9. COMPRESSIBILITY AND SETTLEMENT

9.1 COMPRESSIBILITY OF AN ELASTIC SOLID

Structures such as buildings, bridges, etc. are frequently erected on soil as the founding material. Since soil is a compressible material these structures experience downward movement or settlement. Depending on the soil and the size of the structure these settlements may vary from negligible amounts to several metres in extreme cases. To facilitate adequate design of the structure it is essential to be able to predict the settlement that the structure will experience.

The settlement produced by the application of a stress to the surface of a compressible material depends upon the rigidity of the material and the boundary conditions prevailing. These effects may be examined quantitatively by consideration of the settlement of solid which behaves according to the theory of elasticity. (Timoshenko and Goodier, 1951). In some areas of foundation engineering it is often assumed that soils and rocks behave as elastic solids, particularly when the applied stresses are considerably less than failure values. This assumption is less appropriate with some soils such as soft clays and loose sandy soils.

In the brief discussion that follows the behaviour of an elastic solid can be described by two parameters - the Young’s modulus (E) and the Poisson’s ratio (v). Fig. 9.1(a) represents a laterally unconfined elastic solid of rectangular cross section resting on the surface of a rigid frictionless base. A vertical stress \( \sigma_v \) is applied to the top of the solid, which experiences settlement as a result.

The applied stress conditions for this case are:

\[
\sigma_v = \sigma_1 \text{ and } \sigma_2 = \sigma_3 = 0
\]

where

\( \sigma_1 = \) major principal stress
\( \sigma_2 = \) intermediate principal stress
\( \sigma_3 = \) minor principal stress

compressives stresses and strains being positive

For these conditions the settlement (\( \rho \)) may be calculated from the vertical strain \( \varepsilon_v \) (\( = \varepsilon_1 \))

\[
\rho = \varepsilon_v \ L = (\sigma_1/E) \ L \quad (9.1)
\]

Fig. 9.1(b) represents a laterally confined elastic solid, which is subjected to a vertical stress \( \sigma_v \). As a result of the lateral confinement all horizontal movement is prevented. The confined
boundaries of the solid will be considered frictionless so that vertical movement of the solid is not restricted. This is a case of one dimensional compression in which movement occurs in one direction only.

The stress conditions for this case are

\[ \sigma_v = \sigma_1 \text{ and } \sigma_2 = \sigma_3 \neq 0 \]

and because the horizontal strain is zero

\[ \epsilon_2 = \epsilon_3 = 0 = \frac{1}{E} (\sigma_2 - \nu \sigma_1 - \nu \sigma_2) \]

\[ \therefore \sigma_2 (1-\nu) = \nu \sigma_1 \]

or \[ \frac{\sigma_2}{\sigma_1} = \frac{\nu}{1-\nu} \left( = K_0 \right) \]

(9.2)

also

\[ \epsilon_1 = \frac{1}{E} (\sigma_1 - 2\nu \sigma_2) \]

\[ = \frac{1}{E} (\sigma_1 - 2\nu \cdot \frac{\nu \sigma_1}{1-\nu}) \]

\[ = \frac{\sigma_1}{E} \left( \frac{1-\nu - 2\nu^2}{1-\nu} \right) \]

\[ = \frac{\sigma_1}{E} \left( \frac{(1-2\nu)(1+\nu)}{(1-\nu)} \right) \]

\[ \therefore \frac{\sigma_1}{\epsilon_1} = \frac{E (1-\nu)}{(1-2\nu)(1+\nu)} \]

(9.3)

Equation (9.3) shows that the ratio of vertical stress to vertical strain for the laterally confined case is not equal to the Young’s modulus. The appropriate modulus for this case (equation 9.3) is sometimes referred to as the constrained or dilatational modulus. The settlement or vertical deflection (\( \rho \)) may be calculated from the vertical strain (\( \epsilon_1 \))

\[ \rho = \epsilon_1 L = \sigma_1 L \frac{(1-2\nu)(1+\nu)}{E(1-\nu)} \]

