Chapter 7

Settlement of Shallow Foundations

Omitted parts:

Section 7.7

CAUSES OF SETTLEMENT

Settlement of a structure resting on soil may be caused by two distinct kinds of action within the foundation soils:-

I. Settlement Due to Shear Stress (Distortion Settlement)

In the case the applied load caused shearing stresses to develop within the soil mass which are greater than the shear strength of the material, then the soil fails by sliding downward and laterally, and the structure settle and may tip of vertical alignment. This is what we referred to as BEARING CAPACITY.

II. Settlement Due to Compressive Stress (Volumetric Settlement)

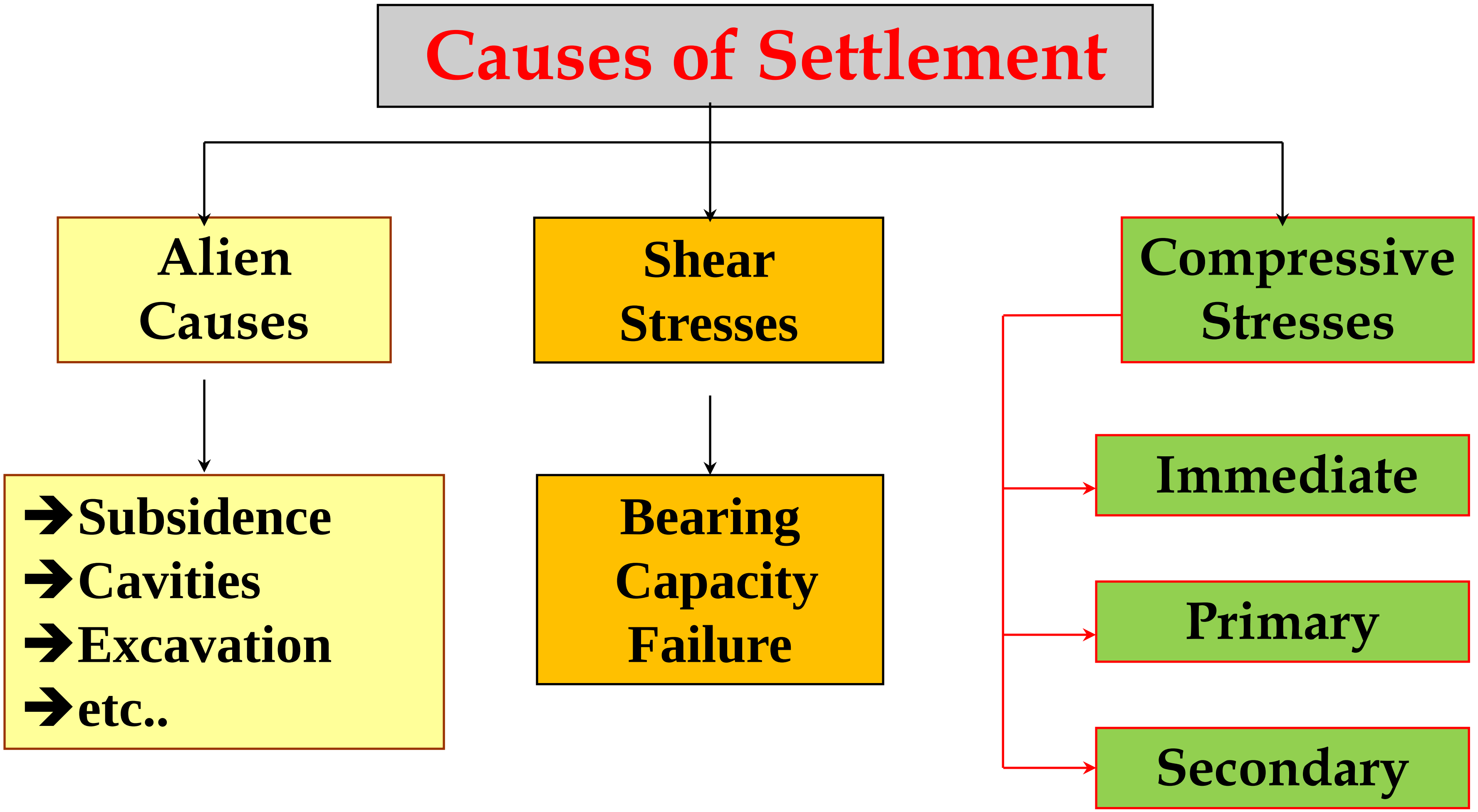
As a result of the applied load a compressive stress is transmitted to the soil leading to compressive strain. Due to the compressive strain the structure settles. This is important only if the settlement is excessive otherwise it is not dangerous.

ALLOWABLE BEARING CAPACITY

The allowable bearing capacity is the smaller of the following two conditions:

$$q_{all} = \text{smallest of}\begin{cases} \dfrac{q_u}{FS} \text{(to control shear failure)} \\ q_{all,settlement} \text{(to control settlement)} \end{cases}$$

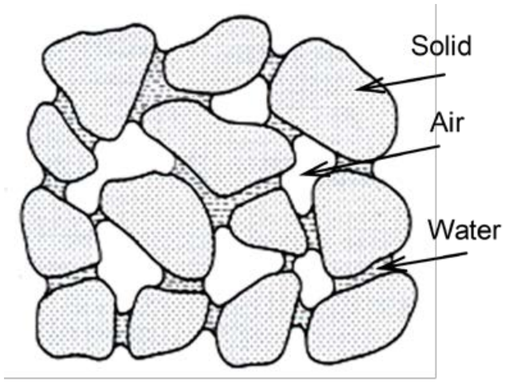
CAUSES OF SETTLEMENT



Mechanisms of Compression

Compression of soil is due to a number of mechanisms:

- **Deformation of soil particles or grains**
- **Relocations of soil particles**
- **Expulsion of water or air from the void spaces**



Components of Settlement

Settlement of a soil layer under applied load is the sum of two broad components or categories:

1. Elastic settlement (or immediate) settlements

2. Consolidation settlement

1. Elastic settlement (or immediate) settlements

Elastic or immediate settlement takes place instantly at the moment of the application of load due to the distortion (but no bearing failure) and bending of soil particles (mainly clay). It is not generally elastic although theory of elasticity is applied for its evaluation. It is predominant in coarse-grained soils.

Consolidation settlement

Consolidation settlement is the sum of two parts or types:

A. Primary consolidation settlement

In this the compression of clay is due to expulsion of water from pores. The process is referred to as primary consolidation and the associated settlement is termed primary consolidation settlement. Commonly they are referred to simply as consolidation and consolidation settlement (CE 481)

B. Secondary consolidation settlement

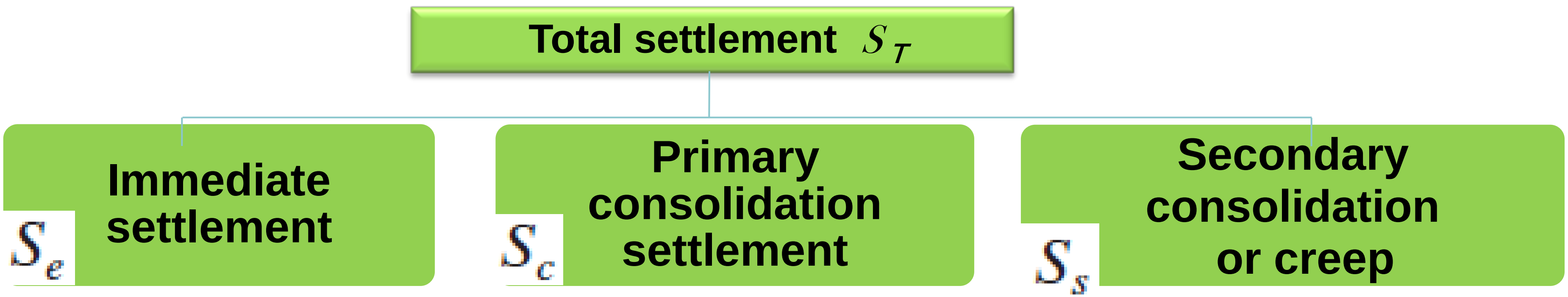
The compression of clay soil due to plastic readjustment of soil grains and progressive breaking of clayey particles and their inter-particles bonds is known as secondary consolidation or secondary compression, and the associated settlement is called secondary consolidation settlement or secondary compression.

Components of Settlement

The total settlement of a foundation can be expressed as:

$$S_T = S_e + S_c + S_s$$

S_T = Total settlement
S_e = Elastic or immediate settlement
S_c = Primary consolidation settlement
S_s = Secondary consolidation settlement



- It should be mentioned that S_c and S_s overlap each other and impossible to detect which certainly when one type ends and the other begins. However, for simplicity they are treated separately and secondary consolidation is usually assumed to begin at the end of primary consolidation.

Components of settlement

The total soil settlement S_T may contain one or more of these types:

Immediate settlement S_e

- Due to distortion or elastic deformation with no change in water content
- Occurs rapidly during the application of load
- Quite small quantity in dense sands, gravels and stiff clays

Primary consolidation settlement S_c

- Decrease in voids volume due to squeeze of pore-water out of the soil
- Occurs in saturated fine grained soils (low coefficient of permeability)

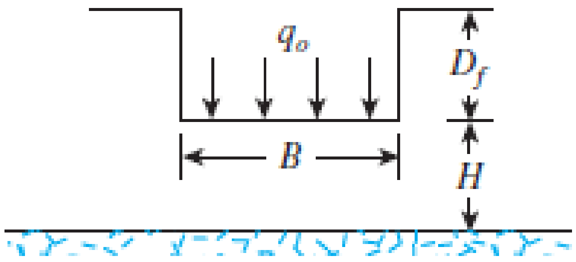
  Time dependent
- Only significant in clays and silts

Secondary consolidation or creep S_s

- Due to gradual changes in the particulate structure of the soil
- Occurs very slowly, long after the primary consolidation is completed

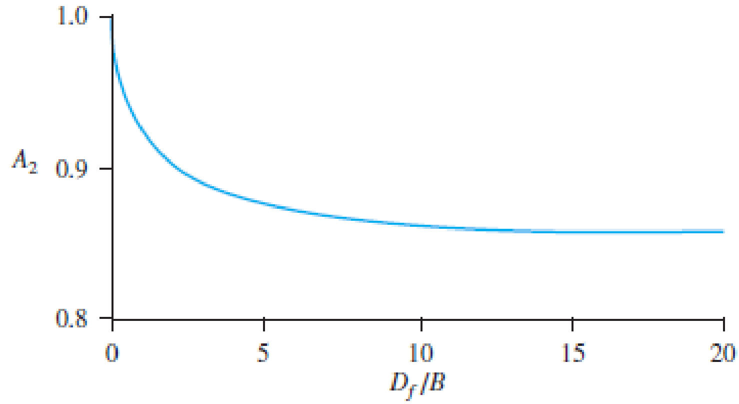
  Time dependent
- Most significant in saturated soft clayey and organic soils and peats

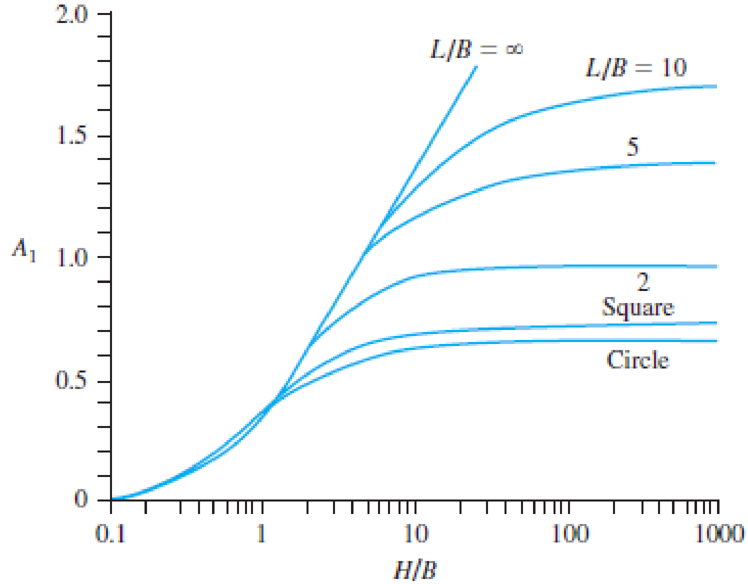
Elastic Settlement of Shallow Foundation on Saturated Clay (μ_s = 0.5)



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

$A_1 = f(H/B, L/B)$
$A_2 = f(D_f/B)$
L = length of the foundation
B = width of the foundation
D_f = depth of the foundation
H = depth of the bottom of the foundation to a rigid layer
q_o = net load per unit area of the foundation





Elastic Settlement of Shallow Foundation on Saturated Clay ($\mu_s = 0.5$)

$$E_s = \beta c_u$$

where c_u = undrained shear strength.

Table 7.1 Range of β for Saturated Clay [Eq. (7.2)][a]

Plasticity Index	β				
	OCR = 1	OCR = 2	OCR = 3	OCR = 4	OCR = 5
<30	1500–600	1380–500	1200–580	950–380	730–300
30 to 50	600–300	550–270	580–220	380–180	300–150
>50	300–150	270–120	220–100	180–90	150–75

[a]Based on Duncan and Buchignani (1976)

Example 7.1

Consider a shallow foundation 2 m × 1 m in plan in a saturated clay layer. A rigid rock layer is located 8 m below the bottom of the foundation. Given:

Foundation: $D_f = 1$ m, $q_o = 120$ kN/m^2
Clay: $c_u = 150$ kN/m^2, OCR = 2, and Plasticity index, PI = 35

Estimate the elastic settlement of the foundation.

