Chapter 9

Settlement of Shallow Foundations

Omitted parts:

Section 9.8 , 9.9, 9.15 ,9.16

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)

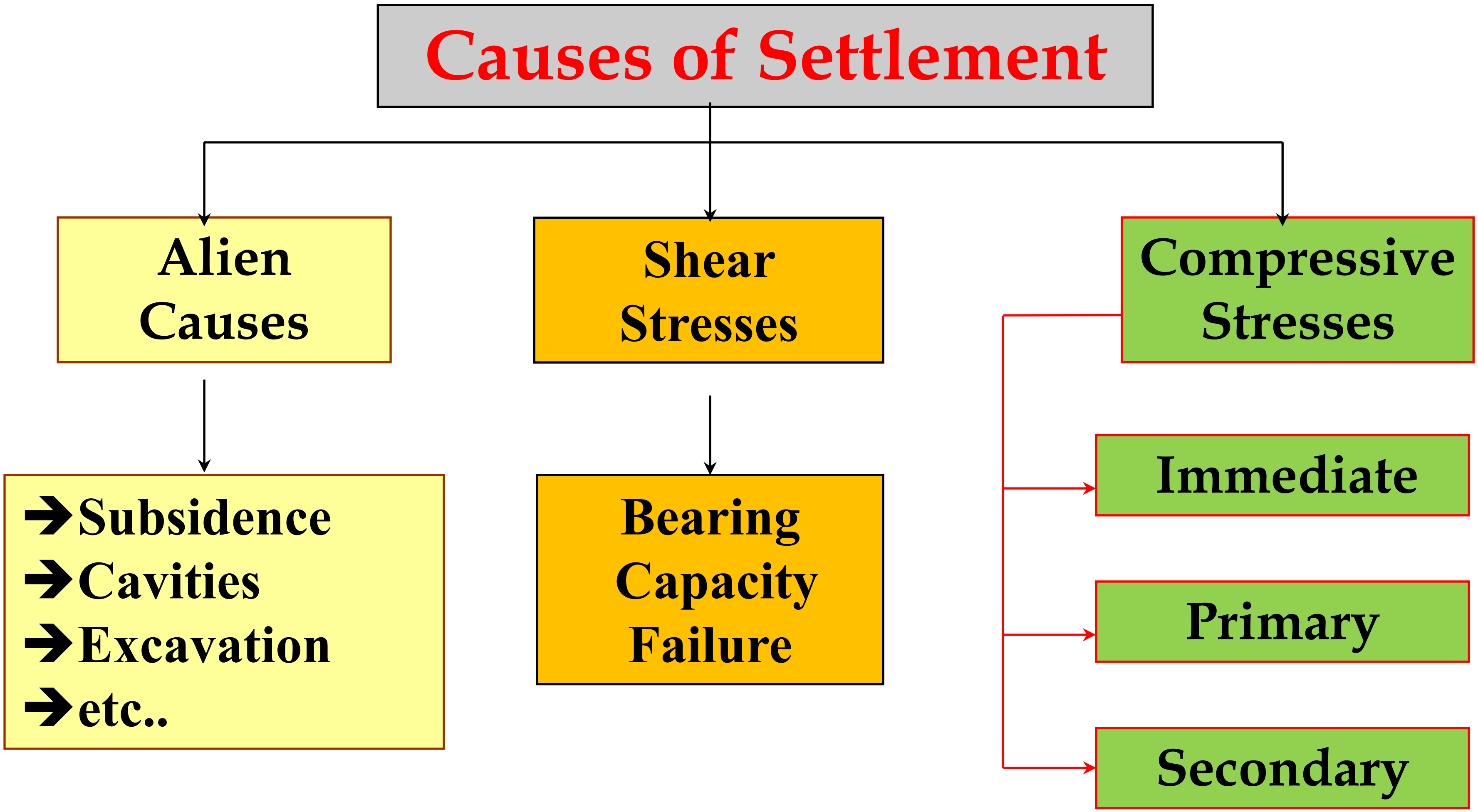
As a result of the applied load a compressive stress is transmitted to the soil leading to compressive strain. Due to the compressive strain the structure settles. This is important only if the settlement is excessive otherwise it is not dangerous.

ALLOWABLE BEARING CAPACITY

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

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

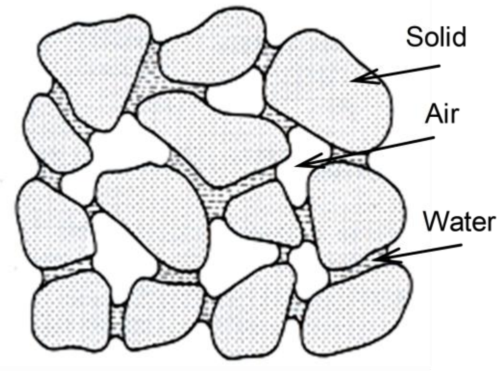
CAUSES OF SETTLEMENT



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

2. 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 or consolidation settlement (CE 481)

B. Secondary consolidation settlement

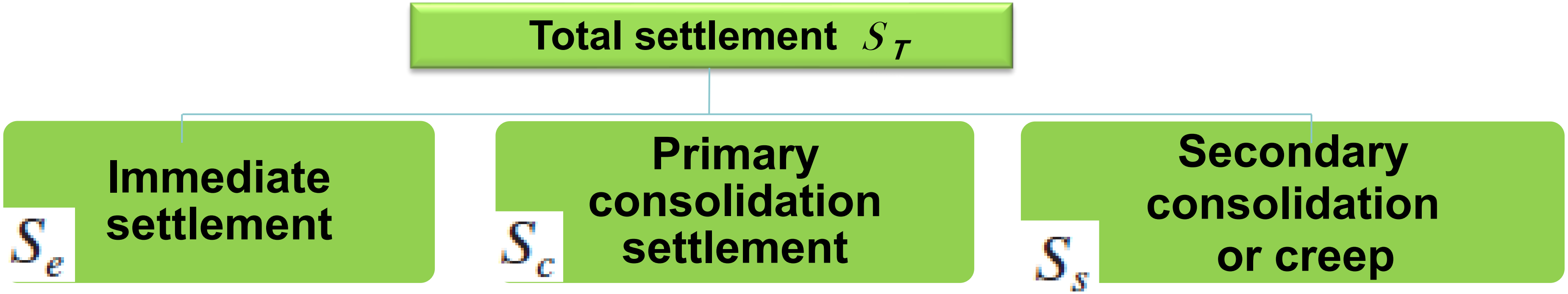
The compression of clay soil due to plastic readjustment of soil grains and progressive breaking of clayey particles and their inter-particles bonds is known as secondary consolidation or secondary compression, and the associated settlement is called secondary consolidation settlement or secondary compression.

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

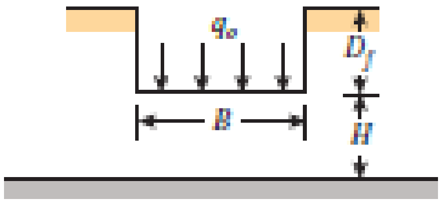
Primary consolidation settlement S_c

- 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

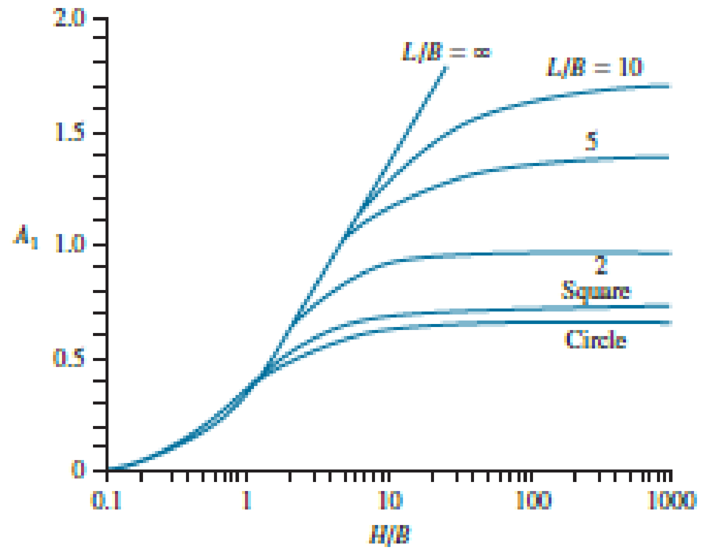
- Due to gradual changes in the particulate structure of the soil
- Occurs very slowly, long after the primary consolidation is completed
  Time dependent
- Most significant in saturated soft clayey and organic soils and peats

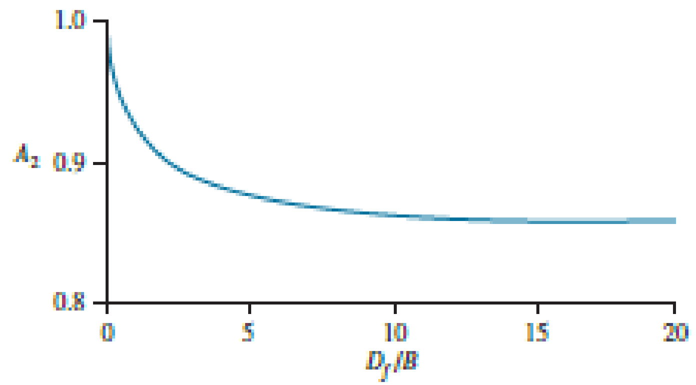
Elastic Settlement of Shallow Foundation on Saturated Clay ($\mu_s = 0.5$)



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

$A_1 = f(H/B, L/B)$
$A_2 = f(D_f/B)$
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 (μ_s = 0.5)

$$E_s = \beta c_u$$

where c_u = undrained shear strength.

TABLE 9.1 Range of β for Saturated Clay [Eq. (9.2)][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 9.1

EXAMPLE 9.1

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. (9.1),

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

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 9.1). Hence,

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

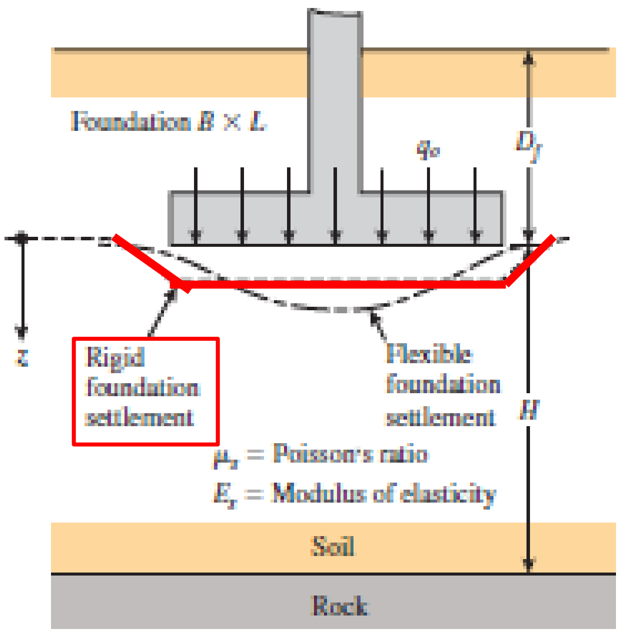
Also, from Figure 9.1, $A_1 = 0.9$ and $A_2 = 0.92$. Hence,

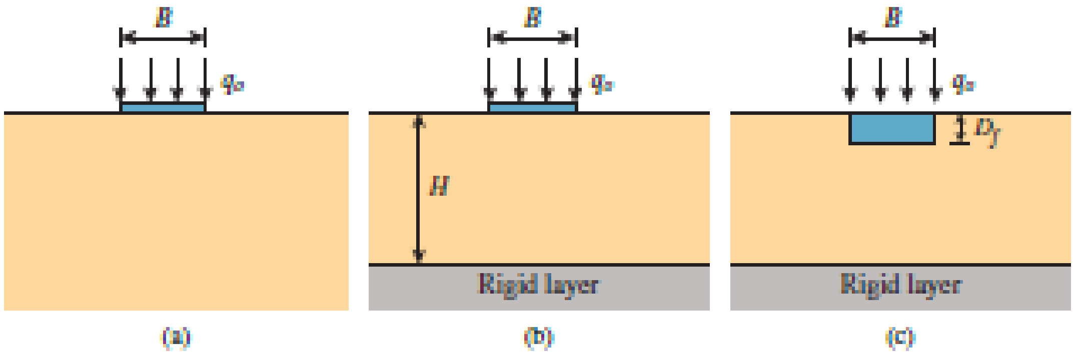
$$S_e = A_1 A_2 \frac{q_o B}{E_s} = (0.9)(0.92)\frac{(120)(1)}{72{,}000} = 0.00138 \text{ m} = \mathbf{1.38 \text{ mm}}$$

Elastic Settlement in Granular Soil

Settlement Based on the Theory of Elasticity

1. Flexible surface foundation ($D_f = 0$) resting on an elastic half-space, as shown in Figure (a)
2. The presence of a rigid layer at a finite depth H as shown in Figure (b)
3. Embedment depth D_f as shown in Figure (c).

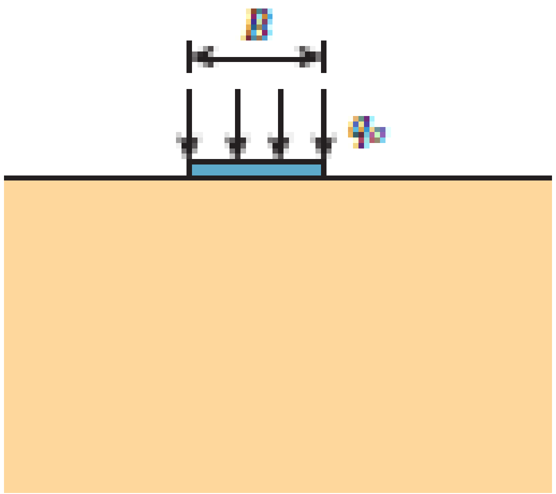




Elastic Settlement in Granular Soil

1. Surface Foundations on Elastic Half-Space

$$S_e = \frac{q_o B}{E_s}(1 - \mu_s^2)I$$



where

I = the influence factor that depends on the location of the point of interest on the foundation.

q_o = the net pressure applied by the foundation to the underlying soil

B = the width of the foundation

E_s = the modulus of elasticity of the soil

μ_s = Poisson's ratio of the soil.

