Module 4
Lecture 14

SHALLOW FOUNDATIONS: ALLOWABLE BEARING CAPACITY AND SETTLEMENT

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SETTLEMENT CALCULATION

ELASTIC SETTLEMENT BASED ON THE THEORY OF ELASTICITY

The elastic settlement of a shallow foundation can be estimated by using the theory of elasticity. Referring to figure 4.16 and using Hooke’s law,

\[ S_e = \int_0^H \varepsilon_z \, dz = \frac{1}{E_s} \int_0^H (\Delta p_z - \mu_s \Delta p_x - \mu_s \Delta p_y) \, dz \]  \[ 4.27 \]

Where

- \( S_e \) = elastic settlement
- \( E_s \) = modulus of elasticity of soil
- \( H \) = thickness of the soil layer
- \( \mu_s \) = Poisson’s ratio of the soil
- \( \Delta p_x, \Delta p_y, \Delta p_z \) = stress increase due to the net applied foundation load in the x, y, and z, directions, respectively
Theoretically, if the depth of foundation $D_f = 0, H = \infty$, and the foundation is perfectly flexible, according to Harr (1966) the settlement may be expressed as (figure 4.17)

$$S_e = \frac{Bq_o}{E_s} (1 - \mu_s^2) \frac{\alpha}{2}$$  \hspace{1cm} \text{(corner of the flexible foundation)} \hspace{1cm} [4.28]$$

$$S_e = \frac{Bq_o}{E_s} (1 - \mu_s^2)\alpha$$  \hspace{1cm} \text{(center of the flexible foundation)} \hspace{1cm} [4.29]$$

Where

$$\alpha = \frac{1}{\pi} \left[ \ln \left( \frac{1 + m_1^2 + m_1}{1 + m_1^2 - m_1} \right) + m \ln \left( \frac{1 + m_1^2 + 1}{1 + m_1^2 - 1} \right) \right]$$ [4.30]$$

$$m_1 = \frac{L}{B}$$ [4.31]$$

B = \text{width of foundation}$$

$L = \text{length of foundation}$

The values of $\alpha$ for various length-to-width ($L/B$) ratios are shown in figure 4.18. The average immediate settlement for a flexible foundation also may be expressed as
Figure 4.18 Values of $\alpha$, $\alpha_{av}$, and $\alpha_r$—equations (28, 29, 32 and 32a)

$$S_e = \frac{Bq_o}{E_s} (1 - \mu_s^2)\alpha_{av} = \text{average for flexible foundation} \quad [4.32]$$

Figure 18 also shows the values of $\alpha_{av}$ for various $L/B$ ratios of foundation.

However, if the foundation shown in figure 17 is rigid, the immediate settlement will be different and may be expressed as

$$S_e = \frac{Bq_o}{E_s} (1 - \mu_s^2)\alpha_r = \text{(rigid foundation)} \quad [4.32a]$$

The value of $\alpha_r$ for various $L/B$ ratios of foundation are shown in figure 4.18.

If $D_f = 0$ and $H < \infty$ due to the presence of a rigid (incompressible) layer as shown in figure 17,

$$S_e = \frac{Bq_o}{E_s} (1 - \mu_s^2)^2 \left[\frac{(1-\mu_s^2)F_1 + (1-\mu_s-2\mu_s^2)F_2}{2}\right] \quad \text{corner of flexible foundation} \quad [4.33a]$$

And

$$S_e = \frac{Bq_o}{E_s} (1 - \mu_s^2)[(1 - \mu_s^2)F_1 + (1 - \mu_s - 2\mu_s^2)F_2]\text{(corner of flexible foundation)} \quad [4.33b]$$

The variation of $F_1$ and $F_2$ with $H/B$ are given in figures 4.19 and 4.20, respectively (Steinbrenner, 1934).
Figure 4.19 Variation of $F_1$ with $H/B$ (based on Steinbrener, 1934)

Figure 4.20 Variation of $F_2$ with $H/B$ (based on Steinbrenner, 1934)
It is also important to realize that the preceding relationships for $S_e$ assume that the depth of the foundation is equal to zero. For $D_f > 0$, the magnitude of $S_e$ will decrease.