Solution

From Eq. (7.1),

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

Given:

$$\frac{L}{B} = \frac{2}{1} = 2$$

$$\frac{D_f}{B} = \frac{1}{1} = 1$$

$$\frac{H}{B} = \frac{8}{1} = 8$$

$$E_s = \beta c_u$$

For OCR = 2 and PI = 35, the value of $\beta \approx 480$ (Table 7.1). Hence,

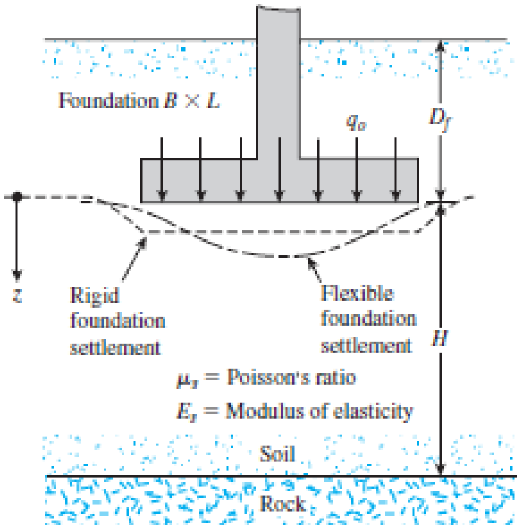
$$E_s = (480)(150) = 72{,}000 \text{ kN/m}^2$$

Also, from Figure 7.1, $A_1 = 0.9$ and $A_2 = 0.92$. Hence,

$$S_e = A_1 A_2 \frac{q_o B}{E_s} = (0.9)(0.92)\frac{(120)(1)}{72{,}000} = 0.00138 \text{ m} = \mathbf{1.38\ mm}$$ ■

Elastic Settlement in Granular Soil

Settlement Based on the Theory of Elasticity



$$S_e = q_o(\alpha B')\frac{1-\mu_s^2}{E_s} I_s I_f$$

where

q_o = net applied pressure on the foundation
μ_s = Poisson's ratio of soil
E_s = average modulus of elasticity of the soil under the foundation, measured from $z = 0$ to about $z = 5B$
$B' = B/2$ for center of foundation
$= B$ for corner of foundation
I_s = shape factor (Steinbrenner, 1934)

$$I_f = \text{depth factor (Fox, 1948)} = f\left(\frac{D_f}{B}, \mu_s, \text{and } \frac{L}{B}\right)$$

α = a factor that depends on the location on the foundation where settlement is being calculated

The elastic settlement of a *rigid foundation* can be estimated as

$$S_{e(\text{rigid})} \approx 0.93 S_{e(\text{flexible, center})}$$

Elastic Settlement in Granular Soil

To calculate settlement at the *center* of the foundation, we use

$$\alpha = 4$$

$$m' = \frac{L}{B}$$

and

$$n' = \frac{H}{\left(\frac{B}{2}\right)}$$

To calculate settlement at a *corner* of the foundation,

$$\alpha = 1$$

$$m' = \frac{L}{B}$$

and

$$n' = \frac{H}{B}$$

$$I_s = F_1 + \frac{1 - 2\mu_s}{1 - \mu_s} F_2$$

Table 7.2 Variation of F_1 with m' and n'

n'	m' = 1.0	1.2	1.4	1.6	1.8	2.0	2.5	3.0	3.5	4.0
0.25	0.014	0.013	0.012	0.011	0.011	0.011	0.010	0.010	0.010	0.010
0.50	0.049	0.046	0.044	0.042	0.041	0.040	0.038	0.038	0.037	0.037
0.75	0.095	0.090	0.087	0.084	0.082	0.080	0.077	0.076	0.074	0.074
1.00	0.142	0.138	0.134	0.130	0.127	0.125	0.121	0.118	0.116	0.115

Table 7.3 Variation of F_2 with m' and n'

n'	m' = 1.0	1.2	1.4	1.6	1.8	2.0	2.5	3.0	3.5	4.0
0.25	0.049	0.050	0.051	0.051	0.051	0.052	0.052	0.052	0.052	0.052
0.50	0.074	0.077	0.080	0.081	0.083	0.084	0.086	0.086	0.0878	0.087
0.75	0.083	0.089	0.093	0.097	0.099	0.101	0.104	0.106	0.107	0.108
1.00	0.083	0.091	0.098	0.102	0.106	0.109	0.114	0.117	0.119	0.120

Table 7.4 Variation of I_f with D_f/B, B/L, and μ_s

μ_s	D_f/B	B/L = 0.2	0.5	1.0
0.3	0.2	0.95	0.93	0.90
	0.4	0.90	0.86	0.81
	0.6	0.85	0.80	0.74
	1.0	0.78	0.71	0.65
0.4	0.2	0.97	0.96	0.93
	0.4	0.93	0.89	0.85
	0.6	0.89	0.84	0.78
	1.0	0.82	0.75	0.69
0.5	0.2	0.99	0.98	0.96
	0.4	0.95	0.93	0.89
	0.6	0.92	0.87	0.82
	1.0	0.85	0.79	0.72

$$E_s = \frac{\Sigma E_{s(i)} \Delta z}{\bar{z}}$$

where

$E_{s(i)}$ = soil modulus of elasticity within a depth Δz

$\bar{z}$ = H or $5B$, whichever is smaller

Example 7.2

Example 7.2

A rigid shallow foundation 1 m × 2 m is shown in Figure 7.4. Calculate the elastic settlement at the center of the foundation.

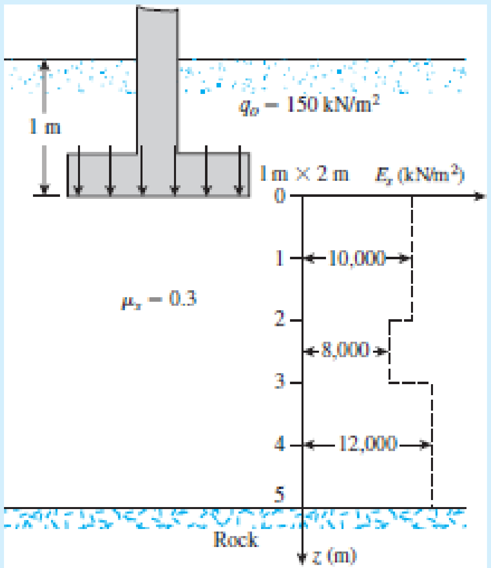


Figure 7.4 Elastic settlement below the center of a foundation

Solution

We are given that $B = 1$ m and $L = 2$ m. Note that $\bar{z} = 5$ m $= 5B$. From Eq. (7.13)

$$E_s = \frac{\Sigma E_{s(i)} \Delta z}{\bar{z}}$$

$$= \frac{(10{,}000)(2) + (8{,}000)(1) + (12{,}000)(2)}{5} = 10{,}400 \text{ kN/m}^2$$

For the *center of the foundation*,

$$\alpha = 4$$

$$m' = \frac{L}{B} = \frac{2}{1} = 2$$

and

$$n' = \frac{H}{\left(\frac{B}{2}\right)} = \frac{5}{\left(\frac{1}{2}\right)} = 10$$

From Tables 7.2 and 7.3, $F_1 = 0.641$ and $F_2 = 0.031$. From Eq. (7.5),

$$I_s = F_1 + \frac{2 - \mu_s}{1 - \mu_s} F_2$$

$$= 0.641 + \frac{2 - 0.3}{1 - 0.3}(0.031) = 0.716$$

Again, $D_f/B = 1/1 = 1$, $B/L = 0.5$, and $\mu_s = 0.3$. From Table 7.4, $I_f = 0.71$.

Hence,

$$S_{e(\text{flexible})} = q_0(\alpha B') \frac{1 - \mu_s^2}{E_s} I_s I_f$$

$$= (150)\left(4 \times \frac{1}{2}\right)\left(\frac{1 - 0.3^2}{10{,}400}\right)(0.716)(0.71) = 0.0133 \text{ m} = 13.3 \text{ mm}$$

Since the foundation is rigid, from Eq.(7.12) we obtain

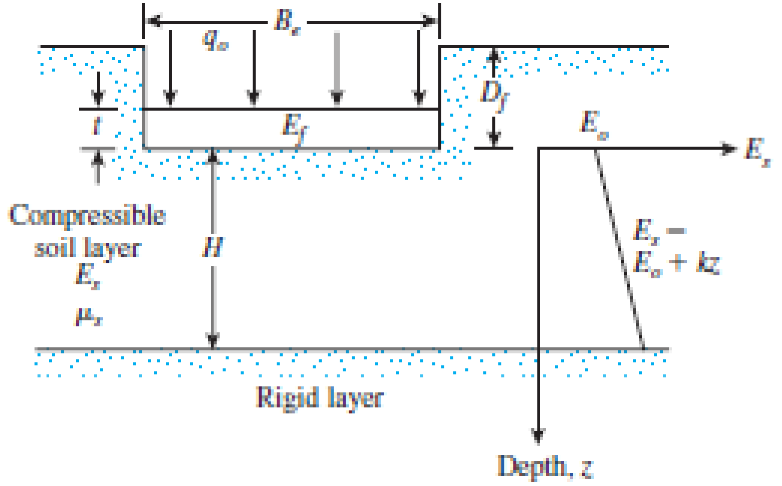
$$S_{e(\text{rigid})} = (0.93)(13.3) = \mathbf{12.4 \text{ mm}}$$

■

Improved Equation for Elastic Settlement

The improved formula takes into account

- the rigidity of the foundation,
- the depth of embedment of the foundation,
- the increase in the modulus of elasticity of the soil with depth, and
- the location of rigid layers at a limited depth



$$S_e = \frac{q_o B_e I_G I_F I_E}{E_o}\left(1 - \mu_s^2\right)$$

where

I_G = influence factor for the variation of E_s with depth

$$= f\left(\beta = \frac{E_o}{kB_e}, \frac{H}{B_e}\right)$$

I_F = foundation rigidity correction factor
I_E = foundation embedment correction factor

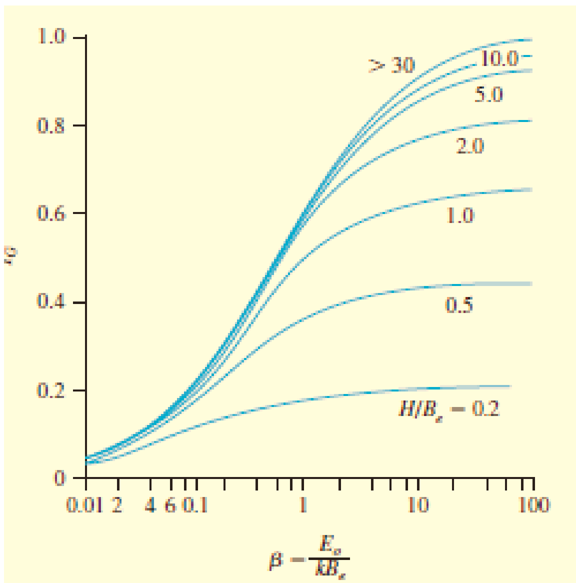
Equivalent diameter ***B*$_e$** of

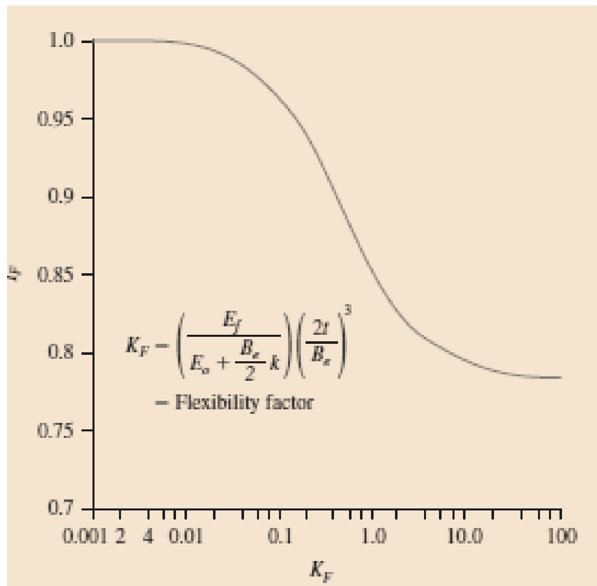
Rectangular foundation $B_e = \sqrt{\frac{4BL}{\pi}}$

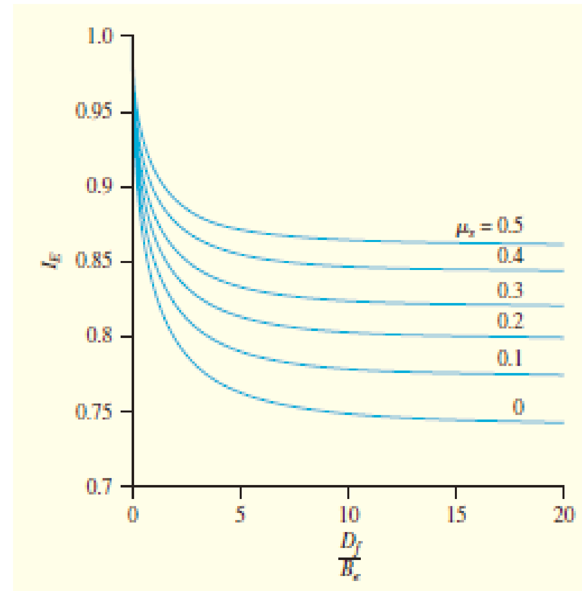
Circular foundation $B_e = B$

$$E_s = E_o + kz$$

Improved Equation for Elastic Settlement







I_G I_F I_E

Example 7.3

Example 7.3

For a shallow foundation supported by a silty sand, as shown in Figure 7.5.