(9.4)

A wide variety of boundary conditions may be imposed on a solid in addition to the two cases that have just been examined. For example the solid may be partially confined with horizontal strain prevented in one direction but no confinement whatever in the other horizontal direction. The stress and strain situation for this case may be expressed as follows:
Fig. 9.1 Compression of an Elastic Solid

Fig. 9.2 Influence of Boundary Conditions on Settlement of an Elastic Solid
\( \sigma_y = \sigma_1, \sigma_2 \neq 0, \sigma_3 = 0 \)

and

\( \varepsilon_2 = 0, \varepsilon_3 \neq 0 \)

For this case it may be shown that

\[
\frac{\sigma_1}{\varepsilon_1} = \frac{E}{(1-\nu^2)}
\]

(9.5)

and the settlement \( \rho \) becomes

\[
\rho = \varepsilon_1 L = \sigma_1 L \frac{(1-\nu^2)}{E}
\]

(9.6)

The settlements calculated from equations (9.4), (9.6) and (9.1) for the laterally confined, partly confined and unconfined boundary conditions respectively are compared in Fig. 9.2. This plot illustrates the importance of boundary conditions in affecting settlement particularly for large values of Poisson’s ratio. The figure shows that greater settlement occurs for the smaller amount of lateral constraint provided. The lateral constraint provided by the lateral stresses is illustrated in Fig. 9.3 which shows that the lateral stress, \( \sigma_2 \) increases as the degree of confinement increases.

Examination of equations (9.1), (9.4) and (9.6) indicates that the general expression for calculation of settlement \( \rho \) is:

\[
\rho = \text{strain} \times \text{stressed length}
\]

\[
= \frac{\text{stress change} \times \text{stressed length}}{\text{modulus}}
\]

where the modulus depends upon the boundary conditions as follows

- unconfined modulus = \( E \) (Young’s modulus)
- partly confined modulus = \( E/(1-\nu^2) \)
- confined modulus = \( \frac{E(1-\nu)}{(1+\nu)(1-2\nu)} \)

Alternatively the inverse of the modulus (compressibility) may be used as follows:

\[
\rho = \text{stress change} \times \text{stressed length} \times \text{compressibility}
\]

where

- unconfined compressibility = \( \frac{1}{E} \)
- partly confined compressibility = \( (1-\nu^2)/E \)
confined compressibility = \frac{(1+\nu) (1-2\nu)}{(1-\nu) E}

The confined (one dimensional) compressibility is also referred to as the coefficient of volume compressibility or the coefficient of volume decrease and the symbol \( m_v \) is widely used to indicate the value of this compressibility.

### 9.2 ELASTIC SETTLEMENT OF FOOTINGS

In cases where a loaded area such as a footing for a building, is located on a soil deposit, which may be idealized as an elastic solid, the settlement caused by the load may be calculated by means of the elastic displacement equation.

\[
\rho = \frac{q B (1-\nu^2) I_\rho}{E}
\]  

(9.7)

where  
\( \rho \) = settlement of the footing  
\( q \) = average pressure applied  
\( B \) = width or diameter of footing  
\( \nu \) = Poisson’s ratio of the soil  
\( E \) = Young’s Modulus of the soil  
\( I_\rho \) = approximate influence coefficient for settlement

The influence coefficient \((I_\rho)\) depends on a number of parameters including footing shape, footing flexibility, distance to a rigid base and footing embedment depth. For example in Fig. 9.4, Das (1984) provides \( I_\rho \) values for a variety of situations. The Young’s modulus \((E)\) of the soil should be determined by appropriate laboratory or field tests. In the absence of such test data Table 9.1 may be used as a rough guide.