Example 5

A foundation is $1 \text{ m} \times 2 \text{ m}$ in plan and carries a net load per unit area, $q_o = 150 \text{ kN/m}^2$. Given, for the soil, $E_s = 10,000 \text{ kN/m}^2$; $\mu_s = 0.3$. Assuming the foundation to be flexible, estimate the elastic settlement at the center of the foundation for the following conditions:

a. $D_f = 0$; $H = \infty$

b. $D_f = 0$; $H = 5 \text{ m}$

Solution

Part a

From equation (29)

$$S_e = \frac{Bq_o}{E_s} (1 - \mu_s^2)\alpha$$

For $L/B = 2/1 = 2$, from figure 18, $\alpha \approx 1.53$, so

$$S_e = \frac{(1)(150)}{10,000} (1 - 0.3^2)(1.53) = 0.0209 \text{ m} = 20.9 \text{ mm}$$

Part b

From equation (33b)

$$S_e = \frac{Bq_o}{E_s} (1 - \mu_s^2)[(1 - \mu_s^2)F_1 + (1 - \mu_s - 2\mu_s^2)F_2]$$

For $L/B = 2$ and $H/B = 5$, from figures 19 and 20, $F_1 \approx 0.525$ and $F_2 \approx 0.06$

$$S_e = \frac{(1)(150)}{10,000} (1 - 0.3^2)[(1 - 0.3^2)(0.525) + (1 - 0.3 - 2 \times 0.3^2)(0.06)] = 0.007 \text{ m} = 7.0 \text{ mm}$$

ELASTIC SETTLEMENT OF FOUNDATION ON SATURATED CLAY

Janbu et al. (1956) proposed an equation for evaluating the average settlement of flexible foundations on saturated clay soils (Poisson’s ratio, $\mu_s = 0.5$). For the notation used in figure 4.21, this equation is
Figure 4.21 Values of $A_1$ and $A_2$ for elastic settlement calculation-equation (34) (after Christian and Carrier, 1978)

$$S_e = A_1 A_2 \frac{q_o B}{E_s} \tag{4.34}$$

Where $A_1$ is a function of $H/B$ and $L/B$ and $A_2$ is a function of $D_1/B$

Christian and Carrier (1978) modified the values of $A_1$ and $A_2$ to some extent, as presented in figure 21.

**SETTLEMENT OF SANDY SOIL: USE OF STRAIN INFLUENCE FACTOR**

Settlement of granular soils can be evaluated by use of a semi-empirical strain influence factor (figure 4.22) proposed by Schmertmann and Hartman (1978). According to this method, the settlement is
\[ S_e = C_1 C_2 (\bar{q} - q) \sum_{0}^{z_2} \frac{l_z}{E_s} \Delta z \]  

Figure 4.22 Elastic settlement calculation by using strain influence factor

Where

\( I_z = \) strain influence factor

\( C_1 = \) a correction factor for the depth of foundation embedment = \( 1 - 0.5[\frac{q}{(\bar{q} - q)}] \)

\( C_2 = \) a correction factor to account for creep in soil = \( 1 + 0.2 \log(\text{time in years}/0.1) \)

\( \bar{q} = \text{stress at the level of the foundation} \)

\( q = \gamma D_f \)

The variation of the strain influence factor with depth below the foundation is shown in figure 22a. Note that, for square or circular foundations,

\( I_z = 0.1 \quad \text{at} \quad z = 0 \)

\( I_z = 0.5 \quad \text{at} \quad z = z_1 = 0.5 \, B \)

\( I_z = 0 \quad \text{at} \quad z = z_2 = 2B \)

Similarly, for foundation with \( L/B \geq 10 \),

\( I_z = 0.2 \quad \text{at} \quad z = 0 \)
\[ I_z = 0.5 \text{ at } z = z_1 = B \]
\[ I_z = 0 \text{ at } z = z_2 = 4B \]

Where

\[ B = \text{width of the foundation and } L = \text{length of the foundation} \]

For values of \( L/B \) between 1 and 10, necessary interpolations can be made.