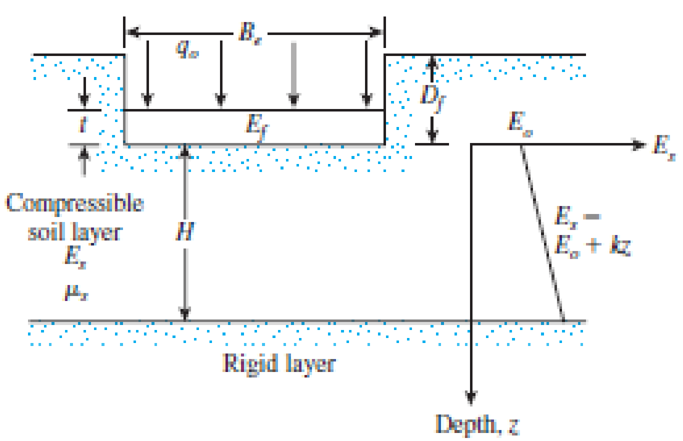
Length $= L = 3$ m
Width $= B = 1.5$ m
Depth of foundation $= D_f = 1.5$ m

Thickness of foundation $= t = 0.3$ m
Load per unit area $= q_o = 240$ kN/m^2
$E_f = 16 \times 10^6$ kN/m^2

The silty sand soil has the following properties:

$H = 3.7$ m
$\mu_s = 0.3$
$E_o = 9700$ kN/m^2
$k = 575$ kN/m^2/m

Estimate the elastic settlement of the foundation.



Solution

From Eq. (7.14), the equivalent diameter is

$$B_e = \sqrt{\frac{4BL}{\pi}} = \sqrt{\frac{(4)(1.5)(3)}{\pi}} = 2.39 \text{ m}$$

so

$$\beta = \frac{E_o}{kB_e} = \frac{9700}{(575)(2.39)} = 7.06$$

and

$$\frac{H}{B_e} = \frac{3.7}{2.39} = 1.55$$

From Figure 7.6, for $\beta = 7.06$ and $H/B_e = 1.55$, the value of $I_G \approx 0.7$. From Eq. (7.18),

$$I_F = \frac{\pi}{4} + \frac{1}{4.6 + 10\left(\dfrac{E_f}{E_o + \dfrac{B_e}{2}k}\right)\left(\dfrac{2t}{B_e}\right)^3}$$

$$= \frac{\pi}{4} + \frac{1}{4.6 + 10\left[\dfrac{16 \times 10^6}{9700 + \left(\dfrac{2.39}{2}\right)(575)}\right]\left[\dfrac{(2)(0.3)}{2.39}\right]^3} = 0.789$$

Example 7.3

From Eq. (7.19),

$$I_E = 1 - \frac{1}{3.5\exp(1.22\mu_s - 0.4)\left(\frac{B_e}{D_f} + 1.6\right)}$$

$$= 1 - \frac{1}{3.5\exp\left[(1.22)(0.3) - 0.4\right]\left(\frac{2.39}{1.5} + 1.6\right)} = 0.907$$

From Eq. (7.17),

$$S_e = \frac{q_o B_e I_G I_F I_E}{E_o}(1 - \mu_s^2)$$

so, with $q_o = 240\ \text{kN/m}^2$, it follows that

$$S_e = \frac{(240)(2.39)(0.7)(0.789)(0.907)}{9700}(1 - 0.3^2) \approx 0.02696\ \text{m} \approx 27\ \text{mm}$$ ■

Settlement of Sandy Soil: Use of Strain Influence Factor

I. Solution of Schmertmann et al. (1978)

$$S_e = C_1 C_2 (\overline{q} - q) \sum_0^{z_2} \frac{I_z}{E_s} \Delta z$$

where

I_z = strain influence factor
C_1 = a correction factor for the depth of foundation embedment = $1 - 0.5\,[q/(\overline{q} - q)]$
C_2 = a correction factor to account for creep in soil
= $1 + 0.2 \log(\text{time in years}/0.1)$
$\overline{q}$ = stress at the level of the foundation
$q = \gamma D_f$ = effective stress at the base of the foundation
E_s = modulus of elasticity of soil

Settlement of Sandy Soil: Use of Strain Influence Factor

The recommended variation of the strain influence factor I_z for square ($L/B = 1$) or circular foundations and for foundations with $L/B >= 10$ is shown in Figure 7.9. The I_z diagrams for $1 < L/B < 10$ can be interpolated.

Note that the maximum value of I_z [that is, $I_{z\,(m)}$] occurs at $z = z_1$ and then reduces to zero at $z = z_2$. The maximum value of I_z can be calculated as:

$$I_{z(m)} = 0.5 + 0.1\sqrt{\frac{\bar{q} - q}{q'_{z(1)}}}$$

where

$q'_{z(1)}$ = effective stress at a depth of z_1 before construction of the foundation

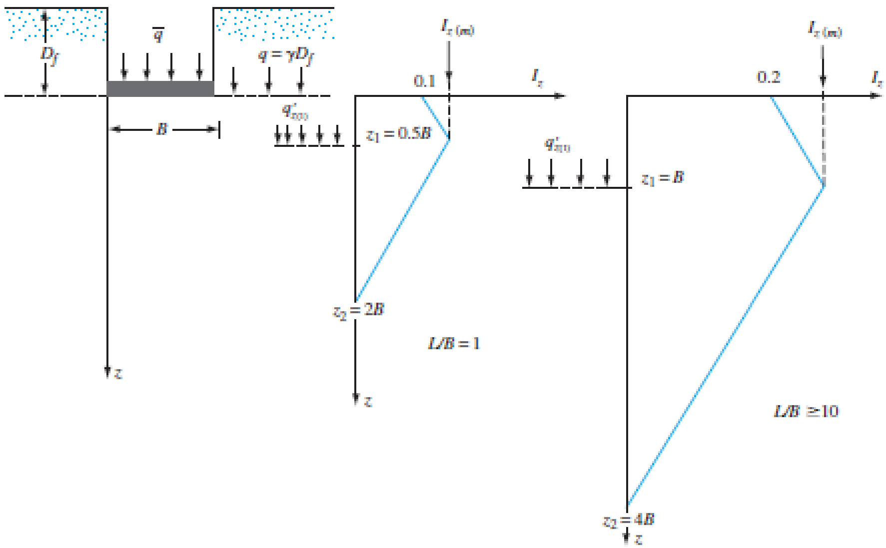


Figure 7.9 Variation of strain influence factor with depth and L/B

Settlement of Sandy Soil: Use of Strain Influence Factor

The following relations are suggested by Salgado (2008) for interpolation of I_z at $z = 0$, z_1/B, and z_2/B for rectangular foundations.

- I_z at $z = 0$

$$I_z = 0.1 + 0.0111\left(\frac{L}{B} - 1\right) \leq 0.2$$

- Variation of z_1/B for $I_{z(m)}$

$$\frac{z_1}{B} = 0.5 + 0.0555\left(\frac{L}{B} - 1\right) \leq 1$$

- Variation of z_2/B

$$\frac{z_2}{B} = 2 + 0.222\left(\frac{L}{B} - 1\right) \leq 4$$

Schmertmann et al. (1978) suggested that

$$E_s = 2.5q_c \text{ (for square foundation)}$$

and

$$E_s = 3.5q_c \text{ (for } L/B \geq 10\text{)}$$

where q_c = cone penetration resistance.

It appears reasonable to write (Terzaghi et al., 1996)

$$E_{s(\text{rectangle})} = \left(1 + 0.4 \log\frac{L}{B}\right)E_{s(\text{square})}$$

Procedure for calculation of S_e using the strain influence factor

Step 1. Plot the foundation and the variation of I_z with depth to scale (Figure 7.10a).
Step 2. Using the correlation from standard penetration resistance (N_{60}) or cone penetration resistance (q_c), plot the actual variation of E_s with depth (Figure 7.10b).
Step 3. Approximate the actual variation of E_s into a number of layers of soil having a constant E_s, such as $E_{s(1)}$, $E_{s(2)}$, . . . , $E_{s(i)}$, . . . $E_{s(n)}$ (Figure 7.10b).
Step 4. Divide the soil layer from $z = 0$ to $z = z_2$ into a number of layers by drawing horizontal lines. The number of layers will depend on the break in continuity in the I_z and E_s diagrams.
Step 5. Prepare a table (such as Table 7.5) to obtain $\Sigma \frac{I_z}{E_s}\Delta z$.
Step 6. Calculate C_1 and C_2.
Step 7. Calculate S_e from Eq. (7.20).

$$S_e = C_1 C_2(\bar{q} - q)\sum_0^{z_2} \frac{I_z}{E_s}\Delta z$$

Table 7.5 Calculation of $\Sigma \frac{I_z}{E_s}\Delta z$

Layer no.	Δz	E_s	I_z at the middle of the layer	$\frac{I_z}{E_s}\Delta z$
1	$\Delta z_{(1)}$	$E_{s(1)}$	$I_{z(1)}$	$\frac{I_{z(1)}}{E_{s(1)}}\Delta z_1$
2	$\Delta z_{(2)}$	$E_{s(2)}$	$I_{z(2)}$	
⋮	⋮	⋮	⋮	
i	$\Delta z_{(i)}$	$E_{s(i)}$	$I_{z(i)}$	$\frac{I_{z(i)}}{E_{s(i)}}\Delta z_i$
⋮	⋮	⋮	⋮	⋮
n	$\Delta z_{(n)}$	$E_{s(n)}$	$I_{z(n)}$	$\frac{I_{z(n)}}{E_{s(n)}}\Delta z_n$
				$\Sigma \frac{I_z}{E_s}\Delta z$

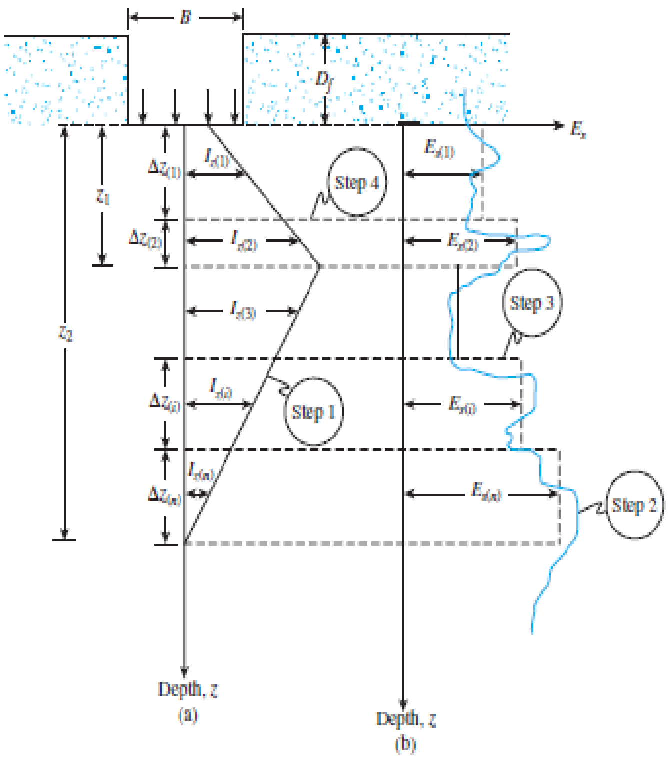


Figure 7.10 Procedure for calculation of S_e using the strain influence factor

Example 7.4

Example 7.4

Consider a rectangular foundation 2 m × 4 m in plan at a depth of 1.2 m in a sand deposit, as shown in Figure 7.11a. Given: $\gamma = 17.5$ kN/m^3; $\bar{q} = 145$ kN/m^2, and the following approximated variation of q_c with z:

z (m)	q_c (kN/m^2)
0–0.5	2250
0.5–2.5	3430
2.5–6.0	2950

Estimate the elastic settlement of the foundation using the strain influence factor method.

Solution

From Eq. (7.23),

$$\frac{z_1}{B} = 0.5 + 0.0555\left(\frac{L}{B} - 1\right) = 0.5 + 0.0555\left(\frac{4}{2} - 1\right) \approx 0.56$$

$$z_1 = (0.56)(2) = 1.12 \text{ m}$$

From Eq. (7.24),

$$\frac{z_2}{B} = 2 + 0.222\left(\frac{L}{B} - 1\right) = 2 + 0.222(2 - 1) = 2.22$$

$$z_2 = (2.22)(2) = 4.44 \text{ m}$$

From Eq. (7.22), at $z = 0$,

$$I_z = 0.1 + 0.0111\left(\frac{L}{B} - 1\right) = 0.1 + 0.0111\left(\frac{4}{2} - 1\right) \approx 0.11$$

From Eq. (7.21),

$$I_{z(m)} = 0.5 + 0.1\sqrt{\frac{\bar{q} - q}{q'_{z(1)}}} = 0.5 + 0.1\left[\frac{145 - (1.2 \times 17.5)}{(1.2 + 1.12)(17.5)}\right]^{0.5} = 0.675$$

The plot of I_z versus z is shown in Figure 7.11c. Again, from Eq. (7.27)

$$E_{s(\text{rectangle})} = \left(1 + 0.4\log\frac{L}{B}\right)E_{s(\text{square})} = \left[1 + 0.4\log\left(\frac{4}{2}\right)\right](2.5 \times q_c) = 2.8q_c$$

Hence, the approximated variation of E_s with z is as follows:

z (m)	q_c (kN/m^2)	E_s (kN/m^2)
0–0.5	2250	6300
0.5–2.5	3430	9604
2.5–6.0	2950	8260

The plot of E_s versus z is shown in Figure 7.11b.