<table>
<thead>
<tr>
<th>Material</th>
<th>Young’s Modulus (E) - MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rock</td>
<td>2,000 - 20,000</td>
</tr>
<tr>
<td>Weathered rock</td>
<td>200 - 5,000</td>
</tr>
<tr>
<td>Dense sand and gravel</td>
<td>50 - 1,000</td>
</tr>
<tr>
<td>Firm clay</td>
<td>5 - 50</td>
</tr>
<tr>
<td>Soft clay</td>
<td>0.5 - 5</td>
</tr>
</tbody>
</table>
Fig. 9.3 Influence of Boundary Conditions on the Intermediate Principal Stress

Fig. 9.4 Influence Factors for Settlement of Footings on the Surface of a Semi-Infinite Elastic Solid (after Das, 1984)
For elastic settlement of embedded flexible footings on saturated clay, Janbu, Bjerrum and Kjaernsli (1956) proposed the following expression for the evaluation of average settlement

\[ \rho = \mu_0 \mu_1 q B/E \]  

(9.8)

where \( \mu_0 \) and \( \mu_1 \) are dimensionless parameters which describe the effect of embedment depth and the effect of depth of the compressible layer respectively. The plots for the parameters \( \mu_0 \) and \( \mu_1 \) originally presented by Janbu, Bjerrum and Kjaernsli have been improved by Christian and Carrier (1978) and the improved chart is given in Fig. 9.5.

9.3 COMPRESSIBILITY OF A REAL SOIL

The solutions of many soil mechanics problems would be greatly simplified if soils behaved like elastic solids. The assumption of elastic behaviour may be reasonable for some soils but this is not so in general. It is found that the compressibility \( (m_v) \) of an apparently uniform soil deposit is generally not constant but decreases with increasing depth below the ground surface because of the increasing degree of confinement of the soil.

Fig. 9.6 represents three soils, each having a different value of compressibility and enclosed within rigid but frictionless boundaries. The settlement \( (\rho) \) of the top of the soil as a result of the imposition of the vertical stress, \( \Delta \sigma_v \) is found by summing the contributions of each of the three soils.

\[ \rho = \Sigma_{\text{stress change} \times \text{stressed length} \times \text{compressibility}} \]

\[ = \Delta \sigma_v (z_A x m_{vA} + z_B x m_{vB} + z_C x m_{vC}) \]  

(9.9)

A further effect, which has hitherto been ignored now needs to be taken into account. This is the effect of stress level on the compressibility which is illustrated in Fig. 9.7. Let \( \rho_A, \rho_B \) and \( \rho_C \) represent the densities of the three soils A, B and C respectively (Fig. 9.6). The initial vertical stresses at the mid depths of each of the three soils are

\[ \sigma_{iA} = \rho_A g \frac{z_A}{2} \]

\[ \sigma_{iB} = \rho_A g z_A + \rho_B g \frac{z_B}{2} \]

\[ \sigma_{iC} = \rho_A g z_A + \rho_B g z_B + \rho_C g \frac{z_C}{2} \]

The final vertical stresses at the mid depths of each of the three soils are

\[ \sigma_{fA} = \sigma_{iA} + \Delta \sigma_v \]
Fig. 9.5 Values of $\mu_0$ and $\mu_1$ for Elastic Settlement on Saturated Clay (after Christian & Carrier, 1978)

Fig. 9.6 Compression of Soil Layers
\[ \sigma_{fB} = \sigma_{iB} + \Delta \sigma_v \]
\[ \sigma_{fC} = \sigma_{iC} + \Delta \sigma_v \]

In most settlement problems the initial and final stresses described above are effective stresses and not total stresses. Effective and total stresses are the same when the pore water pressure is zero. In the case of saturated soils it should be remembered that compression (ie. settlement) occurs only as a result of a change in effective stress and not purely a change in total stress.

The determination of the appropriate compressibility for soil B is illustrated in Fig. 9.7. Since the compressibilities at the initial stress level \( \sigma_{iB} \) and the final stress level \( \sigma_{fB} \) are not equal, the compressibility \( m_{vB} \) for use in equation (9.9) is determined at the average stress level \( \sigma_{av} \) where

\[ \sigma_{av} = \frac{\sigma_{iB} + \sigma_{fB}}{2} \]
\[ = \sigma_{iB} + \frac{\Delta \sigma_v}{2} \quad (9.10) \]

A similar procedure is used for soils A and C to obtain the compressibilities \( m_{vA} \) and \( m_{vC} \). Equation (9.9) can then be used to find the total settlement.

