5-8 IMMEDIATE SETTLEMENTS: OTHER CONSIDERATIONS

We can interpret Eq. (5-16*a*) in terms of the Mechanics of Materials equation

$$\Delta H = \frac{PH}{AE} = \sigma \frac{L}{E} = \epsilon H$$

as previously given (and using symbols consistent with this text) where

$$\epsilon = q_o/E_s \qquad H = B(1 - \mu^2)mI_sI_F$$

The major problems, of course, are to obtain the correct E_s and H. It has already been noted with reference to Table 5-3 that one should use the weighted average value of E_s in the influence depth H. Obviously if H is fairly large and one obtains somehow only one value of E_s the resulting computation for ΔH may not be very reliable unless that one value happens to be the "weighted average" on a chance basis.

It is evident that for the usual range of Poisson's ratio μ of 0.2 to 0.4, this parameter has little effect on ΔH (using the extreme range from 0 to 0.5 only produces a maximum difference of 25 percent).

The influence depth H can be estimated reasonably well as noted with reference to Table 5-3 by taking the smaller of $5B$ or the depth to the hard layer, where the "hard" layer was defined as that where the stress-strain modulus was $\geq$ 10 times E_s of the next adjacent layer. One will have to use some judgment if the soil grades from stiff to stiffer so that a factor of 10 is not clearly defined.

Finally we note that the depth factor I_F can reduce computed settlements considerably for $D/B \rightarrow 1$.

Determination of the Stress-Strain Modulus E_s

Several methods are available for determining (actually estimating) the stress-strain modulus:

1. Unconfined compression tests
2. Triaxial compression tests
3. In situ tests
   a. SPT
   b. CPT
   c. Pressuremeter
   d. Flat dilatometer
   e. Iowa stepped blade
   f. Plate-load tests

Unconfined compression tests tend to give conservative values of E_s; i.e., the computed value (usually the initial tangent modulus) is too small, resulting in computed values of ΔH being large compared with any measured value. If the value of ΔH is excessively[2] large, the selection of foundation type may be adversely affected; that is, a recommendation for piles or caissons might be made when, in fact, spread footings would be satisfactory.

[2]Termed *overly conservative* in engineering lexicon.

Triaxial tests tend to produce more usable values of E_s since any confining pressure "stiffens" the soil so that a larger initial tangent modulus is obtained. Other factors such as whether the triaxial test is a U, CU, or CK_oU tend to affect the E_s obtained (see Sec. 2-14). Generally triaxial tests will also be conservative but not quite so much as unconfined compression tests. This observation was somewhat confirmed by Crawford and Burn (1962), where E_s in situ was estimated to be 4 to 13 times as large as that obtained from laboratory q_u test plots and about 1 to 1.5 times those obtained from triaxial U tests.

The in situ tests of SPT and CPT tend to use empirical correlations to obtain E_s. Other in situ tests such as the pressuremeter, the flat dilatometer, and the Iowa stepped blade tend to obtain more direct measurements of E_s. The value of stress-strain modulus E_s obtained from these tests is generally the horizontal value—but the vertical value is usually needed for settlements. Most soils are anisotropic, so the horizontal E_{sh} value may be considerably different from the vertical value E_{sv}.[3] Overconsolidation may also alter the vertical and horizontal values of stress-strain modulus.

Anisotropy, stress history, natural cementation, and overconsolidation are likely to be very significant factors in determining E_s, especially for cohesionless soils. In cohesionless soils cementation is particularly significant; for individual soil grains the effect can be very small, but the statistical accumulation for the mass can have a large effect. Cementation (also called "aging") can be easily lost in recovered cohesionless samples. Drilling disturbances in cohesionless soils for the purpose of performing pressuremeter, dilatometer, or other tests may sufficiently destroy the cementation/aging in the vicinity of the hole to reduce E_s to little more than an estimate.

Because the laboratory values of E_s are expensive to obtain and are generally not very good anyway owing to sampling disturbance, the standard penetration test (SPT) and cone penetration test (CPT) have been widely used to obtain the stress-strain modulus E_s resulting from empirical equations and/or correlations. Table 5-6 gives a number of equations for possible use in several test methods. The value to use should be based on local experience with that equation giving the best fit for that locality. Referring to Table 5-6, we can see that a good estimate for the SPT is

$$E_s = C_1(N + C_2)$$

where values of $C_2 = 6$ and 15 are shown and C_1 ranges from 250 upward. This equation can also be written (see again Table 5-6) as

$$E_s = C_2' + C_1 N \qquad C_2' = C_1 C_2$$

For best results one should attempt to determine the C_i constants for the local area. The increase for $E_{s,\mathrm{OCR}}$ using the multiplier $\sqrt{\mathrm{OCR}}$ seems to be reasonably valid (and substantially used), although again local materials/practice might produce a slightly better multiplier.

For the CPT test the stress-strain modulus in Table 5-6 is of the general form

$$E_s = C_3 + C_4 q_c$$

where C_3 ranges from 0 upward and C_4 may be one of the values also shown in Table 5-6. Values of $C_3 = 0$ and $C_4 = 2.5$ to 3.0 for normally consolidated sands seem rather widely used.

[3]Always used as E_s in this text unless specifically noted otherwise.

A significant factor for the CPT is that there may be some critical depth below which the cone resistance q_c is nearly constant. This has a theoretical basis in that, below this depth, a local bearing failure develops in a small zone around the tip of the cone. Obviously the soil stiffens with depth (but not beyond bound). Depth increases may not be very large owing to "local" failure around the cone tip. Thus, the use of an equation of the general form

$$E_s = C_3 + C_4 q_c \tan^{-1}\left(\frac{z}{D}\right)^n$$

may be necessary to maintain reasonable values for E_s at the several depth increments z through the test zone depth of D.

For this reason values of E_s obtained using N values from the SPT may be more reliable than those from the CPT. We also note that the cone test is essentially a measure of ultimate bearing capacity on the cone tip (which has an area of only 10 cm^2). This phenomenon is illustrated on Figs. 3-14, 3-17, and in the cone data of Table P3-11, where nearly constant q_c values are shown at large D/B ratios. This observation means that one may not obtain very good estimates of E_s at depths beyond the critical depth (usually in the form of a depth ratio such as 15 to 100 D/B)[4] of the cone unless the overburden pressure over the depth of interest is somehow included, perhaps by using a variable C_5 ranging from 0 to 100 as follows:

$$C_4 = \left(\frac{C_5 + p'_o}{p'_o}\right)^n \qquad \text{or} \qquad C_4 = C_5 + \log p'_o$$

where p'_o = the effective overburden pressure at the depth D (or D/B) of interest as previously defined in Chap. 2

n = exponent with a value usually ranging from 0.4 to 0.7 (but other values might be used)

The Effect of the Overconsolidation Ratio (OCR) on E_s

Table 5-6 gives the commonly used multiplier $\sqrt{\text{OCR}}$ used to increase the normally consolidated value of stress-strain modulus $E_{s,\text{nc}}$. By using the square root of OCR the effect is certainly not so great as using OCR as the multiplier. When the soil is *overconsolidated* the following occur:

1. The soil $E_{s,\text{OCR}}$ should be larger than $E_{s,\text{nc}}$. However, we are usually concerned with the vertical value, so that the "OCR" value may not be much larger than the normally consolidated vertical value of E_s.
2. If in situ tests are used, the horizontal value of E_{sh} is obtained. For an overconsolidated soil this value may be very much larger than the vertical value, but this estimate depends heavily on how much soil disturbance (or lateral expansion) occurred when the hole was drilled and/or test device inserted.
3. In overconsolidated soils if the soil is excavated (as for a large and/or deep basement) and expands from loss of overburden, the resulting E_s is smaller than before and may be very much smaller, perhaps requiring a new test(s).

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[4]Noting that the cone diameter B ($= 35.6$ mm) is not great, only a shallow depth D will produce a large D/B ratio for a cone test.

TABLE 5-6

Equations for stress-strain modulus E_s by several test methods

E_s in kPa for SPT and units of q_c for CPT; divide kPa by 50 to obtain ksf. The N values should be estimated as N_{55} and not N_{70}. Refer also to Tables 2-7 and 2-8.

Soil	SPT	CPT
Sand (normally consolidated)	$E_s = 500(N + 15)$	$E_s = (2 \text{ to } 4)q_u$
	$= 7000\sqrt{N}$	$= 8000\sqrt{q_c}$
	$= 6000N$	— — —
	— — —	$E_s = 1.2(3D_r^2 + 2)q_c$
	$\ddagger E_s = (15\,000 \text{ to } 22\,000) \cdot \ln N$	$*E_s = (1 + D_r^2)q_c$
Sand (saturated)	$E_s = 250(N + 15)$	$E_s = Fq_c$
		$e = 1.0 \quad F = 3.5$
		$e = 0.6 \quad F = 7.0$
Sands, all (norm. consol.)	$\P E_s = (2600 \text{ to } 2900)N$	
Sand (overconsolidated)	$\dagger E_s = 40\,000 + 1050N$	$E_s = (6 \text{ to } 30)q_c$
	$E_{s(\mathrm{OCR})} \approx E_{s,\mathrm{nc}}\sqrt{\mathrm{OCR}}$	
Gravelly sand	$E_s = 1200(N + 6)$	
	$= 600(N + 6) \quad N \le 15$	
	$= 600(N + 6) + 2000 \quad N > 15$	
Clayey sand	$E_s = 320(N + 15)$	$E_s = (3 \text{ to } 6)q_c$
Silts, sandy silt, or clayey silt	$E_s = 300(N + 6)$	$E_s = (1 \text{ to } 2)q_c$
	If $q_c < 2500$ kPa use $\quad {}^{\$}E_s' = 2.5q_c$	
	$2500 < q_c < 5000$ use $\quad E_s' = 4q_c + 5000$	
	where	
	$E_s' = \text{constrained modulus} = \dfrac{E_s(1-\mu)}{(1+\mu)(1-2\mu)} = \dfrac{1}{m_v}$	
Soft clay or clayey silt		$E_s = (3 \text{ to } 8)q_c$

4. It is not easy to determine if a cohesionless deposit is overconsolidated or what the OCR might be. Cementation may be less difficult to discover, particularly if during drilling or excavation sand "lumps" are present. Carefully done consolidation tests will aid in obtaining the OCR of cohesive deposits as noted in Chap. 2.