To use equation (35) first requires evaluation of the approximate variation of the modulus of elasticity with depth (figure 4.22). This evaluation can be made by using the standard penetration numbers or cone penetration resistances. The soil layer can be divided into several layers to a depth of \( z = z_2 \), and the elastic settlement of each layer can be estimated. The sum of the settlement of all layers equals \( S_z \), Schmertmann (1970) provided a case history of a rectangular foundation (Belgian bridge pier) having \( L = 23 \) m and \( B = 2.6 \) m and being supported by a granular soil deposit. For this foundation we may assume that \( L/B \approx 10 \) for plotting the strain influence factor diagram. Figure 4.23 shows the details of the foundation along with the approximate variation of the cone penetration resistance, \( q_c \), with depth. For this foundation [equation (35)], note that

![Figure 4.23 Variation of \( I_z \) and \( q_r \) below the foundation](image-url)
\[ q = 178.54 \text{ kN/m}^2 \]

\[ q = 31.39 \text{ kN/m}^2 \]

\[ C_1 = 1 - 0.5 \frac{q}{q-q} = 1 - (0.5) \left(\frac{31.39}{178.54-31.39}\right) = 0.893 \]

\[ C_2 = 1 + 0.2 \log \left(\frac{t_{yr}}{0.1}\right) \]

For \( t = 5 \text{ yr} \)

\[ C_2 = 1 + 0.2 \log \left(\frac{5}{0.1}\right) = 1.34 \]

The following table shows the calculation of \( \sum_0^{z_2} \left(\frac{I_z}{E_s}\right) \Delta z \) in conjunction with figure 4.23.

<table>
<thead>
<tr>
<th>Layer</th>
<th>( \Delta z \text{(m)} )</th>
<th>( q_c \text{(kN/m}^2)</th>
<th>( E_s^a \text{(kN/m}^2)</th>
<th>( z \text{ to the center of the layer (m)} )</th>
<th>( I_z \text{ at the center of the layer} )</th>
<th>( \left(\frac{I_z}{E_s}\right) \Delta z \text{(m}^2/\text{kN)} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>2,450</td>
<td>8,575</td>
<td>0.5</td>
<td>0.258</td>
<td>3.00 \times 10^{-5}</td>
</tr>
<tr>
<td>2</td>
<td>1.6</td>
<td>3,430</td>
<td>12,005</td>
<td>1.8</td>
<td>0.408</td>
<td>5.43 \times 10^{-5}</td>
</tr>
<tr>
<td>3</td>
<td>0.4</td>
<td>3,430</td>
<td>12,005</td>
<td>2.8</td>
<td>0.487</td>
<td>1.62 \times 10^{-5}</td>
</tr>
<tr>
<td>4</td>
<td>0.5</td>
<td>6,870</td>
<td>24,045</td>
<td>3.25</td>
<td>0.458</td>
<td>0.95 \times 10^{-5}</td>
</tr>
<tr>
<td>5</td>
<td>1.0</td>
<td>2,950</td>
<td>10,325</td>
<td>4.0</td>
<td>0.410</td>
<td>3.97 \times 10^{-5}</td>
</tr>
<tr>
<td>6</td>
<td>0.5</td>
<td>8,340</td>
<td>29,190</td>
<td>4.75</td>
<td>0.362</td>
<td>0.62 \times 10^{-5}</td>
</tr>
</tbody>
</table>
Hence the immediate settlement is calculated as

\[ S_e = \sum \frac{z}{E_s} \Delta z \]

\[ = (0.893)(1.34)(178.54 - 31.39)(18.95 \times 10^{-5}) \]

\[ = 0.03336 \approx 33 \text{ mm} \]

After five years, the actual maximum settlement observed for the foundation was about 39 mm.

**RANGE OF MATERIAL PARAMETERS FOR COMPUTING ELASTIC SETTLEMENT**

Section 8-10 presented the equations for calculating immediate settlement of foundations. These equations contain the elastic parameters, such as \( E_s \) and \( \mu_s \). If the laboratory test results for these parameters are not available, certain realistic assumptions have to be made. Table 5 shows the approximate range of the elastic parameters for various soils.

