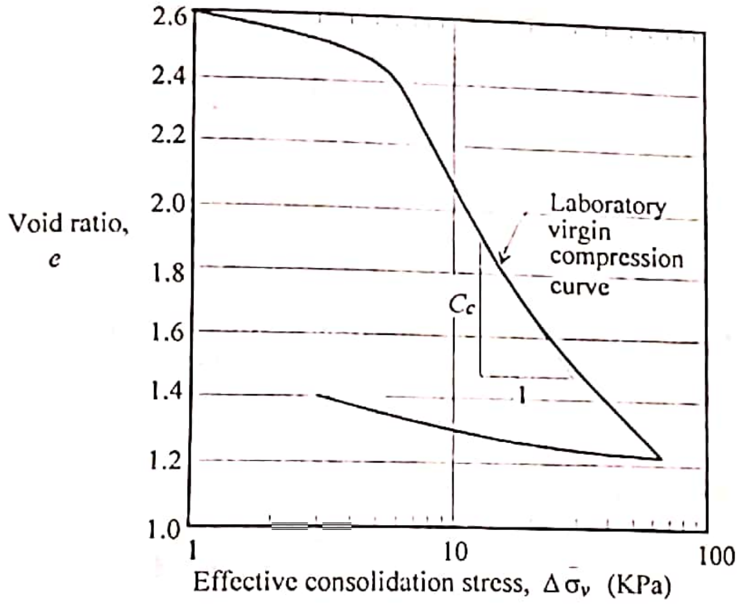


(b)

Figure 7.3 Two ways to present consolidation test data: (a) Percent consolidation (or strain) versus effective stress: (b) Void ratio versus effective stress.



(a)



(b)

Figure 7.4 Consolidation test data presented as: (a) Percent consolidation (or strain) versus log effective stress and (b) Void ratio versus log effective stress.

- Time deformation curve

Time deformation curves are used to determine the co-efficient of consolidation C_v which is used in settlement rate analysis.and since both the curves fitting methods (i.e. Casagrande's and Taylor's methods) are approximations to theory, you should not expect them to agree exactly. Often C_v as determined by the $\sqrt{t}$ method is slightly greater than by the log$t$ fitting method. Brief descriptions of these methods are as under:

Two empirical methods known as *curve fitting methods* were developed by Casagrande and Taylor. These methods were developed to fit approximately the laboratory test data to the Tergazhi theory of consolidation.

(a) Casagrande's log time fitting method (1938)

In this method the deformation dial readings are plotted versus the log of time, as shown in Fig. 7.5. In this Fig. R_o and R_{100} represent dial readings for zero % degree of consolidatin, U_o and 100% consolidation, U_{100} respectively. According to Casagrande, the times for U_o and U_{100} are estimated using the following procedure:

- To determine U_{100}, draw two tangents to the laboratory consolidation curve (Fig 7.5). The intersection of these tagents would fix the position of U_{100} and the time for 100% primary consolidation, t_{100} can be read against this position from the graph directly.
- To determine R_o or U_o proceed as below:
  - (i) Choose any two times t_1 and t_2, in the ratio of 4 to 1 (i.e. $t_2 = 4t_1$) and record dial readings R_1 and R_2 corresponding to t_1 and t_2 respectively.
  - (ii) Mark off a distance above R_1 equal to the difference $R_2 - R_1 = x$ (say). This defines the correct zero point R_o for U_o. In equation form

$$R_o = R_1 - (R_2 - R_1) \qquad 7.4$$

Several trails are usually advisible to obtain a good average value of R_o (U_o).

Once U_o and U_{100} are determined, the t_{50}, the time for 50% degree of consolidation can be read directly from the Fig. 7.5 and C_v is calculated using:

$$T = \frac{C_v\, t_{50}}{H^2} \qquad 7.5$$

Where,

T = time factor, for t_{50}= 0.197

(b) Taylor's square root of time fitting method (1948)

Fig. 7.6 represents a curve of deformation dial reading plotted against square root of time, $\sqrt{t}$. Taylor observed that the abscissa of the curve at 90% consolidation (U_{90}) was about 1.15 times the abscissa of the extension of the straight line and, therefore, this property of the plot helps in locating the position of U_{90} and t_{90} (Fig. 7.6). To determine U_o or R_o, he recommended to proceed as follows:

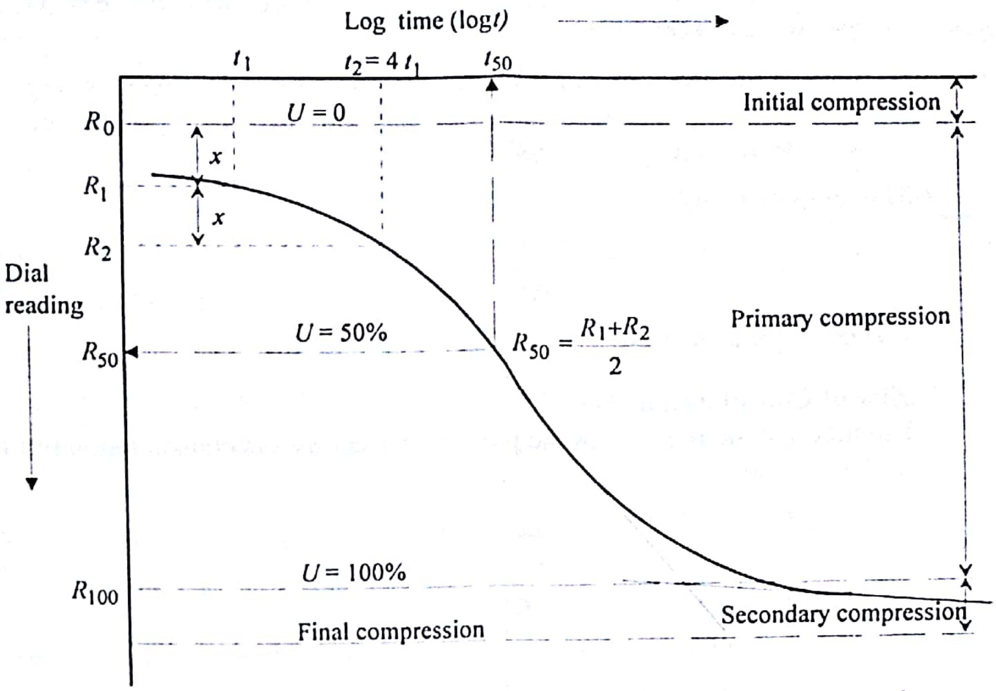


Figure 7.5 Determination of t_{50} by the Casagrande method

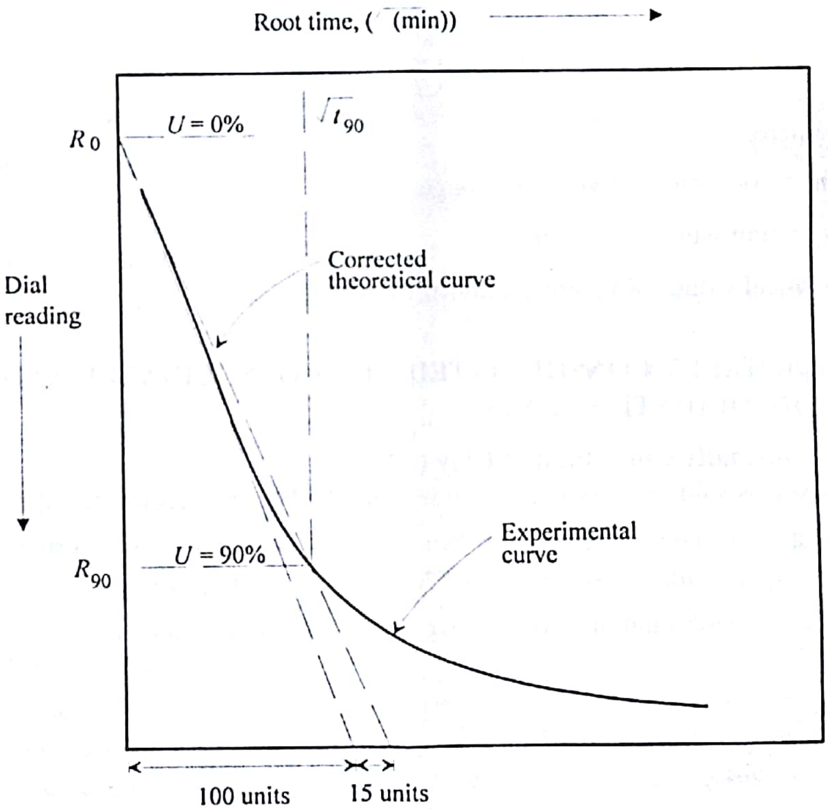


Figure 7.6 *Determination of C_v using Taylor's square root of time method.*

(i) Project the straight line portion of the initial part of curve (Fig. 7.6) backword to zero time to define R_o or U_o.

(ii) Draw a second line from R_o with all abscissas 1.15 times as large as the corresponding values on the first line. The intersection of the second line with the laboratory curve defines R_{90} or U_{90}.

(iii) Compute C_v using

$$T = \frac{C_v\, t_{90}}{H^2} \qquad 7.6$$

Where T for $t_{90} = 0.848$.

Utility of Consolidation Test

To summarize the findings of the preceding sections, the consolidation test data is used for the following:

(i) Load-deformation curve (i.e. void ratio versus $\log\Delta\overline{\sigma}_v$ plot) is utilized to compute compression index, C_c, which is used for the computation of total settlement in settlement analysis.

(ii) Time-deformation curves (i.e. degree of consolidation, U, versus time plots) are used to compute coefficient of consolidation, C_v which is utilized in rate of settlement analysis.

(iii) Consolidation test data can be used to calculate the co-efficient of permeability, k from the following relation:

$$C_v = \frac{k}{m_v \gamma_w} \qquad 7.7$$

Where,

m_v = coefficient of volume change

γ_w = unit weight of water

Typical values of C_v are summarized in Table 7.3.

7.5 NORMALLY CONSOLIDATED, PRE-CONSOLIDATED AND UNDER CONSOLIDATED CLAYS

- **Normally Consolidated Clay (NCC)**

A soil is said to be *normally consolidated* when the preconsolidation pressure $\overline{\sigma}_p$ is approximately equal to the existing effective vertical overburden pressure, $\overline{\sigma}_{vo}$ (i.e. $\overline{\sigma}_{vo}$ is within about ±10% of $\overline{\sigma}_p$). Thus a NCC has never been subjected to a stress greater than the existing overburden pressure in the past.