In general, with an OCR > 1 you should carefully ascertain the site conditions that will prevail at the time settlement becomes the design concern. This evaluation is, of course, true for any site, but particularly so if OCR > 1.

5-9 SIZE EFFECTS ON SETTLEMENTS AND BEARING CAPACITY

5-9.1 Effects on Settlements

A major problem in foundation design is to proportion the footings and/or contact pressure so that settlements between adjacent footings are nearly equal. Figure 5-9 illustrates the problem

TABLE 5-6

Equations for stress-strain modulus E_s by several test methods (*continued*)

E_s in kPa for SPT and units of q_c for CPT; divide kPa by 50 to obtain ksf. The N values should be estimated as N_{55} and not N_{70}. Refer also to Tables 2-7 and 2-8.

Soil

Use the undrained shear strength s_u in units of s_u

Clay and silt	$I_P > 30$ or *organic*	$E_s = (100 \text{ to } 500)s_u$
Silty or sandy clay	$I_P < 30$ or *stiff*	$E_s = (500 \text{ to } 1500)s_u$ Again, $E_{s,\mathrm{OCR}} \approx E_{s\,\mathrm{nc}}\sqrt{\mathrm{OCR}}$ Use smaller s_u-coefficient for highly plastic clay.

Of general application in clays is

$$E_s = Ks_u \qquad \text{(units of } s_u\text{)} \qquad (a)$$

where K is defined as

$$K = 4200 - 142.54 I_P + 1.73 I_P^2 - 0.0071 I_P^3 \qquad (b)$$

and I_P = plasticity index in **percent.** Use $20\% \leq I_P \leq 100\%$ and round K to the nearest multiple of 10.

Another equation of general application is

$$E_s = 9400 - 8900 I_P + 11\,600 I_c - 8800 S \qquad \text{(kPa)}$$
$$I_P, I_c, S = \text{previously defined above and/or in Chap. 2} \qquad (c)$$

*Vesić (1970).

†Author's equation from plot of D'Appolonia et al. (1970).

‡USSR (may not be standard blow count N).

¶Japanese Design Standards (lower value for structures).

$Senneset et al. (1988)

General sources: First European Conference on Standard Penetration Testing (1974), vol. 2.1, pp. 150–151; *CGJ,* November 1983, pp. 726–737; *Use of In Situ Tests in Geotechnical Engineering,* ASCE (1986), p. 1173; Mitchell and Gardner (1975); *Penetration Testing (Second European Conference)* (1982), vol. 1, p. 160; 11th ICSMFE (1985), vol. 2, pp. 462, 765; vol. 4, p. 2185; *International Symposium on Penetration Testing* (1988), 2 vols.

Notes:

1. For q_c generally use (2.5 to 3)q_c for normally consolidated sand and about 4 to 6 q_c for overconsolidated sand.
2. Can use Eqs. (a) and (b) above for all clay. They are particularly applicable for OCR > 1. Probably should use both Eqs. (a) and (c), and if results differ significantly either use an average or compute another E_s using a different equation.
3. For sands try to use more than one equation or else use one of the equations and compare the computed E_s to published table (see Table 2-8) values.
4. For silts use any of the above equations, but if the equations are given for sand use smaller coefficients.
5. For sand, using $E_s = 250$ or $500(N + 15)$ may give a modulus that is too small (but conservative). Suggest when you use equations of this form you compute E_s by one or more additional equations and average the results.
6. Note: Using $\sqrt{\mathrm{OCR}}$ is the same as $(\mathrm{OCR})^{1/2}$, so that exponent $n = 0.5$. You can use other values for the exponent from about 0.3 to 0.5. However, since all the equations for E_s are approximations the use of $n = 0.5$ is sufficiently accurate unless you have good-quality field or laboratory test values.

(and why plate load tests have little real value). It is evident that if the depth of influence is $H = 5B$, a 0.3-m square plate has an influence depth of $5 \times 0.3 = 1.5$ m, whereas a 2-m prototype would have a depth of $5 \times 2 = 10$ m. Considerable changes in the soil can occur in that amount of depth increase.

To address this problem theoretically, let us rewrite Eq. (5.16a) [taking $(1-\mu^2)/E_s = E_s'$] as

$$\Delta H_1 = q_{o1} B_1' m I_{s1} I_{F1} E_{s1}' \tag{a}$$

$$\Delta H_2 = q_{02} B_2' m I_{s2} I_{F2} E_{s2}' \tag{b}$$

where q_{oi} = base contact pressure (usually using the allowable bearing pressure q_a)
B_i' = base widths as defined with Eq. (5-16a)
I_{si} = settlement influence factors based on $H/B_{i,i}'$ and L'/B'
m = number of I_{si} contributions, 1, 4, etc.
I_{Fi} = factors based on the D/B_i ratio
E_{si}' = average stress-strain modulus over the effective depths H (= $5B$ or actual H to hard stratum). In general, $E_{s2}' < E_{s1}'$ for $B_2' > B_1'$ but the increase will not usually be linear.

Dividing Eq. (b) by Eq. (a) we obtain

$$\frac{\Delta H_2}{\Delta H_1} = \frac{q_{o2}}{q_{o1}} \frac{B_2'}{B_1'} \frac{m I_{s2}}{m I_{s1}} \frac{I_{F2}}{I_{F1}} \frac{E_{s2}'}{E_{s1}'} \tag{5-18}$$

This equation is as theoretically correct as the basic settlement equations. What has been done in the past is this:

1. For clay soils assume constant E_{si}', I_{Fi}, and mI_{si} so that we have

$$\frac{\Delta H_2}{\Delta H_1} = \frac{q_{o2}}{q_{o1}} \frac{B_2'}{B_1'} \tag{c}$$

which simplifies for constant contact pressure $q_o (= q_{o1} = q_{o2})$ to

$$\Delta H_2 = \Delta H_1 (B_2'/B_1') \tag{d}$$

This equation has been very widely used for clay soils. It simply states in equation form that the settlement of a footing of width B_2 is the settlement of a footing of width

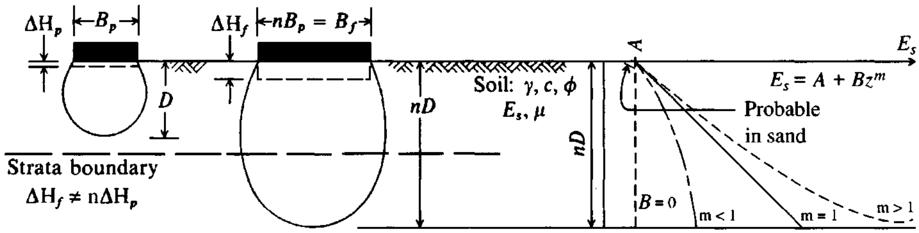

Figure 5-9 Influence of footing size on the depth of the stress zone and E_S. Note that, with an underlying stratum of different soil, the plate settlement does not reflect stresses in this material; thus, the settlement of the full-size footing can be seriously underestimated.

$B_1(= \Delta H_1)$ times the ratio of the footing widths B_2/B_1. Experience indicates the use of this approximation has been reasonably satisfactory.

2. For sand soils the same assumptions of constant values except for B_i' were made but this procedure did not predict very well. Multipliers were sought, and one of the most popular [Terzaghi and Peck (1967), p. 489] was

$$\Delta H_2 = \Delta H_1 \left(\frac{2B_2'}{B_2' + B_1'} \right)^2 \qquad (e)$$

Usually B_1' was a load test plate of size 1×1 ft or 0.3×0.3 m and B_2' was the prototype footing of dimension B. The influence of this equation can be seen with the bearing-capacity equation [Eq. (4-12)]. This equation did not provide very good estimates, so another proposal changes the ΔH_1 multiplier to

$$\left(\frac{B_2'}{B_1'} \right)^n \quad \text{or} \quad \left(\frac{A_2}{A_1} \right)^n$$

where A_i = base areas and values of 0.4 to 0.7 are often suggested for the exponent n (0.5 is most common).

It should be evident that there is little chance of producing a reasonable multiplier—particularly if the B_2/B_1 ratio is very large, as for using a 0.3-m square plate to extrapolate to a 2- to 3-m square base (or to a 20- or 30-m square mat). The reason is that sand requires confinement to develop strength (or E_s). If we assume that 75 mm (or 3 in.) around the perimeter of any size plate provides the "confinement" to the interior sand, then only one-fourth of a 0.3-m square plate is effective. Thus, the apparent E_s is too small at the surface compared to the prototype, which may be of size 2×2 m and which, with the edge loss, is about 93 percent effective. Therefore, the E_{s2}'/E_{s1}' ratio would be in error and the anticipated settlements of the large plate B_2' too large (but conservative). A literature survey by the author indicates that for large B_2'/B_1' ratios the increased settlement ΔH_2 should not exceed about $1.6(\Delta H_1)$ or the reduced allowable bearing capacity q_{a2} should not be less than about $0.4q_{a1}$. For small footing ratios of about 1.1 to 3 the settlement ratios should be about 1.1 to 1.2 and the pressure ratios about 0.9 to 0.8.