Several investigators have correlated the values of the modulus of elasticity, \( E_s \), with the field standard penetration number, \( N_F \) and the cone penetration resistance, \( q_c \). Mitchell and Gardner (1975) compiled a list of these correlations, Schmertmann (1970) indicated that the modulus of elasticity of sand may be given by

\[ E_s (kN/m^2) = 766 N_F \]  \[ \text{[4.36]} \]
Where

\( N_F = \text{field standard penetration number} \)

In English units

\[ E \ (U.S.\ ton/ft^2) = 8N_F \]  

[4.37]

**Table 5 Elastic Parameters of Various Soils**

<table>
<thead>
<tr>
<th>Type of soil</th>
<th>Modulus of elasticity, ( E_s )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( lb/in^2 )</td>
</tr>
<tr>
<td>Loose sand</td>
<td>1,500-3,500</td>
</tr>
<tr>
<td>Medium dense sand</td>
<td>2,500-4,000</td>
</tr>
<tr>
<td>Dense sand</td>
<td>5,000-8000</td>
</tr>
<tr>
<td>Silty sand</td>
<td>1,500-2,500</td>
</tr>
<tr>
<td>Sand and gravel</td>
<td>10,000-25,000</td>
</tr>
<tr>
<td>Soft clay</td>
<td>600-3,000</td>
</tr>
<tr>
<td>Medium clay</td>
<td>3,000-6,000</td>
</tr>
<tr>
<td>Stiff clay</td>
<td>6,000-14,000</td>
</tr>
</tbody>
</table>

Similarly,

\[ E_s = 2q_c \]  

[4.38]

Where

\( Q_c = \text{static cone penetration resistance} \)

Schmertmann and Hartman (1978) further suggested that the following correlations may be used with the strain influence factors described in section 10:

\[ E_s = 2.5q_c \quad (\text{for square and circular foundations}) \]  

[4.39]

And

\[ E_s = 3.5q_c \quad (\text{for strip foundations}) \]  

[4.40]

Note: Any consistent set of units may be used in equation (38)-(40).
The modulus of elasticity of normally consolidated clays may be estimated as

\[ E_s = 250c \text{ to } 500c \]  \hspace{1cm} [4.41]

And for overconsolidated clays as

\[ E_s = 750c \text{ to } 1000c \]  \hspace{1cm} [4.42]

Where

c = undrained cohesion of clay soil

**CONSOLIDATION SETTLEMENT**

As mentioned before, consolidation settlement occurs over time, and it occurs in saturated clayey soils when they are subjected to increased load caused by foundation construction (figure 4.24). Based on the one-dimensional consolidation settlement equations given in chapter 1, we write

\[ S_c = \int \varepsilon_z \, dz \]

Where

\[ \varepsilon_z = \text{vertical strain} \]

\[ = \frac{\Delta e}{1+e_o} \]
\[ \Delta e = \text{change of void ratio} \]

\[ = f(p_o, p_c, \text{and } \Delta p) \]

So

\[ S_c = \frac{C_c H_c}{1 + e_o} \log \frac{p_o + \Delta p_{av}}{p_o} \]  \hspace{1cm} \text{(for normally consolidated clays)}

(From chapter 1) \hspace{1cm} [64]

\[ S_c = \frac{C_c H_c}{1 + e_o} \log \frac{p_o + \Delta p_{av}}{p_o} \]  \hspace{1cm} \text{(for overconsolidated clays)}

(From chapter 1) \hspace{1cm} [66]

\[ S_c = \frac{C_c H_c}{1 + e_o} \left( \log \frac{p_o + \Delta p_{av}}{p_o} + \frac{C_c H_c}{1 + e_o} \log \frac{p_o + \Delta p_{av}}{p_o} \right) \]  \hspace{1cm} \text{(for overconsolidated clays with } p_o < p_c < p_o + \Delta p_{av})

(From chapter 1) \hspace{1cm} [66]

Where

\[ p_o = \text{average effective pressure on the clay layer before the construction of the foundation} \]

\[ \Delta p_{av} = \text{average increase of pressure on the clay layer caused by the foundation construction} \]

\[ p_c = \text{preconsolidation pressure} \]

\[ e_o = \text{initial void ratio of the clay layer} \]

\[ C_c = \text{compression index} \]

\[ C_s = \text{swelling index} \]

\[ H_c = \text{thickness of the clay layer} \]

The procedures for determining the compression and swelling indexes were discussed in chapter 1.