Table 7.3 Typical values of the coefficient of consolidation C_v

Soil	C_v	
	$cm^2/s \times 10^{-4}$	m^2/yr
•Boston blue clay (CL) (Ladd and Luscher, 1965)	40 ± 20	12 ± 6
•Organic silt (OH) (Lowe, Zaccheo, and Feldman, 1964)	2–10	0.6–3
•Glacial lake clays (CL) (Wallace and Otto, 1964)	6.5–8.7	2.0–2.7
•Chicago silty clay (CL) (Terzaghi and Peck, 1967)	8.5	2.7
•Swedish medium sensitive clays (CL–CH) (Holtz and Broms, 1972)		
1. laboratory	0.4–0.7	0.1–0.2
2 field	0.7–3.0	0.2–1.0
•San Francisco Bay Mud (CL)	2–4	0.6–1.2
•Mexico City clay (MH) (Leonards and Girault, 1961)	0.9–1.5	0.3–0.5

- **Pre-consolidated Clay (PCC)**

On the other hand, if the pre-consolidation pressure, $\bar{\sigma}_p$ is greater than the existing overburden pressure, $\bar{\sigma}_{vo}$ (i.e. $\bar{\sigma}_p > \bar{\sigma}_{vo}$) then the soil is called as *pre-consolidated or over-consolidated clay* (OCC).

- **Over-consolidation Ratio, (OCR)**

OCR is a tool, generally used to distinguish amongst NCC, OCC and UCC soils. This is defined as:

$$OCR = \bar{\sigma}_p / \bar{\sigma}_{vo} \qquad 7.8$$

For,

NCC OCR = 1

OCC OCR > 1

UCC OCR < 1 (Under-consolidated clay)

- **Under-consolidated Clay (UCC)**

Under-consolidation can occur, for example, in soils that have only recently been deposited, either geologically or by man. Under these conditions, the soil layer has not yet come to equilibrium under the weight of the overburden load that is the pore pressure is in a hydro excess state.

Comparison of NCC and OCC

Description of Parameter	NCC	OCC
Natural Moisture content, w_n	High, usually close to liquid limit	Relatively low, usually less than plastic limit
Density, γ	Relatively low	Usually high compared to NCC
Liquidity index	0.6 – 1 and over	0 – 0.6
SPT-resistance	Low with consistency between very soft and firm	High, with consistency stiff to hard
Compressibility	Under a given intensity of loading, settlement is comparatively higher	Settlement is relatively small for the same intensity of loading
Unconfined compression strength, q_u	Very low to low	Medium to high

Causes of Preconsolidation

There can be many reasons for preconsolidation. Some of these are summarized below:

Reason	Remarks
• Change in total stress due to: - Removal of overburden - Past structures - glacination	Geologic erosion or excavation by men
• Change in pore water pressure due to: - W/T fluctuations - Artesian pressures - Deep pumping; flow into tunnels - Desication due to surface drying - Desication due to plant life	Common in city areas
• Change in soil structure due to - Secondary compression (aging)	OCR as high as about 1.9 has been reported by different researchers
• Environmental chages such as pH, temperature, and salt concentration	Lambe (1958)
• Chemical alterations due to *weathering*, precipitation, cementing agents, ion exchange etc.	Bjerrum (1967)
• Change of strain rate on loading	Lowe (1974)

Preconsolidation Pressure

Several procedures have been proposed to determine $\overline{\sigma}_p$. Some of the most commonly used are briefly discussed below:-

(a) Casagrande's method (1936)

Casagrande proposed to utilize *e* versus log$\overline{\sigma}_v$ graph for the determination of $\overline{\sigma}_p$. Fig. 7.7 explains this procedure. A brief description of this procedure is given below:

(i) Choose by eye the point of minimum raduis (or maximum curvature) on *e*-log$\overline{\sigma}_v$ plot (point *A*, Fig. 7.7)