For these reasons, and because Eq. (5-18) is theoretically exact, its use is recommended.

5-9.2 Effects on Bearing Capacity

Another use of Eq. (5-18) is for bearing capacity. Here we take $\Delta H_1 = \Delta H_2$ so settlements are equal and replace $q_{o1} = q_{a1}$; $q_{o2} = q_{a2}$. Rearranging terms we obtain

$$q_{a2} = q_{a1} \frac{B_1'}{B_2'} \frac{E_{s1}'}{E_{s2}'} \frac{mI_{s1}}{mI_{s2}} \frac{I_{F1}}{I_{F2}} \qquad (5\text{-}19)$$

The analogy of Eq. (e), taking settlement directly proportional to q_a, gives

$$q_{a2} = q_{a1} \left(\frac{B_2 + B_1'}{2B} \right)^2 \qquad (f)$$

The effect of base width was included in Eq. (4-12), somewhat similarly to Eq. (f). Equation (f) tends to be too conservative—particularly for extrapolating plate-load tests to prototype bases—and is not much used at present. The author recommends using Eq. (5-19) for theoretical accuracy, and the additional parameters seldom produce great difficulty because q_a is usually obtained from SPT or CPT data and it is a trivial exercise to obtain the stress-strain modulus additionally from tables such as Table 5-6.

Example 5-9. The geotechnical consultant on a foundation project has obtained the soil data and profile as shown on Fig. E5-9. A best average of N values (they were nearly constant as in Fig. P3-10) gave $N'_{70} = 20$ shown. Column loads including dead and live loads are estimated in the range of 450 to 900 kN (100 to 200 kips).

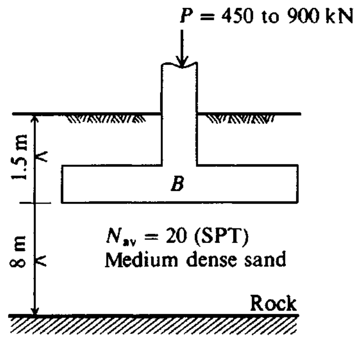

Figure E5-9

Required. Recommend q_a for this project so that ΔH is limited to not over 25 mm.

Solution.

Step 1. Find a tentative q_a using Eq. (4-12). Convert N_{70} to N_{55}, giving $N_{55} = 20(70/55) = 25.45$. Use $N_{55} = 25$.

From Eq. (4-12),

$$q_a = \frac{N_{55}}{0.08}\left(\frac{B+0.3}{B}\right)^2\left(1+0.33\frac{D}{B}\right) \quad \text{but} \quad 1+0.33\frac{D}{B} \le 1.33$$

***B*, m**	**$1 + 0.33\frac{D}{B}$**	**q_a, kPa (rounded)**
1.2	1.33	650 [probably no $B < 1.2$ m]
2.0	1.25	515
3.0	1.17	440

The actual soil pressure q for the given range of column loads and for $B = 1.5$ m is from

$$q = \frac{P}{A} = \frac{450}{1.5 \times 1.5} = 200 \text{ kPa}$$

to

$$q = \frac{900}{2.25} = 400 \text{ kPa}$$

Both of these soil pressures are much less than q_a in the foregoing table. *Tentatively recommend* q_a = **250** kPa. The maximum allowable soil pressure, as an *approximate* average of the three table values, is about **500** kPa (actual average = 535) with a maximum settlement $\Delta H \approx 25$ mm.

Step 2. Check settlement for $q_a = 250$ kPa.

$$B^2 q_a = P_{\text{av}}$$

$$B = \sqrt{\frac{450 + 900}{2 \times 250}} = 1.6 \text{ m as the average width } B$$

For $B = 1.6 \times 1.6$ m we have $L/B = 1$

$$B' = \frac{1.6}{2} = 0.8 \qquad \text{and} \qquad \frac{H}{B'} = \frac{8}{0.8} = 10 \qquad \left(\text{or } \frac{H}{B} = 5\right)$$

From Table 5-2 at $H/B' = 10$ and $L/B = 1$ we obtain

$$I_1 = 0.498 \qquad I_2 = 0.016 \qquad \text{For sand, estimate } \mu = 0.3$$

$$I_s = I_1 + \frac{1 - 2\mu}{1 - \mu} I_2 \qquad I_s = 0.498 + \frac{0.4}{0.7}(0.016) = 0.507$$

From Fig. 5-7 at $D/B = 1.5/1.6 = 0.94$ we obtain $I_F = 0.65$ (using program FFACTOR we obtain 0.66). From Table 5-6 we estimate E_s for a normally consolidated sand as

$$E_s = 500(N + 15) = 500(25 + 15) = 20\,000 \text{ kPa} \qquad (\text{note use of } N_{55})$$

Using $E_s = 2600\,N$, we write

$$E_s = 2600 \text{ N} = 2600(25) = 65\,000 \text{ kPa} \qquad (\text{also } N_{55})$$

and if $E_s = 7000\sqrt{\text{N}}$, we have

$$E_s = 7000\,N = 7000\sqrt{25} = 35\,000 \text{ kPa}$$

From Table 2-7 the value of 20 MPa appears reasonable (and conservative). Substituting values into Eq. (5-16a) with $q_a = q_o$, we have

$$\Delta H = q_o B' \frac{1 - \mu^2}{E_s} m I_s I_F$$

and, noting $m = 4$ for the center settlement we have

$$\Delta H = 250(0.8)\frac{1 - 0.3^2}{20\,000}(4 \times 0.507)(0.65)(1000) = \mathbf{12} \text{ mm}$$

The factor 1000 converts ΔH in m to mm. For $E_s = 65\,000$,

$$\Delta H = 12\left(\frac{20}{65}\right) = 3.7 \text{ mm}$$

Here we can also ratio q_a (maximum $q_a \approx 500$ kPa for $\Delta H = 25$ mm) to obtain

$$\frac{\Delta H}{25 \text{ mm}} = \frac{q_{a,\text{used}}}{q_{a,\text{max}}} \rightarrow \Delta H = 25(250/500) = \mathbf{12.5} \text{ mm}$$

It would appear that in the range of $B = 1.5$ to 2.5 m the settlements will be well under 25 mm and differential settlements (difference in settlements between adjacent footings of different size) will be acceptable. An "averaged" E_s could have been used but was not needed as the minimum value gives acceptable ΔH and great computational refinement is not needed at this preliminary stage of design.

$$\text{Recommend:} \qquad q_a = 250 \text{ kPa (about 5 ksf)}$$
$$\Delta H = \text{under 25 mm}$$

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Example 5-10.

Given. Spread footings on an overconsolidated (or very heavily compacted) dune sand [D'Appolonia et al. (1968) and in Table 5-3].

Required. Estimate the probable footing settlements.

Solution. From careful reading of the reference we obtain the average $B = 12.5$ ft and $L/B = 1.6$; also $\mu = 0.33$ was given.

From the boring log of Fig. 6 and soil profile of Fig. 2 of the reference we can estimate $H = 4B$. Also take $N_{55} = 25$ as the estimated weighted average in depth $H = 4B$, noting that borings stopped at approximately $N_{55} = 40$ before the full depth of $4B$. From the data given the preconsolidation was from dunes to elevations of 650 and 700 from the base elevations of 607 ft. Using $\gamma = 0.110$ kcf and an average depth of 6 ft below footing base we can estimate the OCR at between 7 and 15. We will take OCR = 9 as a reasonable "average." The footing load q_o at the time settlement measurements were taken was approximately 3.4 ksf (about 55 percent of the design load). Finally, the D/B ratio was given as 0.5 on average.

With these data we can proceed with a solution.

For $H/B' = 2(4B)/B = 8$ and $L/B = 1.6$ we obtain from Table 5-2

$$I_1 = 0.573 \quad \text{and} \quad I_2 = 0.031$$

Also for $D/B = 0.5$ we obtain $I_F = 0.75$ from Fig. 5-7. Then

$$I_s = 0.573 + \frac{1 - 2(0.33)}{1 - 0.33}(0.031) = 0.589$$

For E_s use Table 5-5 with OCR = 9

$$E_s = 10(N + 15)\text{OCR}^{1/2} \qquad (\text{obtain } 10 = 500/50 \text{ for ksf})$$

$$E_s = 10(25 + 15)(9)^{1/2} = 1200 \text{ ksf}$$

$$\Delta H = 3.4\left(\frac{12.5}{2}\right)\left(\frac{1 - 0.33^2}{1200}\right)(4 \times 0.589)(0.75)(12) = \mathbf{0.335} \text{ in.}$$

The "measured" values as shown in Table 5-3 ranged from 0.3 to 0.4 inches.

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Example 5-11. What is the expected *corner* settlement of the footing of Ex. 5-9?