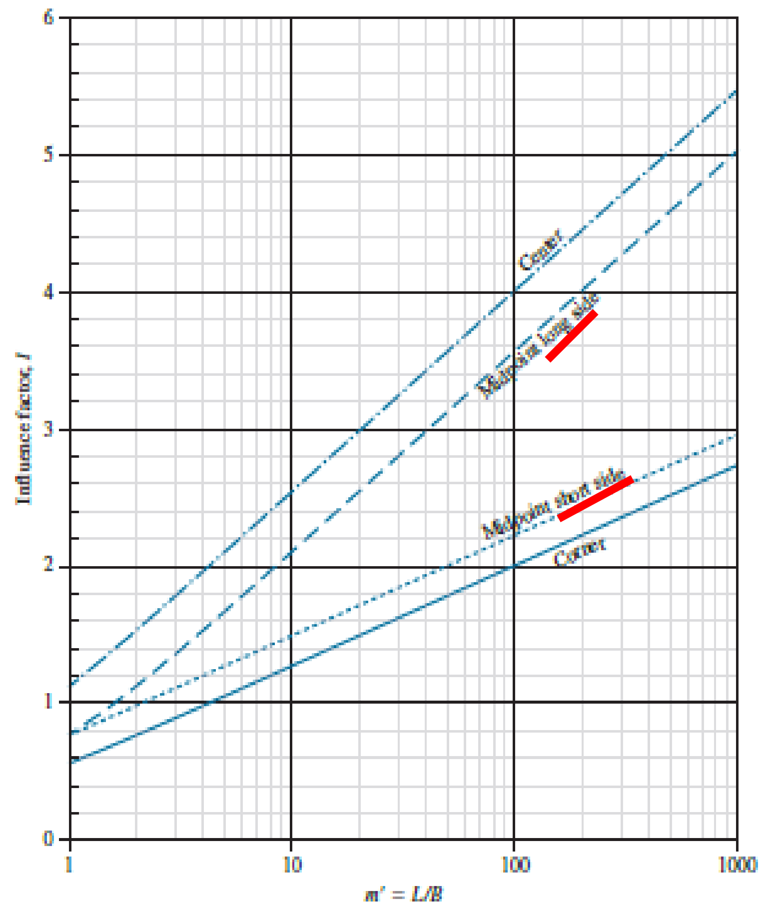
Elastic Settlement in Granular Soil

1. Surface Foundations on Elastic Half-Space

Schleicher (1926) expressed the influence factor for the *corner of a flexible foundation* as

$$I_{corner} = \frac{1}{\pi}\left[m' \ln\left(\frac{1+\sqrt{m'^2+1}}{m'}\right) + \ln\left(m' + \sqrt{m'^2+1}\right)\right]$$

where $m' = L/B$



The influence factors for the other locations on the foundation can be determined by dividing the foundation into four rectangles and using the principle of superposition

Elastic Settlement in Granular Soil

1. Surface Foundations on Elastic Half-Space

Giroud (1968) and Poulos and Davis (1974)

Corner:	$I = 0.7283 \log m' + 0.5469$
Center:	$I = 1.4566 \log m' + 1.0939$
Midpoint short side:	$I = 0.7318 \log m' + 0.7617$
Midpoint long side:	$I = 1.4357 \log m' + 0.6894$

The settlements under the center and the perimeter of a uniformly loaded *flexible circular foundation* of diameter B are given by:

Center: $$S_e = \frac{q_o B}{E_s}(1 - \mu_s^2)$$ I =1.0

Perimeter: $$S_e = \frac{q_o B}{E_s}(1 - \mu_s^2)\frac{2}{\pi}$$ I = 2/π

Elastic Settlement in Granular Soil

1. Surface Foundations on Elastic Half-Space

- Flexible foundations apply uniform pressure and settle non-uniformly.
- Rigid foundations apply non-uniform pressure and settle uniformly.
- Some influence factors for estimating the average values of the settlements of flexible and rigid foundations, are given in Table 9.2.
- Bowles (1987) suggested that the settlement of the rigid foundation can be estimated as 93% of the settlement computed for a flexible *foundation under the center*.
- The average values of *I* reported by Giroud (1968) and Poulos and Davis (1974) suggest that the average value of the settlement of a flexible foundation can be computed using 84–88% of the *I*-value for the center.

TABLE 9.2 Influence Factors to Compute *Average* Settlement of Flexible and Rigid Foundation

$m' = L/B$	Flexible	Rigid
Circle	0.85	0.79
1	0.95	0.82
1.5	1.20	1.07
2	1.30	1.21
3	1.52	1.42
5	1.82	1.60
10	2.24	2.00
100	2.96	3.40

Elastic Settlement in Granular Soil

2. Effects of a Rigid Layer on the Settlements of Surface Foundations

When the soil is underlain by a rigid layer, the settlement has to be reduced

The influence factor for the corner of a uniformly loaded flexible rectangular surface foundation is:

$$I_s = F_1 + \left(\frac{1 - 2\mu_s}{1 - \mu_s}\right)F_2$$

where

$$F_1 = \frac{1}{\pi}(A_0 + A_1)$$

$$F_2 = \frac{n'}{2\pi}\tan^{-1}A_2$$

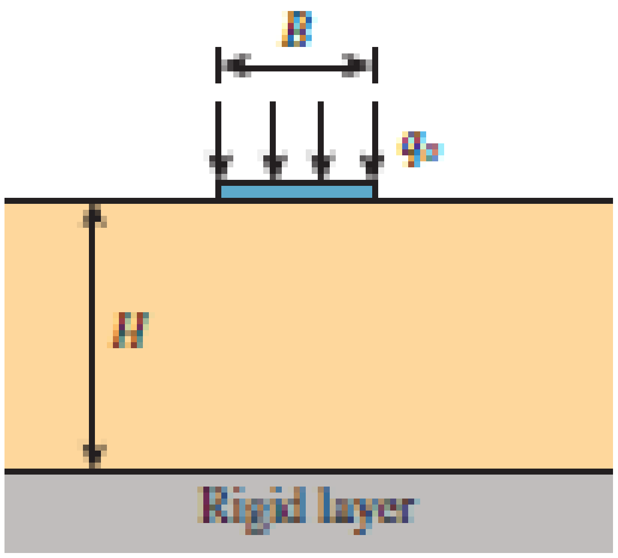
A_0, A_1, and A_2 are given by

$$A_0 = m'\ln\frac{\left(1 + \sqrt{m'^2 + 1}\right)\sqrt{m'^2 + n'^2}}{m'\left(1 + \sqrt{m'^2 + n'^2 + 1}\right)}$$

$$A_1 = \ln\frac{\left(m' + \sqrt{m'^2 + 1}\right)\sqrt{1 + n'^2}}{m' + \sqrt{m'^2 + n'^2 + 1}}$$

$$A_2 = \frac{m'}{n' + \sqrt{m'^2 + n'^2 + 1}}$$

$$n' = H/B.$$



Elastic Settlement in Granular Soil

2. Effects of a Rigid Layer on the Settlements of Surface Foundations

$$F_1 = \frac{1}{\pi}(A_0 + A_1)$$

$$F_2 = \frac{n'}{2\pi}\tan^{-1}A_2$$

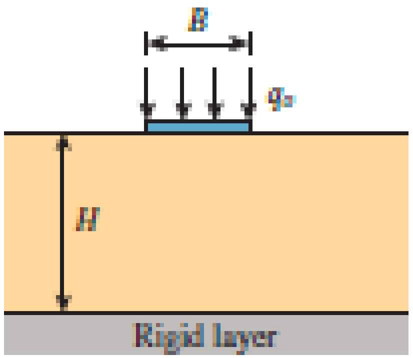


TABLE 9.3 Variation of F_1 with m' and n'

	m'									
n'	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
1.25	0.186	0.183	0.179	0.176	0.173	0.170	0.165	0.161	0.158	0.157
1.50	0.224	0.224	0.222	0.219	0.216	0.213	0.207	0.203	0.199	0.197
1.75	0.257	0.259	0.259	0.258	0.255	0.253	0.247	0.242	0.238	0.235
2.00	0.285	0.290	0.292	0.292	0.291	0.289	0.284	0.279	0.275	0.271
2.25	0.309	0.317	0.321	0.323	0.323	0.322	0.317	0.313	0.308	0.305
2.50	0.330	0.341	0.347	0.350	0.351	0.351	0.348	0.344	0.340	0.336
2.75	0.348	0.361	0.369	0.374	0.377	0.378	0.377	0.373	0.369	0.365
3.00	0.363	0.379	0.389	0.396	0.400	0.402	0.402	0.400	0.396	0.392
3.25	0.376	0.394	0.406	0.415	0.420	0.423	0.426	0.424	0.421	0.418
3.50	0.388	0.408	0.422	0.431	0.438	0.442	0.447	0.447	0.444	0.441
3.75	0.399	0.420	0.436	0.447	0.454	0.460	0.467	0.458	0.466	0.464
4.00	0.408	0.431	0.448	0.460	0.469	0.476	0.484	0.487	0.486	0.484
4.25	0.417	0.440	0.458	0.472	0.481	0.484	0.495	0.514	0.515	0.515
4.50	0.424	0.450	0.469	0.484	0.495	0.503	0.516	0.521	0.522	0.522
4.75	0.431	0.458	0.478	0.494	0.506	0.515	0.530	0.536	0.539	0.539
5.00	0.437	0.465	0.487	0.503	0.516	0.526	0.543	0.551	0.554	0.554

TABLE 9.4 Variation of F_2 with m' and n'

	m'									
n'	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
1.25	0.080	0.089	0.096	0.102	0.107	0.111	0.118	0.122	0.125	0.127
1.50	0.075	0.084	0.093	0.099	0.105	0.110	0.118	0.124	0.128	0.130
1.75	0.069	0.079	0.088	0.095	0.101	0.107	0.117	0.123	0.128	0.131
2.00	0.064	0.074	0.083	0.090	0.097	0.102	0.114	0.121	0.127	0.131
2.25	0.059	0.069	0.077	0.085	0.092	0.098	0.110	0.119	0.125	0.130
2.50	0.055	0.064	0.073	0.080	0.087	0.093	0.106	0.115	0.122	0.127
2.75	0.051	0.060	0.068	0.076	0.082	0.089	0.102	0.111	0.119	0.125
3.00	0.048	0.056	0.064	0.071	0.078	0.084	0.097	0.108	0.116	0.122
3.25	0.045	0.053	0.060	0.067	0.074	0.080	0.093	0.104	0.112	0.119
3.50	0.042	0.050	0.057	0.064	0.070	0.076	0.089	0.100	0.109	0.116
3.75	0.040	0.047	0.054	0.060	0.067	0.073	0.086	0.096	0.105	0.113
4.00	0.037	0.044	0.051	0.057	0.063	0.069	0.082	0.093	0.102	0.110
4.25	0.036	0.042	0.049	0.055	0.061	0.066	0.079	0.090	0.099	0.107
4.50	0.034	0.040	0.046	0.052	0.058	0.063	0.076	0.086	0.096	0.104
4.75	0.032	0.038	0.044	0.050	0.055	0.061	0.073	0.083	0.093	0.101
5.00	0.031	0.036	0.042	0.048	0.053	0.058	0.070	0.080	0.090	0.098

Elastic Settlement in Granular Soil

2. Effects of a Rigid Layer on the Settlements of Surface Foundations

For $H = \infty$, $n' = \infty$,

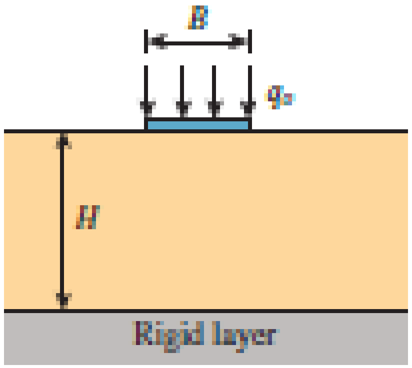
$$A_0 = m' \ln \frac{1 + \sqrt{m'^2 + 1}}{m'}$$

$$A_1 = \ln\left(m' + \sqrt{m'^2 + 1}\right)$$

$$A_2 = 0$$

$$F_1 = \frac{1}{\pi}(A_0 + A_1)$$

$$F_2 = \frac{n'}{2\pi}\tan^{-1} A_2$$



$$F_1 = \frac{1}{\pi}\left[m' \ln\left(\frac{1 + \sqrt{m'^2 + 1}}{m'}\right) + \ln\left(m' + \sqrt{m'^2 + 1}\right)\right]$$

$$F_2 = 0$$

Elastic Settlement in Granular Soil

3. Effect of Embedment

When the foundation base is located at some depth beneath the ground level, the embedment reduces the settlement.

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

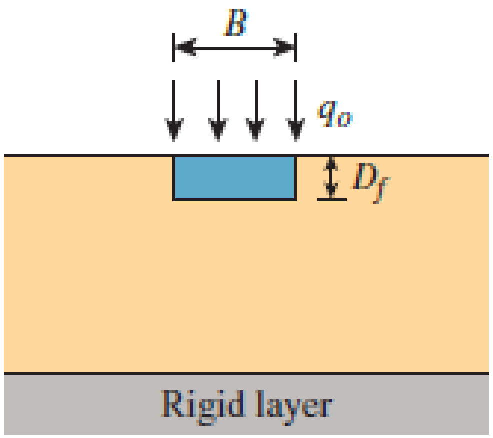
embedment

$$I_s = F_1 + \left(\frac{1 - 2\mu_s}{1 - \mu_s}\right) F_2$$

I_f next page

$$S_e = \frac{q_o B}{E_s}(1 - \mu_s^2) I$$

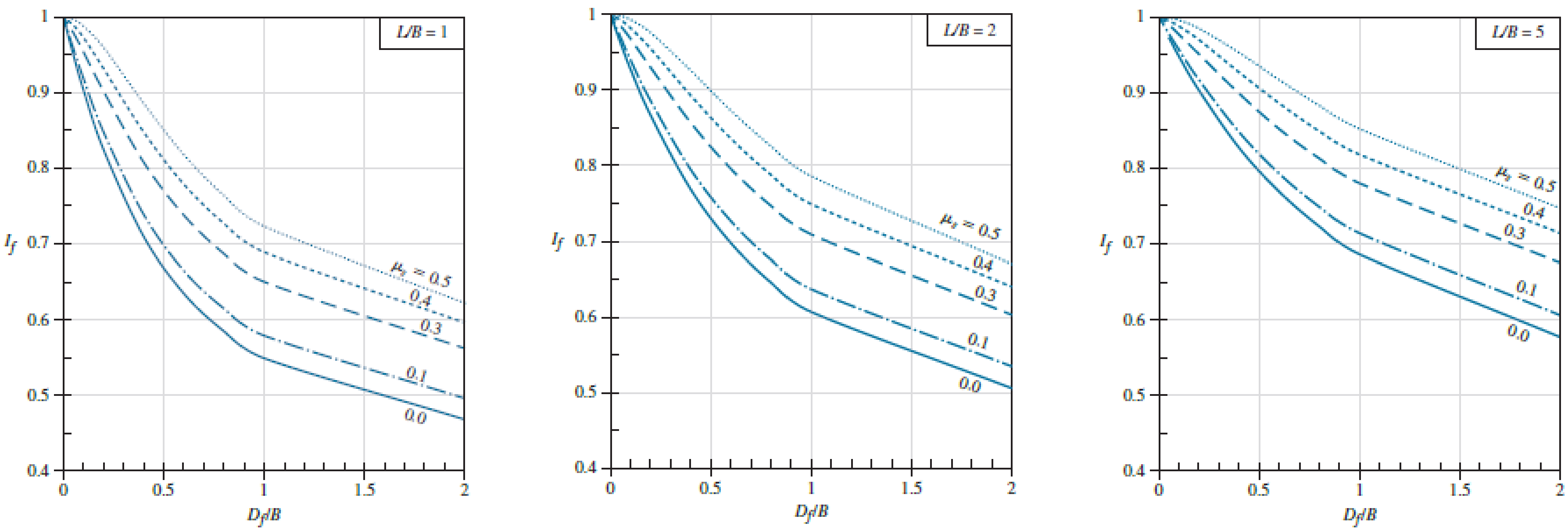
Surface



Elastic Settlement in Granular Soil

3. Effect of Embedment

I_f



Elastic Settlement in Granular Soil

3. Effect of Embedment

E_s

Due to the nonhomogeneous nature of soil deposits, the magnitude of E_s may vary with depth. For that reason, Bowles (1987) recommended using a weighted average of E_s