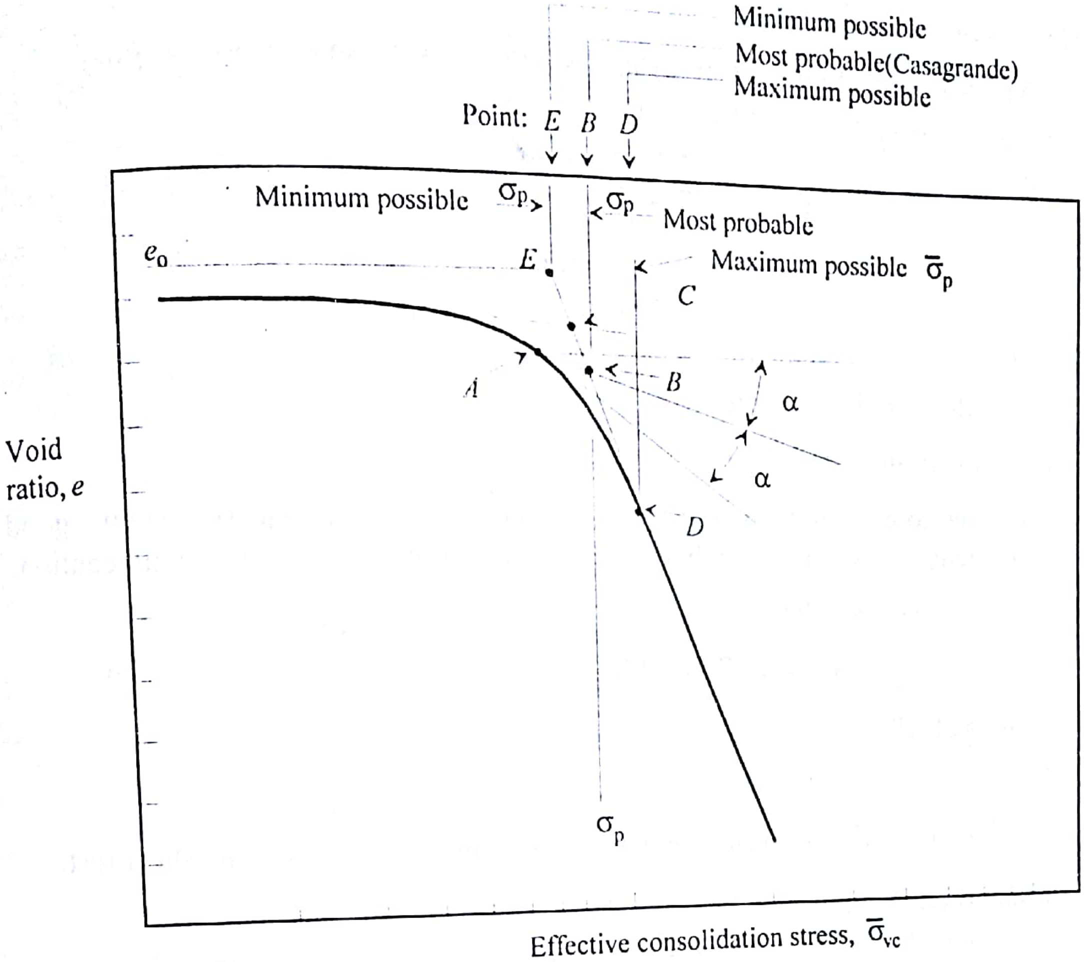


Figure 7.7 The Casagrande (1936b) construction for determining the pre-consolidation stress. Also known are the minimum possible, the most probable, and the maximum possible pre-consolidation stresses.

(ii) Draw a horizontal line from point A.

(iii) Draw a targent to the curve at point A

(iv) Bisect the angle made between the horizontal line through A and the tagent at A (steps ii and iii above).

(v) Extend the straight line portion of the virgin compression curve up to where it meets the biscator line obtained in step iv. The point of intersection of these two lines is the preconsolidation streess (point B in Fig. 7.7).

An even simpler method for estimating the $\bar{\sigma}_p$ is used by some engineers. The two straight lines portions of the e-log$\bar{\sigma}_v$ curve are extended; this intersection defines another *most probable* preconsolidation pressure (point C of Fig 7.7). If you think about it, the maximum possible $\bar{\sigma}_p$ is at point D, the minimum possible $\bar{\sigma}_p$ is at point E, the intersection of the virgin compression curve with a horizontal line drawn form e_o.

(b) Some Emperical Methods

• **Nagaraj and Murthy (1985, 1986) modified by Bowles (1988) method**
For soils preconsolidated by overburden pressure:

$$\log \bar{\sigma}_p = \frac{5.97 - 5.32\, w_n}{LL - 0.25 \log \bar{\sigma}_{v_o}} \qquad 7.9$$

in the units of kPa.

Where,

w_n = natural moisture content

LL = liquid limit of the soil

The over-consolidation pressure (OCP) computed by this equation shows good agreement with values given by Worth (1979), but it should be used with caution.

For soils preconsolidated due to cementation and shrinkage:

$$\bar{\sigma}_p = 3.78 S_u - 2.9 \qquad 7.10$$

in units of KPa.

Where,

S_u = in-situ undrained shear strength as determined by the field vane shear test.

• **Skempton Method (1957)**
According to Skempton, the preconsolidation pressure, $\bar{\sigma}_p$ is computed using the following empirical relation:

$$\bar{\sigma}_p = \frac{q_u}{0.11 + 0.0037(PI)} \qquad 7.11$$

Where,

q_u= unconfined compression strength of the clay

PI = plasticity index of the clay.

7.6 FIELD CONSOLIDATION CURVE (CORRECTION FOR DISTURBANCE EFFECT)

Schmertmann (1955) developed a graphical procedure to correct for sample disturbance effect and to determine the correct field consolidation curve.

Fig. 7.8 represents this procedure for *NCC* and *OCC* respectively.