Solution. For $q_a = 250$ kPa $= q_o$; $\mu = 0.3$; $E_s = 20\,000$ kPa, and using the "average" $B = 1.6 \times 1.6$ m of step 2, we have

$$D/B = 1.5/1.6 = 0.94 \qquad \text{and} \qquad I_F = 0.65 \qquad (\text{as before})$$

$$H/B' = H/B = 8/1.6 = 5 \qquad (\text{with } L/B = 1)$$

using program FFACTOR, obtain

$$I_1 = 0.437 \quad \text{and} \quad I_2 = 0.031 \qquad I_s = 0.437 + \frac{0.4}{0.7}(0.031) = \mathbf{0.455}$$

Substituting into Eq. (5-16*a*) using $B' = B$ for the corner and noting with a corner there is only one contribution ($m = 1$), we obtain

$$\Delta H = 250(1.6)\left(\frac{1 - 0.3^2}{20\,000}\right)(1 \times 0.455)(0.65)(1000) = \mathbf{5.4}\text{ mm}$$

Observe that the corner settlement is not equal to the center settlement divided by four (12/4 = 3 mm < 5.4 mm computed here).

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5-10 ALTERNATIVE METHODS OF COMPUTING ELASTIC SETTLEMENTS

Since the elastic settlement is simply

$$\Delta H = \int_0^H \epsilon\, dh = \sum_{i=1}^{n} \epsilon_i H_i$$

any method that accurately gives the strains in the identified influence depth H would give an accurate evaluation of the settlement ΔH. As can be seen in Table 5-3 there is at present no better procedure than that proposed using Eq. (5.16*a*); however, in foundation engineering local practice sometimes prevails over any "best" method. For this reason the following two alternatives are given—not as any author recommendation—so that the reader has familiarity with the procedures.

One method is that proposed by Schmertmann (1970) wherein the change in the Boussinesq pressure bulb was interpreted as related to the strain. Since the pressure bulb changes more rapidly from about 0.4 to $0.6B$, this depth is interpreted to have the largest strains. Schmertmann then proposed using a triangular relative-strain diagram to model this strain distribution with ordinates of 0, 0.6, and 0 at $0B$, $0.5B$, and $2B$, respectively. The area of the diagram is related to the settlement, and for constant E_s, which is the same assumption used to develop the strain profile, one may directly compute the settlement as the area of the triangle × strain to obtain

$$\Delta H = 0.6B\frac{\Delta q}{E_s} = 0.6B\epsilon \tag{5-20}$$

Schmertmann also incorporated two correction factors for embedment depth and time as follows:

$$\text{For embedment} \qquad C_1 = 1 - 0.5\frac{\overline{q}}{q_o - \overline{q}}$$

$$\text{For time} \qquad C_2 = 1 + 0.2\log\frac{t}{0.1}$$

where $\overline{q}$ and q_o have been previously defined and t is time > 0.1 in years. With these correction factors Eq. (5-20) now is written as

$$\Delta H = C_1C_2(0.6B)\epsilon \tag{5-20a}$$

If E_s is not constant, Schmertmann proposed to plot the strain profile and obtain influence factors I_z at the center of each change in E_s over a depth increment Δz to obtain

$$\Delta H = C_1C_2\Delta q\sum\frac{I_z\Delta z}{E_s} \tag{5-20b}$$

This calculation would obviously give a conservative ΔH if E_s is constant or increases with depth. If lower layers have a much smaller E_s the solution could give ΔH that is underpredicted. With these two correction factors and $E_s = 2q_c$ (using cone data), Schmertmann computed a number of cases from the literature (some of which are used by the author in Table 5-3) and obtained only fair agreement between computed and measured values of ΔH.

Another procedure is to use the stress path method of Sec. 2-13. In this method one performs a series of triaxial tests at in situ CK_oUC conditions and plots $2q = \sigma_1 - \sigma_3$ versus the strain ϵ for points along the vertical center line of the foundation at depths of, say,

$$B/4, \ B/2, \ B, \ 1.5B, \ 2B, \ 3B, \text{ and } 4B, \text{ or similar}$$

Fewer tests can be used, but confinement ($K_o\sigma_1$) is a significant parameter that has a considerable effect on the strain ϵ, requiring that enough tests be made in the upper depth of $z = 0$ to $4B$ to provide a reliable strain profile so one can use

$$\Delta H = \sum_1^n \epsilon_i H_i$$

This method requires careful construction of sand samples or use of good-quality "undisturbed" clay samples. It may give good results for normally consolidated sands but not for overconsolidated and/or cemented sands because sample reconstruction will be impossible. According to Lambe and Whitman (1979, p. 218) the settlement can be rather well predicted, but their example used eight triaxial tests in a medium to fine sand that apparently was not preconsolidated ($K_o = 0.4$) to find the displacement beneath a round tank. D'Appolonia et al. (1968) in the overconsolidated dune sand (of Example 5-9) used this procedure with two series of footings with seven triaxial tests each at the minimum and maximum estimated OCRs on the site with only fair correlation.

Since we start the triaxial tests from in situ K_o consolidated conditions it is evident that the triaxial test stress $\Delta\sigma_1$ has a 1 : 1 correspondence to the footing stress Δq at that depth. The Boussinesq method is commonly used to estimate Δq. Unless the stress path procedure is perceived to give substantially better settlement estimates, its cost will be far out of proportion to results because of the large number of triaxial tests required.

Example 5-12. Compute the immediate elastic settlement for the soil–footing system shown in Fig. E5-12*a*.

Preliminary work. A series of triaxial (or direct shear) tests must be run to establish ϕ. With ϕ the K_o soil pressure can be computed so that the triaxial tests are performed at that value of cell pressure σ_3. Plot the initial part of the stress-strain curve to a large scale as shown in Fig. E5-12*b*. For cyclic tests plot the last cycle and shift the ordinate so the curve passes through the origin. For this example take

$$\phi = 35^\circ \qquad \gamma_1 = 17.3 \qquad \gamma_2 = 19.1 \text{ kN/m}^3$$
$$K_o = 1 - \sin 35^\circ = 0.426$$

Use a single value of ϕ even though it has been previously noted that ϕ varies with soil density.

Test 1: $p_o = 2(17.3) = 34.6$ kPa $\qquad \sigma_3 = 0.426(34.6) = 14.7$ kPa

Use cell pressure = 20 kPa (approx. 3 psi)

Test 2: $p_o = 3(17.3) + 1.5(19.1) = 80.6$ kPa (estimating density)

Use cell pressure = 40 kPa

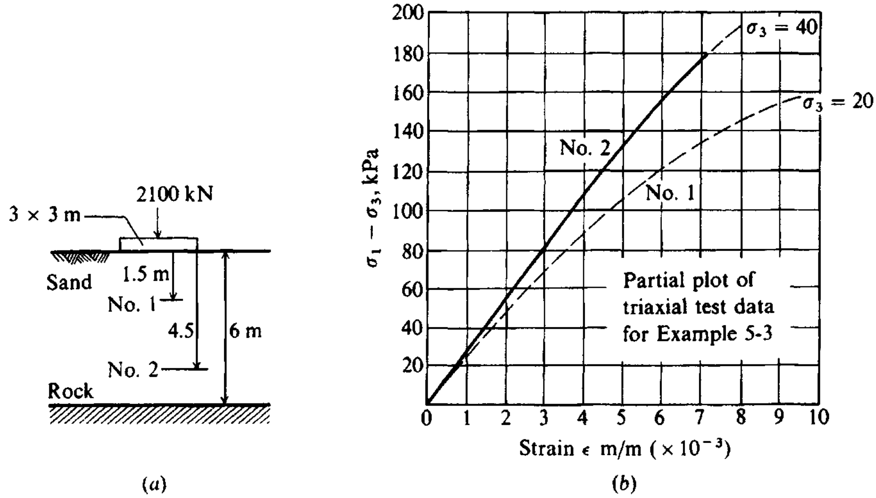

Figure E5-12

It is not a simple matter to test reliably at very low cell pressures. Usually it is not easy to build sand samples to specific densities. At low cell pressures the vacuum used to hold the sample in place until the cell pressure can be applied can "preconsolidate" the sample some amount. Probably three or four tests would be better for this foundation but two are sufficient to illustrate the procedure.

Required. Estimate footing settlement using

a. Stress path method.
b. $\Delta H = \Delta\sigma_1 L/E_s$. Use a secant modulus of elasticity passing through the origin and stress point.

Solution. Divide the 6-m stratum into four increments and make Table E5-12. Obtain q/q_o from Fig. 5-4;

$$q_h = q_v K_o \qquad \text{obtain } \epsilon \text{ from stress-strain plot at } \Delta\sigma_1$$

$$\Delta\sigma_1 = q_v - q_h = q_v(1 - K_o) = \sigma_1 - \sigma_3$$

$$q_o = \frac{2100}{9} = 233.3 \text{ kPa}$$

At $D/B = 0.0$, $\Delta\sigma_1 = q_v(1 - K_o) = 233.3(1 - 0.426) = 133.9$ kPa. From the stress-strain plot (curve 1) in Fig. E5-12*b*, we obtain $\epsilon_1 = 7 \times 10^{-3}$. The corresponding secant modulus $E_s = \Delta\sigma_1/\epsilon_1 = 133.9/0.007 = 19\,130$ kPa, etc.