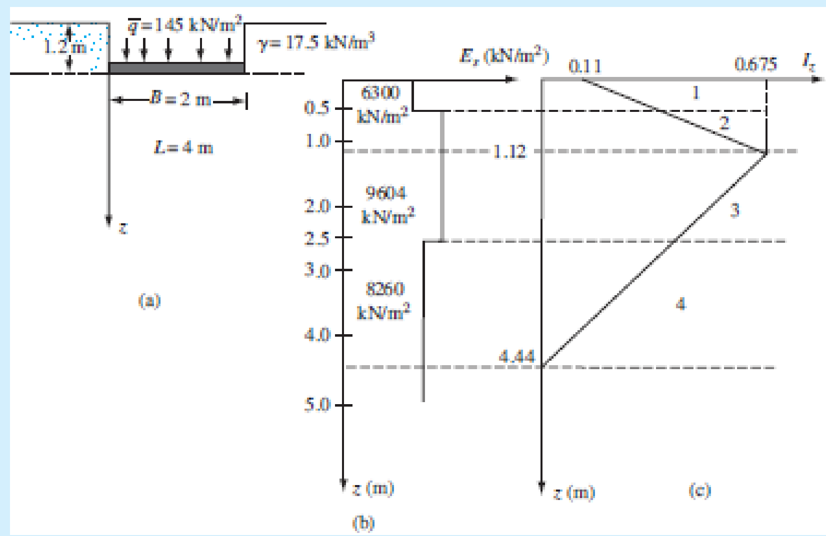


Figure 7.11

Example 7.4

The soil layer is divided into four layers as shown in Figures 7.11b and 7.11c. Now the following table can be prepared.

Layer no.	Δz (m)	E_s (kN/m²)	I_z at middle of layer	$\frac{I_z}{E_s}\Delta z$ (m³/kN)
1	0.50	6300	0.236	1.87×10^{-5}
2	0.62	9604	0.519	3.35×10^{-5}
3	1.38	9604	0.535	7.68×10^{-5}
4	1.94	8260	0.197	4.62×10^{-5}
				$\Sigma 17.52 \times 10^{-5}$

$$S_e = C_1 C_2(\bar{q} - q)\sum \frac{I_z}{E_s}\Delta z$$

$$C_1 = 1 - 0.5\left(\frac{q}{\bar{q} - q}\right) = 1 - 0.5\left(\frac{21}{145 - 21}\right) = 0.915$$

Assume the time for creep is 10 years. So,

$$C_2 = 1 + 0.2 \log\left(\frac{10}{0.1}\right) = 1.4$$

Hence,

$$S_e = (0.915)(1.4)(145 - 21)(17.52 \times 10^{-5}) = 2783 \times 10^{-5}\ \text{m} = \mathbf{27.83\ mm}$$

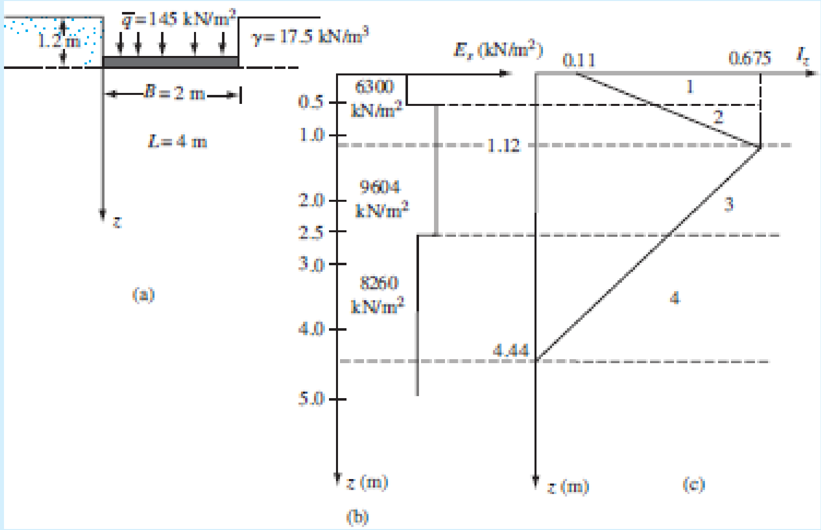


Figure 7.11

Settlement of Sandy Soil: Use of Strain Influence Factor

II. Solution of Terzaghi et al. (1996)

$$S_e = C_d(\bar{q} - q)\sum_{0}^{z_2}\frac{I_z}{E_s}\Delta z + \underbrace{0.02\left[\frac{0.1}{\frac{\sum(q_c\Delta z)}{z_2}}\right]z_2 \log\left(\frac{t\text{ days}}{1\text{ day}}\right)}_{\text{Post-construction settlement}}$$

Terzaghi, Peck, and Mesri (1996) proposed a slightly different form of the strain influence factor diagram, as shown in Figure 7.12. According to Terzaghi et al. (1996),

At $z = 0$, $I_z = 0.2$ (for all L/B values)
At $z = z_1 = 0.5B$, $I_z = 0.6$ (for all L/B values)
At $z = z_2 = 2B$, $I_z = 0$ (for $L/B = 1$)
At $z = z_2 = 4B$, $I_z = 0$ (for $L/B \geq 10$)

For L/B between 1 and 10 (or > 10),

$$\frac{z_2}{B} = 2\left[1 + \log\left(\frac{L}{B}\right)\right]$$

In Eq. (7.29), q_c is in MN/m^2.

The relationships for E_s are

$$E_s = 3.5q_c \text{ (for square and circular foundations)}$$

and

$$E_{s(\text{rectangular})} = \left[1 + 0.4\left(\frac{L}{B}\right)\right]E_{s(\text{square})} \quad (\text{for } L/B \geq 10)$$

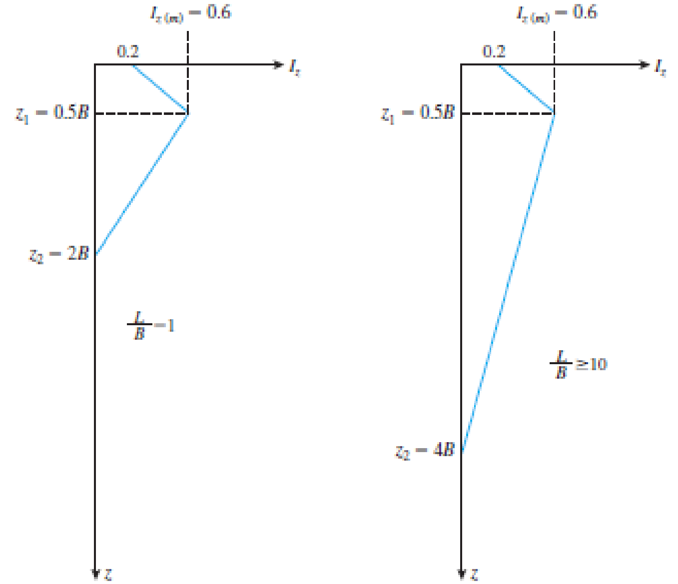


Figure 7.12 Strain influence factor diagram proposed by Terzaghi, Peck, and Mesri (1996)

Table 7.6 Variation of C_d with D_f/B*

D_f/B	C_d
0.1	1
0.2	0.96
0.3	0.92
0.5	0.86
0.7	0.82
1.0	0.77
2.0	0.68
3.0	0.65

*Based on data from Terzaghi et al. (1996)

Example 7.5

Example 7.5

Solve Example 7.4 using the method of Terzaghi et al. (1996).

Solution

Given: $L/B = 4/2 = 2$

Figure 7.13a shows the plot of I_z with depth below the foundation. Note that

$$\frac{z_2}{B} = 2\left[1 + \log\left(\frac{L}{B}\right)\right] = 2[1 + \log(2)] = 2.6$$

or

$$z_2 = (2.6)(B) = (2.6)(2) = 5.2 \text{ m}$$

Also, from Eqs. (7.30) and (7.31),

$$E_s = \left[1 + 0.4\left(\frac{L}{B}\right)\right](3.5q_c) = \left[1 + 0.4\left(\frac{4}{2}\right)\right](3.5q_c) = 6.3q_c$$

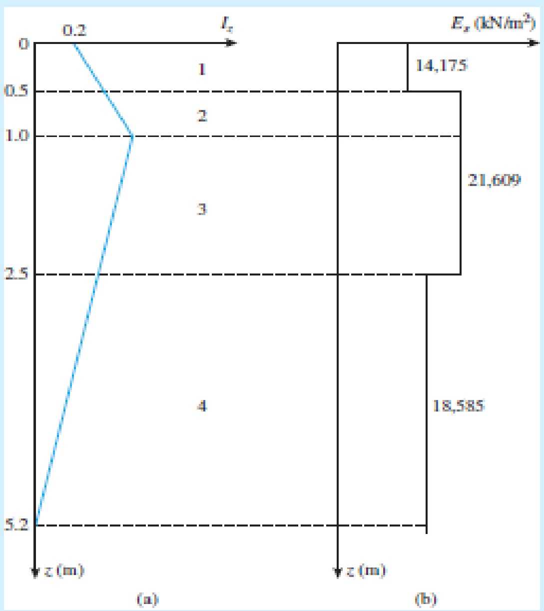


Figure 7.13

z (m)	q_c (kN/m²)	E_s (kN/m²)
0–0.5	2250	14,175
0.5–2.5	3430	21,609
2.5–6	2950	18,585

Again, $D_f/B = 1.2/2 = 0.6$. From Table 7.6, $C_d \approx 0.85$.

The following is the table to calculate $\sum_0^{z_2} \frac{I_z}{E_s}\Delta z$.

Layer No.	Δz (m)	E_s (kN/m²)	I_z at the middle of the layer	$\frac{I_z}{E_s}\Delta z$ (m²/kN)
1	0.5	14,175	0.3	1.058×10^{-5}
2	0.5	21,609	0.5	1.157×10^{-5}
3	1.5	21,609	0.493	3.422×10^{-5}
4	2.7	18,585	0.193	2.804×10^{-5}
				$\Sigma\ 8.441 \times 10^{-5}$ m²/kN

Thus,

$$C_d(\bar{q} - q)\sum_0^{z_2}\frac{I_z}{E_s}\Delta_z = (0.85)(145 - 21)(8.441 \times 10^{-5}) = 889.68 \times 10^{-5} \text{ m}$$

Post-construction creep is

$$0.02\left[\frac{0.1}{\frac{\sum(q_c\Delta z)}{z_2}}\right]z_2 \log\left(\frac{t \text{ days}}{1 \text{ day}}\right)$$

$$\frac{\sum(q_c\Delta z)}{z_2} = \frac{(2250 \times 0.5) + (3430 \times 2) + (2950 \times 2.7)}{5.2}$$
$$= 3067.3 \text{ kN/m}^2 \approx 3.07 \text{ MN/m}^2$$

Hence, the elastic settlement is

$$S_e = 889.68 \times 10^{-5} + 0.02\left[\frac{0.1}{3.07}\right](5.2)\log\left(\frac{10 \times 365 \text{ days}}{1 \text{ day}}\right)$$
$$= 2096.68 \times 10^{-5} \text{ m}$$
$$\approx \mathbf{20.97 \text{ mm}}$$

Example 7.5

Note: The magnitude of S_e is about 75% of that found in Example 7.4. In Example 7.4, the elastic settlement was about 19.88 mm and settlement due to creep was about 7.95 mm. However, in Example 7.5, elastic settlement is 8.89 mm and the settlement due to creep is about 12.07 mm. Thus the magnitude of creep settlement is about 50% more in Example 7.5. However, the magnitude of elastic settlement in Example 7.4 is about twice that compared to that in Example 7.5. This is because of the assumption of the $E_s - q_c$ relationship. ■

Settlement of Foundation on Sand Based on Standard Penetration Resistance

I. Meyerhof's Method

$$S_e(\text{mm}) = \frac{1.25 q_{net}(\text{kN/m}^2)}{N_{60} F_d} \quad (\text{for } B \le 1.22 \text{ m})$$

$$S_e(\text{mm}) = \frac{2 q_{net}(\text{kN/m}^2)}{N_{60} F_d} \left(\frac{B}{B + 0.3} \right)^2 \quad (\text{for } B > 1.22 \text{ m})$$

$$q_{net}(\text{kN/m}^2) = \frac{N_{60}}{0.05} F_d \left(\frac{S_e}{25} \right) \quad (\text{for } B \le 1.22 \text{ m})$$

$$q_{net}(\text{kN/m}^2) = \frac{N_{60}}{0.08} \left(\frac{B + 0.3}{B} \right)^2 F_d \left(\frac{S_e}{25} \right) \quad (\text{for } B > 1.22 \text{ m})$$

The N_{60} is the standard penetration resistance between the bottom of the foundation and $2B$ below the bottom.

Settlement of Foundation on Sand Based on Standard Penetration Resistance

II. Burland and Burbidge's Method

1. Variation of Standard Penetration Number with Depth:

Obtain the field penetration numbers N_{60} with depth at the location of the foundation The following adjustments of N_{60} may be necessary:

For gravel or sandy gravel

$$N_{60(a)} = 1.25\, N_{60}$$

For fine sand or silty sand below the groundwater table and N_{60} >15,

$N_{60(a)} = 15+0.5(N_{60} - 15)$

where $N_{60(a)}$ = adjusted N_{60} value.

2. Determination of Depth of Stress Influence (z'):

In determining the depth of stress influence, the following three cases may arise:

Case I. If N_{60} [or $N_{60(a)}$] is approximately constant with depth, calculate z' from

$$\frac{z'}{B_R} = 1.4\left(\frac{B}{B_R}\right)^{0.75}$$

where

$$B_R = \text{reference width}\begin{cases} = 1 \text{ ft (if } B \text{ is in ft)} \\ = 0.3 \text{ m (if } B \text{ is in m)} \end{cases}$$

B = width of the actual foundation

Case II. If N_{60} [or $N_{60(a)}$] is increasing with depth, use the above Equation.

Case III. If N_{60} [or $N_{60(a)}$] is decreasing with depth, $z' = 2B$ or to the bottom of soft soil layer measured from the bottom of the foundation (whichever is smaller).