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

where

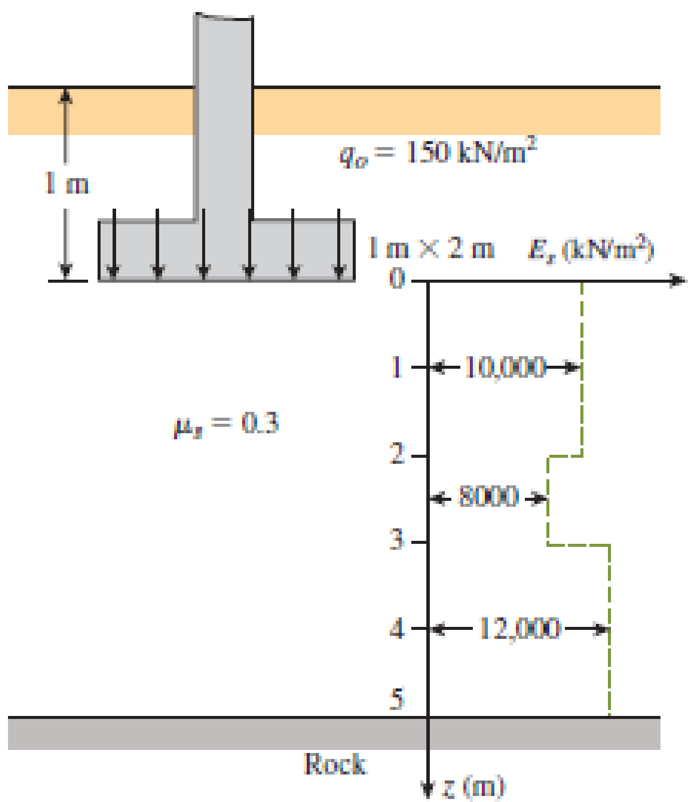
$E_{s(i)}$ = soil modulus of elasticity within a depth Δz

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

EXAMPLE 9.2

EXAMPLE 9.2

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



EXAMPLE 9.2

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

SOLUTION

We are given that $B = 1$ m and $L = 2$ m. Note that $\bar{z} = 5$ m $= 5B$. From Eq. (9.23),

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

$$= \frac{(10{,}000)(2) + (8000)(1) + (12{,}000)(2)}{5} = 10{,}400 \text{ kN/m}^2$$

For one of the four quarters of the foundation, $B = 0.5$ m and $L = 1.0$ m. Also, $H = 6.0$ m (*Note:* The Steinbrenner factors in Tables 9.3 and 9.4 are for surface foundations with $D_f = 0$.)

H=6.0 m

$$m' = L/B = 2.0 \text{ and } n' = H/B = 12.0$$

From Table 9.3, $F_1 = 0.653$, and from Table 9.4, $F_2 = 0.028$.

From Eq. (9.11), with $\mu_s = 0.3$,

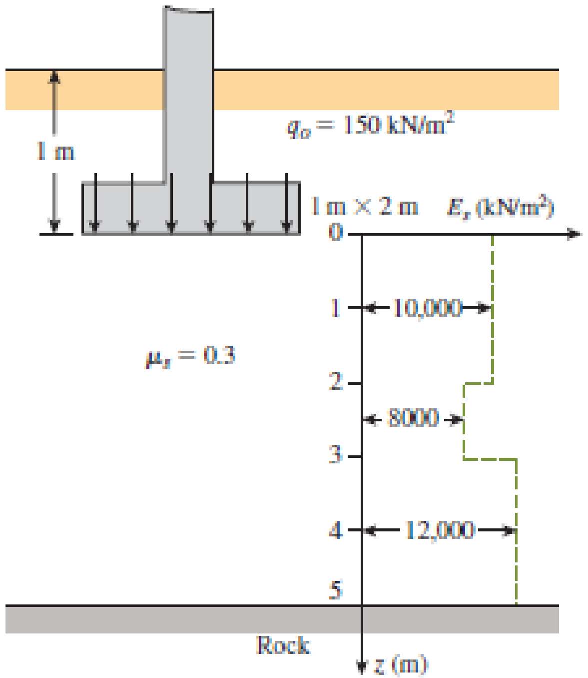
$$I_s = F_1 + \left(\frac{1 - 2\mu_s}{1 - \mu_s}\right)F_2 = 0.653 + \left(\frac{1 - 2 \times 0.3}{1 - 0.3}\right)(0.028) = 0.669$$

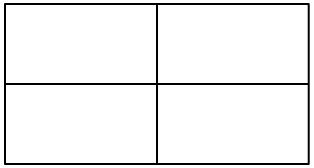
For $\mu_s = 0.3$, $L/B = 2$ and $D_f/B = 1$ (using $B = 1$ m for the entire foundation); from Figure 9.5b, $I_f = 0.71$.

From Eq. (9.22) and considering the four quarters,

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

$$= \frac{(150)(0.5)}{(10{,}400)}(1 - 0.3^2)(0.669 \times 4)(0.71) = 0.0124 \text{ m} = \mathbf{12.4 \text{ mm}}$$



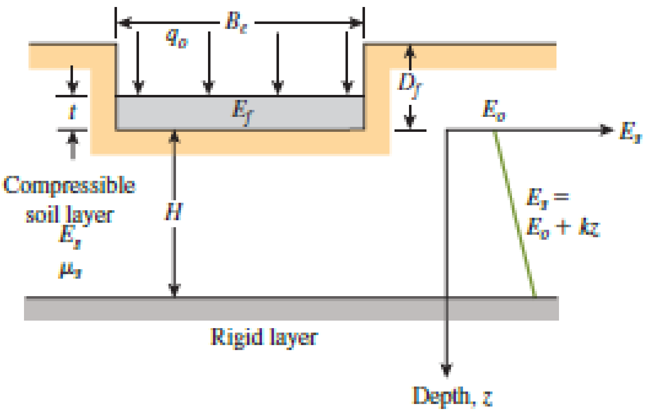


B= 0.5 m
L =1.0 m

Improved Equation for Elastic Settlement

The improved formula takes into account

1. The rigidity of the foundation
2. The depth of embedment of the foundation
3. The increase in the modulus of elasticity of the soil with depth
4. The location of rigid layers at a limited depth



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

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

B_e = the equivalent diameter

B_e of a rectangular foundation,

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

B = width of foundation

L = length of foundation

For circular foundations,

$$B_e = B$$

B = diameter of foundation.

Improved Equation for Elastic Settlement

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

I_G = influence factor for the variation of E_s with depth

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

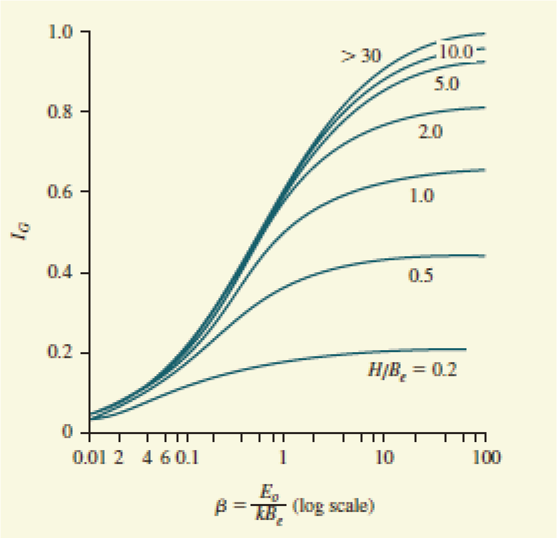
I_F = foundation rigidity correction factor

I_E = foundation embedment correction factor

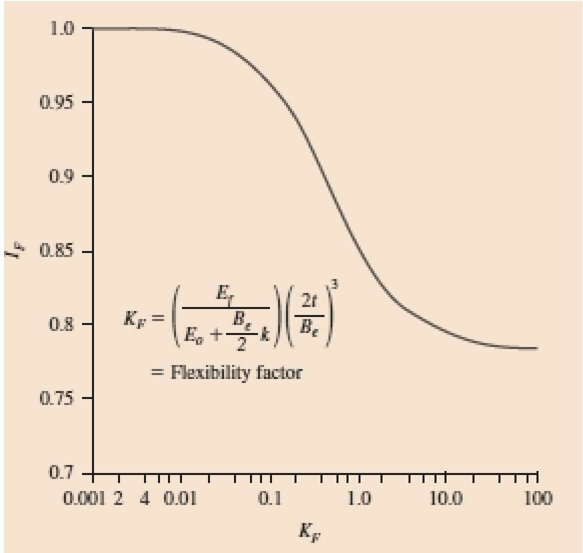
$$I_F = \frac{\pi}{4} + \frac{1}{4.6 + 10\left(\dfrac{E_f}{E_o + \dfrac{B_e}{2}k}\right)\left(\dfrac{2t}{B_e}\right)^3}$$

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

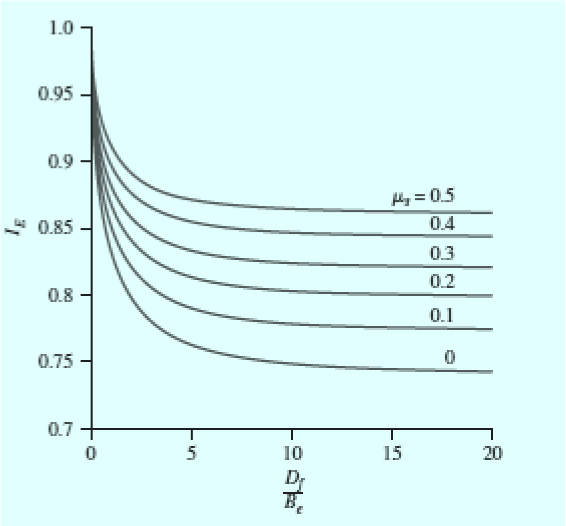
Improved Equation for Elastic Settlement



I_G



I_F



I_E

EXAMPLE 9.3

EXAMPLE 9.3

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

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:

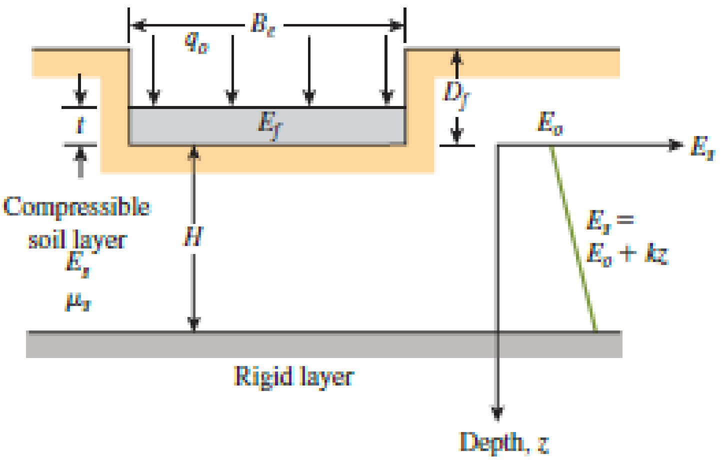
H = 3.7 m

μ_s = 0.3

E_o = 9700 kN/m^2

k = 575 kN/m^2/m

Estimate the elastic settlement of the foundation.



Example 9.3

SOLUTION

From Eq. (9.24), 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 9.8, for $\beta = 7.06$ and $H/B_e = 1.55$, the value of $I_G \approx 0.7$. From Eq. (9.28),

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

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

Example 9.3

From Eq. (9.29),

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

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

From Eq. (9.27),

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

so, with $q_o = 240$ kN/m^2, it follows that

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

Strain Influence Factor

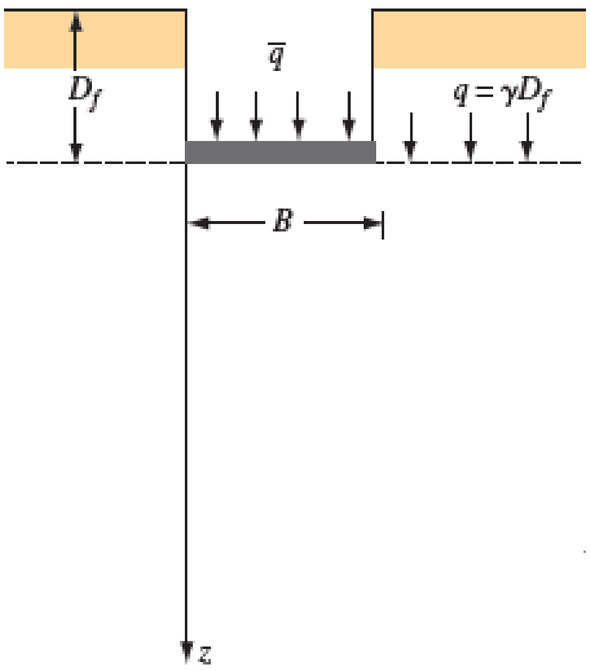
I. Solution of Schmertmann et al. (1978)

II. Solution of Terzaghi et al. (1996)

Settlement of Sandy Soil: Use of Strain Influence Factor

I. Solution of Schmertmann et al. (1978)

$$S_e = C_1 C_2 (\bar{q} - q) \sum_{0}^{z_2} \frac{I_z}{E_s} \Delta z$$



I_z = strain influence factor

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

C_2 = a correction factor to account for creep in soil

$= 1 + 0.2 \log$ (time in years/0.1)

$\bar{q}$ = stress at the level of the foundation

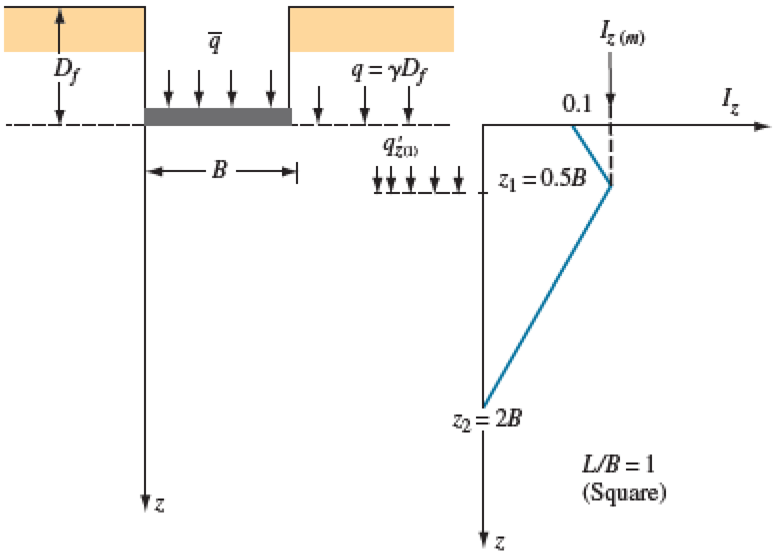
$q = \gamma D_f$ = effective stress at the base of the foundation