TABLE E5-12

Curve	D	D/B	q/q_o	q_v, kPa	$\Delta\sigma_1$	$\epsilon \times 10^{-3}$	$E_s \times 10^3$ kPa
1	0	0	1	233.3	133.9	7.0	19.13
1	1.5	0.5	0.7	163.3	93.7	4.6	20.4
1	3.0	1.0	0.33	77.0	44.1	1.8	24.5
2	4.5	1.5	0.19	44.0	25.3	1.0	25.3
2	6.0	2.0	0.12	28.0	16.1	0.6	26.8

We can now compute the settlement using the stress path method by using the strains and the contributory depths (from a depth plot not shown) as

$$\Delta H = 0.75 \text{ m} \times 7.0 + 1.5 \text{ m} \times (4.6 + 1.8 + 1.0) + 0.75 \text{ m} \times 0.6 = \mathbf{16.8} \text{ mm}$$

We note that m $\times$ 1000 $\times$ 10^{-3} cancels, so this computation directly gives the settlement in mm.

For the secant modulus of elasticity method we will numerically integrate the modulus of elasticity using Eq. (5-22) of Sec. 5-12 to find the average E_s as

$$E_s = \frac{1.5}{6}\left(\frac{19.13 + 26.8}{2} + 20.4 + 24.5 + 25.3\right)10^3 = 23.29 \times 10^3 \text{ kPa}$$

A similar computation for $\Delta\sigma_1$ gives 59.525 (using $\Delta\sigma_1$ to be compatible with E_s):

$$\Delta H = \frac{\Delta\sigma L}{E_s} = \frac{(59.525)(6)}{23\,290} = 15.3 \times 10^{-3} \text{ m} = \mathbf{15.3} \text{ mm}$$

This small discrepancy between the two methods is principally due to using the secant instead of the tangent modulus of elasticity. How this compares with a field ΔH will depend on how realistic K_o is compared to field lateral restraint beneath the base. If we used Eq. (5-1a) to modify $E_{s,\text{tr}}$ (and strain) both ΔH values would be reduced approximately 1/1.6 = 0.62 (10.4 and 9.5 mm).

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5-11 STRESSES AND DISPLACEMENTS IN LAYERED AND ANISOTROPIC SOILS

There are numerous elastic solutions for special cases of stresses and displacements in layered or anisotropic soils. Special cases are sometimes useful to obtain an indication of probable (or possible) magnitude of error from using an idealized soil mass (isotropic, homogeneous, etc.). Generally, the special cases in the literature [Poulos and Davis (1974) summarize a large amount of curves, charts, and tables] are not found in nature, or by the time the necessary interpolations from curves and tables are made, the problem would be solved.

The author proposes that one of the best uses of the finite-element method (FEM) is to solve this type of problem. A computer program FEM2D is noted on your program diskette. One solves this type of problem as follows:

1. Model a reasonable size of half-space, once for all, and use a data generator to develop the data to define the x, y coordinates of the nodes and the node numbers defining each element and the soil for that element. The model should have provision for about five different layers of soil (for fewer layers one simply uses the same soil properties for more than one layer).
2. Solve the problem for a point load at one node where the footing is placed and for a "one" soil mass. This is either in the ground or at the ground surface (or both) depending on whether it is desired to obtain depth effects.
3. Re-solve the problem with the point load at the same location but with the correct soil stratification.
4. From the Boussinesq pressure bulbs obtain the stress at the desired point beneath the footing (now we are incorporating the shape and three-dimensional effect of the load into the problem).
5. From steps 2 and 3 find the point load stress at the same point as obtained in step 4.

6. Compute the stress due to stratification at any depth z as a proportion to obtain

$$q_{fL} = q_b \left(\frac{q_3}{q_2} \right) \tag{5-21}$$

where q_b = Boussinesq value for a footing of same dimension and applicable corrections for depth, etc., in a homogeneous soil mass at the depth of interest

q_{fL} = stresses due to footing in layered soil at the depth of interest

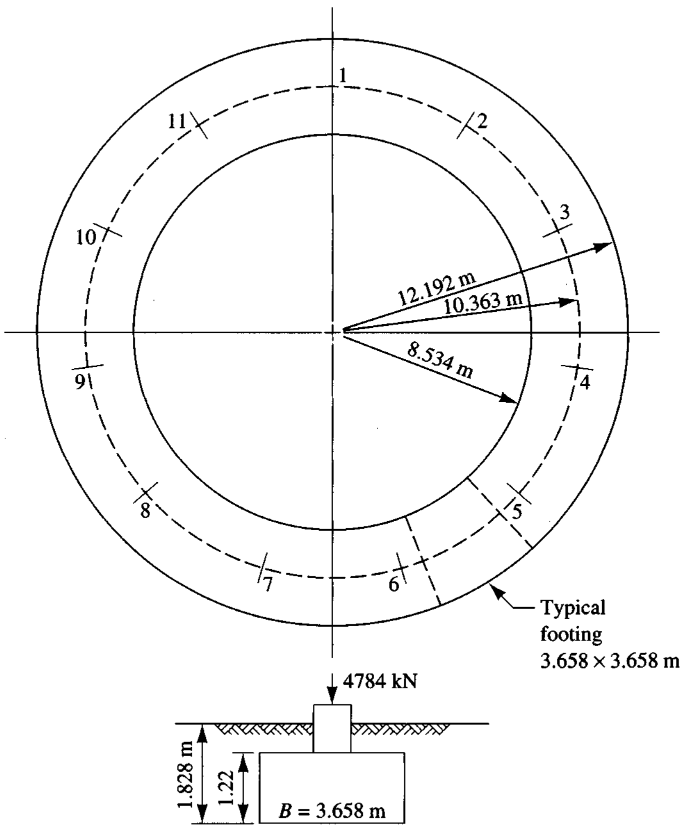
q_3, q_2 = stresses from the FEM solutions for the layered (step 3) and homogeneous (step 2) cases at the same depth of interest.

This solution is at least as good as the soil parameters E_s and μ used in the FEM. This method allows using a simpler two-dimensional plane-strain or plane-stress solution rather than a much more complex three-dimensional analysis. Deflections can be computed in an analogous manner.

Example 5-13.

Given. A 3.7 m wide × 24.4 m O.D. foundation ring as shown in Fig. E5-13*a*. This example is taken from Bhushan and Boniadi (1988) and some units have been converted to SI but the field

Figure E5-13*a* Ring foundation geometry and other data. Uses 11 equally spaced columns with pedestals located on center line of ring (not on centerline of area).



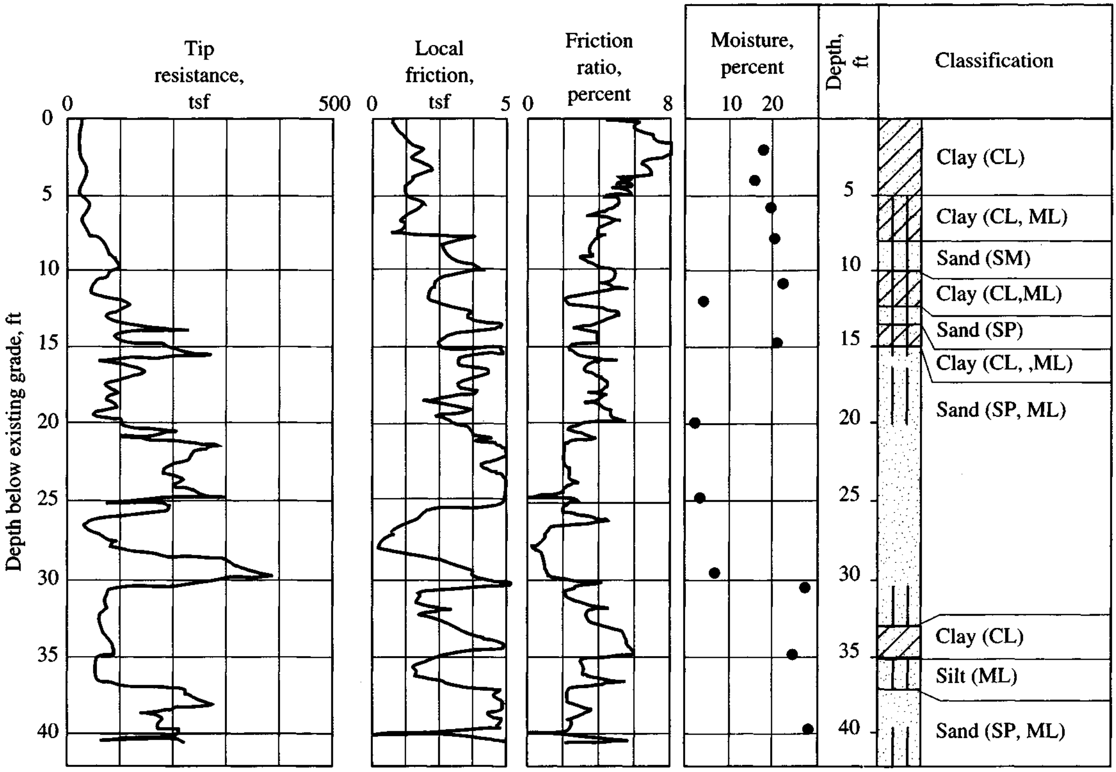


Figure E5-13*b* Typical subsurface exploration (boring) log. Note use of Fps units (1 tsf = 96 kPa).

log is retained in the manner obtained and presented by them. Appropriate conversions to SI will be made as necessary. The measured settlement during preload was 10 to 17 mm and the average given by the reference was 15.2 mm.

Required. Estimate the settlement under the preload stress of 252.8 kPa given by the reference. The preload stress is somewhat larger than the working load stress but will only be temporary.

Solution. We will use a modification of the method given by Bowles (1987) and in the previous edition of this textbook.

Assumptions.