Settlement of Foundation on Sand Based on Standard Penetration Resistance

II. Burland and Burbidge's Method

3. Calculation of Elastic Settlement S_e

The elastic settlement of the foundation, S_e, can be calculated from

$$\frac{S_e}{B_R} = \alpha_1 \alpha_2 \alpha_3 \left[\frac{1.25\left(\frac{L}{B}\right)}{0.25 + \left(\frac{L}{B}\right)} \right]^2 \left(\frac{B}{B_R}\right)^{0.7} \left(\frac{q'}{p_a}\right)$$

where

α_1 = a constant
α_2 = compressibility index
α_3 = correction for the depth of influence
p_a = atmospheric pressure = 100 kN/m^2
L = length of the foundation

Table 7.7 Summary of q', α_1, α_2, and α_3

Soil type	q'	α_1	α_2	α_3
Normally consolidated sand	q_{net}	0.14	$\frac{1.71}{[\bar{N}_{60} \text{ or } \bar{N}_{60(a)}]^{1.4}}$	$\alpha_3 = \frac{H}{z'}\left(2 - \frac{H}{z'}\right)$ (if $H \le z'$)
Overconsolidated sand ($q_{net} \le \sigma'_c$)	q_{net}	0.047	$\frac{0.57}{[\bar{N}_{60} \text{ or } \bar{N}_{60(a)}]^{1.4}}$	or $\alpha_3 = 1$ (if $H > z'$)
where σ'_c = preconsolidation pressure				where H = depth of compressible layer
Overconsolidated sand ($q_{net} > \sigma'_c$)	$q_{net} - 0.67\sigma'_c$	0.14	$\frac{0.57}{[\bar{N}_{60} \text{ or } \bar{N}_{60(a)}]^{1.4}}$	

Table 7.7 summarizes the values of q', α_1, α_2, and α_3 to be used in Equation for various types of soils. Note that, in this table, $[\bar{N}_{60} \text{ or } \bar{N}_{60(a)}]$ = average value of N_{60} [or $N_{60(a)}$ in the depth of stress influence.

Example 7.6 & 7.7

Example 7.6

A shallow foundation measuring 1.75 m × 1.75 m is to be constructed over a layer of sand. Given $D_f = 1$ m; N_{60} is generally increasing with depth; $\overline{N}_{60}$ in the depth of stress influence = 10, $q_{net} = 120$ kN/m^2. The sand is normally consolidated. Estimate the elastic settlement of the foundation. Use the Burland and Burbidge method.

Solution

From Eq. (7.44),

$$\frac{z'}{B_R} = 1.4\left(\frac{B}{B_R}\right)^{0.75}$$

Depth of stress influence,

$$z' = 1.4\left(\frac{B}{B_R}\right)^{0.75} B_R = (1.4)(0.3)\left(\frac{1.75}{0.3}\right)^{0.75} \approx 1.58 \text{ m}$$

From Eq. (7.45),

$$\frac{S_e}{B_R} = \alpha_1\alpha_2\alpha_3\left[\frac{1.25\left(\frac{L}{B}\right)}{0.25 + \left(\frac{L}{B}\right)}\right]^2\left(\frac{B}{B_R}\right)^{0.7}\left(\frac{q'}{p_a}\right)$$

For normally consolidated sand (Table 7.6),

$$\alpha_1 = 0.14$$
$$\alpha_2 = \frac{1.71}{(\overline{N}_{60})^{1.4}} = \frac{1.71}{(10)^{1.4}} = 0.068$$
$$\alpha_3 = 1$$
$$q' = q_{net} = 120 \text{ kN/m}^2$$

So,

$$\frac{S_e}{0.3} = (0.14)(0.068)(1)\left[\frac{(1.25)\left(\frac{1.75}{1.75}\right)}{0.25 + \left(\frac{1.75}{1.75}\right)}\right]^2\left(\frac{1.75}{0.3}\right)^{0.7}\left(\frac{120}{100}\right)$$

$$S_e \approx 0.0118 \text{ m} = \mathbf{11.8\ mm}$$

Example 7.7

Solve Example 7.6 using Meyerhof's method.

Solution

From Eq. (7.41),

$$S_e = \frac{2q_{net}}{(\overline{N}_{60})(F_d)}\left(\frac{B}{B+0.3}\right)^2$$

$$F_d = 1 + 0.33(D_f/B) = 1 + 0.33(1/1.75) = 1.19$$

$$S_e = \frac{(2)(120)}{(10)(1.19)}\left(\frac{1.75}{1.75+0.3}\right)^2 = \mathbf{14.7\ mm}$$

Effect of the Rise of Water Table on Elastic Settlement

Terzaghi (1943) suggested that the submergence of soil mass reduces the soil stiffness by about half, which in turn doubles the settlement. In most cases of foundation design, it is considered that, if the ground water table is located 1.5*B* to 2*B* below the bottom of the foundation, it will not have any effect on the settlement. The total elastic settlement (S'_e) due to the rise of the ground water table can be given as:

$$S'_e = S_e C_w$$

where

S_e = elastic settlement before the rise of ground water table

C_w = water correction factor

- Peck, Hansen, and Thornburn (1974):

$$C_w = \frac{1}{0.5 + 0.5\left(\dfrac{D_w}{D_f + B}\right)} \geq 1$$

- Teng (1982):

$$C_w = \frac{1}{0.5 + 0.5\left(\dfrac{D_w - D_f}{B}\right)} \leq 2 \quad \left(\begin{array}{l}\text{for water table below the} \\ \text{base of the foundation}\end{array}\right)$$

- Bowles (1977):

$$C_w = 2 - \left(\frac{D_w}{D_f + B}\right)$$

Example 7.9

Example 7.9

Consider the shallow foundation given in Example 7.6. Due to flooding, the ground water table rose from $D_w = 4$ m to 2 m (Figure 7.19). Estimate the total elastic settlement S_e' after the rise of the water table. Use Eq. (7.60).

Solution

From Eq. (7.59),

$$S_e' = S_e C_w$$

From Eq. (7.60),

$$C_w = \frac{1}{0.5 + 0.5\left(\dfrac{D_w}{D_f + B}\right)} = \frac{1}{0.5 + 0.5\left(\dfrac{2}{1 + 1.75}\right)} = 1.158$$

Hence,

$$S_e' = (11.8 \text{ mm})(1.158) = \mathbf{13.66 \text{ mm}}$$

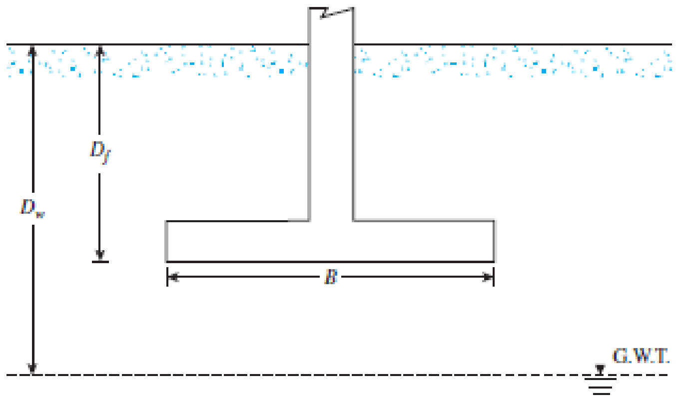
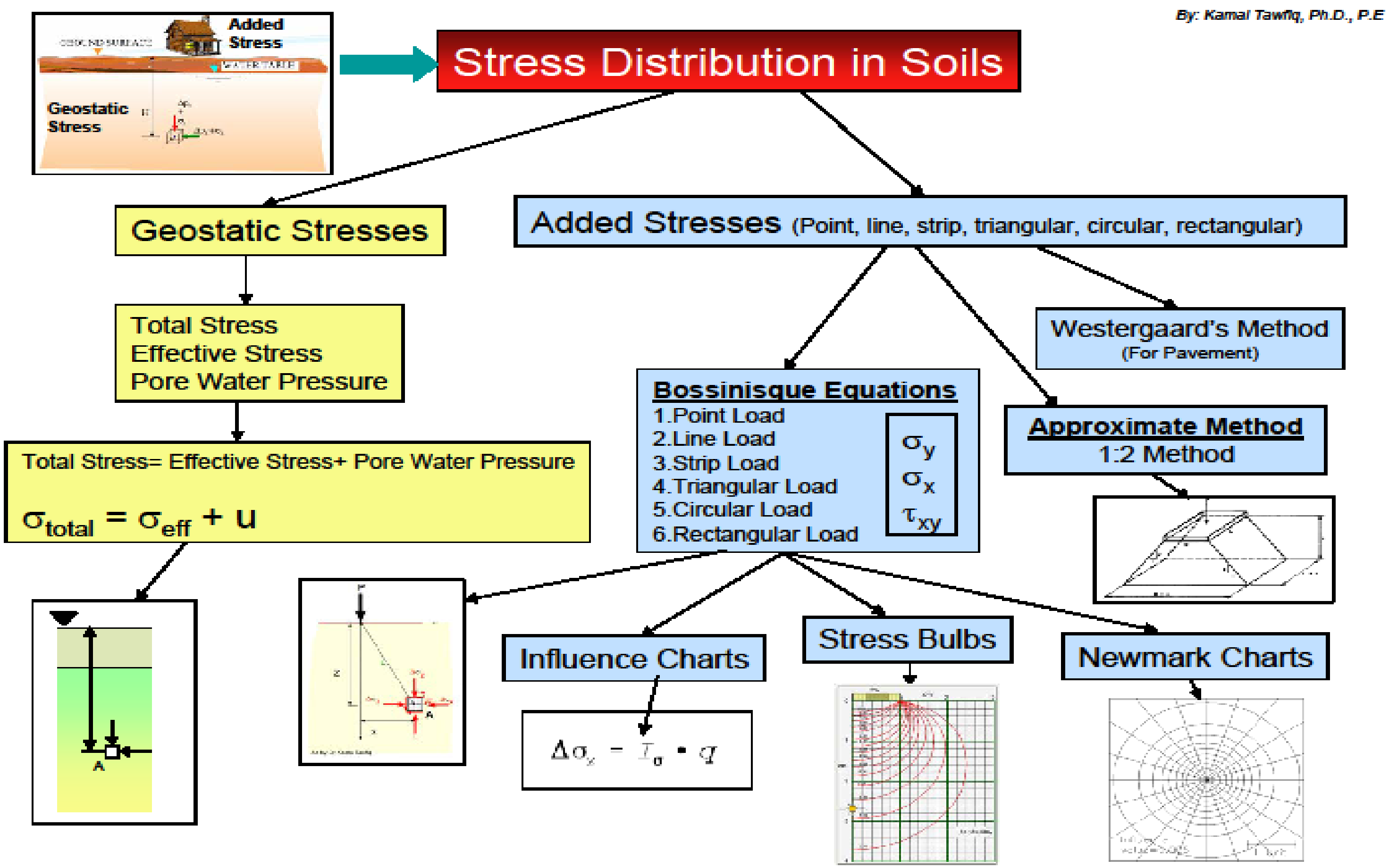


Figure 7.19 Effect of rise of ground water table on elastic settlement in granular soil

Stresses Distribution in Soils



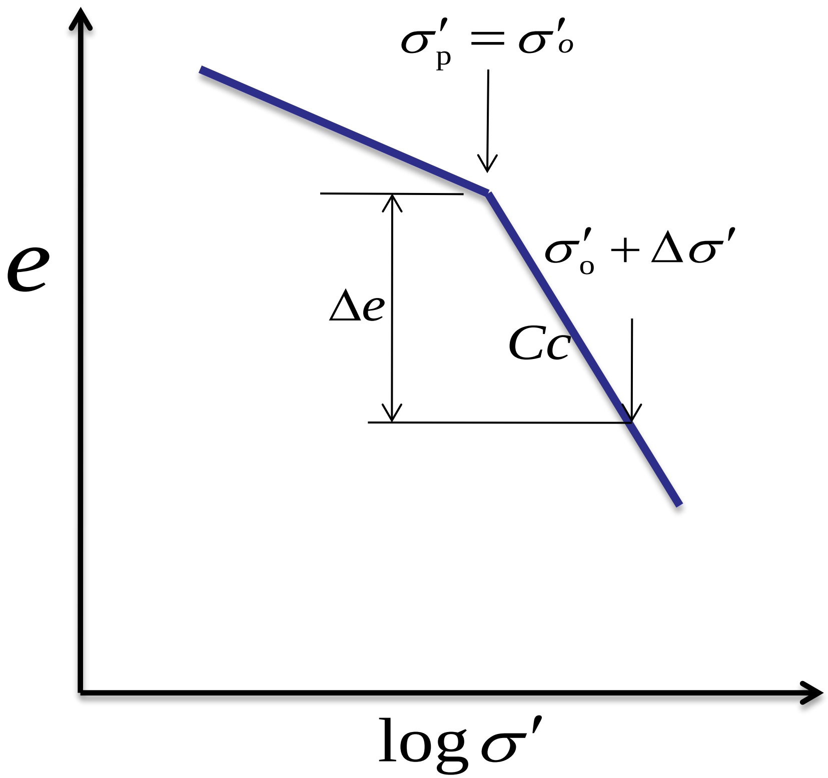
Calculation of Primary Consolidation Settlement

a) Normally Consolidated Clay ($\sigma'_0 = \sigma'_c$)

$$S_c = \frac{\Delta e}{1+e_o} H$$

$$\Delta e = C_c \log\left(\frac{\sigma'_o + \Delta\sigma}{\sigma'_o}\right)$$

$$\boxed{S_c = \frac{C_c H}{1 + e_o} \log\left(\frac{\sigma'_o + \Delta\sigma'}{\sigma'_o}\right)}$$