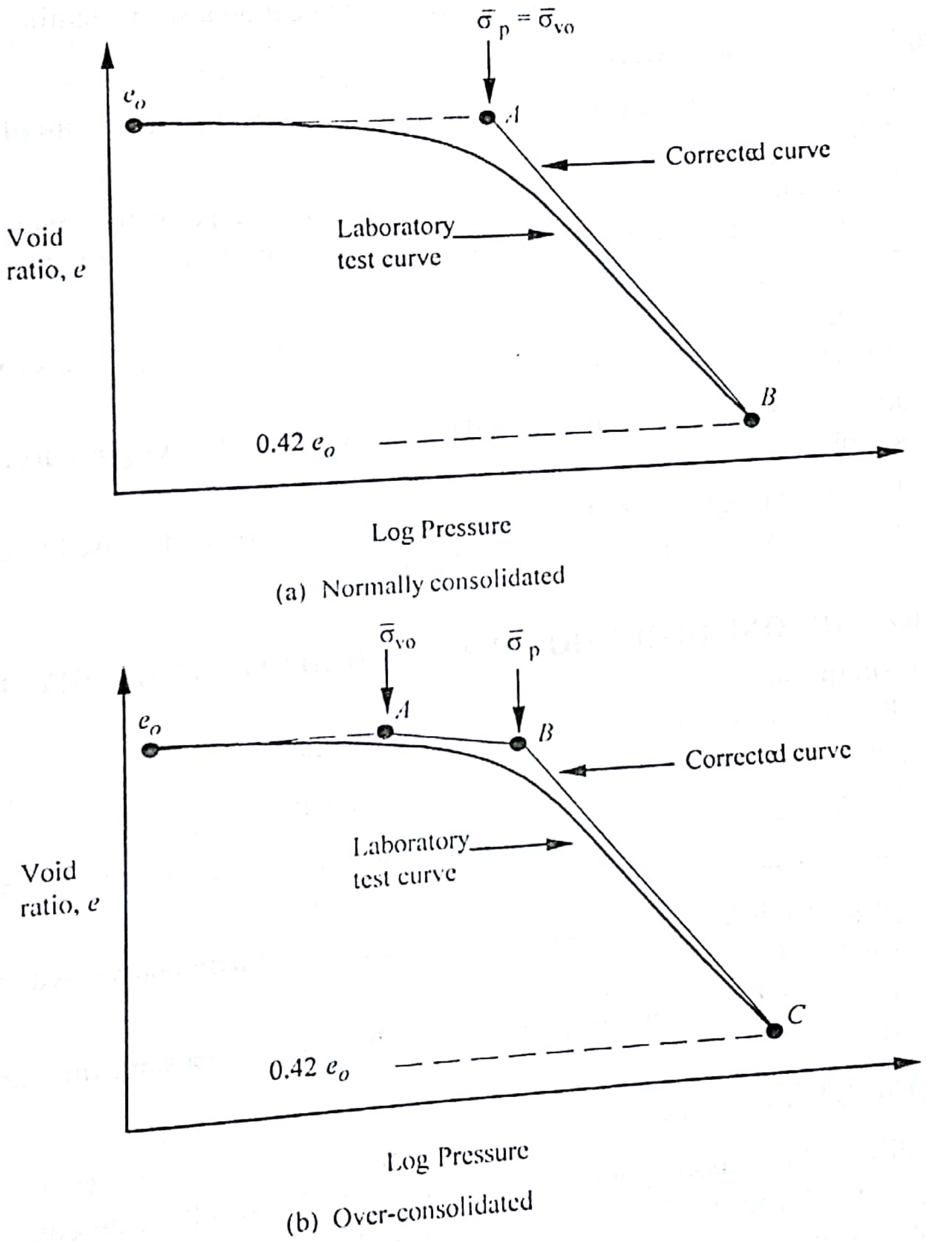


Figure 7.8 Correction for disturbance effects using e-logPressure curve: (a) For NCC (b) For OCC

(a) Normally Consolidated Clay

- For *NCC* locate point *A* at the intersection of the effective existing overburden pressure, $\bar{\sigma}_{vo}$ at the sample depth and the in-situ void ratio e_o.

- Locate point B on the virgin curve, or the extension of the virgin curve, where it crosses the e-log$\overline{\sigma}_v$ line for $e = 0.42\ e_o$
- Join points A and B located in steps above. This line will be the corrected field curve. The virgin curve is usually a straight line. For some sensitive soils, however, it may be a curved line (concave up) being some what steeper at the preconsolidation pressure than at higher pressures. If the laboratory plot has this type of curve, the corrected curve should be drawn with a similar shape.

(b) Pre-consolidated Clay

- For pre-consolidated clays, locate point A following the procedure of NCC as described above.
- From Point A draw a line parallel to the average slope of the rebound curve and locate a point B where this line intersects the pre-consolidation pressure $\overline{\sigma}_p$.
- Locate a point C on the virgin curve or its extension where it crosses the laboratory curve for $e = 0.42\ e_o$.
- Join points B and C which should form the corrected virgin curve for the sample.
- The line through B and A is the recompression curve, and is used to calculate the settlements for pressures smaller than $\overline{\sigma}_p$

7.7 TERZAGHI ONE DIMENSIONAL CONSOLIDATION THEORY (1925)

- **Assumptions**

(i) The soil is assumed to be homogeneous material.

(ii) Soil is fully saturated i.e. all voids are full of water with no air.

(iii) Water in the voids is assumed to be incompressible so that the change in volume is due to the change in volume of voids.