E_s = modulus of elasticity of soil

I. Solution of Schmertmann et al. (1978)

Square ($L/B = 1$) or circular foundations

z	I_z
0	0.1
$Z_1 = 0.5$ B	$I_{z(m)}$
$Z_2 = 2$ B	0



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

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

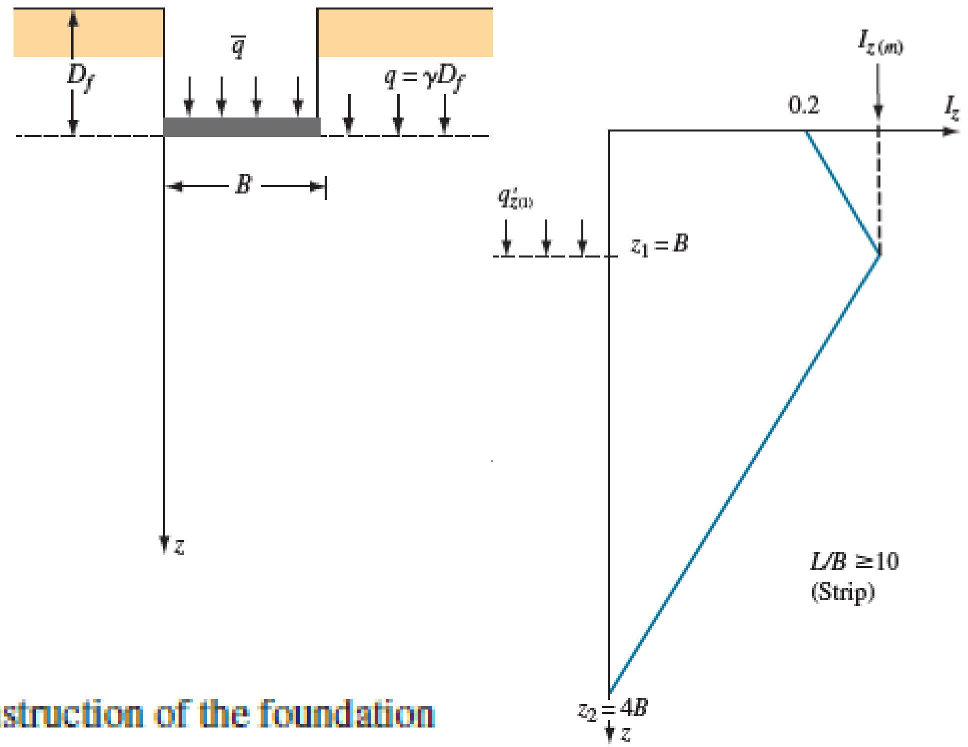
I. Solution of Schmertmann et al. (1978)

Foundation with *L / B* >= 10

z	I_z
0	0.2
Z_1 = B	$I_{z(m)}$
Z_2 = 4 B	0

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

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



I. Solution of Schmertmann et al. (1978)

Rectangular foundations

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 $z = 0$

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

- z_1/B

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

- z_2/B

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

I. Solution of Schmertmann et al. (1978)

E_s Modulus of Elasticity of Soil

Schmertmann et al. (1978) suggested that

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

and

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

where q_c is the cone penetration resistance.

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

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

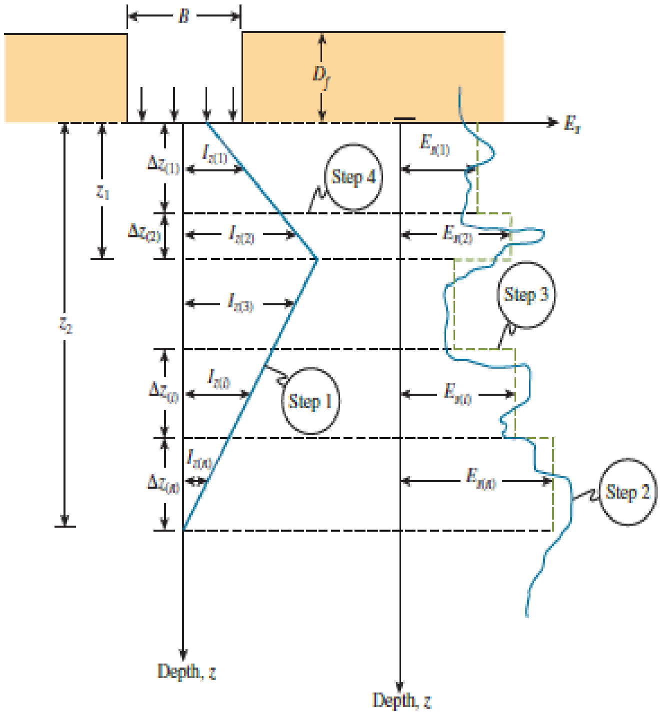
Procedure for calculation of S_e using the strain influence factor

Step 1. Plot the foundation and the variation of I_z with depth to scale

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

Step 3. Approximate the actual variation of E_s into a number of layers of soil having a constant E_s, such as $E_{s(1)}, E_{s(2)}, \ldots, E_{s(i)}, \ldots E_{s(n)}$

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.



Procedure for calculation of S_e using the strain influence factor

Step 5. Prepare a table (such as Table 9.5) to obtain $\Sigma \frac{I_z}{E_s} \Delta z$.

Step 6. Calculate C_1 and C_2.

Step 7. Calculate S_e

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

TABLE 9.5 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	$\Delta z_{(n)}$	$E_{s(n)}$	$I_{z(n)}$	$\frac{I_{z(n)}}{E_{s(n)}} \Delta z_n$
				$\Sigma \frac{I_z}{E_s} \Delta z$

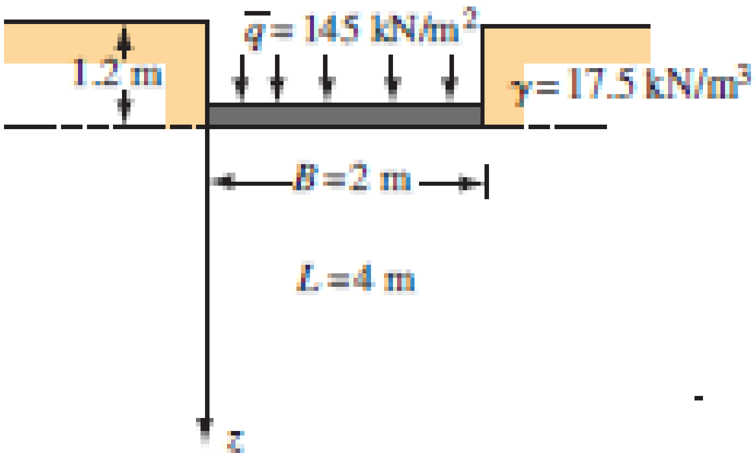
EXAMPLE 9.4

EXAMPLE 9.4

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 9.13a. Given: $\gamma = 17.5\ \text{kN/m}^3$, $\bar{q} = 145\ \text{kN/m}^2$, 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.



EXAMPLE 9.4

Step 1. Plot the foundation and the variation of I_z with depth to scale

Rectangular Foundation

- I_z at $z = 0$

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

- z_1/B

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

- z_2/B

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

at $z = 0$,

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

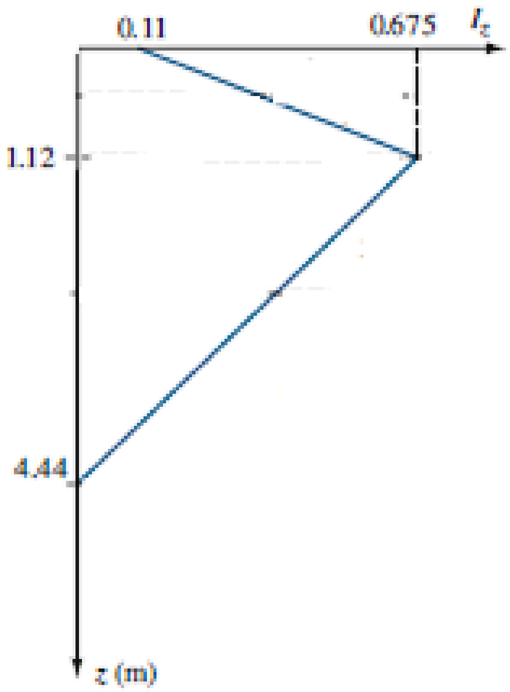
$$\frac{z_1}{B} = 0.5 + 0.0555\left(\frac{L}{B} - 1\right) = 0.5 + 0.0555\left(\frac{4}{2} - 1\right) \approx 0.56$$

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

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

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

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



EXAMPLE 9.4

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

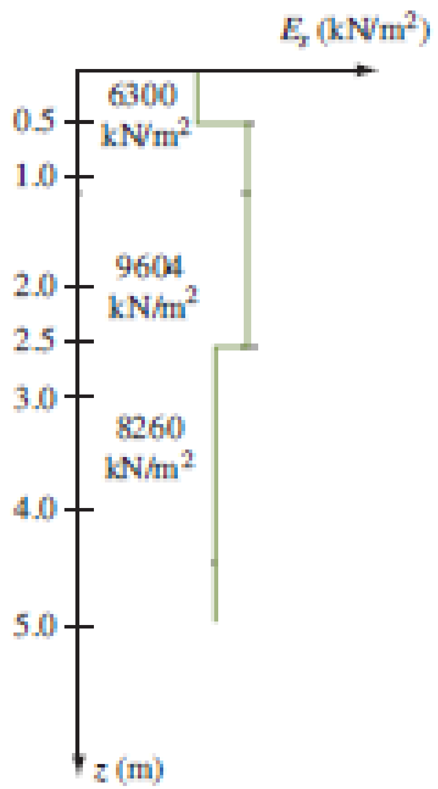
$$E_{s(\text{rectangle})} = \left(1 + 0.4 \log\frac{L}{B}\right)E_{s(\text{square})} = \left[1 + 0.4\log\left(\frac{4}{2}\right)\right](2.5 \times q_c) = 2.8q_c$$

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

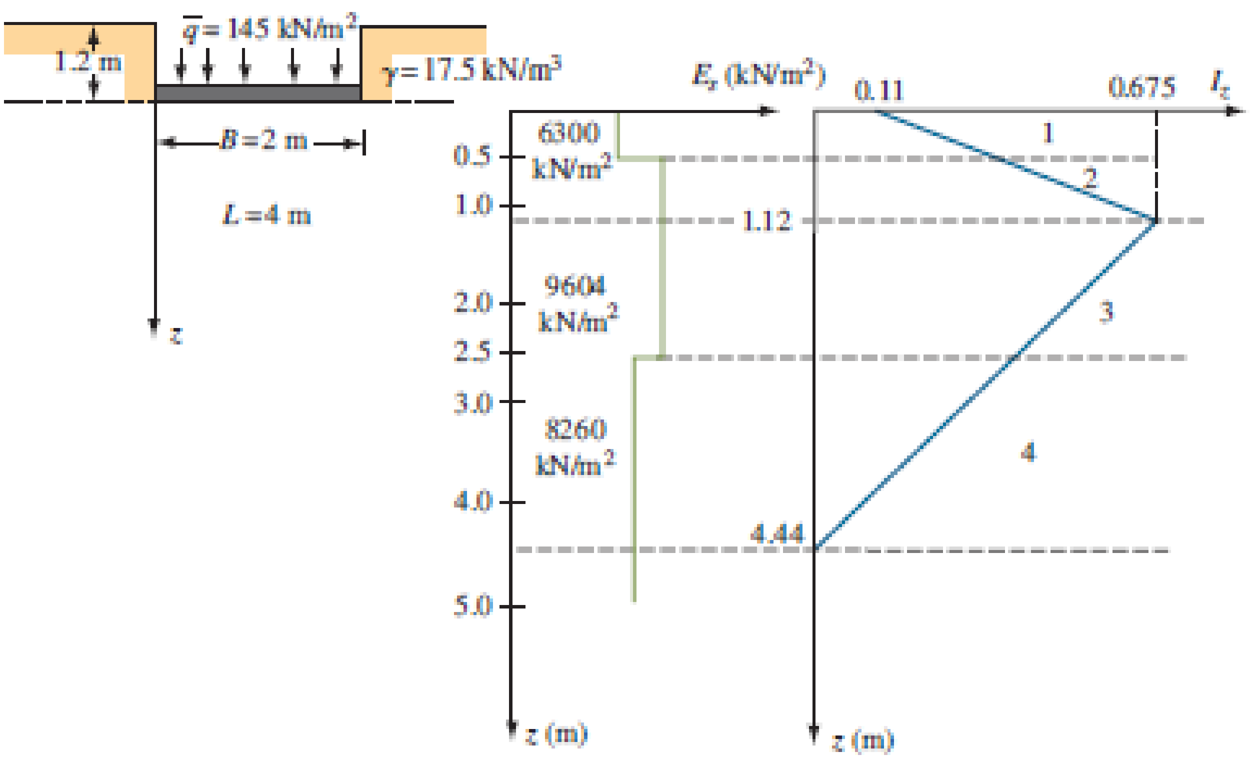
EXAMPLE 9.4

Step 3. Approximate the actual variation of E_s into a number of layers of soil having a constant E_s, such as $E_{s(1)}, E_{s(2)}, \ldots, E_{s(i)}, \ldots E_{s(n)}$



EXAMPLE 9.4

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.



EXAMPLE 9.4

Step 5. Prepare a table to obtain $\Sigma \frac{I_z}{E_s} \Delta z$.

Layer no.	Δz (m)	E_s (kN/m^2)	I_z at middle of layer	$\frac{I_z}{E_s}\Delta z$ (m^3/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}
				$\Sigma 17.52 \times 10^{-5}$

EXAMPLE 9.4

Step 6. Calculate C_1 and C_2.

Step 7. Calculate S_e

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

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

Assume the time for creep is 10 years. So,

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

Hence,

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

II. Solution of Terzaghi et al. (1996)

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

, q_c is in MN/m^2.