Calculation of Primary Consolidation Settlement

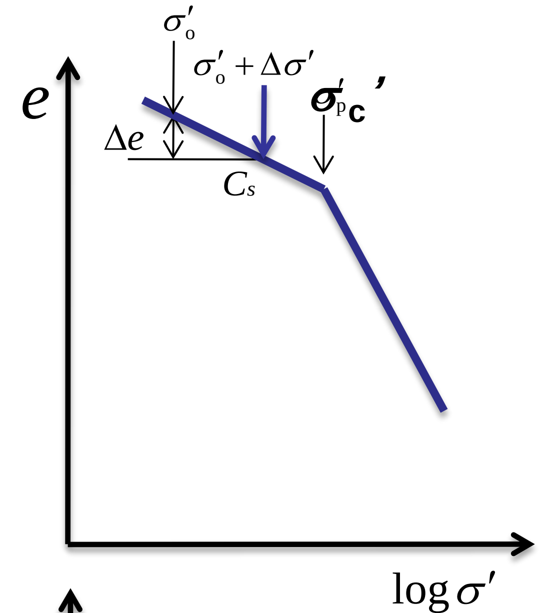
b) Overconsolidated Clays

$$S_c = \frac{\Delta e}{1+e_o} H$$

Case I: $\sigma'_0 + \Delta\sigma' \leq \sigma_c'$

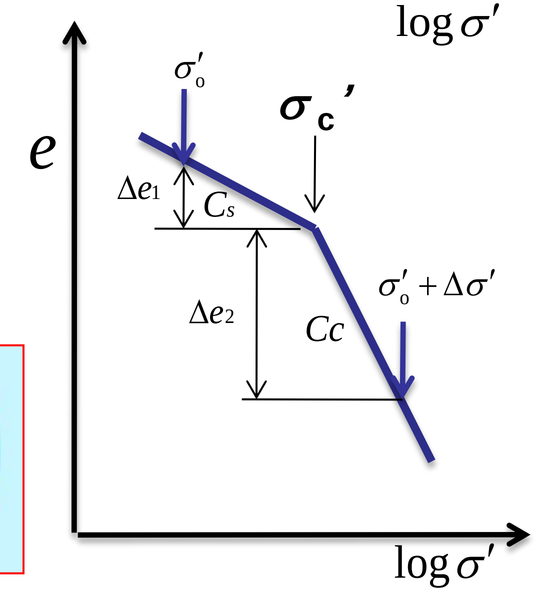
$$\Delta e = C_s[\log(\sigma'_o + \Delta\sigma') - \log \sigma'_o]$$

$$\boxed{S_c = \frac{C_s H}{1 + e_o} \log\left(\frac{\sigma'_o + \Delta\sigma'}{\sigma'_o}\right)}$$



Case II: $\sigma'_0 + \Delta\sigma' > \sigma_c'$

$$\boxed{S_c = \frac{C_s H}{1 + e_o} \log \frac{\sigma'_c}{\sigma'_o} + \frac{C_c H}{1 + e_o} \log\left(\frac{\sigma'_o + \Delta\sigma'}{\sigma'_c}\right)}$$



Summary of calculation procedure

1. Calculate σ'_o at the middle of the clay layer
2. Determine σ'_c from the e-log σ' plot (if not given)
3. Determine whether the clay is N.C. or O.C.
4. Calculate $\Delta\sigma$
5. Use the appropriate equation

- If N.C.

$$S_c = \frac{C_c H}{1 + e_o} \log\left(\frac{\sigma'_o + \Delta\sigma'}{\sigma'_o}\right)$$

- If O.C.

$$S_c = \frac{C_s H}{1 + e_o} \log\left(\frac{\sigma'_o + \Delta\sigma'}{\sigma'_o}\right)$$

$\underline{If\ \ \sigma'_o + \Delta\sigma \le \sigma'_c}$

$$S_c = \frac{C_s H}{1 + e_o} \log\frac{\sigma'_c}{\sigma'_o} + \frac{C_c H}{1 + e_o} \log\left(\frac{\sigma'_o + \Delta\sigma'}{\sigma'_c}\right)$$

$\underline{If\ \ \sigma'_o + \Delta\sigma > \sigma'_c}$

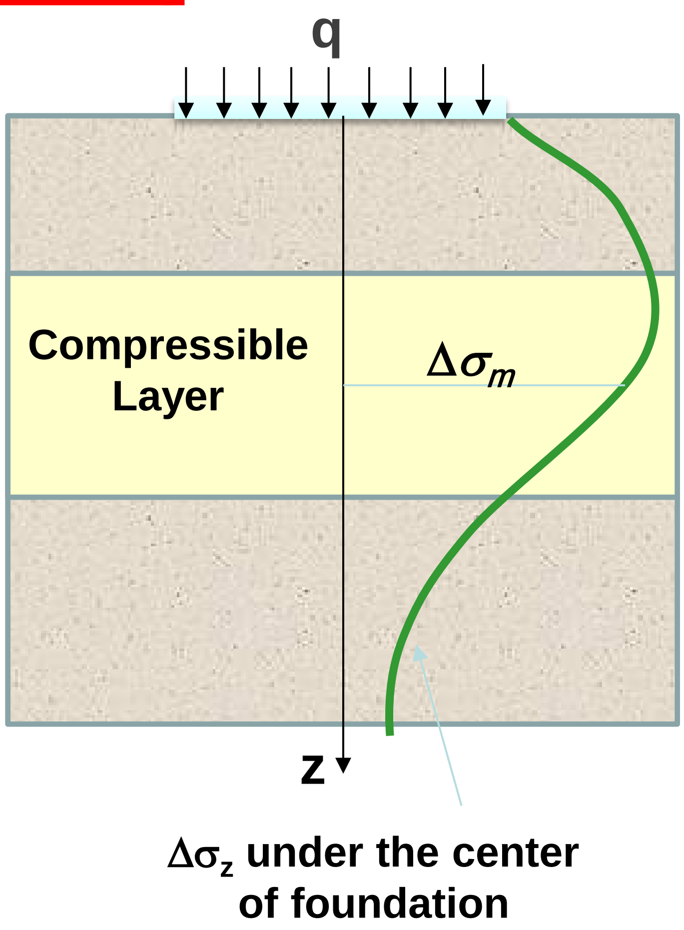
Nonlinear pressure increase

Approach 1: Middle of layer (midpoint rule)

- For settlement calculation, the pressure increase $\Delta\sigma_z$ can be approximated as :

$$\boxed{\Delta\sigma_z = \Delta\sigma_m}$$

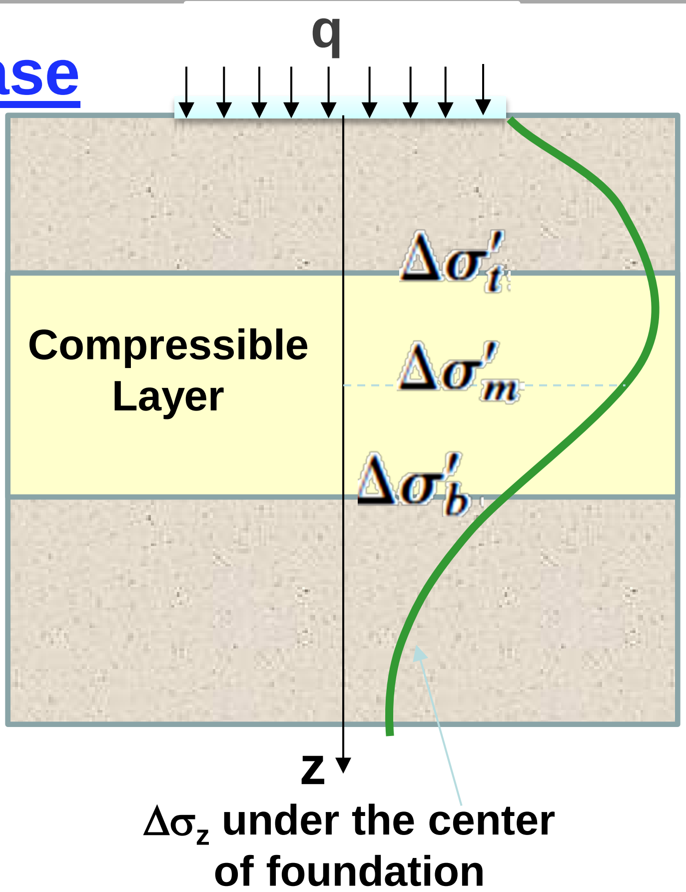
where $\Delta\sigma_m$ represent the increase in the effective pressure in the middle of the layer.



Nonlinear pressure increase

Approach 2: Average pressure increase

- **For settlement calculation we will use the average pressure increase $\Delta\sigma_{av}$, using weighted average method (Simpson's rule):**



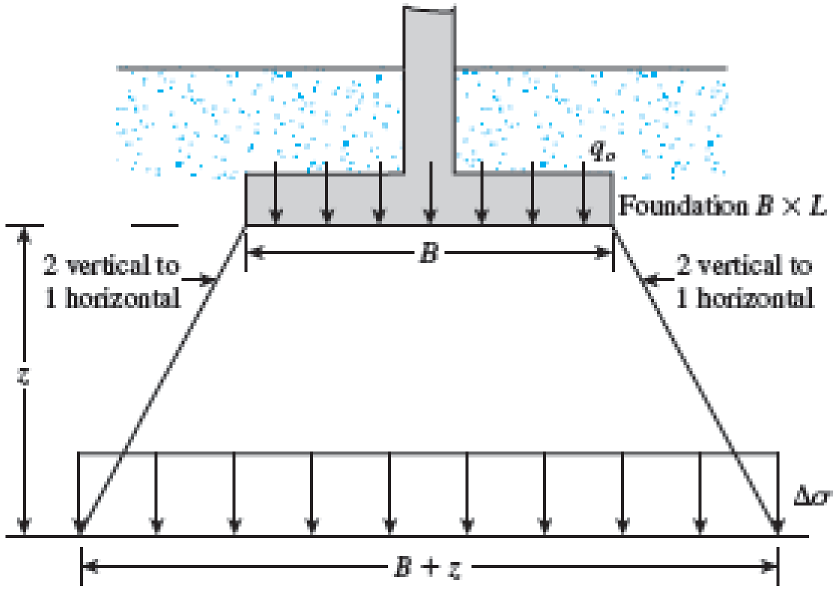
$$\Delta\sigma'_{av} = \frac{\Delta\sigma'_t + 4\Delta\sigma'_m + \Delta\sigma'_b}{6}$$

where $\Delta\sigma_t$, $\Delta\sigma_m$ and $\Delta\sigma_b$ represent the increase in the pressure at the top, middle, and bottom of the clay, respectively, under the center of the footing.

Stress from Approximate Method

2:1 Method

$$\Delta\sigma = \frac{q_o \times B \times L}{(B + z)(L + z)}$$



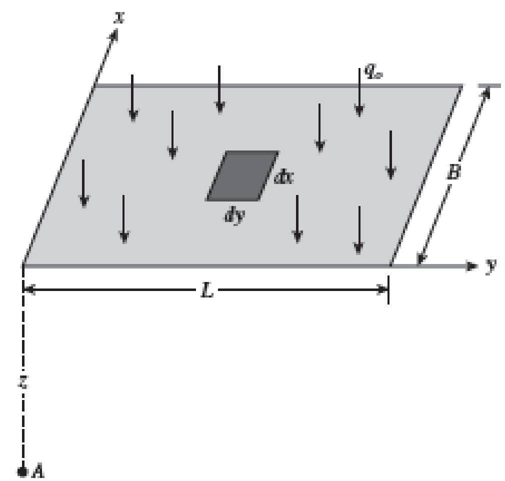
Stress below a Rectangular Area

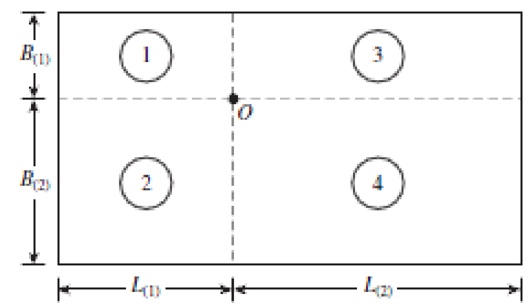
$$\Delta\sigma = \int_{y=0}^{L}\int_{x=0}^{B} \frac{3q_o\,(dx\,dy)z^3}{2\pi(x^2+y^2+z^2)^{5/2}} = q_o I$$

$$I = \text{influence factor} = \frac{1}{4\pi}\left(\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+m^2n^2+1}\cdot\frac{m^2+n^2+2}{m^2+n^2+1} + \tan^{-1}\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+1-m^2n^2}\right)$$

$$m = \frac{B}{z}$$

$$n = \frac{L}{z}$$





Stress below a Rectangular Area

Table 6.4 Variation of Influence Value I [Eq. (6.10)][a]