(iv) The sample is laterally confined, only vertical settlement and vertical drainage are allowed.

(v) The co-efficient of permeability k is assumed to be constant throughout the soil.

(vi) Darcy's law of permeability is valid.

In reality all these assumptions are not fully met. The results from consolidation study, however, reveal that large discrepancies between theory and nature are due to the pressure of large volume of air in the voids of the soil.

Detailed mathematical treatment of the Terzaghi's theory of one dimensional consolidation is beyond the scope of this book. Brief end results only are presented in this section. In the derivation, volume of water flowing in and out of a soil element is considered and the volume change of the element is considered to be equal to the difference of inflow and outflow in time dt.

Mathematically:

$$\frac{-k}{\gamma_w}\frac{\partial^2 u}{\partial z^2}dz\,dt \qquad 7.12$$

Where,

z = the depth of soil element. Partial differentials is used because the pore pressure, u is a function of both z and time, t.

The other part of the equation is obtained by relating the volume change (change in void ratio) to the change in effective stress by means of the co-efficient of compressibility, $a_v = \frac{\Delta e}{\Delta \sigma}$ obtained from the consolidation test.

Mathematically:

$$\frac{-a_v}{1+e_o}\cdot\frac{\partial u}{\partial t}dtdz \qquad 7.13$$

Equating equations 7.12 and 7.13

$$\frac{-k}{\gamma_\omega}\frac{\partial^2 u}{\partial z^2}dzdt = \frac{-a_v}{1+e_o}\frac{\partial u}{\partial t}dtdz \qquad \text{or}$$

$$C_v\frac{\partial^2 u}{\partial z^2} = \frac{\partial u}{\partial t} \qquad 7.14$$

Where,

C_v = co-efficient of consolidation $= \frac{k}{\gamma_w}\cdot\frac{1+e_o}{a_v}$

Equation 7.14 is the Terzaghi one-dimensional consolidation equation.

There are a variety of ways to solve this partial differential equation such as Hass's (1966) approximate solution, Taylor (1948) solution, Terzaghi (1925) solution etc.

Terzaghi presented the solution in terms of a Fourier series expansion as follows:

$$u = (\overline{\sigma}_2 - \overline{\sigma}_1)\sum_{n=0}^{\infty} f_1(Z)f_2(T) \qquad 7.15$$

Where Z and T are dimensionless parameters. Z is a geometry parameter, and it is equal to Z/H. T is called as the time factor and is given by

$$T = C_v\frac{t}{H^2} \qquad 7.16$$

Where,

t = time for a certain degree of consolidation

H = length of drainage path, for double drainage = $2H/2$, $2H$ being the total thickness of the layer.

The progress of consolidation after some time t at any depth Z in the consolidation layer is called compression ratio, U_Z or %age consolidation or degree of consolidation and is given by:

$$U_z = \frac{e_1 - e}{e_1 - e_2} = \frac{\bar{\sigma} - \bar{\sigma}_1}{\bar{\sigma}_2 - \bar{\sigma}_1} = \frac{u_i - u}{u_i} = 1 - \frac{u}{u_i} \qquad 7.17$$

(See Fig. 7.9)

Using equation 7.17, equation 7.15 is reduced to:

$$U_z = 1 - \sum_{n=0}^{\infty} f_1(Z) f_2(T) \qquad 7.18$$

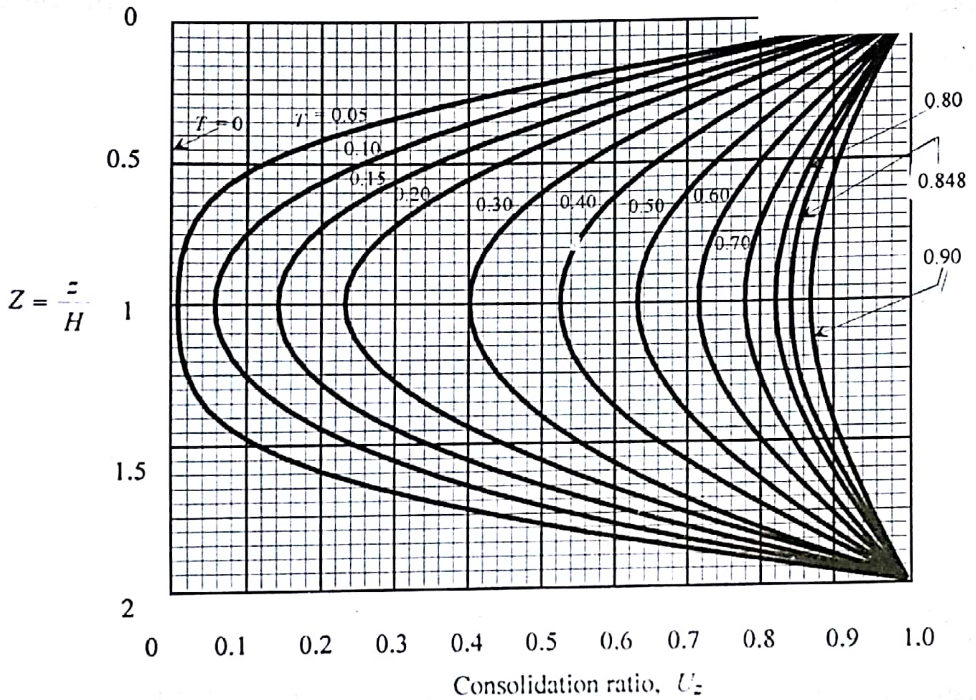


Figure 7.9 Consolidation for any location and time factor in a doubly drained layer (after Taylor, 1948)

The solution of this equation is given in Fig 7.9. The lines for constant T are called as isochrones or lines of constant time. These lines give %age consolidation for a given time factor (T) through the compressible layer.