C_d is the depth factor

II. Solution of Terzaghi et al. (1996)

Terzaghi et al. (1996) proposed a slightly different form of the strain influence factor diagram

At $z = 0$, $I_z = 0.2$ (for all L/B values)

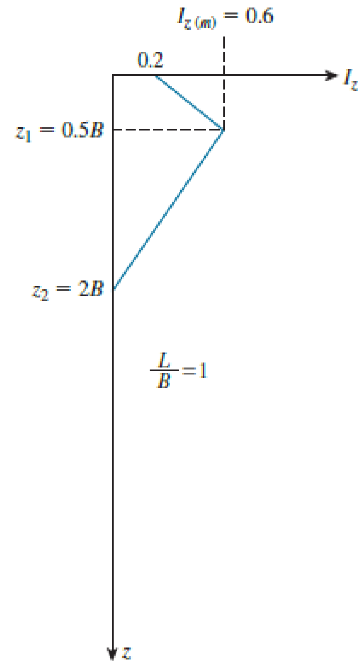
At $z = z_1 = 0.5B$, $I_z = 0.6$ (for all L/B values)

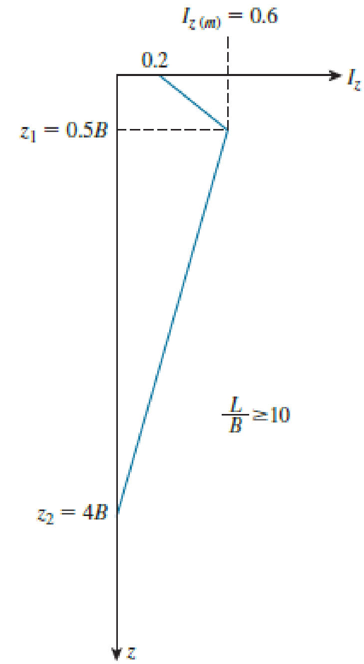
At $z = z_2 = 2B$, $I_z = 0$ (for $L/B = 1$)

At $z = z_2 = 4B$, $I_z = 0$ (for $L/B \geq 10$)

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

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





II. Solution of Terzaghi et al. (1996)

$E_s = 3.5q_c$ (for square and circular foundations)

$$E_{s(\text{rectangular})} = \left[1 + 0.4\log\left(\frac{L}{B}\right)\right]E_{s(\text{square})} \leq 1.4E_{s(\text{square})}$$

C_d is the depth factor

TABLE 9.6 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 9.5

EXAMPLE 9.5

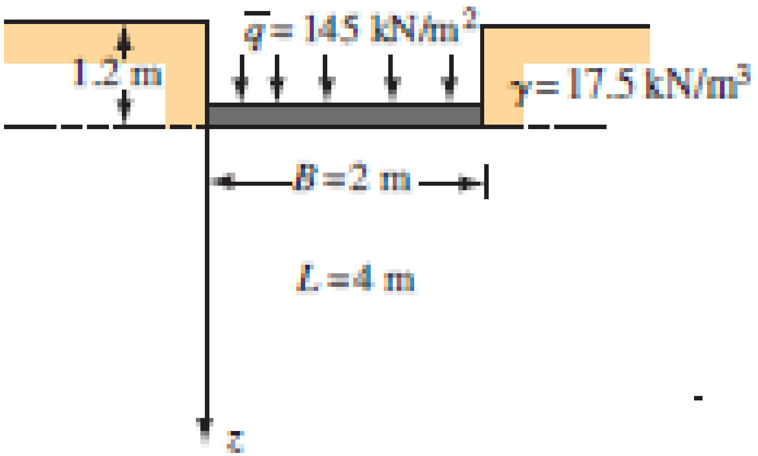
Solve Example 9.4 using the method of Terzaghi et al. (1996).

EXAMPLE 9.4

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 9.13a. Given: $\gamma = 17.5\ \text{kN/m}^3$, $\bar{q} = 145\ \text{kN/m}^2$, 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.



Example 9.5

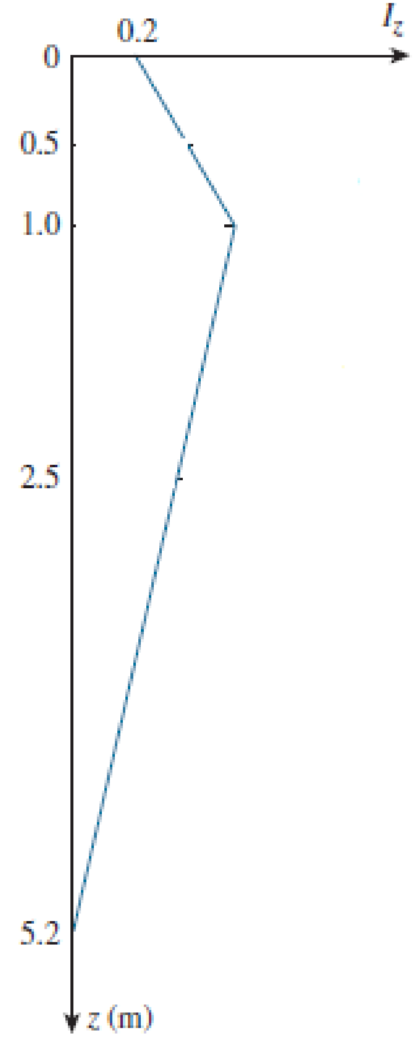
Given: $L/B = 4/2 = 2$.

Figure 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

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

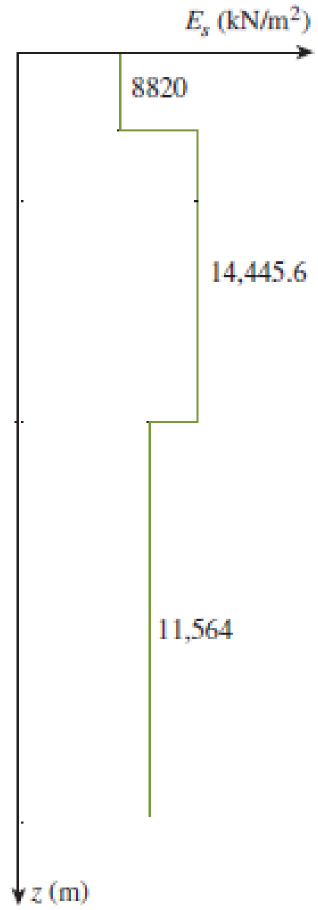


Example 9.5

$$E_s = \left[1 + 0.4 \log\left(\frac{L}{B}\right)\right](3.5q_c) = \left[1 + 0.4 \log\left(\frac{4}{2}\right)\right](3.5q_c) = 3.92q_c$$

The following table can be prepared and shows the variation of E_s with depth

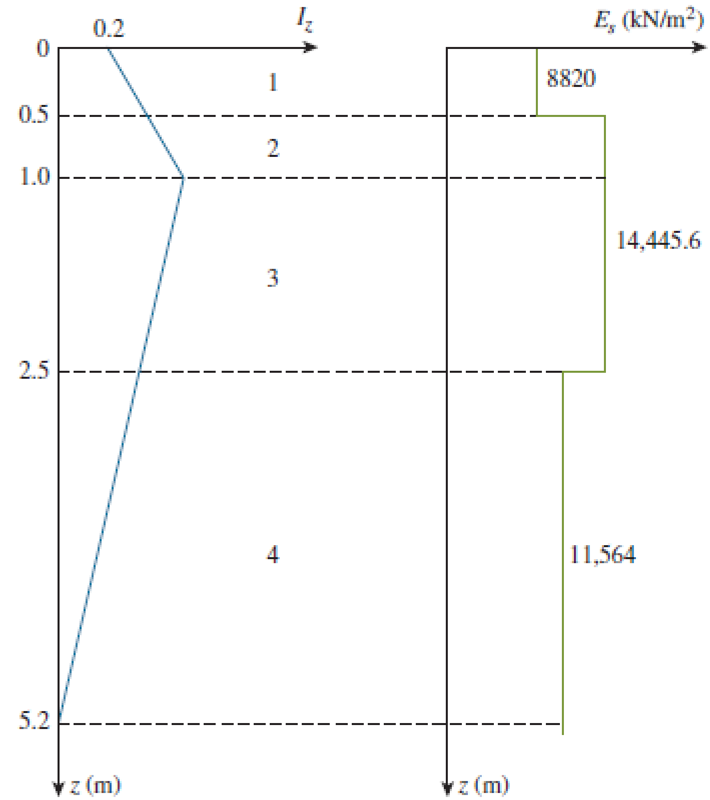
z (m)	q_c (kN/m^2)	E_s (kN/m^2)
0–0.5	2250	8820
0.5–2.5	3430	14,445.6
2.5–6	2950	11,564



Example 9.5

The following table is used 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	8820	0.3	1.7×10^{-5}
2	0.5	14,445.6	0.5	1.73×10^{-5}
3	1.5	14,445.6	0.493	5.12×10^{-5}
4	2.7	11,564	0.193	4.5×10^{-5}
				$\Sigma\ 13.06 \times 10^{-5}$ m²/kN



Example 9.5

$D_f/B = 1.2/2 = 0.6$. From Table 9.6, $C_d \approx 0.85$.

$$C_d(\bar{q} - q)\sum_0^{z_2} \frac{I_z}{E_s}\Delta_z = (0.85)(145 - 21)(13.06 \times 10^{-5}) = 1376.5 \times 10^{-5} \text{ m}$$

Postconstruction creep is

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

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

$$S_e = 1376.5 \times 10^{-5} + 0.02\left[\frac{0.1}{3.07}\right](5.2)\log\left(\frac{10 \times 365 \text{ days}}{1 \text{ day}}\right)$$
$$= 2583.3 \times 10^{-5} \text{ m}$$
$$\approx \mathbf{25.83 \text{ mm}}$$

Examples 9.4 & 9.5

Example	Method	S_e (mm)
Example 9.4	Schmertmann et al. (1978)	27.83
Example 9.5	Terzaghi et al. (1996)	25.83

Notes:

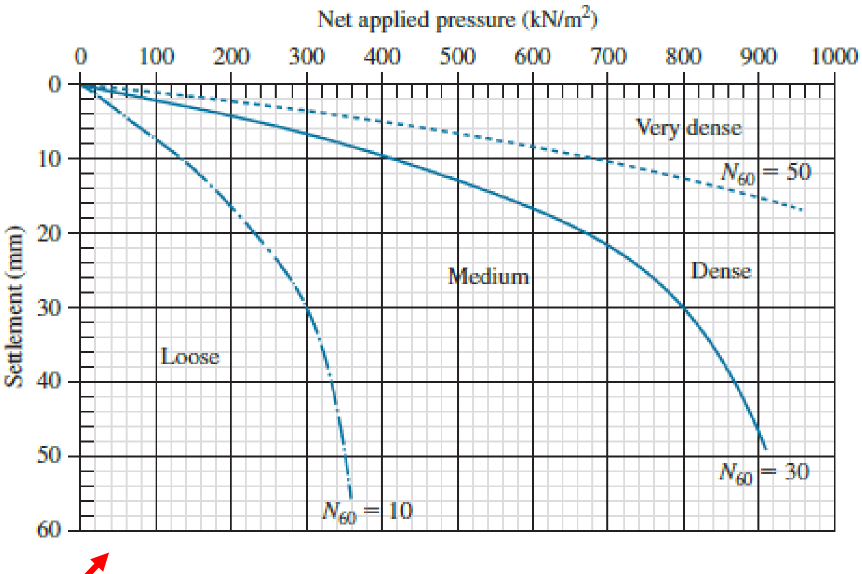
- The magnitude of S_e in Example 9.5 is about 93% of that found in Example 9.4.
- In Example 9.4, the elastic settlement was about 19.88 mm, and settlement due to creep was about 7.95 mm.
- In Example 9.5, elastic settlement is about 13.77 mm, and the settlement due to creep is about 12.07 mm.
- The magnitude of creep settlement is about 50% more in Example 9.5.
- The magnitude of elastic settlement in Example 9.4 is about 30% more compared to that in Example 9.5. This is because of the assumption of the $E_s - q_c$ relationship.

Settlement of Foundation on Sand Based on Standard Penetration Resistance

I. Terzaghi and Peck's Method

$$S_{e,\text{ foundation}} = S_{e,\text{ plate}}\left(\frac{2B}{B+0.3}\right)^2\left(1-\frac{1}{4}\frac{D_f}{B}\right)$$

where B is in meters



Settlement of 300 mm × 300 mm plate

- Leonards (1986) suggested replacing ¼ by 1/3, based on additional load test data.
- The values of $S_{e,\text{ plate}}$ can be obtained from Figure
- These load tests were carried out on thick deposits of normally consolidated drained sand.
- This method was originally proposed for square foundations but can be applied to rectangular and strip foundations with caution.
- The deeper influence zone and increase in the stresses within the underlying soil mass in the case of rectangular or strip foundations are compensated by the increase in the soil stiffness.

EXAMPLE 9.6

EXAMPLE 9.6

A 2.5 m square foundation placed at a depth of 1.5 m within a sandy soil applies a net pressure of 120 kN/m^2 to the underlying ground. The sand has $\gamma = 18.5$ kN/m^3 and $N_{60} = 25$. What would be the settlement?

SOLUTION

For net applied pressure = 120 kN/m^2 and $N_{60} = 25$; from Figure , $S_{e,\text{ plate}} = 4$ mm.

$$S_{e,\text{ foundation}} = S_{e,\text{ plate}}\left(\frac{2B}{B+0.3}\right)^2\left(1-\frac{1}{3}\frac{D_f}{B}\right)$$

$$= (4)\left(\frac{2\times 2.5}{2.5+0.3}\right)^2\left(1-\frac{1}{3}\times\frac{1.5}{2.5}\right) = \mathbf{10.2\ mm}$$

Settlement of Foundation on Sand Based on Standard Penetration Resistance

II. Meyerhof's Method

Meyerhof (1956) proposed a correlation for the *net bearing pressure* for foundations with the standard penetration resistance, N_{60}. The net pressure has been defined as

$$q_{net} = \overline{q} - \gamma D_f$$

where $\overline{q}$ is the stress at the level of the foundation.

For 25 mm of estimated maximum settlement

$$q_{net}(\text{kN/m}^2) = \frac{N_{60}}{0.08} \quad (\text{for } B \leq 1.22 \text{ m})$$

$$q_{net}(\text{kN/m}^2) = \frac{N_{60}}{0.125}\left(\frac{B + 0.3}{B}\right)^2 \quad (\text{for } B > 1.22 \text{ m})$$

Meyerhof (1965) suggested that the net allowable bearing pressure should be increased by about 50%.

Settlement of Foundation on Sand Based on Standard Penetration Resistance

II. Meyerhof's Method

Bowles (1977) proposed modified form of the bearing equations

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

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

F_d = depth factor = $1 + 0.33(D_f/B)$

B = foundation width, in meters

S_e = settlement, in mm

Settlement of Foundation on Sand Based on Standard Penetration Resistance

II. Meyerhof's Method

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

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

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

III. 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.

Settlement of Foundation on Sand Based on Standard Penetration Resistance

III. Burland and Burbidge's Method

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 = reference width = 0.3 m (if B is in m)

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

III. 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 9.7 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	$\dfrac{1.71}{[\overline{N}_{60} \text{ or } \overline{N}_{60(a)}]^{1.4}}$	$\alpha_3 = \dfrac{H}{z'}\left(2 - \dfrac{H}{z'}\right)$ (if $H \le z'$) or $\alpha_3 = 1$ (if $H > z'$) where H = depth of compressible layer
Overconsolidated sand ($q_{net} \le \sigma'_c$) where σ'_c = preconsolidation pressure	q_{net}	0.047	$\dfrac{0.57}{[\overline{N}_{60} \text{ or } \overline{N}_{60(a)}]^{1.4}}$	
Overconsolidated sand ($q_{net} > \sigma'_c$)	$q_{net} - 0.67\sigma'_c$	0.14	$\dfrac{0.57}{[\overline{N}_{60} \text{ or } \overline{N}_{60(a)}]^{1.4}}$	

Table 9.7 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, $[\overline{N}_{60} \text{ or } \overline{N}_{60(a)}]$ = average value of N_{60} or $N_{60(a)}$ in the depth of stress influence.