1. Take effective $H = 5B' = 9.144$ m, giving $H/B' = 5$.
2. Since a ring closes on itself an L value has no significance so use an approximate square as shown by dashed lines on the ring in Fig. E5-13*a*. This gives $B \times B = 3.658 \times 3.658$ m ($B' = 1.829$ m). If the inside diameter of the ring were smaller than 17.1 m we might be justified in using B = outside diameter but not here.
3. From an inspection of the "typical" cone penetration resistance q_c profile of Fig. E5-13*b* estimate an "average" $q_c = 150$ tsf, which converts to

$$q_{c,\mathrm{SI}} = 150(2)(47.88) = 14\,364 \text{ kPa}$$

Then estimate $E_s = 3q_c$ since the zone of interest from -6 ft to -36 ft (1.83 to 11 m) for a depth of $5B$ includes both clay and sand layers. This process gives

$$E_s = 3 \times 14364 = 43092 \rightarrow \mathbf{44000}\ \text{kPa}$$

4. $D/B = 1.82/3.66 = 0.5$ (given, not an assumption).
5. I will use Eq. (5-16a) with $\mu = 0.3$ and with the Fox embedment reduction factor I_F.

With these data and using program FFACTOR for $D/B = 0.5$, $\mu = 0.3$, and $L/B = 1$, obtain $I_F = 0.77$ and the Steinbrenner influence factor $I_s = 0.455$. One could also have used Table 1 of Bowles (1987) for I_s and Table 5-7 for I_F. Making a direct substitution into Eq. (5-16a), we have

$$\Delta H = qB'\frac{(1-\mu^2)}{E_s}mI_sI_F \tag{5-16}$$

$$= 252.8(1.83)\frac{(1-0.3^2)}{44000}(4)(0.455)(0.77)(1000) = 13.4 \text{ mm}$$

This result compares to the average $\Delta H = 15.2$ mm (0.6 in., and in the range of displacements) reported in the reference. The reader should redo this example using $E_s = 2q_c$ and also inspect Fig. E5-13b and see if the author made a selection for the average $q_c = 150$ tsf that is reasonable.

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5-12 CONSOLIDATION SETTLEMENTS

The settlements of fine-grained, saturated cohesive soils will be time-dependent, and consolidation theory is usually used, although elastic methods can be, and sometimes are, used. Equation (2-44) or (2-45) is usually used for consolidation settlements, however, the alternate form given by Eq. (2-43) as

$$\Delta H = m_v \Delta p H = \epsilon H$$

is also used. Some authorities routinely use this latter equation format for settlement computations both for clay and fine-to-medium sand since $m_v = 1/E_s$ (the constrained modulus of elasticity) where m_v is determined in a consolidation test. The sample, being on the order of only 20 to 25 mm thick, may give results that are not very representative; and in sands, the SPT or CPT is generally preferable since a large number of values can be obtained at relatively low cost compared with the effort in a consolidation test—even if the loads can be changed rapidly.

In applying consolidation theory to compute settlements in clay we have three factors to consider:

1. Whether the soil is normally consolidated or preconsolidated (OCR > 1)
2. Estimating the in situ void ratio e_o and obtaining sufficient compression indexes to profile the clay layer(s) adequately
3. Estimating the average stress increase Δq in the stratum of thickness H

Section 2-10 has adequately considered what to do for preconsolidated strata. That section also detailed obtaining e_o and the compression indexes. Here we are primarily concerned with practical application of the theory.

The in situ void ratio e_o can usually be determined reasonably well using w_N and G_s and/or volumetric-gravimetric data from the soil sample in the consolidation ring used for the test. It is usual to use values at the midheight of the consolidating layer, so if the consolidation test sample were at a different location, the void ratio at midheight can be computed from rearranging Eq. (2-42) and defining $\Delta e = e_o - e$ and $p_2 = p'_o + \Delta p'_o$ to obtain

$$e = e_o - C_c \log \frac{p'_o + \Delta p'_o}{p'_o}$$

where e_o = void ratio test depth z
$p'_o = \gamma' z$ = effective overburden pressure at depth z
$\Delta p'_o = \gamma'(dz)$ = increase or decrease in p'_o from depth z
dz = depth from test depth z to midheight of stratum and may be (+) if below or (−) if above

It can be seen that the void ratio is not linear (and probably the compression indexes are not either), so one should not use a very large stratum thickness H over which Δq, e_o, and C_c are averaged at $H/2$.

The average pressure increase in the stratum of thickness H from the foundation load can be obtained by simply averaging the top and bottom value from Boussinesq theory for H values up to about 1 m. For greater thickness one should use a numerical integration process. The trapezoidal-rule formula is well suited for this (and other numerical integration) where a depth (or space) increment Δh = constant is taken with end values p_1, p_n and interior points at Δh spacing. This gives the area A of the pressure profile as

$$A = H\Delta p = \Delta h\left(\frac{p_1 + p_n}{2} + p_2 + p_3 + \cdots + p_{n-1}\right) \tag{5-22}$$

from which the average pressure increase Δp in stratum thickness H is

$$\Delta p = \frac{A}{H}$$

It is, of course, necessary to compute p'_o at the midheight of the layer as well. Where the layer(s) are over about 2-m thick, one should give consideration to obtaining additional values of C_c and e_o so that the layer can be subdivided into layers of thickness H_i and the total settlement computed as

$$\Delta H = \sum_1^n \Delta H_i$$

These additional values can result in a large number of computations, and it may be worthwhile to program the steps so that the work is semiautomated.

One may question the validity of using the Boussinesq method when the actual case is one or more layers of clay soils with different C_c (or one or more layers of soils where immediate settlements occur) overlying one or more consolidating clay layers. Although the method is certainly not exact, unless there is a significant difference, say by a factor of five times or more in the stress-strain modulus of the two materials, more refined computation will improve the computed stress increase very little [see Morgan and Gerrard (1971)].

Example 5-14.

Given. The consolidation test, soil profile, and other data shown in Fig. E5-14. Note that original data are given in Fps units and not converted, as emphasis is on procedures.

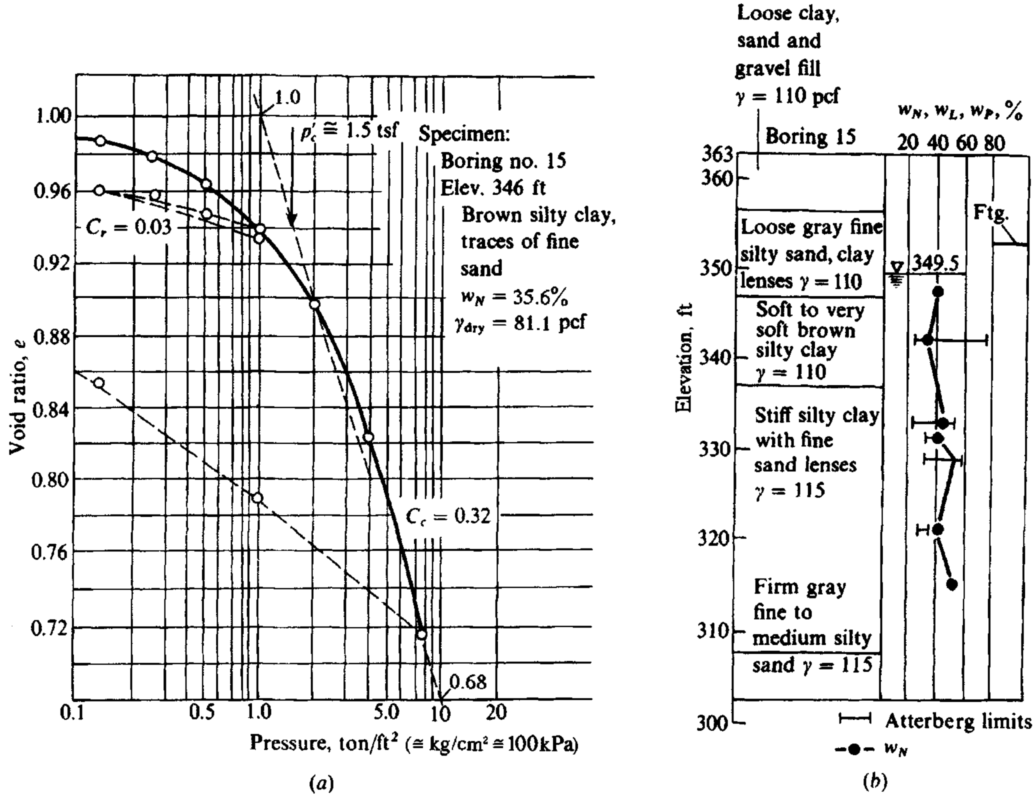

Figure E5-14

Required. Estimate the settlement of an 8 × 8 ft footing carrying 375 kips at elevation 353 ft on the "soft to very soft brown silty clay" (elevation 347 ft to 337 ft).

Solution. Note that the author of this book estimated p'_c using as a guide both the first and second reload cycles since the e versus $\log p$ curve does not have a distinct "sharp-curved" portion. It is possible that a better estimate might have been made using either Method 3 or Method 4 of Sec. 2-10.3. The Casagrande method would not be any better than the "eye" method used by the author of this book, since a sharply curving part of the curve is not clearly identified. Even the "virgin" curve part of this e versus $\log p$ plot is somewhat curved, and the slope for computing C_c is some approximate. With these comments we shall continue with a solution.