m	n = 0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4
0.1	0.00470	0.00917	0.01323	0.01678	0.01978	0.02223	0.02420	0.02576	0.02698	0.02794	0.02926	0.03007
0.2	0.00917	0.01790	0.02585	0.03280	0.03866	0.04348	0.04735	0.05042	0.05283	0.05471	0.05733	0.05894
0.3	0.01323	0.02585	0.03735	0.04742	0.05593	0.06294	0.06858	0.07308	0.07661	0.07938	0.08323	0.08561
0.4	0.01678	0.03280	0.04742	0.06024	0.07111	0.08009	0.08734	0.09314	0.09770	0.10129	0.10631	0.10941
0.5	0.01978	0.03866	0.05593	0.07111	0.08403	0.09473	0.10340	0.11035	0.11584	0.12018	0.12626	0.13003
0.6	0.02223	0.04348	0.06294	0.08009	0.09473	0.10688	0.11679	0.12474	0.13105	0.13605	0.14309	0.14749
0.7	0.02420	0.04735	0.06858	0.08734	0.10340	0.11679	0.12772	0.13653	0.14356	0.14914	0.15703	0.16199
0.8	0.02576	0.05042	0.07308	0.09314	0.11035	0.12474	0.13653	0.14607	0.15371	0.15978	0.16843	0.17389
0.9	0.02698	0.05283	0.07661	0.09770	0.11584	0.13105	0.14356	0.15371	0.16185	0.16835	0.17766	0.18357
1.0	0.02794	0.05471	0.07938	0.10129	0.12018	0.13605	0.14914	0.15978	0.16835	0.17522	0.18508	0.19139
1.2	0.02926	0.05733	0.08323	0.10631	0.12626	0.14309	0.15703	0.16843	0.17766	0.18508	0.19584	0.20278
1.4	0.03007	0.05894	0.08561	0.10941	0.13003	0.14749	0.16199	0.17389	0.18357	0.19139	0.20278	0.21020
1.6	0.03058	0.05994	0.08709	0.11135	0.13241	0.15028	0.16515	0.17739	0.18737	0.19546	0.20731	0.21510
1.8	0.03090	0.06058	0.08804	0.11260	0.13395	0.15207	0.16720	0.17967	0.18986	0.19814	0.21032	0.21836
2.0	0.03111	0.06100	0.08867	0.11342	0.13496	0.15326	0.16856	0.18119	0.19152	0.19994	0.21235	0.22058
2.5	0.03138	0.06155	0.08948	0.11450	0.13628	0.15483	0.17036	0.18321	0.19375	0.20236	0.21512	0.22364
3.0	0.03150	0.06178	0.08982	0.11495	0.13684	0.15550	0.17113	0.18407	0.19470	0.20341	0.21633	0.22499
4.0	0.03158	0.06194	0.09007	0.11527	0.13724	0.15598	0.17168	0.18469	0.19540	0.20417	0.21722	0.22600
5.0	0.03160	0.06199	0.09014	0.11537	0.13737	0.15612	0.17185	0.18488	0.19561	0.20440	0.21749	0.22632
6.0	0.03161	0.06201	0.09017	0.11541	0.13741	0.15617	0.17191	0.18496	0.19569	0.20449	0.21760	0.22644
8.0	0.03162	0.06202	0.09018	0.11543	0.13744	0.15621	0.17195	0.18500	0.19574	0.20455	0.21767	0.22652
10.0	0.03162	0.06202	0.09019	0.11544	0.13745	0.15622	0.17196	0.18502	0.19576	0.20457	0.21769	0.22654
∞	0.03162	0.06202	0.09019	0.11544	0.13745	0.15623	0.17197	0.18502	0.19577	0.20458	0.21770	0.22656

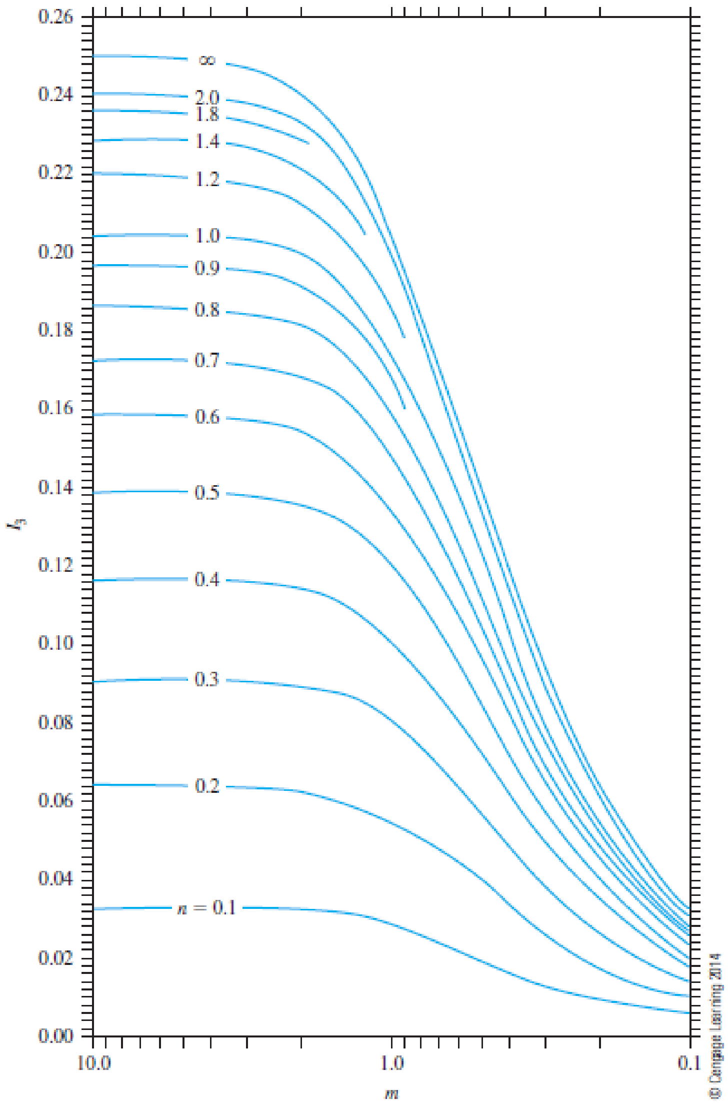
$$m = \frac{B}{z}$$

$$n = \frac{L}{z}$$

Stress below a Rectangular Area

$$m = \frac{B}{z}$$

$$n = \frac{L}{z}$$



Vertical Stress below the Center of a Uniformly Loaded Circular Area

$$\Delta\sigma_z = q\left\{1 - \frac{1}{[(R/z)^2 + 1]^{3/2}}\right\}$$

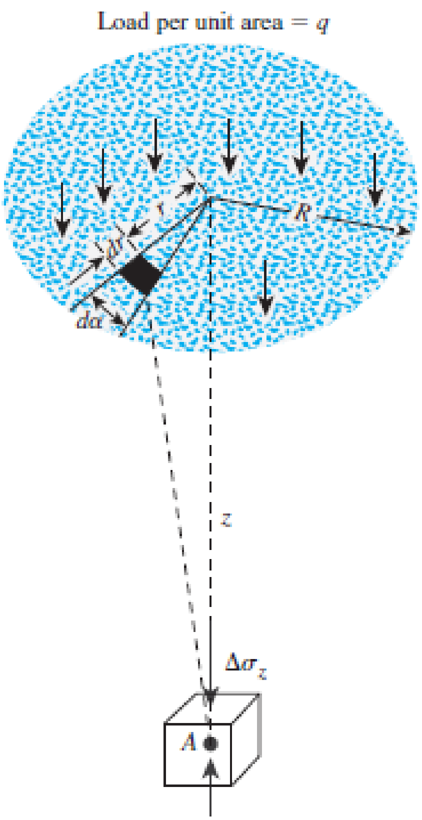


Table 10.6 Variation of $\Delta\sigma_z/q$ with z/R [Eq. (10.26)]

z/R	$\Delta\sigma_z/q$	z/R	$\Delta\sigma_z/q$
0	1	1.0	0.6465
0.02	0.9999	1.5	0.4240
0.05	0.9998	2.0	0.2845
0.10	0.9990	2.5	0.1996
0.2	0.9925	3.0	0.1436
0.4	0.9488	4.0	0.0869
0.5	0.9106	5.0	0.0571
0.8	0.7562		

Influence Chart for Vertical Pressure

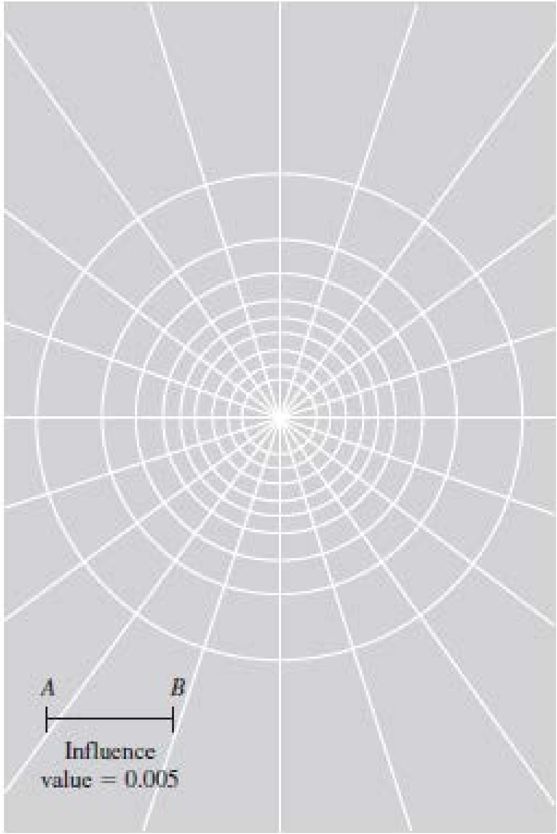
The procedure for obtaining vertical pressure at any point below a loaded area is as follows:

1. Determine the depth z below the uniformly loaded area at which the stress increase is required.
2. Plot the plan of the loaded area with a scale of z equal to the unit length of the chart ($\overline{AB}$).
3. Place the plan (plotted in step 2) on the influence chart in such a way that the point below which the stress is to be determined is located at the center of the chart.
4. Count the number of elements (M) of the chart enclosed by the plan of the loaded area.

The increase in the pressure at the point under consideration is given by

$$\Delta\sigma_z = (IV)qM$$

where IV = influence value
q = pressure on the loaded area



Three-Dimensional Effect on Primary Consolidation Settlement

$$S_{c(p)} = K_{\mathrm{cr}(} \; S_{c(p)-\mathrm{oed}}$$

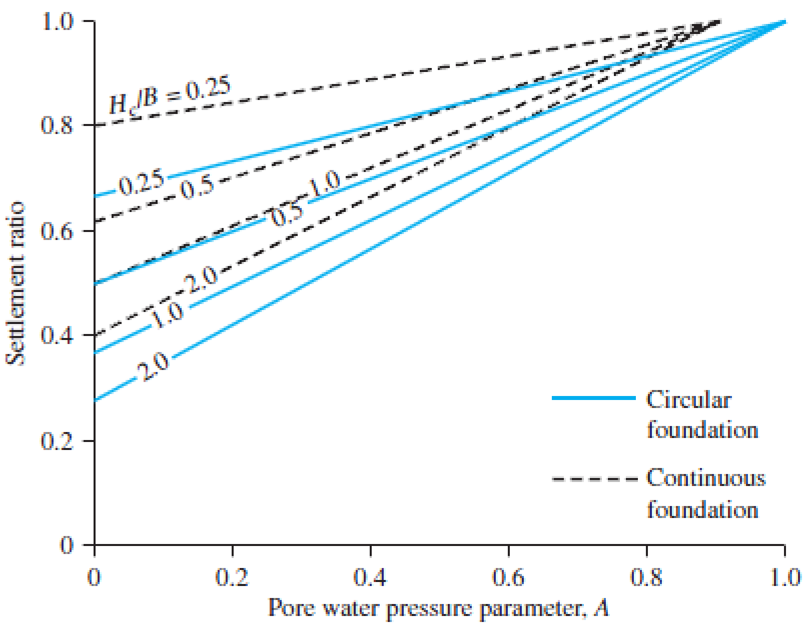


Figure 7.22 Settlement ratios for circular (K_{cir}) and continuous (K_{str}) foundations

$$S_{c(p)} = K_{\mathrm{cr(OC)}} \, S_{c(p)-\mathrm{oed}}$$

Table 7.9 Variation of $K_{\mathrm{cr(OC)}}$ with OCR and B/H_c

	$K_{\mathrm{cr(OC)}}$		
OCR	$B/H_c = 4.0$	$B/H_c = 1.0$	$B/H_c = 0.2$
1	1	1	1
2	0.986	0.957	0.929
3	0.972	0.914	0.842
4	0.964	0.871	0.771
5	0.950	0.829	0.707
6	0.943	0.800	0.643
7	0.929	0.757	0.586
8	0.914	0.729	0.529
9	0.900	0.700	0.493
10	0.886	0.671	0.457
11	0.871	0.643	0.429
12	0.864	0.629	0.414
13	0.857	0.614	0.400
14	0.850	0.607	0.386
15	0.843	0.600	0.371
16	0.843	0.600	0.357

Example 7.10

Example 7.10

A plan of a foundation 1 m × 2 m is shown in Figure 7.23. Estimate the consolidation settlement of the foundation, taking into account the three-dimensional effect. Given: $A = 0.6$.