EXAMPLE 9.7

EXAMPLE 9.7

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

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

Depth of stress influence,

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

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

For normally consolidated sand (Table 9.7),

$$\alpha_1 = 0.14$$

$$\alpha_2 = \frac{1.71}{(\overline{N}_{60})^{1.4}} = \frac{1.71}{(10)^{1.4}} = 0.068$$

$$\alpha_3 = 1$$

$$q' = q_{net} = 120 \text{ kN/m}^2$$

So,

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

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

EXAMPLE 9.8

EXAMPLE 9.8

Solve Example 9.6 using Meyerhof's method.

SOLUTION

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.

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

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

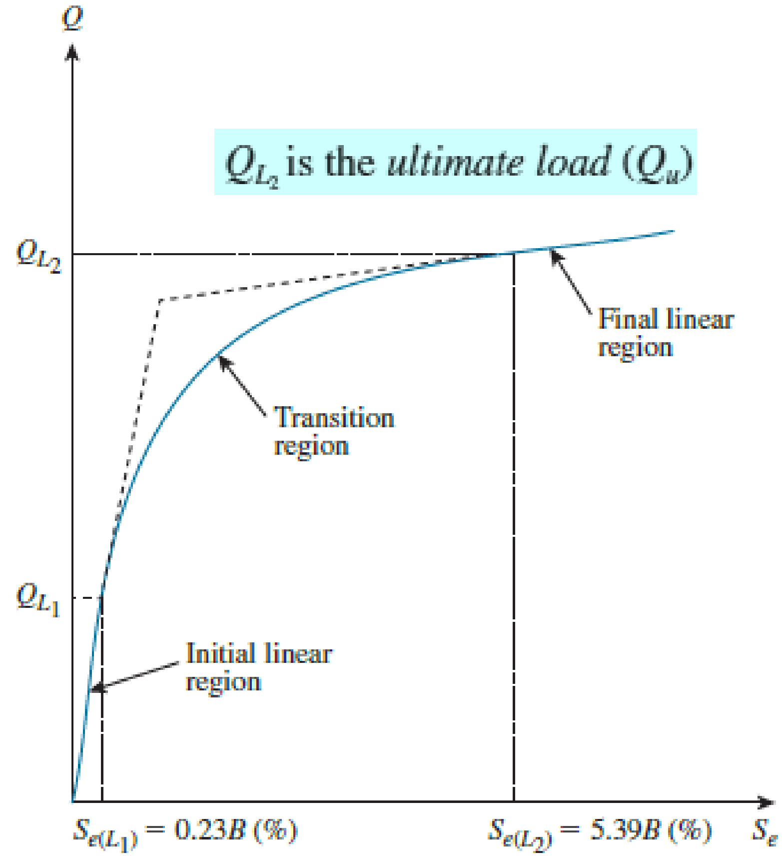
$$S_e = \frac{(2)(120)}{(10)(1.19)}\left(\frac{1.75}{1.75+0.3}\right)^2 = \mathbf{14.7\ mm}$$

Settlement Estimation Using the $L1 - L2$ Method

Akbas and Kulhawy (2009) evaluated 167 load–displacement relationships obtained from field tests.

$$\frac{Q}{Q_{L_2}} = \frac{\left(\frac{S_e}{B}\right)}{0.69\left(\frac{S_e}{B}\right) + 1.68}$$

S_e/B is in percent.



<u>**Akbas and Kulhawy (2009) recommended that**</u>

- For $B > 1$ m [from Eq. (6.42) with $c' = 0$]

$$Q_{L_2} = Q_u = \left[\frac{1}{2}\gamma B N_\gamma F_{\gamma s} F_{\gamma d} F_{\gamma c} + q N_q F_{qs} F_{qd} F_{qc}\right] A$$

where

A = area of the foundation

- For $B \leq 1$ m,

$$Q_{L_2} = \left[\frac{1}{2}\gamma N_\gamma F_{\gamma s} F_{\gamma d} F_{\gamma c} + q N_q F_{qs} F_{qd} F_{qc}\right] A$$

EXAMPLE 9.11

EXAMPLE 9.11

For a square foundation supported by a sand layer, the following are given:

Foundation: $B = 1.5$ m; $D_f = 1$ m

Sand: $\gamma = 16.5$ kN/m^3; $\phi' = 35°$; $G_s = 280$ kN/m^2

Load on foundation: $Q = 800$ kN

Estimate:

a. $S_{e(L_1)}$
b. $S_{e(L_2)}$
c. Settlement S_e with application of load $Q = 800$ kN

SOLUTION

Part a

$$S_{e(L_1)} = 0.23B\ (\%) = \frac{(0.23)(1.5 \times 1000)}{100} = \mathbf{3.45\ mm}$$

Part b

$$S_{e(L_2)} = 5.39B\ (\%) = \frac{(5.39)(1.5 \times 1000)}{100} = \mathbf{80.85\ mm}$$

EXAMPLE 9.11

Part c

B is greater than 1 m.

$$Q_{L_2} = \left[\frac{1}{2}\gamma B N_\gamma F_{\gamma s} F_{\gamma d} F_{\gamma c} + q N_q F_{qs} F_{qd} F_{qc}\right] A$$

$$\gamma = 16.5 \text{ kN/m}^3;\ B = 1.5 \text{ m};\ q = \gamma D_f = (16.5)(1) = 16.5 \text{ kN/m}^2$$

From Table 6.2, for $\phi' = 35°$, $N_\gamma = 48.03$ and $N_q = 33.3$. From Table 6.3,

$$F_{qs} = 1 + \left(\frac{B}{L}\right)\tan\phi' = 1 + \left(\frac{1.5}{1.5}\right)\tan 35 = 1.7$$

$$F_{\gamma s} = 1 - 0.4\left(\frac{B}{L}\right) = 1 - (0.4)\left(\frac{1.5}{1.5}\right) = 0.6$$

$$F_{qd} = 1 + 2\tan\phi'(1 - \sin\phi')^2\left(\frac{D_f}{B}\right) = 1 + 2\tan 35(1 - \sin 35)^2\left(\frac{1}{1.5}\right) \approx 1.17$$

$$F_{\gamma d} = 1$$

In order to calculate F_{qc} and $F_{\gamma c}$, refer to Eq. (6.43) (with $c' = 0$):

$$I_r = \frac{G_s}{q\tan\phi'} = \frac{280}{(16.5)(\tan 35)} = 24.23$$

$$I_{r(cr)} = \frac{1}{2}\left\{\exp\left[\left(3.3 - 0.45\frac{B}{L}\right)\cot\left(45 - \frac{\phi'}{2}\right)\right]\right\}$$

$$= \frac{1}{2}\left\{\exp\left[\left(3.3 - 0.45\frac{1.5}{1.5}\right)\cot\left(45 - \frac{35}{2}\right)\right]\right\} = 119.3$$

EXAMPLE 9.11

So, $I_r < I_{r(cr)}$.

$$F_{\gamma c} = F_{qc} = \exp\left\{\left[-4.4 + 0.6\left(\frac{B}{L}\right)\right]\tan\phi' + \frac{(3.07\sin\phi')(\log 2I_r)}{1 + \sin\phi'}\right\}$$

$$= \exp\left\{\left[-4.4 + 0.6\left(\frac{1.5}{1.5}\right)\right]\tan 35 + \frac{(3.07\sin 35)(\log 2\times 24.23)}{1 + \sin 35}\right\}$$

$$= 0.461$$

Thus,

$$Q_{L_2} = \begin{bmatrix} \left(\frac{1}{2}\right)(16.5)(1.5)(48.03)(0.6)(1)(0.461) \\ + (16.5)(33.3)(1.7)(1.7)(0.461) \end{bmatrix}(1.5\times 1.5) = 1467.4\text{ kN}$$

Substituting the values of Q and Q_{L_2} in Eq. (9.76),

$$\frac{800}{1467.4} = \frac{\left(\frac{S_e}{B}\right)}{0.69\left(\frac{S_e}{B}\right) + 1.68}; \quad \frac{S_e}{B} = 1.467\%$$

$$S_e = (1.467)\left(\frac{1.5\times 1000}{100}\right) \approx \mathbf{22.0\ mm}$$

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

Effect of the Rise of Water Table on Elastic Settlement

- Peck, Hansen, and Thornburn (1974):

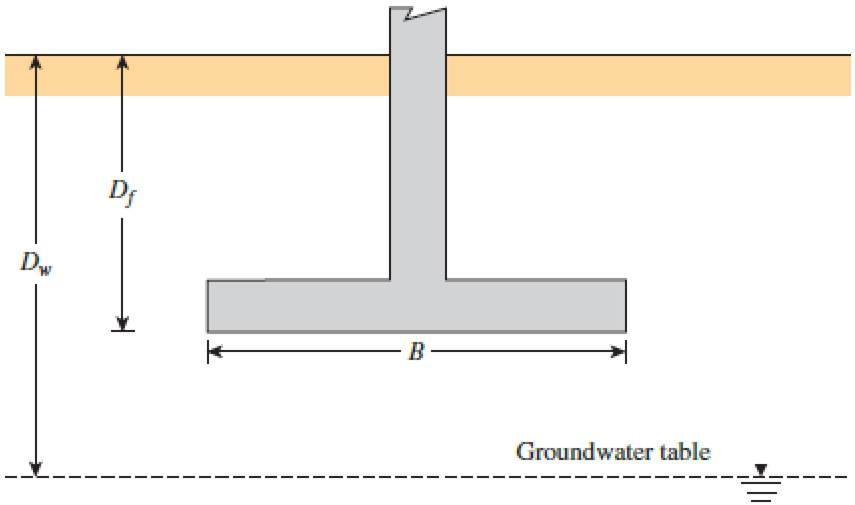
$$C_w = \frac{1}{0.5 + 0.5\left(\frac{D_w}{D_f + B}\right)} \geq 1$$

- Teng (1982):

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

- Bowles (1977):

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



Effect of the Rise of Water Table on Elastic Settlement

Method of Shahriar et al. (2014)

- **When the water table is present in the vicinity of the foundation, the unit weight of the soil has to be reduced for calculation of bearing capacity.**
- **Any future rise in the water table can reduce the ultimate bearing capacity. A future water table rise in the vicinity of the foundation in granular soil can reduce the soil stiffness and, hence, produce additional settlement.**
- **Terzaghi (1943) concluded that when the water table rises from very deep to the foundation level, the settlement will be doubled in granular soil.**

Effect of the Rise of Water Table on Elastic Settlement

Method of Shahriar et al. (2014)

Provided that the settlement is doubled when the entire sand layer beneath the foundation is submerged, laboratory model test results and numerical modeling work by Shahriar et al. (2014) show that the additional settlement produced by the rise of water table to any height can be expressed as

$$S_{e,\ \text{additional}} = \frac{A_w}{A_t} S_e$$

where S_e is the elastic settlement computed in dry soil, A_w is the area of the strain influence diagram submerged due to water table rise, and A_t is the total area of the strain influence diagram under the foundation.

EXAMPLE 9.12

EXAMPLE 9.12

Consider the shallow foundation given in Example 9.7. Due to flooding, the groundwater table rose from $D_w = 4$ m to 2 m (. Estimate the total elastic settlement S'_e after the rise of the water table.

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.

SOLUTION

$$S'_e = S_e C_w$$

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

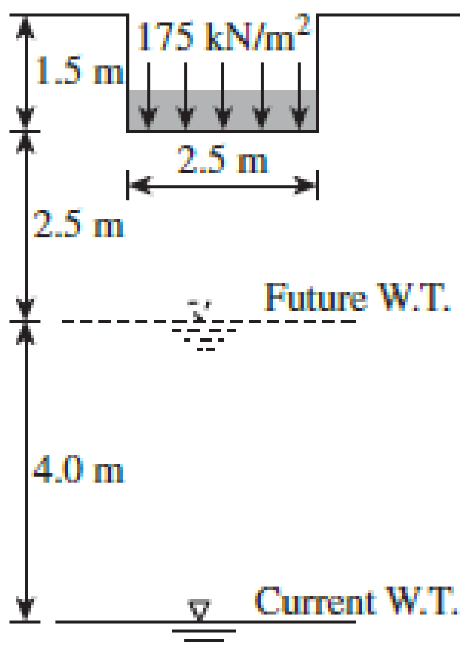
Hence,

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

EXAMPLE 9.13

EXAMPLE 9.13

A pad foundation 2.5 m × 2.5 m in plan, when placed at a depth of 1.5 m in sand, applies 175 kN/m^2 pressure to the underlying ground. Given: $\gamma = 18.0$ kN/m^3. Currently the water table is at 6.5 m below the foundation, and the expected settlement is 15.0 mm. In the future, as the worst-case scenario, it is expected that the water table could rise by 4.0 m, as shown in Figure   What would be the total settlement of the foundation if this occurs?



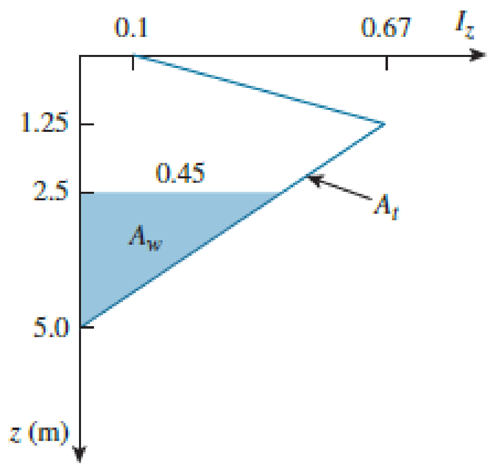
EXAMPLE 9.13

SOLUTION

The influence factor diagram needs to be drawn first.

$$I_{z(m)} = 0.5 + 0.1\sqrt{\frac{\bar{q} - q}{q'_{z(1)}}} = 0.5 + 0.1\sqrt{\frac{175 - (18.0)(1.5)}{(18.0)\left[1.5 + \left(\frac{2.5}{2}\right)\right]}} = 0.67$$

The I_z versus z diagram is shown in Figure   Currently, the water table is below the influence zone. $S_e = 15.0$ mm. The total area of the influence diagram A_t is given by