Estimate the initial (or in situ) void ratio e_o. The value at the first plotted point (0.985) is high since the soil has expanded from loss of overburden pressure. Obtain the value of 0.96 at the end of the first rebound cycle as a better estimate. We will check this estimated e_o, since the soil is approximately saturated, using an equation from Chap. 2. This equation requires the specific gravity G_s (estimated 2.70) and the natural water content (35.6% from Fig. E5-14a):

$$e_o = \frac{w_N G_s}{100} \approx 0.356(2.70) = \mathbf{0.961} \qquad \text{(coincidence ??)}$$

Compute the slope of the rebound curve C_r as a best estimate of the slope, which the user should lightly pencil in but is not shown here, to obtain the void ratio values and pressure change. A better value might have been obtained using the average of both the initial and rebound "slopes," but that task is left as a reader exercise.

$$C_r = \frac{\Delta e}{\log p_2/p_1} = \frac{0.960 - 0.930}{\log 1/0.14} = \frac{0.030}{0.854} = \mathbf{0.035}$$

(Note that this slope could have been extended across one log cycle, but points will be used to illustrate alternative).

Compute C_c as the slope of the curve beyond p'_c; extend dashed line shown on Fig. E5-14*a* across one log cycle and obtain

$$C_c = \frac{1.00 - 0.68}{\log 10/1} = \frac{0.32}{1} = \mathbf{0.32}$$

As a check use equations from Table 2-5:

$$C_c = 0.009(w_L - 10) = 0.009(78 - 10) = 0.612 \qquad (a)$$

$$C_c = 0.37(e_o + 0.003w_L + 0.004w_N - 0.34)$$
$$= 0.37[0.96 + 0.003(78) + 0.004(35.6) - 0.34] = 0.37 \qquad (b)$$

Eq. (*a*) is probably in error because the soil is preconsolidated. Eq. (*b*) differs from the plot value because of plot interpretation, but it is not a bad estimate, because it somewhat accounts for preconsolidation by taking into account the liquid and natural water contents as well as the initial void ratio.

Now find the *average* increase in stratum pressure Δp from base load [contact pressure $q_o = 375/(8 \times 8) = 5.859$ ksf (rather high)]:

1. Use the 2 : 1 method [see Eqs. (5-2*a, b*)]. With the footing at elevation 353, the depth to the top of the clay layer is $353 - 347 = 6.0$ ft; to the bottom, the depth is $353 - 337 = 16$ ft. Thus,

$$\Delta pH = \int_6^{16} \frac{375}{(8+z)^2}\,dx = \left[-\frac{375}{8+z}\right]_6^{16}$$

   Inserting the limits, we have

$$\Delta p = \frac{1}{10}\left(-\frac{375}{24} + \frac{375}{14}\right) = \mathbf{1.12}\ \text{ksf}$$

2. Using the Boussinesq pressure bulbs (Fig. 5-4) and computer program SMBWVP we can construct the following table:

Elevation, ft	*D/B*	$\Delta q/q_o$	Fig. 5-4 Δq	Δq^* (SMBWVP)
−6.0	6/8 = 0.75	0.50	2.93	2.87
8.5	1.06	0.33	1.93	1.82
11.0	1.375	0.23	1.35	1.22
13.5	1.68	0.16	0.94	0.86
−16.0	2.00	0.12	0.70	0.64

Compute the average stress increase $\Delta p(= \Delta q)$ using Eq. 5-22 and the computer-generated values (but the pressure bulb values are reasonable considering the small text scale—and probably about as accurate):

$$A = \Delta qH = 2.5\left(\frac{2.87 + 0.64}{2} + 1.82 + 1.22 + 0.86\right) = 14.14$$

with $H = 10$ ft; $\Delta p = 14.14/10 = \mathbf{1.41}$ ksf (pressure bulbs $= 1.51$)

Next find the *effective* overburden pressure at midheight of the consolidating stratum (refer to Fig. E5-14*b*) referenced to the ground surface, not the footing base:

$$\begin{aligned} p'_o &= 0.110(363.0 - 349.5) &&= 1.485 \\ &\quad + (0.110 - 0.624)(349.5 - 342.0) &&= \underline{0.356} \end{aligned}$$

$$\text{Total } \textit{effective} \text{ pressure } p'_o = \mathbf{1.841} \text{ ksf}$$

From the e versus $\log p$ plot we obtain (method previously noted)

$$p'_c = 1.5 \text{ tsf} = 3.00 \text{ ksf} \qquad \text{OCR} = 3.00/1.84 = 1.6$$

and

$$\begin{aligned} p'_o + \Delta p &= 1.84 + 1.41 = 3.25 \text{ ksf} \\ \Delta p_2 &= 0.25 \text{ ksf} \qquad p'_o = p'_c = 3.00 \text{ ksf} \\ \Delta p_1 &= 1.41 - 0.25 = 1.16 \text{ ksf} \qquad p'_o = 1.84 \text{ ksf} \qquad C_c = C_r \end{aligned}$$

Inserting values into Eqs. (2-45*a*), we have

$$\Delta H_1 = \frac{0.035(10)}{1 + 0.96} \log \frac{1.84 + 1.16}{1.84} = \mathbf{0.038} \text{ ft}$$

$$\Delta H_2 = \frac{0.32(10)}{1.96} \log \frac{3.00 + 0.25}{3.00} = \mathbf{0.057}$$

$$\Delta H_{total} = \mathbf{0.095} \text{ ft} \qquad (0.095 \times 12 \approx 1.14 \text{ in.})$$

This settlement is probably a little too large, and it is quite possible that the soil below elevation 337 ft ("stiff silty clay . . . ") would contribute additional consolidation settlement. The contact pressure $q_o = 5.86$ ksf is rather high, and the base should probably be rechecked for settlement using dimensions of either 9×9 or 10×10 ft.

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5-12.1 Proportioning Footings for Equal Consolidation Settlement

We considered the problem of sizing footings for equal immediate settlements in developing Eq. (5-18). For footings located over a consolidating clay layer, finding the dimensions of $B \times L$ to obtain equal settlements becomes a trial procedure, as illustrated in the following example.

Example 5-15. Proportion a footing such that the consolidation settlement is not over 40 mm for the given conditions of Fig. E5-15*a*.

Solution. Assume that the net increase in soil pressure due to the concrete displacement of the soil is negligible. Since the settlement depends on the contact pressure and footing size and is nonlinear,

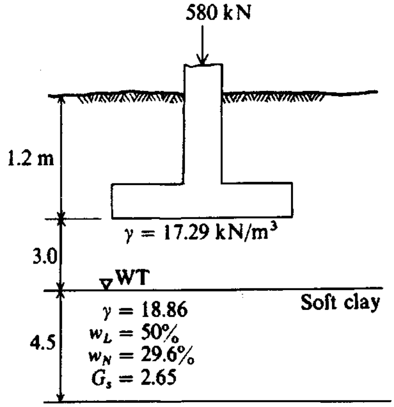

Figure E5-15*a*

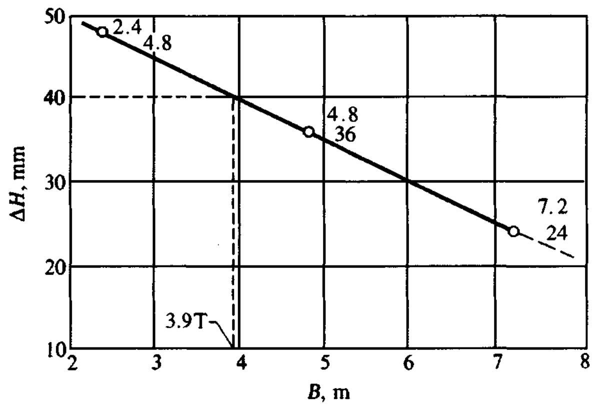

Figure E5-15*b*

several trials will be required, and it will be most convenient to use the average stress increase in the stratum Δp. The results of ΔH versus B will be plotted to find the required footing size.

$$p'_o = (3.0 + 1.2)(17.29) + \frac{4.5}{2}(18.86 - 9.807) = \mathbf{93}\text{ kPa}$$

Take $C_c = 0.009(w_L - 10) = 0.009(50 - 10) = \mathbf{0.36}$ (but soil may have OCR > 1). Also:

$$e_o = wG_s = \frac{29.6}{100}(2.65) = \mathbf{0.784} \qquad \text{assuming } S = 100 \text{ percent}$$

$$\Delta H = \frac{C_c H}{1 + e_o}\log\frac{p'_o + \Delta p}{p'_o} = \frac{0.36(4.5)}{1.784}\log\frac{p'_o + \Delta p}{p'_o} = 0.91\log\frac{93 + \Delta p}{93}$$

Use the Boussinesq method (Fig. 5-4), and obtain data in Table E5-15.