**EXAMPLE**

Fig. 9.8(a) represents a layer of compressible clay sandwiched between relatively incompressible sand deposits. Determine the settlement of the ground surface if a load of 50kN/m\(^2\) is placed over a large area of the ground. The compressibility of the clay is given by Fig. 9.8(b). The densities of the dry sand, saturated sand and clay are 2000kg/m\(^3\), 2200kg/m\(^3\) and 1600kg/m\(^3\) respectively.

In order to determine the relevant compressibility of the clay it is necessary to find the average stress as in equation (9.10). It is assumed that the capillary rise in the sand is zero so that the sand above the water table is dry and that below the water table is saturated. The initial vertical effective stress, \( \sigma'_i \) at the mid depth of the clay layer is

\[ \sigma'_i = 2000 \times 9.81 \times 3 + 1200 \times 9.81 \times 2 + 600 \times 9.81 \times 1 \text{ N/m}^2 \]
\[ = 88.29 \text{ kN/m}^2 \]
Since the surface load ($\Delta \sigma_v$) is placed over a large area, the clay layer will experience a stress increase equal to $\Delta \sigma_v$. Hence the average effective stress ($\sigma'_{av}$) at the mid depth of the clay layer is, from equation (9.10)

\[
\sigma'_{av} = \sigma_i + \frac{\Delta \sigma_v}{2}
\]

\[
= 88.29 + \frac{50}{2}
\]

\[
= 113.29 \text{ kN/m}^2
\]

From Fig. 9.8(b) the corresponding compressibility is .0005m$^2$/kN. The settlement ($\rho$) of the ground surface can now be calculated as follows

\[
\rho = \text{compressibility} \times \text{stress change} \times \text{stressed length}
\]

\[
= 0.0005 \times 50 \times 2
\]

\[
= 0.05 \text{m}
\]
9.4 THE OEDOMETER

The compressibility of a soil is often measured in a laboratory device known as an oedometer or consolidometer. Fig. 9.9 shows a cross sectional outline of an oedometer in which the cylindrical soil sample is confined inside a ring in order to prevent lateral strain. Porous stones are placed on both sides of the soil to permit escape of water. The vertical load is applied to the soil in one of a variety of ways such as by application of weights to a hanger, by means of weights applied through a lever system to the top of the soil or by means of air pressure applied to a piston. The amount of vertical compression experienced by the soil as a result of the application of load is measured by means of a dial gauge or a displacement transducer. The conventional
testing technique, which is described in most books on soil testing, consists of applying successive increments of load and observing the deflection after each increment until the movement ceases. In a saturated sample of soil the application of the vertical load results in the development of a pore pressure (equal to the vertical stress applied) within the soil. This pore pressure gradually dissipates as water is expelled from the soil through the porous stones. Movement of the soil continues until the pore pressure has fully dissipated. Typical time-deflection plots for a clay soil are illustrated in Fig. 9.10. This figure shows soil deflection continuing until approx. 24 hr. Valuable information relating to prediction of rate of settlement of structures may be extracted from data such as that shown in Fig. 9.10 and this matter will be explored in Chapter 10.

The results obtained from an oedometer test may be presented as shown in Fig. 9.11 which shows the vertical strain ($\varepsilon_1$) at the end of each load increment plotted against the vertical effective stress ($\sigma'_v$). Clearly the slope of the resulting curve is the one dimensional or confined compressibility ($m_v$) and as illustrated in the figure the magnitude of $m_v$ decreases as the vertical effective stress increases.