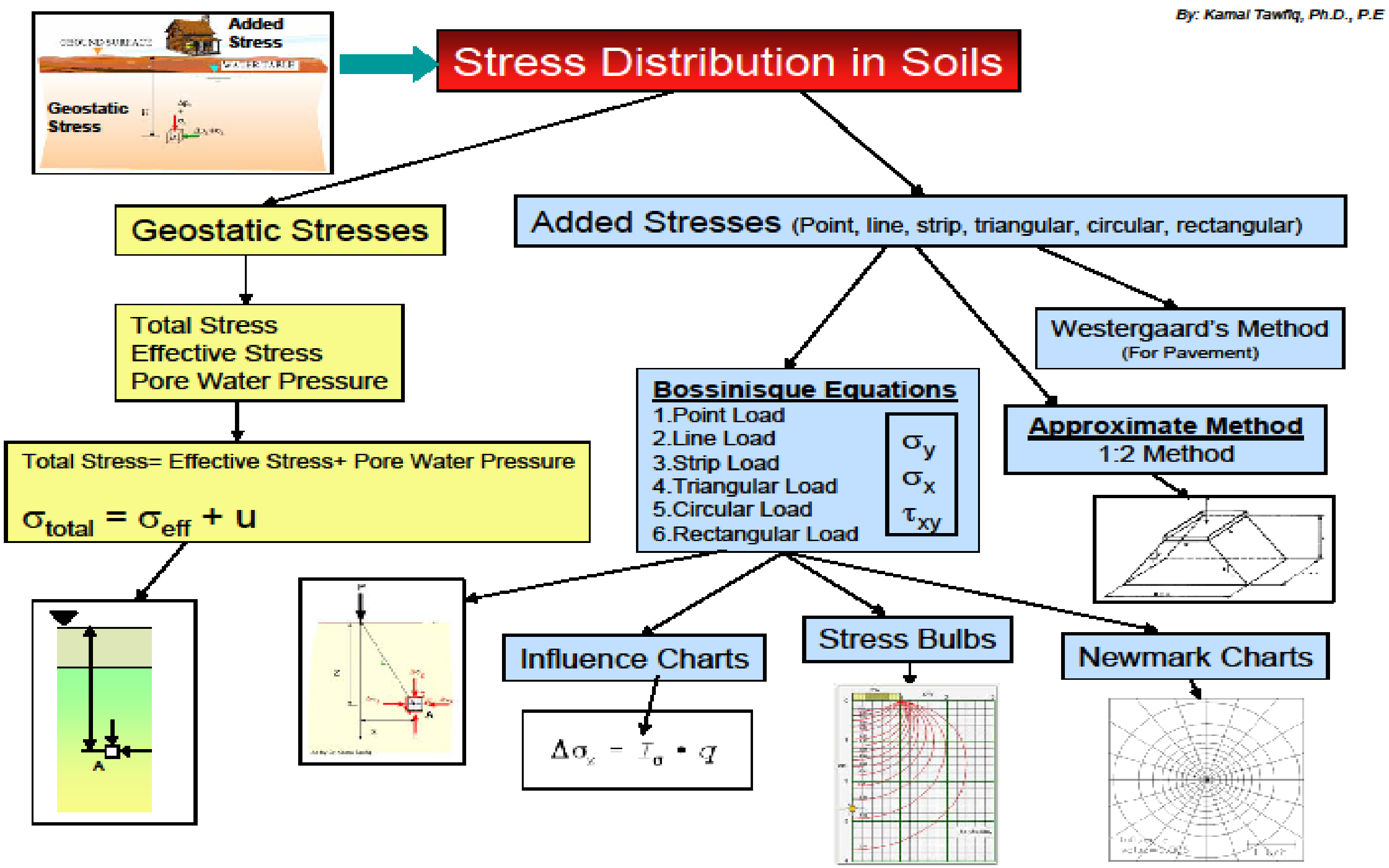
$$A_t = \left(\frac{0.10 + 0.67}{2}\right) \times 1.25 + \frac{1}{2} \times 0.67 \times 3.75 = 1.738 \text{ m}$$

$$A_w = \frac{1}{2} \times 2.5 \times 0.45 = 0.563 \text{ m}$$

$$S_{e,\text{ additional}} = \frac{A_w}{A_t} S_e = \frac{0.563}{1.738} \times 15.0 = 4.9 \text{ mm}$$

The total settlement would be 15.0 + 4.9 = **19.9 mm.**

Stresses Distribution in Soils



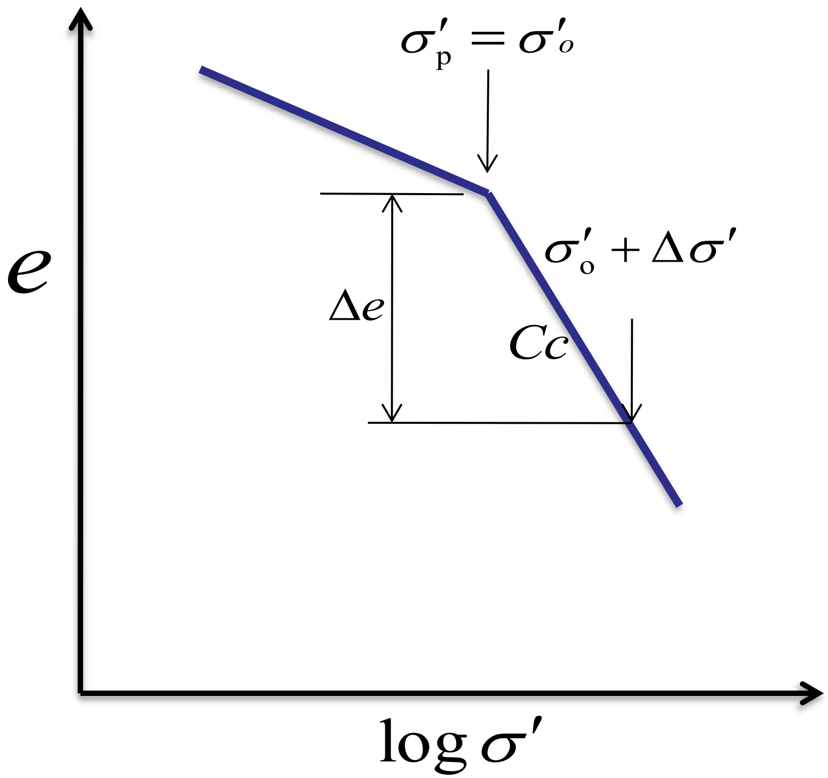
Calculation of Primary Consolidation Settlement

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

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

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

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



Calculation of Primary Consolidation Settlement

b) Overconsolidated Clays

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

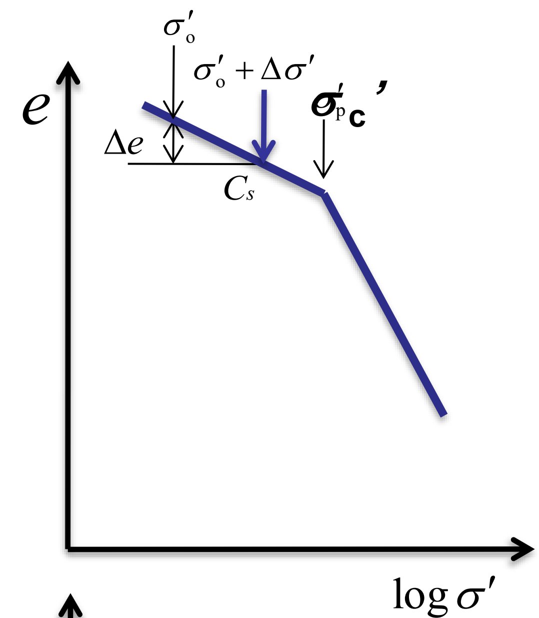
Case I: $\sigma'_0 + \Delta\sigma' \leq \sigma_c'$

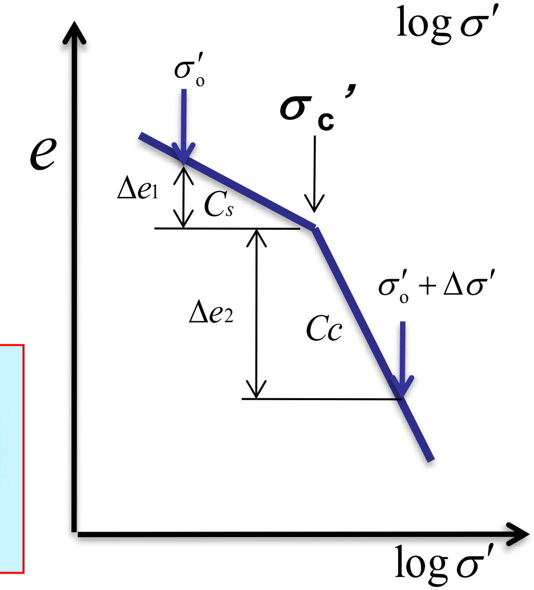
$$\Delta e = C_s[\log(\sigma'_o + \Delta\sigma') - \log \sigma'_o]$$

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

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

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





Calculation of Primary Consolidation Settlement

$$S_{c(p)} = \frac{C_c H_c}{1 + e_o} \log \frac{\sigma'_o + \Delta\sigma'_{av}}{\sigma'_o}$$ (for normally consolidated clays)

$$S_{c(p)} = \frac{C_s H_c}{1 + e_o} \log \frac{\sigma'_o + \Delta\sigma'_{av}}{\sigma'_o}$$ for overconsolidated clays with $\sigma'_o + \Delta\sigma'_{av} < \sigma'_c$

$$S_{c(p)} = \frac{C_s H_c}{1 + e_o} \log \frac{\sigma'_c}{\sigma'_o} + \frac{C_c H_c}{1 + e_o} \log \frac{\sigma'_o + \Delta\sigma'_{av}}{\sigma'_c}$$ for overconsolidated clays with $\sigma'_o < \sigma'_c < \sigma'_o + \Delta\sigma'_{av}$

where

σ'_o = average effective pressure on the clay layer before the construction of the foundation

$\Delta\sigma'_{av}$ = average increase in effective pressure on the clay layer caused by the construction of the foundation

σ'_c = preconsolidation pressure

e_o = initial void ratio of the clay layer

C_c = compression index

C_s = swelling index

H_c = thickness of the clay layer

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.

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

$If\ \ \sigma'_0 + \Delta\sigma \le \sigma'_c$

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

$If\ \ \sigma'_0 + \Delta\sigma > \sigma'_c$

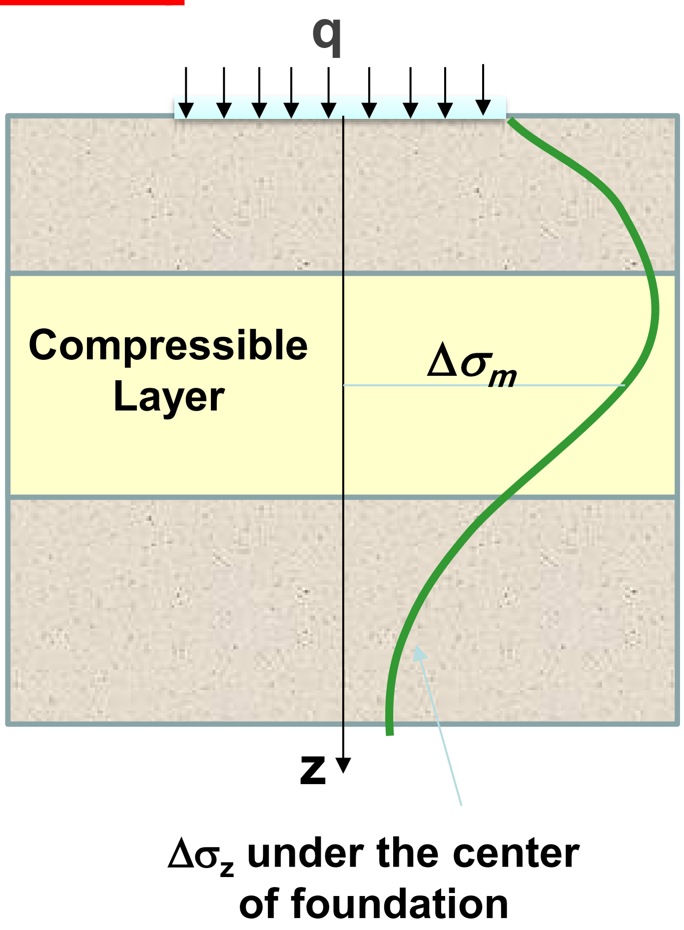
Nonlinear pressure increase

Approach 1: Middle of layer (midpoint rule)

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

$$\Delta\sigma_z = \Delta\sigma_m$$

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

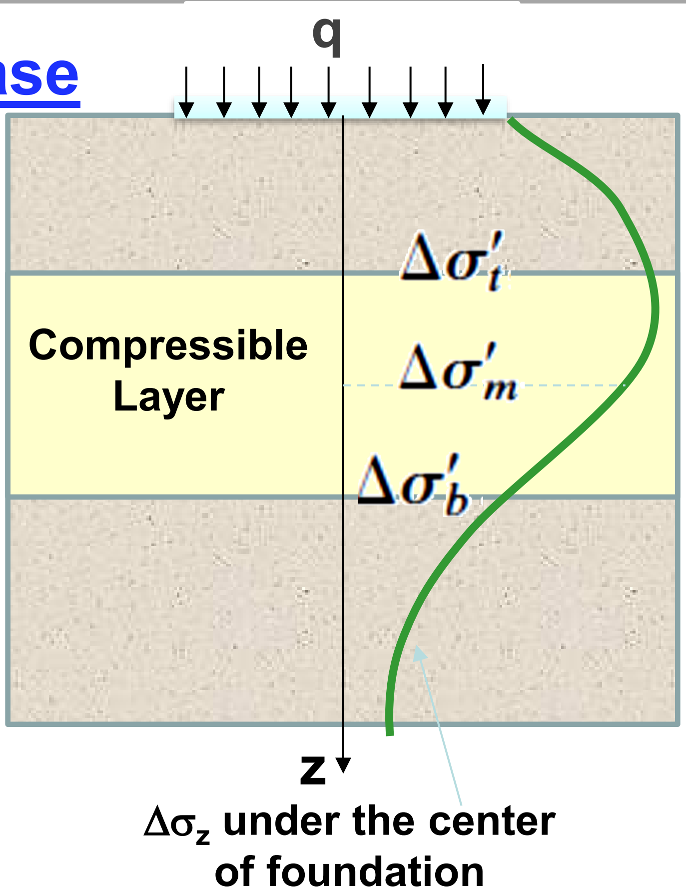


Nonlinear pressure increase

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):

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

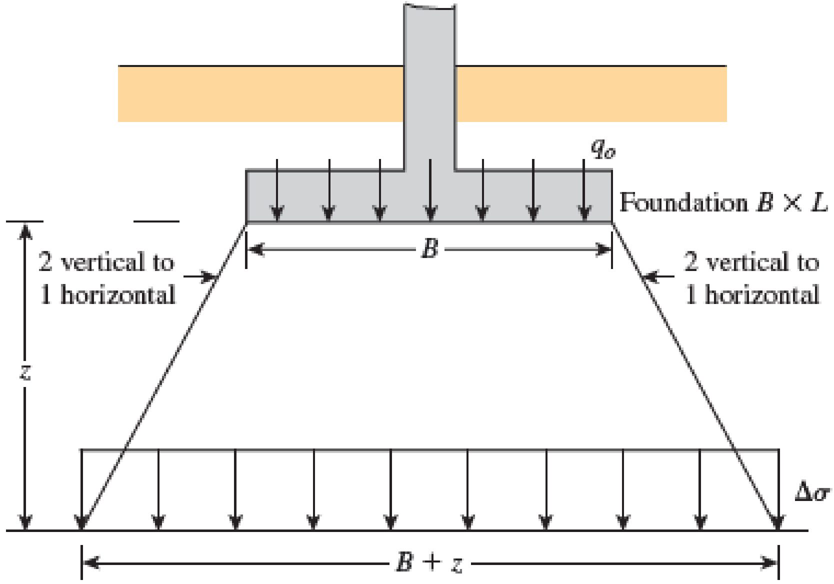


where $\Delta\sigma_t$, $\Delta\sigma_m$ and $\Delta\sigma_b$ represent the increase in the pressure at the top, middle, and bottom of the clay, respectively, under the center of the footing.