CHAPTER-9

SETTLEMENT ANALYSIS

9.1 INTRODUCTION

Foundation designed using the knowledge of modern Soil Mechanics seldom undergo catastrophic failure due to inadequate bearing capacity as usually ample FOS. against shear is applied in computing the allowable baring capacity (ABC). On the other hand, settlement still remains to be the constant source of trouble for most of the civil engineering structures. Excessive settlement may cause structural as well as other damage, especially if such settlement occurs rapidly.

Foundation settlements must be estimated with great care for important buildings like bridges, high rise towers, power plants and other similar high cost structures. Settlement for structures such as embankments, earth dams, levees, braced sheeting and retaining walls can usually be estimated with a greater margin of error.

Settlement computations are only best estimates due to the following two reasons:

(1) Errors in obtaining the reliable values of soil parameters used in settlement calculations.

Problems of obtaining *truly undisturbed* soil samples means laboratory values are often in great error.

(2) Error in computing the reliable stress from the applied load:

This chapter will discuss how to obtain reasonable good estimates of settlement for various soil types (i.e. granular soils and/or clays).

9.2 SETTLEMENT

When a soil deposit is loaded, deformation will occur due to the change in stress. The total vertical downward deformation at the surface resulting from the load is called *settlement*. Similarly when the load is decreased (e.g. during excavation) the deformation may be vertically upward and is known as: *swelling*. Estimate of settlements and swellings are made using identical procedures.

Settlement Types

1. Types with respect to Permanency:

(i) Permanent settlement (or irreversible settlements)

This type of settlement is caused due to distortion brought about by sliding and rolling of particles under the action of the applied stresses. The sliding and rolling will reduce the voids resulting in reduction of volume of soil deposit. The increased stresses may also crush the soil particles while alter the material and produce some settlement. This type of settlement is permanent and undergo very insignificant recovery upon removal of load. Settlements due to consolidation (both primary and secondary) generally fall under this category.

(ii) Temporary Settlement
Settlement due to elastic compression of the soils and plastic flow are usually reversible and recover a major part on load release. Immediate settlement and plastic settlement due to lateral flow fall under this category. This settlement is generally small in soils.

(2) Types with respect to Mode of Occurrence

(i) Primary Consolidation Settlement (s_c)
These settlements are time-dependent or long-term settlements, completion time varies from 1–5 year or more. This is also known as primary consolidation i.e. the settlement caused due to expulsion of water from the pores of saturated fine-grained soils (clays). This type of settlement is predominant in saturated inorganic fine-grained soil (clays).

(ii) Secondary consolidation (s_s)
This is the consolidation under constant effective stress causing no drainage. This is very predominant in certain organic soils, but insignificant for inorganic soils. This is similar to creep in concrete.

(iii) Immediate Settlement (s_i)
This type of settlement is predominant in coarse-grained soils of high permeability and in unsaturated fine-grained soils of low permeability. The completion time for these is usually few days (say about 7 days). Usually this type of settlement is completed during the construction period and is called built in settlement. This is also known as short term settlement.

(iv) Settlement produced due to inadequate shear strength of the soil mass is caused due to bearing capacity failure of soil. Settlement due to lateral expulsion of soils from underneath the foundation is an example of this category. This type of settlement cannot be estimated using present knowledge of soil Mechanics but can be controlled easily by controlling bearing capacity. Fig. 9.1 represents different types of deformations in soils.

The total settlement (s_t) is given by:

$$s_t = s_i + s_c + s_s \qquad 9.1$$

(3) Type with respect to Uniformity

(i) Uniform Settlement
When all the points settle with equal amount, the settlement is known as uniform settlement. This type of settlement is possible only under relatively rigid foundation loaded with uniform pressure and resting on uniform soil deposit, which is a very rare possibility. This type of settlement may not endanger the structural stability but generally affects the utility of the structure by jamming doors/windows, damaging the utility lines (sewer, water supply mains etc.). Magnitude of total uniform settlement depends upon the type of structure (See Tables 9.1 and 9.2).

(ii) Differential Settlement

When different parts of the structure settle by different magnitude, the settlement is called differential settlement. This is very important as it may endanger the structural stability and may cause catastrophic failures. Tables 9.1 and 9.2 present the allowable values of tolerable differential settlement.