![Fig. 9.9 Cross Section of an Oedometer](image)
Fig. 9.10 Typical Time-Deflection Plot in an Oedometer Test

Fig. 9.11 Stress - Strain Curve from an Oedometer Test
Fig. 9.12  Phase Diagrams for a Loaded Soil

Fig. 9.13  Void Ratio-Stress Plot for Oedometer Data
An alternative method of presenting the data from an oedometer test involves the use of the void ratio of the soil. This may be demonstrated by means of phase diagrams as shown in Fig. 9.12. As a result of the application of a vertical stress ($\Delta \sigma'_v$) the voids decrease in volume by an amount of $\Delta V_v$. If $e_i$ and $e_f$ represent the initial and final void ratios respectively then the change in void ratio ($\Delta e$) as a result of the application of stress is given by

$$\Delta e = e_i - e_f = \frac{V_v}{V_s} - \frac{V_v - \Delta V_v}{V_s} = \frac{\Delta V_v}{V_s}$$

Since lateral strain is prevented in the oedometer test the changes in vertical and volumetric strains will be identical.

$$\therefore \Delta \varepsilon_1 = \frac{\Delta V_v}{V_s + V_v} = \frac{\Delta V_v}{V_s} \cdot \frac{V_s}{V_s + V_v} = \frac{\Delta e}{1 + e_i} \quad (9.11)$$

where $\Delta \varepsilon_1$ is the change in vertical strain.

Equation (9.11) demonstrates that the two methods of presenting oedometer data; one in terms of vertical strain change and the other in terms of changes in void ratio are equivalent.

The compressibility ($m_v$) may also be related to void ratio change as follows

$$m_v = \frac{\Delta \varepsilon_1}{\Delta \sigma'_v} = \frac{\Delta e}{(1+e_i)\Delta \sigma'_v} = \frac{\Delta n}{\Delta \sigma'_v} \quad (9.12)$$

where $\Delta n$ indicates the change in porosity as a result of the application of stress $\Delta \sigma'_v$.

The conventional method of plotting oedometer data using void ratios involves the use of a logarithmic scale for the stress as shown in Fig. 9.13. The plotted line for the first loading of the soil is often linear so the equation of the line can be expressed simply as follows

$$e_f = e_i - C_c \log_{10} \left( \frac{\sigma'_v + \Delta \sigma'_v}{\sigma'_v} \right) \quad (5.13)$$

where the slope of the line, $C_c$, known as the compression index is an alternative measure of compressibility of the soil. The $\sigma'_v$ in equation (9.13) is the initial value of the effective vertical stress.

In the absence of test data various empirical expressions have been suggested for the estimation of the compression index ($C_c$). Rendon - Herrero (1980) has summarised a number of these expressions which are shown in Table 9.2.
TABLE 9.2

EXPRESSIONS FOR COMPRESSION INDEX

<table>
<thead>
<tr>
<th>Equation</th>
<th>Reference</th>
<th>Region of applicability</th>
</tr>
</thead>
<tbody>
<tr>
<td>$C_c = 0.007 \ (s_o - 7)$</td>
<td>Skempton</td>
<td>Remolded clays</td>
</tr>
<tr>
<td>$C_c = 0.01 \ w_s$</td>
<td></td>
<td>Chicago clays</td>
</tr>
<tr>
<td>$C_c = 1.15 \ (e_o - 0.35)$</td>
<td>Nishida</td>
<td>All clays</td>
</tr>
<tr>
<td>$C_c = 0.30 \ (e_o - 0.27)$</td>
<td>Hough</td>
<td>Inorganic cohesive soil; silt, silty clay, clay</td>
</tr>
<tr>
<td>$C_c = 0.0115 \ w_N$</td>
<td></td>
<td>Organic soils, peats, organic silt and clay</td>
</tr>
<tr>
<td>$C_c = 0.0046 \ (w_i - 9)$</td>
<td></td>
<td>Brazilian clays</td>
</tr>
<tr>
<td>$C_c = 0.009 \ (w_i - 10)$</td>
<td>Terzaghi and Peck</td>
<td>Normally consolidated clays</td>
</tr>
<tr>
<td>$C_c = 0.75 \ (e_o - 0.50)$</td>
<td></td>
<td>Soils with low plasticity</td>
</tr>
<tr>
<td>$C_c = 0.208 \ e_o + 0.0083$</td>
<td></td>
<td>Chicago clays</td>
</tr>
<tr>
<td>$C_c = 0.156 \ e_o + 0.0107$</td>
<td></td>
<td>All clays</td>
</tr>
</tbody>
</table>