Stress from Approximate Method

2:1 Method

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



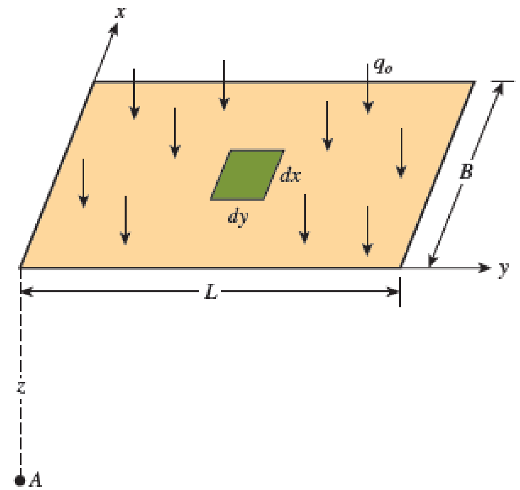
Stress below a Rectangular Area

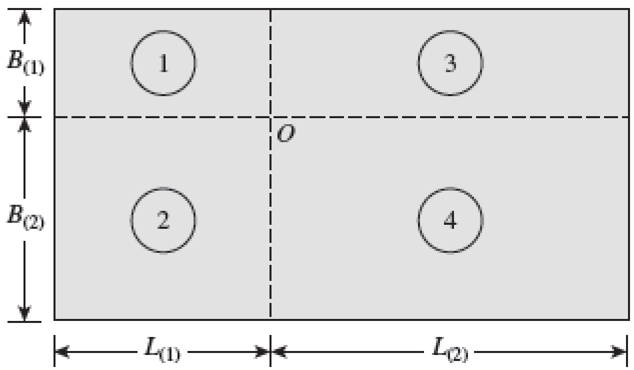
$$\Delta\sigma = \int_{y=0}^{L}\int_{x=0}^{B} \frac{3q_o\,(dx\,dy)z^3}{2\pi(x^2+y^2+z^2)^{5/2}} = q_o I$$

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

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

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





Stress below a Rectangular Area

TABLE 8.5 Variation of Influence Value I [Eq. (8.11)][a]

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

Three-Dimensional Effect on Primary Consolidation Settlement

Skempton–Bjerrum Modification **(1957) for a consolidation settlement calculation**

$$S_{c(p)-\text{oed}} = \int \frac{\Delta e}{1+e_o}\,dz = \int m_v \Delta\sigma'_{(1)}\,dz$$

$S_{c(p)-\text{oed}}$ = one-dimensional consolidation settlement calculated by using oedometer data

$\Delta\sigma'_{(1)}$ = effective vertical stress increase

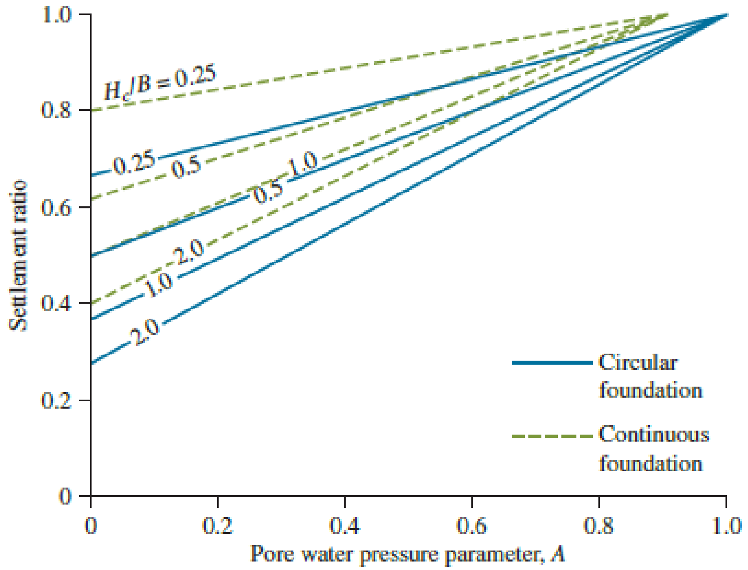
m_v = volume coefficient of compressibility

$$\Delta u = \Delta\sigma_{(3)} + A[\Delta\sigma_{(1)} - \Delta\sigma_{(3)}]$$

where A = pore water pressure parameter

$$K_{\text{cir}} = \frac{S_{c(p)}}{S_{c(p)-\text{oed}}}$$

where K_{cir} = settlement ratio for circular foundations.



Three-Dimensional Effect on Primary Consolidation Settlement

Leonards (1976) examined the correction factor K_{cr} for a three-dimensional consolidation effect in the field for a circular foundation located over *overconsolidated clay*.

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

$$K_{cir(OC)} = f\left(\text{OCR}, \frac{B}{H_c}\right)$$

$$\text{OCR} = \text{overconsolidation ratio} = \frac{\sigma'_c}{\sigma'_o}$$

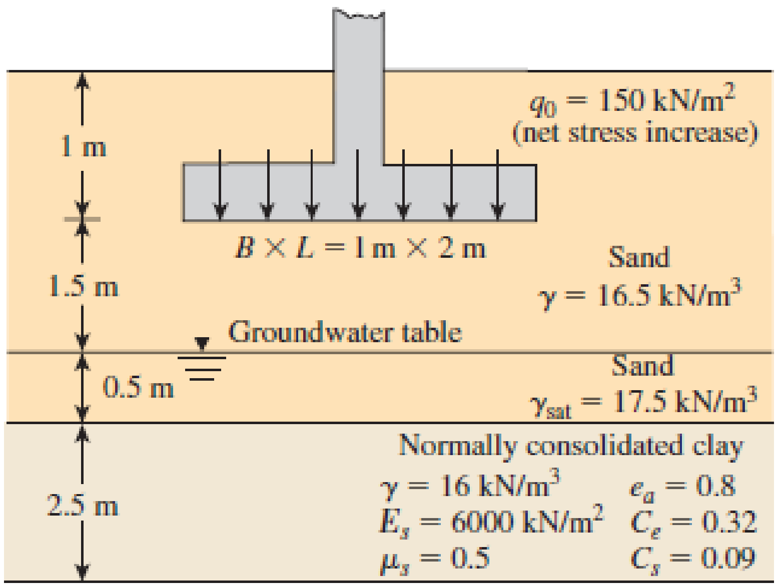
TABLE 9.9 Variation of $K_{cir(OC)}$ with OCR and B/H_c

	$K_{cir(OC)}$		
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 9.14

EXAMPLE 9.14

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



EXAMPLE 9.14

The clay is normally consolidated. Thus,

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

so

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

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

We divide the foundation into four quarters, compute the stress increase under a corner of each quarter and multiply by four. For each quarter, $B = 0.5$ m and $L = 1.0$ m.

Location	z (m)	$m = B/z$	$n = L/z$	I	$\Delta\sigma' = 4q_o I$
Top	2.0	0.25	0.50	0.0475	$28.5 = \Delta\sigma'_t$
Middle	3.25	0.154	0.308	≈ 0.0213	$12.75 = \Delta\sigma'_m$
Bottom	4.5	0.111	0.222	≈ 0.0113	$6.75 = \Delta\sigma'_b$

Now,

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

so

$$S_{c(p)-\text{oed}} = \frac{(0.32)(2.5)}{1 + 0.8} \log\left(\frac{52.84 + 14.38}{52.84}\right) = 0.0465 \text{ m}$$
$$= 46.5 \text{ mm}$$

EXAMPLE 9.14

Now assuming that the 2:1 method of stress increase holds, 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 9.32, for $A = 0.6$ and $H_c/B_{eq} = 0.64$, the magnitude of $K_{cr} \approx 0.78$. Hence,

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

Secondary Consolidation Settlement

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

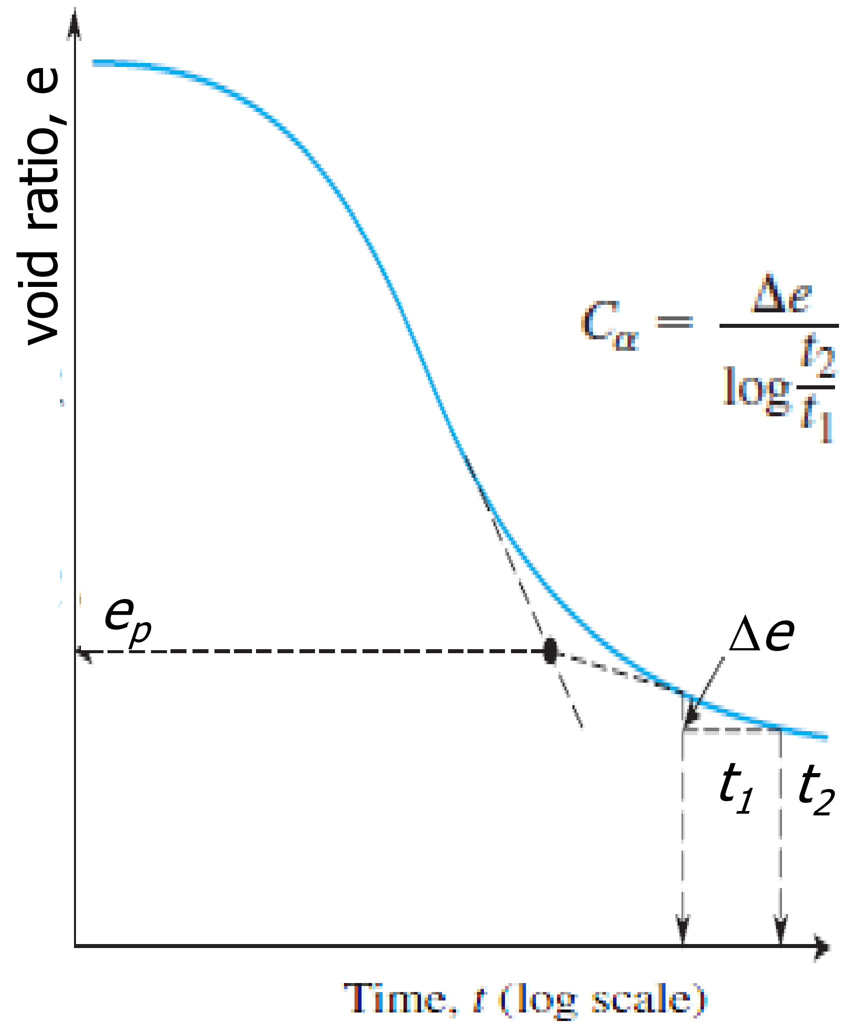
$$S_s = \frac{H}{1 + e_p} \Delta e$$

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

$$\Delta e = C_\alpha log\,(t_2/t_1)$$

C_α = coefficient of secondary compression

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



Secondary Consolidation Settlement

$$C_\alpha = \frac{\Delta e}{\log t_2 - \log t_1} = \frac{\Delta e}{\log(t_2/t_1)}$$

where

C_α = secondary compression index

Δe = change of void ratio

t_1, t_2 = time

The magnitude of the secondary consolidation can be calculated as

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

where

$C'_\alpha = C_\alpha/(1 + e_p)$

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 soil, from which it appears that

$$C'_\alpha \approx 0.0001w$$

where w = natural moisture content, in percent. For most overconsolidated soil, C'_α varies between 0.0005 to 0.001.

Secondary Consolidation Settlement

Mesri and Godlewski (1977) compiled the magnitude of C_α/C_c (C_c = compression index) for a number of soil. 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:

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

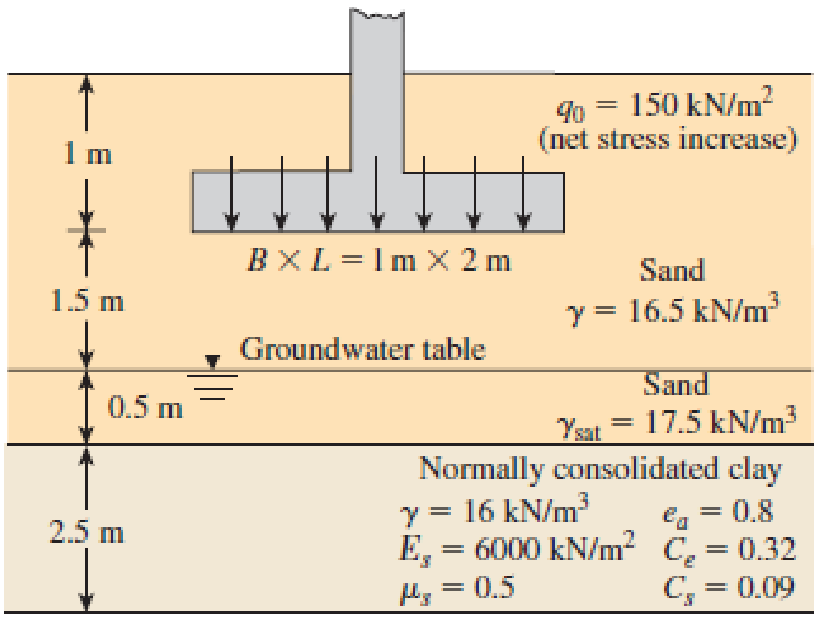
- For peats:

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

EXAMPLE 9.15

EXAMPLE 9.15

Refer to Example 9.14. 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.)



EXAMPLE 9.15

SOLUTION

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

For this problem, $e_1 - e_2 = \Delta e$. Referring to Example 9.14, 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,

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

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

EXAMPLE 9.15

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

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

Thus,

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

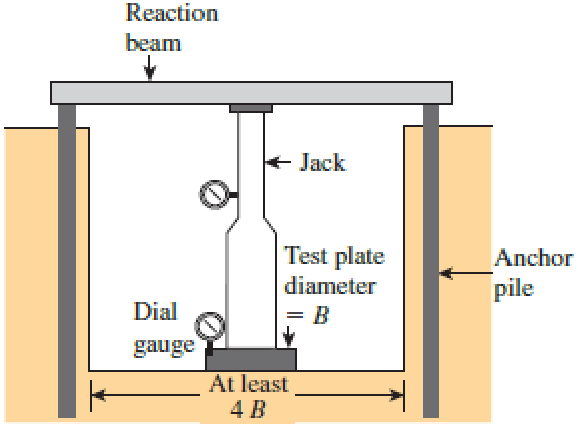
Total consolidation settlement is

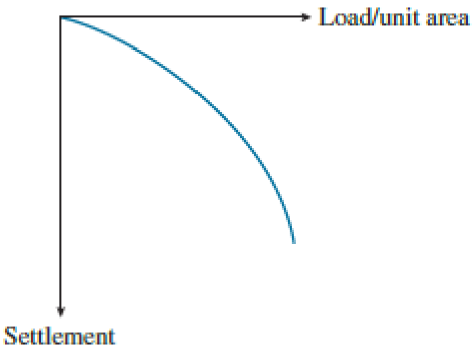
$$\underbrace{36.3 \text{ mm}}_{\substack{\uparrow \\ \text{Example 9.14} \\ \text{(primary} \\ \text{consolidation} \\ \text{settlement)}}} + 19.4 = \mathbf{55.7 \text{ m}}$$

Field Load Test

The Plate Load Test (PLT)

Plate load test in the field





Field 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

\- the ultimate bearing capacity in clay is virtually independent of the size of the plate.

For tests in sandy soil,

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

where

B_F = width of the foundation

B_P = width of the test plate

Field Load Test

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

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

and

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

THE END