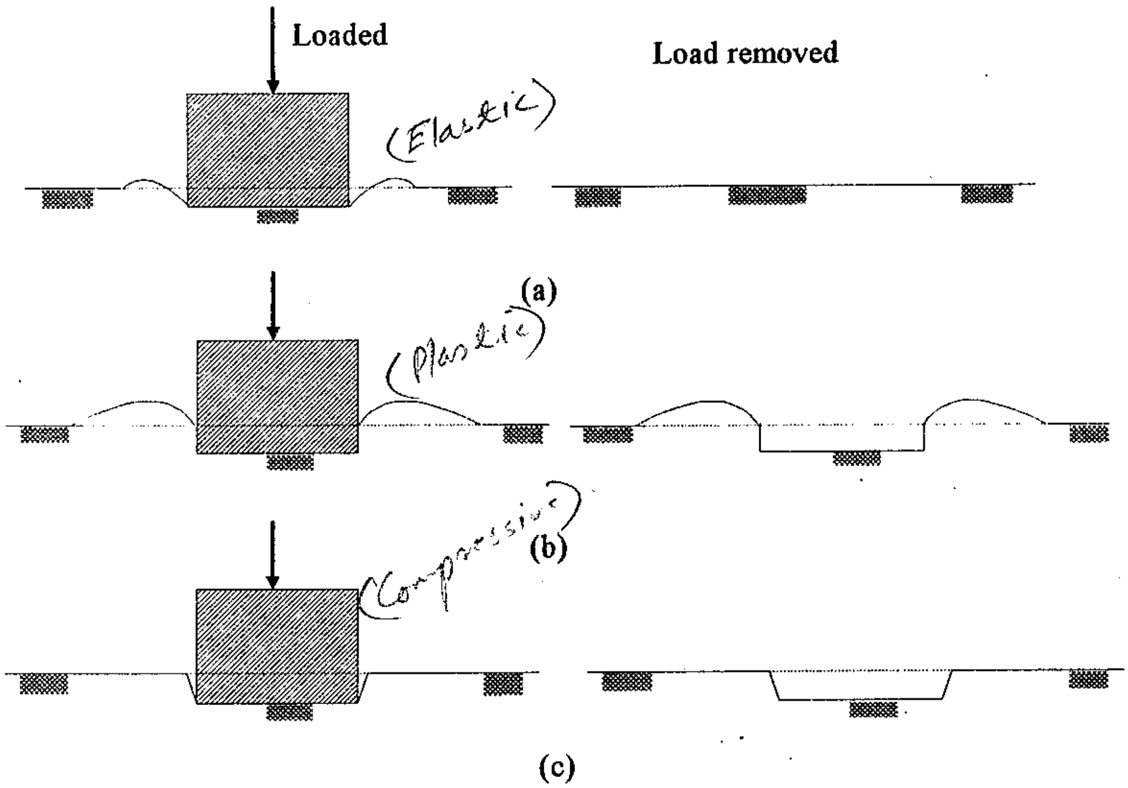


Figure 9.1 Deformation in soil: (a) Elastic (b) Plastic (c) Compressive

9.3 CAUSES OF SETTLEMENT AND REMEDIAL MEASURES

Causes

Following are the major causes of Settlement:

(1) Changes in stress due to:

(a) applied structural load or excavations,
(b) movement of ground water table,
(c) glacination; and
(d) vibrations due to machines, earthquake etc.

(2) Desication due to surface drying and/or plant life.

(3) Changes due to structure of soil (secondary compression)

(4) Adjacent excavation

(5) Mining subsidence

(6) Swelling and Shrinkage

To make intolerable settlements harmless, some times the following constructive measures are applied:

(i) Support the structures on statically determinate foundation system.

(ii) Design structure and their foundations as rigid, stereo-metric unit (for example silos, on continuous slab).

(iii) Divide long structures into small separate units.

(iv) Use three point support system and provide jacking provisions.

(v) Provide flexible connections to various service lines (utility lines).

10.4 CALCULATION OF MAGNITUDE OF SETTLEMENT

The students must refresh their knowledge of chapters-7 and 9 prior to attempt settlement analysis.

Consolidation Settlement (s_c)

Following are the steps for calculating total consolidation settlement:

(1) Sub-soil Profile and Sub-soil Parameters

Perform detailed geotechnical investigation (soil exploration) at the site proposed for the project, plot the subsoil profile, run field/laboratory tests on the representative soil samples recovered from the field and perform the following tests:

- Soil identification & Classification Tests.
  Grain size analysis, plasticity index, moisture content, and specific gravity tests.
- One Dimensional Consolidation Test.

(2) Data Reduction

(i) Analyze the identification and classification tests and classify the soils in accordance with USCS. From these tests results, compute the following properties for use in preliminary settlement analysis:

- Void ratio, $e = \dfrac{wG_s}{S}$
- Compression index, $C_c = 0.009(LL\text{-}10)$
- Liquidity index, $LI = \dfrac{w_n - PL}{LL - PL}$

(ii) Analyze the consolidation test to differentiate whether the compressible soil stratum is normally or pre-consolidated and obtain the following:

- Magnitude of pre-consolidation pressure, you may use Casagrande's method for this purpose.
- Calculate the values of C_c and C_r from plots of log of pressure $\bar{\sigma}$ vs. void ratio (e).
- Correct the $\log\bar{\sigma}$ vs e graph using Schmertmann's method (1955) and plot field curve.
- Calculate for each load increment, C_v using Casagrande's log t vs. compression dial reading (DR) or Taylor's $\sqrt{t}$ vs DR method.