Note: $e_o$ = in situ void ratio, $w_s$ = in situ water content; and $w_i$ = liquid limit

The settlement ($\rho$) may be calculated by means of the compression index by use of the following expression

$$\rho = \text{layer thickness} \times \text{strain}$$

$$= \text{thickness} \times \frac{\Delta e}{1 + e_i}$$

$$= \text{thickness} \times C_c \log_{10} \left( \frac{\sigma_{v}' + \Delta \sigma_{v}'}{\sigma_{v}'} \right) / (1 + e_i)$$

(9.14)

(9.15)

The implicit assumption in the use of equation (9.15) for the calculation of settlement of a structure is that the compression index obtained by means of a laboratory oedometer test on an undisturbed sample of the soil, gives an accurate representation of the behaviour of the soil in the field when loaded by the structure. However, it is found that most soils are in some degree sensitive to disturbance which may occur during field sampling or during laboratory preparation (Rutledge, 1944). The void ratio - stress plots from oedometer tests vary with the degree of
disturbance of the soil sample and values of the compression index have been found to decrease as the degree of sample disturbance increases.

**EXAMPLE**

Determine the settlement for the situation depicted in Fig. 9.8(a) if the compression index and the specific gravity for the clay are 0.3 and 2.70 respectively.

From the saturated density of the clay of 1600kg/m$^3$ given in the example in section 9.3 it may be shown that the initial void ratio is 1.84. In this same example the initial vertical effective stress at the mid depth of the clay layer has been determined

\[
\sigma'_v = 88.29\text{kN/m}^2
\]

The settlement may now be found by substitution into equation (9.15)

\[
\rho = 2.0 \times 0.3 \times \frac{\log_{10} \left( \frac{(88.29+50)}{88.29} \right)}{1 + 1.84}
= 0.041\text{m}
\]

**9.5 NORMALLY CONSOLIDATED AND OVER-CONSOLIDATED SOILS**

In Fig. 9.14(a) P represents a point in a soil deposit which is being increased in thickness by the gradual deposition of further soil over a long period of time. As the deposit becomes thicker the vertical stress on the soil at point P increases. The compression of the soil (decrease in void ratio) as a result of this stress increase, is represented in Fig. 9.15 by line AB. At point B the element of soil P is located a distance (z) below the ground surface and is subjected to a vertical effective stress of $\sigma'_m$. Because the soil has experienced a stress no greater than $\sigma'_m$ in its previous history, the soil is referred to as being normally consolidated.

At this stage, further deposition of soil ceases and erosion begins, resulting in the gradual removal of some of the soil previously deposited. Suppose that up to the present time a depth $z_o$ of soil has been removed by erosion so that the element of soil P is now a distance of $(z-z_o)$ below the ground surface as shown in Fig. 9.14(b). The vertical effective stress at P due to the reduced depth of overburden is $\sigma'_v$. Because of the decrease in stress from $\sigma'_m$ to $\sigma'_v$ the soil will experience a rebound (increase in void ratio) as indicated by line BC in Fig. 9.15. The soil at point P in Fig. 9.14(b), which is represented by point C in Fig. 9.15 is now referred to as overconsolidated since it has previously experienced a greater stress than that which exists at the present time ($\sigma'_v < \sigma'_m$). The ratio of $\sigma'_m$ to $\sigma'_v$ is referred to as the overconsolidation ratio (OCR).
Further deposition of soil would produce further compression of the soil at point P as indicated by line CD in Fig. 9.15.