Note that the increase of pressure, \( \Delta p \), on the clay layer is not constant with depth. The magnitude of \( \Delta p \) will decrease with the increase of depth measured from the bottom of the foundation. However, the average increase of pressure may be approximated by

\[ \Delta p_{av} = \frac{1}{5}(\Delta p_c + 4\Delta p_m + \Delta p_b) \] \hspace{1cm} [4.43]
where $\Delta p_t, \Delta p_m, \text{and } \Delta p_b$ are the pressure increases at the top, middle, and bottom of the clay layer that are caused by the foundation construction.

The method of determining the pressure increase caused by various types of foundation load is discussed in sections 2-7, $\Delta p_{av}$ can also be directly obtained from the method presented in section 5.

**Example 6**

A foundation $1 \ m \times 2m$ in plan is shown in figure 4.25. Estimate the consolidation settlement of the foundation.

![Figure 4.25](image)

**Solution**

The clay is normally consolidated. Thus

$$S_c = \frac{c_{c}H}{1+e_{0}} \log \frac{\frac{p_o+\Delta p_{av}}{p_o}}{\frac{p_o}{p_o}}$$

$$p_o = (2.5)(16.5) + (0.5)(17.5 - 9.81) + (1.25)(16 - 9.81) = 41.25 + 3.85 + 7.74 = 52.84 kN/m^2$$

From equation (43),

$$\Delta p_{av} = \frac{1}{e} (\Delta p_t + 4\Delta p_m + \Delta p_b)$$

Now the following table can be prepared (note: $L = 2 \ m; B = 1 \ m$):

<table>
<thead>
<tr>
<th>$\dot{m}_1 = L/B$</th>
<th>$z(m)$</th>
<th>$z/(B/2) = n_1$</th>
<th>$I_c$</th>
<th>$\Delta p = q_o I_c$</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>2</td>
<td>4</td>
<td>0.190</td>
<td>$28.5 = \Delta p_t$</td>
</tr>
<tr>
<td>2</td>
<td>$2 + 2.5/2 = 3.25$</td>
<td>6.5</td>
<td>$\approx 0.085$</td>
<td>$12.75 = \Delta p_m$</td>
</tr>
</tbody>
</table>
\[ \Delta p_{av} = \frac{1}{6}(28.5 + 4 \times 12.75 + 6.75) = 14.38 \text{ kN/m}^2 \]

So

\[ S_c = \frac{(0.32)(2.5)}{1+0.8} \log \left( \frac{52.84+14.38}{52.84} \right) = 0.0465 = 46.5 \text{ mm} \]

**SKEMPTON-BJERRUM MODIFICATION FOR CONSOLIDATION SETTLEMENT**

The consolidation settlement calculation presented in the preceding section is based on equations (64, 66, and 68). These equations, as shown in chapter 1, are based on one-dimensional laboratory consolidation tests. The underlying assumption for these equations is that the increase of pore water pressure, \( \Delta u \), immediately after the load application equals the increase of stress, \( \Delta p \), at any depth. For this case

\[ S_{c(oed)} = \int \frac{\Delta e}{1+e_o} \, dz = \int m_v \, \Delta p(1) \, dz \]

Where

\[ S_{c(oed)} = \text{consolidation settlement calculated by using equation (64, 66, and 68 from chapter 1)} \]

\[ \Delta p(1) = \text{vertical stress increase (note the change of notation from } \Delta p) \]

\[ m_v = \text{volume coefficient of compressibility (see chapter 1)} \]

In the field, however, when load is applied over a limited area on the ground surface, this assumption will not be correct. Consider the case of a circular foundation on a clay layer as shown in figure 4.26. The vertical and the horizontal stress increases at a point in the clay layer immediately below the center of the foundation are \( \Delta p(1) \) and \( \Delta p(3) \), respectively. For saturated clay, the pore water pressure increase at that depth (chapter 1) is

\[ \Delta u = \Delta p(3) + A[\Delta p(1) - \Delta p(3)] \]  [4.44]
Where

\( A = \text{pore water pressure parameter} \)