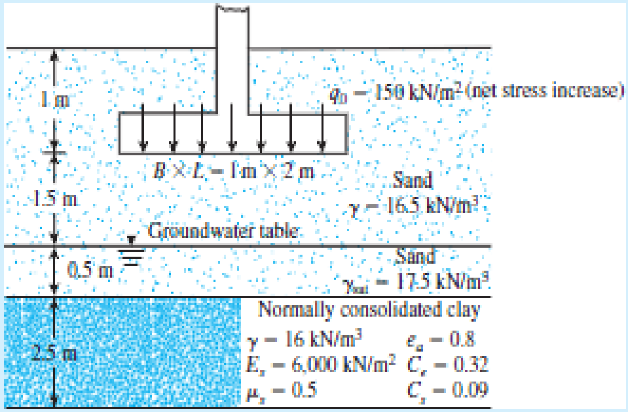


Figure 7.23 Calculation of primary consolidation settlement for a foundation

Solution

The clay is normally consolidated. Thus,

$$S_{c(p)-\text{oed}} = \frac{C_c H_c}{1 + e_o} \log \frac{\sigma'_o + \Delta\sigma'_{av}}{\sigma'_o}$$

so

$$\sigma'_o = (2.5)(16.5) + (0.5)(17.5 - 9.81) + (1.25)(16 - 9.81)$$
$$= 41.25 + 3.85 + 7.74 = 52.84\ \text{kN/m}^2$$

From Eq. (6.29),

$$\Delta\sigma'_{av} = \tfrac{1}{6}(\Delta\sigma'_t + 4\Delta\sigma'_m + \Delta\sigma'_b)$$

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

$m_1 = L/B$	z(m)	$z/(B/2) = n_1$	I_c[a]	$\Delta\sigma' = q_o I_c$[b]
2	2	4	0.190	$28.5 = \Delta\sigma'_t$
2	2 + 2.5/2 = 3.25	6.5	≈ 0.085	$12.75 = \Delta\sigma'_m$
2	2 + 2.5 = 4.5	9	0.045	$6.75 = \Delta\sigma'_b$

[a]Table 6.5
[b]Eq. (6.14)

Now,

$$\Delta\sigma'_{av} = \tfrac{1}{6}(28.5 + 4 \times 12.75 + 6.75) = 14.38\ \text{kN/m}^2$$

Example 7.10

so

$$S_{c(p)-\text{oed}} = \frac{(0.32)(2.5)}{1 + 0.8} \log\left(\frac{52.84 + 14.38}{52.84}\right) = 0.0465 \text{ m}$$

$$= \mathbf{46.5 \text{ mm}}$$

Now assuming that the 2:1 method of stress increase (see Figure 6.7) holds good, the area of distribution of stress at the top of the clay layer will have dimensions

$$B' = \text{width} = B + z = 1 + (1.5 + 0.5) = 3 \text{ m}$$

and

$$L' = \text{width} = L + z = 2 + (1.5 + 0.5) = 4 \text{ m}$$

The diameter of an equivalent circular area, B_{eq}, can be given as

$$\frac{\pi}{4} B_{eq}^2 = B'L'$$

so that

$$B_{eq} = \sqrt{\frac{4B'L'}{\pi}} = \sqrt{\frac{(4)(3)(4)}{\pi}} = 3.91 \text{ m}$$

Also,

$$\frac{H_c}{B_{eq}} = \frac{2.5}{3.91} = 0.64$$

From Figure 7.22, for $A = 0.6$ and $H_c/B_{eq} = 0.64$, the magnitude of $K_{cr} \approx 0.78$. Hence,

$$S_{e(p)} = K_{cr} S_{e(p)-\text{oed}} = (0.78)(46.5) \approx \mathbf{36.3 \text{ mm}}$$

Secondary Consolidation Settlement

- **The magnitude of the secondary consolidation can be calculated as:**

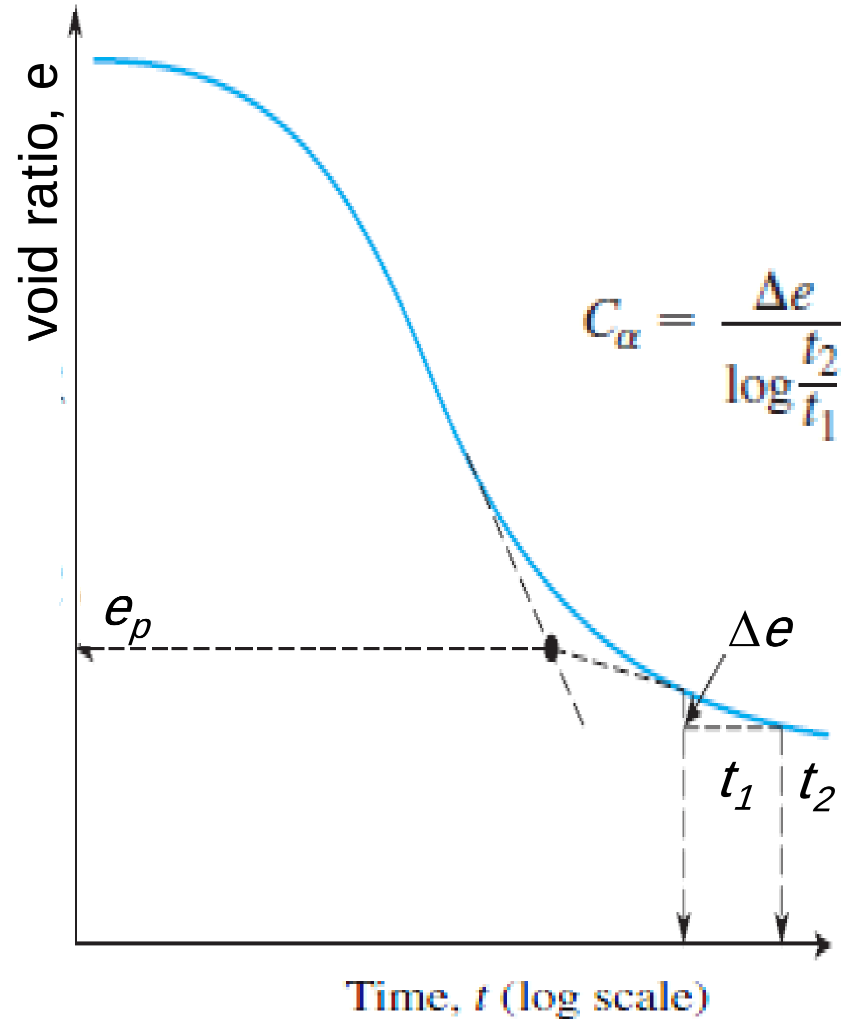
$$\boxed{S_s = \frac{H}{1+e_p}\Delta e}$$

e_p void ratio at the end of primary consolidation,
H thickness of clay layer.

$$\Delta e = C_{\propto} log\,(t_2/t_1)$$

C_α = coefficient of secondary compression

$$\boxed{S_s = \frac{C_\alpha H}{1+e_p} log\left(\frac{t_2}{t_1}\right)}$$



Secondary Consolidation Settlement

$$S_{c(s)} = C'_{\alpha} H_c \log(t_2/t_1)$$

$C'_{\alpha} = C_{\alpha}/(1 + e_p)$
e_p = void ratio at the end of primary consolidation
H_c = thickness of clay layer

Mesri (1973) correlated C'_{α} with the natural moisture content (w) of several soils, from which it appears that

$$C'_{\alpha} \approx 0.0001w \qquad (7.72)$$

where w = natural moisture content, in percent. For most overconsolidated soils, C'_{α} varies between 0.0005 to 0.001.

Mesri and Godlewski (1977) compiled the magnitude of C_{α}/C_c (C_c = compression index) for a number of soils. Based on their compilation, it can be summarized that

- For inorganic clays and silts:

$$C_{\alpha}/C_c \approx 0.04 \pm 0.01$$

- For organic clays and silts:

$$C_{\alpha}/C_c \approx 0.05 \pm 0.01$$

- For peats:

$$C_{\alpha}/C_c \approx 0.075 \pm 0.01$$

Example 7.11

Example 7.11

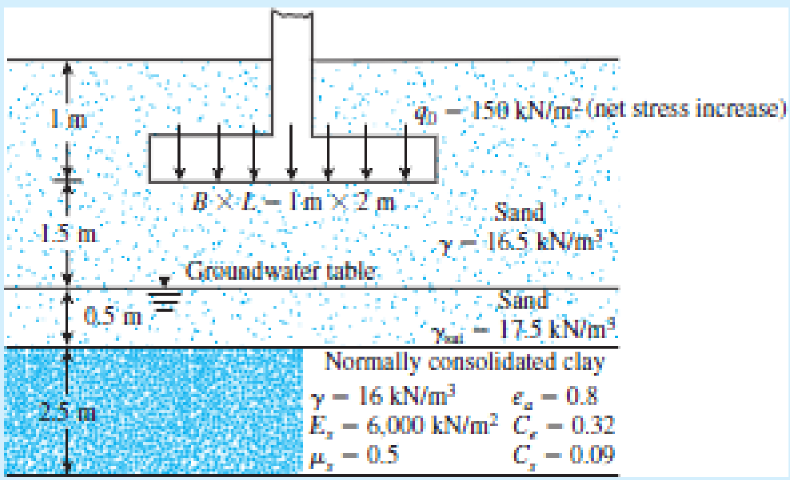
Refer to Example 7.10. Given for the clay layer: $C_\alpha = 0.02$. Estimate the total consolidation settlement five years after the completion of the primary consolidation settlement. (*Note:* Time for completion of primary consolidation settlement is 1.3 years).

Solution

From Eq. (2.53),

$$C_c = \frac{e_1 - e_2}{\log\left(\frac{\sigma_2'}{\sigma_1'}\right)}$$

For this problem, $e_1 - e_2 = \Delta e$.



Referring to Example 7.10, we have

$$\sigma_2' = \sigma_o' + \Delta\sigma' = 52.84 + 14.38 = 67.22 \text{ kN/m}^2$$
$$\sigma_1' = \sigma_o' = 52.84 \text{ kN/m}^2$$
$$C_c = 0.32$$

Hence,

$$\Delta e = C_c \log\left(\frac{\sigma_o' + \Delta\sigma}{\sigma_o'}\right) = 0.32 \log\left(\frac{67.22}{52.84}\right) = 0.0335$$

Given: $e_o = 0.8$. Hence,

$$e_p = e_o - e = 0.8 - 0.0335 = 0.7665$$

From Eq. (7.71),

$$C_\alpha' = \frac{C_\alpha}{1 + e_p} = \frac{0.02}{1 + 0.7665} = 0.0113$$

From Eq. (7.70),

$$S_{c(s)} = C_\alpha' H_c \log\left(\frac{t_2}{t_1}\right)$$

Note: $t_1 = 1.3$ years; $t_2 = 1.3 + 5 = 6.3$ years.
Thus,

$$S_{c(s)} = (0.0113)(2.5 \text{ m}) \log\left(\frac{6.3}{1.3}\right) = 0.0194 \text{ m} = 19.4 \text{ mm}$$

Total consolidation settlement is

$$\underbrace{36.3 \text{ mm}}_{\substack{\uparrow \\ \text{Example 7.10} \\ \text{(Primary} \\ \text{consolidation} \\ \text{settlement)}}} + 19.4 = \mathbf{55.7 \text{ m}}$$

■

The Plate Load Test

The ultimate bearing capacity of foundations, as well as the allowable bearing capacity based on tolerable settlement considerations, can be **effectively** determined from the field load test, generally referred as **plate load test.**

Plate Properties:

The plate used in this test is made of steel and have the following dimensions:
If the plate is **circular**, the diameter will be (150mm to 762mm) with 25mm thickness.
If the plate is **square**, the dimensions are (305mm x 305mm) with 25mm thickness.

Test Mechanism:

- ❑ To conduct a test, a hole is excavated with a minimum diameter of 4B (B is the diameter of the test plate) to depth D_f (depth of proposed foundation).
- ❑ The plate is at the center of the hole, and the load is applied on the plate and increased gradually.
- ❑ As the load increase, the settlement of the plate is observed on dial gauge.
- ❑ The test should be conducted until failure, or the settlement of the plate became 25mm.
- ❑ The value of load at which the test is finished is the ultimate load can be resisted by the plate
- ❑ Divide the ultimate load on the plate area to get ultimate bearing capacity of the plate $q_{u(P)}$.

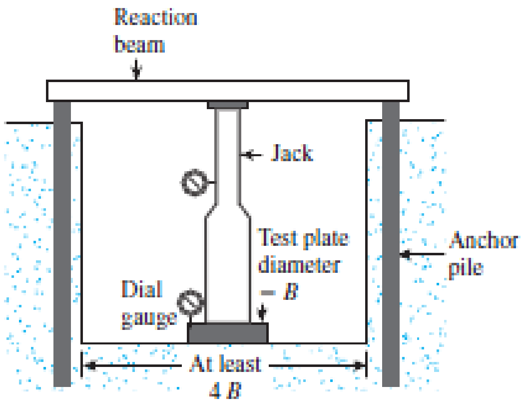
The Plate Load Test

For tests in clay,

$$q_{u(F)} = q_{u(P)}$$

where

$q_{u(F)}$ = ultimate bearing capacity of the proposed foundation
$q_{u(P)}$ = ultimate bearing capacity of the test plate



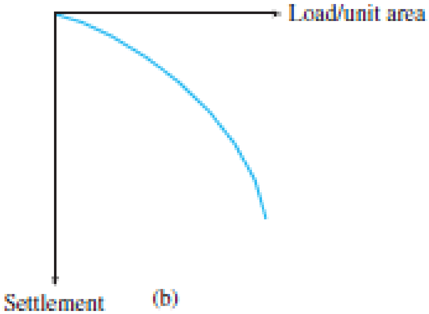
(a)

For tests in sandy soils,

$$q_{u(F)} = q_{u(P)} \frac{B_F}{B_P}$$

where

B_F = width of the foundation
B_P = width of the test plate



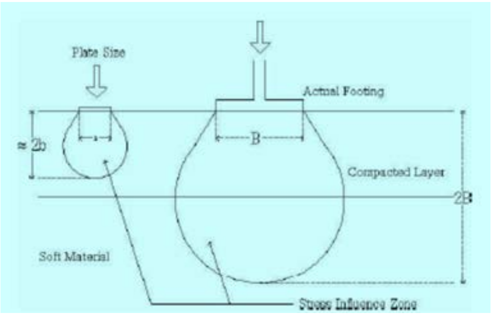
(b)

The allowable bearing capacity of a foundation, based on settlement considerations and for a given intensity of load, q_o, is

$$S_F = S_P \frac{B_F}{B_P} \quad \text{(for clayey soil)} \qquad (7.75)$$

and

$$S_F = S_P \left(\frac{2B_F}{B_F + B_P} \right)^2 \quad \text{(for sandy soil)} \qquad (7.76)$$



Presumptive Bearing Capacity

$$\left.\begin{array}{l}\text{Presumptive Bearing Capacity} \\ \\ \text{Tolerable Settlement of Buildings}\end{array}\right\} \text{Building Codes}$$