Fig. 9.14 Representation of Soil Removal by Erosion

Fig. 9.15 Void Ratio - Stress Diagram for Loading and Unloading of a Soil
If a field sample of the normally consolidated soil (represented by point B in Fig. 9.15) was obtained for a laboratory oedometer test, the rebound of the soil as a result of the removal of the field stresses may also be represented by a line such as BC in Fig. 9.15. The laboratory void ratio - stress curve would follow a path such as line CD in Fig. 9.15. This means that the true field value of the compression index would be given by the slope of the high stress portion of the curve near point D and not by the initial slope of the curve near point C. The value of the compression index \( (C_c) \) is normally defined as the slope of the virgin compression part of the \( e - \log(\sigma'_v) \) curve, that is, the slope ABD in Fig. 9.15.

The slope of the line BC in Fig. 9.15 is referred to as the swell index \( (C_s) \) and is normally evaluated by means of laboratory tests. In most cases the ratio of the compression index \( (C_c) \) to the swell index \( (C_s) \) is within the range of 5 to 10.

The preceding comments indicate the desirability of possessing a technique with which to demonstrate whether a soil is overconsolidated or normally consolidated. The vertical effective stress \( (\sigma'_v) \) for a soil located a known depth below the ground surface may be calculated as previously discussed. A technique is needed to enable the maximum previous vertical effective stress \( (\sigma'_m) \) that the sample may have experienced in the past, to be estimated. An approximate technique which is widely used has been suggested by Casagrande (1936) and is illustrated in Fig. 9.16.

The technique involves the use of a conventional oedometer test result which has been plotted as line ABCD in Fig. 9.16. The point of maximum curvature, point B is estimated. The horizontal line BE and the line BF which is tangential to the curve are drawn. Line BG bisects the angle EBF. The straight line portion CD of the curve is projected upwards to intersect line BG at point H. The abscissa of point H provides an estimate of value of the maximum previous value of the effective vertical stress \( (\sigma'_m) \). The soil is normally consolidated or overconsolidated depending upon whether the present field value of \( \sigma'_v \) is equal to \( \sigma'_m \) or less than \( \sigma'_m \) respectively.

For the calculation of settlement for one dimensional conditions (that is, no horizontal strain), equation (9.14) may be used with any soil - whether normally consolidated or overconsolidated. However, if the compression index \( (C_c) \) is interpreted to be the slope of the high stress portion of the void ratio - log stress curve (this is the usual interpretation) then equation (9.15), or any equation involving \( (C_c) \) may be used only for normally consolidated soils. A discussion of settlement calculation methods applicable to the more commonly encountered field situations, is presented in Geomechanics 2.
Fig. 9.16  Casagrande Construction to Find $\sigma'_m$

Fig. 9.17
EXAMPLE

Using the 2:1 stress transmission, estimate the settlement of the surface strip footing shown in Fig. 9.17. The compressibilities for the three soils are as follows:

Soil A \( m_v = 0.001 \) m\(^2\)/kN  
Soil B \( m_v = 0.0004 \) m\(^2\)/kN  
Soil C \( m_v = 0.0002 \) m\(^2\)/kN

The calculations will be carried out for the mid depth of each layer using equations (9.9) and (3.9).

\[
\begin{array}{cccc}
 z(m) & z/B & (1+z/B) & \Delta \sigma_z (kN/m^2) \\
 Soil A & 0.5 & 0.25 & 1.25 & 80.0 \\
 Soil B & 2.0 & 1.00 & 2.00 & 50.0 \\
 Soil C & 4.0 & 2.00 & 3.00 & 33.3 \\
\end{array}
\]

The settlement \( \rho \) is calculated as follows:

\[
\rho = \sum (m_v \times \Delta \sigma_z \times \text{layer thickness}) \\
= 0.001 \times 80 \times 1.0 + 0.0004 \times 50.0 \times 2.0 \\
+ 0.0002 \times 33.3 \times 2.0 \\
= 0.080 + 0.040 + 0.013 \\
= 0.133 \text{m}.
\]
REFERENCES