For this case

\[
S_c = \int m_v \Delta u \, dz = \int (m_v)(\Delta p_{(3)} + A[\Delta p_{(1)} - \Delta p_{(3)}]) \, dz
\]

Thus we can write

\[
K_{cir} = \frac{S_c}{S_{c,\text{eod}}} = \frac{\int_0^{H_c} m_v \Delta u \, dz}{\int_0^{H_c} m_v \Delta p_{(1)} \, dz} = A + (1 - A) \left[ \frac{\int_0^{H_c} \Delta p_{(3)} \, dz}{\int_0^{H_c} \Delta p_{(1)} \, dz} \right]
\]

Where

\( K_{cir} = \) settlement ratios for circular foundations

The settlement ratio for a continuous foundation (\( K_{str} \)) can be determined in a manner similar to that of a circular foundation. The variation of \( K_{cir} \) and \( K_{str} \) with \( A \) and \( H_c/B \) is given in \textbf{figure 4.27}. (Note: \( B = \) diameter of a circular foundation, and \( B = \) width of a continuous foundation).
Following is the procedure for determining consolidation settlement according to Skempton and Bjerrum (1957).

1. Determine the consolidation settlement, $S_{c(\text{oed})}$, using the procedure outlined in section 12. (Note the change of notation from $S_c$.
2. Determine the pore water pressure parameter, $A$.
3. Determine $H_c/B$.
4. Obtain the settlement ratio-in this case, from figure 27.
5. Calculate the actual consolidation settlement:

\[ S_c = S_{c(\text{oed})} \times \text{settlement ratio} \]  \[ 4.47 \]

This technique is generally referred to as the Skempton-Bjerrum modification for consolidation settlement calculation.

CONSOLIDATION SETTLEMENT-GENERAL COMMENTS AND A CASE HISTORY

In predicting the consolidation settlement and the time rate of settlement for actual field conditions, an engineer has to make several simplifying assumptions. They include the
compression index, coefficient of consolidation, preconsolidation pressure, drainage conditions, and thickness of the clay layer. Soil layering is not always uniform with ideal properties; hence field performance may deviate from the prediction, requiring adjustments during construction. The following case history on consolidation, as reported by Schnabel (1972), illustrates this reality.

**Figure 4.28** shows the subsoil conditions for the construction of a school building in Waldorf, Maryland. Upper Pleistocene sand and gravel soils are underlain by deposits of very loose fine silty sand, soft silty clay, and clayey silt. The softer surface layers are underlain by various layers of stiff to firm silty clay, clayey silt, and sandy silt to a depth of 50 ft. before construction of the building began; a compacted fill having a thickness of 8-10 ft was placed on the ground surface. This fill initiated the consolidation settlement in the soft silty clay and clayey silt.

![Subsoil profile](image)

**Figure 4.28 Subsoil conditions for the construction of school building (note: SPT N value are uncorrected, i.e., after Schnabel, 1972)**

To predict the time rate of settlement, based on the laboratory test results, the engineers made the following approximations:
a. The preconsolidation pressure, \( p_c \) was 1600 to 2800 lb/ft\(^2\) in excess of the existing overburden pressure.
b. The swell index, \( C_s \), was 0.01 to 0.03.
c. For the more compressive layers \( C_v \approx 0.36 \) ft\(^2\)/day, and for the stiffer soil layers \( C_v \approx 3.1 \) ft\(^2\)/day.

The total consolidation settlement was estimated to be about 3 in. under double drainage conditions, 90% settlement was expected to occur in 114 days.

**Figure 4.29** shows a comparison of measured and predicted settlement with time, with indicates that

a. \( \frac{S_c(\text{observed})}{S_c(\text{estimated})} \approx 0.47 \)
b. 90% of the settlement occurred in about 70 days; hence \( \frac{t_{90(\text{observed})}}{t_{90(\text{estimated})}} \approx 0.58 \).

The reality rapid settlement in the field is believed to be due to the presence of a fine sand layer within the Miocene deposits.