TABLE E5-15

	B = 2.4 m		**B = 4.8 m**		**B = 7.2 m**	
***D*, m**	***D/B***	***q/q_o***	***D/B***	***q/q_o***	***D/B***	***q/q_o***
−3.0	1.25	0.25	0.62	0.6	0.42	0.77
−4.5	1.87	0.13	0.94	0.4	0.62	0.60
−6.0	2.5	0.08	1.25	0.25	0.83	0.40
−7.5	3.12	0.06	1.56	0.17	1.04	0.34

Computing the average stress Δp by the trapezoidal rule [Eq. (5-22)], we find

$$\Delta p = \frac{1}{15}\left(\frac{15}{3}\right)\left(\frac{0.25 + 0.06}{2} + 0.13 + 0.08\right) = \frac{0.36}{3} = 0.12q_o \qquad B = 2.4\text{ m}$$

$$= \frac{1}{3}\left(\frac{0.6 + 0.17}{2} + 0.4 + 0.25\right) = 0.35q_o \qquad B = 4.8\text{ m}$$

$$= \frac{1}{3}\left(\frac{0.77 + 0.34}{2} + 0.6 + 0.40\right) = 0.52q_o \qquad B = 7.2\text{ m}$$

$$q_{2.4} = 0.12\frac{580}{2.4^2} = 12\text{ kPa} \qquad q_{4.8} = 0.35\frac{580}{4.8^2} = 8.8\text{ kPa}$$

$$q_{7.2} = 0.52\frac{580}{7.2^2} = 5.8 \text{ kPa}$$

$$\Delta H_{2.4} = 0.91 \log \frac{93 + 12}{93} = 0.91(0.053) = 0.048 \text{ m or } \mathbf{48} \text{ mm}$$

$$\Delta H_{4.8} = 0.91 \log \frac{93 + 8.8}{93} = 0.91(0.039) = 0.036 \text{ m or } \mathbf{36} \text{ mm}$$

$$\Delta H_{7.2} = 0.91 \log \frac{93 + 5.8}{93} = 0.91(0.026) = 0.024 \text{ m or } \mathbf{24} \text{ mm}$$

Plotting these three points to obtain Fig. E5-15*b*, we can interpolate to obtain $B = 4$ m. Although it might appear that $B = 2, 3$, and 4 m might be better trials, the best choices are not known initially and larger values will more rapidly bracket B with at least as good accuracy as the known settlement data. Note the nearly linear plot, which somewhat justifies Eq. (*d*) of Sec. 5-9.1.

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It should be evident at this point that it is impossible to proportion footings so that the settlements will be exactly equal unless the footings are the same size and with the same contact pressure. The following points are important:

1. If the footings are of different size, and with the same contact pressure, the larger base will settle more.
2. The stress profile is based on a depth of approximately $5B$, so clearly there is a greater depth undergoing strain (and ΔH) for larger bases.
3. If the layer H is the same depth beneath two footings of q_o = same but with different B, the larger B will settle more, as there is a larger concentration of Boussinesq settlement (the H/B is smaller for the larger footing). For immediate settlements the influence factor is smaller but B' is larger.

5-12.2 Secondary Compression Settlements

In addition to the primary compression of a base as illustrated in Example 5-14, secondary compression (or creep) also occurs. This phenomenon is associated with both immediate and consolidation-type settlements, although it is usually not of much significance with immediate settlements.

At least a part of the settlement causing the Leaning Tower of Pisa to tilt is probably due to secondary compression, with consolidation providing the remainder of the vertical (and differential) movement.

As previously stated in Chap. 2, secondary compression is the continuing readjustment of the soil grains into a closer (or more dense) state under the compressive load. It occurs after the excess pore pressure has dissipated and may continue for many years.

Secondary compression may be the larger component of settlement in some soils, *particularly in soils with a large organic content.* It can be estimated using Eq. (2-49) of Sec. 2-10.6.[5] The major problem is obtaining the secondary compression index C_α of Eq. (2-49).

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[5]Stinnette (1992) made an extensive study of organic soils in Florida (USA) and provided an extensive literature survey. Both Eq. (2-49) and the methods of Tan et al. (1991) were shown to provide reasonable results but several other methods were also given.

High-quality consolidation tests, if continued for a sufficient time for the appropriate load increment, may give the best value. These are often not done and an estimated value is used, either from one of the equations given in Table 2-5 or from a lesser-quality consolidation test (if any are done).

Example 5-16.

Given. The data of Example 5-14 and a laboratory value of $t_{100} \approx 100$ minutes (from a plot of ΔH versus log time, not shown).

Required. Compute an estimate of secondary consolidation.

Solution. We will use the value from Table 2-5 of

$$C_\alpha / C_c = 0.032$$

and from Example 5-14 we have $C_c = 0.32$, giving

$$C_\alpha = 0.032 C_c = 0.032(0.32) = \mathbf{0.010}$$

Now we need some preliminary computations:

1. $t_{\text{lab}} = 100$ min. There are $24 \times 60 \times 365 = 525\,600$ min in 1 year.
2. Use the following:

$$\frac{t_{\text{field}}}{t_{\text{lab}}} = \frac{H^2_{\text{field}}}{H^2_{\text{lab}}}$$

   This ratio is obtained from using Eq. (2-38), cancelling T_i and c_v and using the appropriate subscripts. The ratio is needed to estimate when secondary compression begins.
3. For a lab sample of $H_{\text{lab}} = 0.75/2$ inches (*two*-way drainage) and a field $H_{\text{field}} = 10$ ft $= 120$ inches (*one*-way drainage from inspection of boring log), the time for 100 percent consolidation before secondary compression starts (using $t_{\text{lab}} = 100$ min)—at least in theory—is

$$t_{\text{field}} = 100(120/0.375)^2 = 10\,240\,000 \text{ min}$$
$$= 10\,240\,000/525\,600 = \mathbf{19.5} \text{ years}$$

   Using Eq. (2-49), we have

$$\Delta H_s = H_s C_\alpha \log \frac{t_2}{t_1}$$

and using $t_2 = 30$ yr (arbitrary), $t_1 = 19.5$ yr, and the consolidating layer as 10 ft (given), we have an *estimated secondary compression* of

$$\Delta H_s = 10(0.010)\log(30/19.5) = \mathbf{0.019} \text{ ft} = 0.23 \text{ in.}$$

which is almost negligible.

It is very likely that the secondary compression will be larger than this, as some will occur during primary consolidation. Theoretically, at the end of 19.5 years there is no excess pore pressure anywhere in the 10-ft layer; however, during this time period dissipation occurs from the top down, with secondary compression beginning before 19.5 years have elapsed in the upper regions. No easily developed theory that is practical to use is currently available to take this into account. It is therefore quite possible that there could be as much as 1 inch of secondary compression, and it could occur well before the time when it is supposed to start, at 19.5 years.

This example and discussion, together with the observation that the consolidation settlement from Example 5-14 is 1.14 in., indicates that there should be more than one consolidation test done in this layer—that is, use at least two 5-foot-thick layers with a test in each. It also would be most prudent to obtain samples and perform one or more additional tests within the $5B$ depth region that penetrates into the "stiff silty clay" underlying this soft clay layer.

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5-13 RELIABILITY OF SETTLEMENT COMPUTATIONS

Settlements are generally made up of immediate, consolidation, and secondary compression (or creep) components as

$$\Delta H = \Delta H_i + \Delta H_c + \Delta H_s$$

In cohesionless soils and unsaturated clays the immediate settlement predominates with perhaps some creep ΔH_s. The consolidation settlement predominates for saturated cohesive soils unless the soil is very organic, in which case the creep term may predominate.

Immediate settlement computations can vary widely but as shown in Table 5-3 can, with some care, be used to predict the settlement ΔH_i quite satisfactorily.

Consolidation theory tends to predict the amount of settlement ΔH_c rather well if care is taken to obtain representative soil parameters. In most cases the settlement prediction is conservative (i.e., is overpredicted) but within acceptable limits. A study of recent Geotechnical Division Journals and papers given at the ASCE conventions (too numerous to cite specifically) gives an overview that consolidation settlements are adequately predicted. The predictions are better for inorganic, insensitive clays than for others. The prediction requires much care if the e versus $\log p$ curve is curved throughout or the clay is very sensitive. Much care is also required if the clay is highly organic, as the creep component will be substantial.

The time rate for consolidation settlement is not well-predicted because the coefficient of permeability is a significant factor. In the laboratory a thin sample with any compression undergoes a large void ratio change relative to in situ. Since the coefficient of consolidation c_v depends on the void ratio [$c_v = f(e)$], the laboratory value tends to be too small, so the time for consolidation is overpredicted; e.g., based on a laboratory test to obtain c_v, the field prediction for a site is 6 years using Eq. (2-38), whereas actual measurements give about 3 years for most of the settlement to occur. While overpredicted times are usually acceptable, there will be cases in which, if the consolidation occurs too rapidly, the superstructure members will crack rather than "creep" into a deformed position.

5-14 STRUCTURES ON FILLS

It is often advantageous, and sometimes necessary, to place the structure or parts of it on filled-in areas. These sites may be sanitary landfills, rubble dumps from demolished buildings, or fills constructed according to engineering criteria. In the situations where sanitary fills or rubble dumps are used, it is doubtful that a structure can be placed on this material and not undergo detrimental settlement unless the fill has had time to decompose and fully consolidate. For most cases of foundations on fills the loads will have to be carried through the fill material utilizing piles or caissons of a noncorrosive material (usually concrete or treated wood).

A well-constructed earth fill, using quality control with regard to both material and compaction, often produces a better foundation base than the original material underlying the fill. Many persons have been reluctant to place a footing on or in fills because of two main factors:

1. Unpleasant results from placing footings on poorly placed fills. With no quality control it is not unusual to get a fill with a hard crust over 0.5 to 1 or more meters of loose fill, as a result of compacting only the last lift, or from placing a lift too thick to be compacted with the available equipment.
2. Placing a footing in the fill with unpleasant results obtained not from the fill settlement but from settlement of the underlying soil due to the weight of both the fill and the structure.

There are precautions one must take with a fill, in addition to exercising compaction control, such as eliminating soils of large volume change; providing adequate drainage; and, if construction is to proceed relatively soon after the fill is placed, making sure that consolidation settlements have been considered. Under consolidation processes the structure and fill will subside from the weight of the fill alone; and this will take place whether the footings are placed on the natural soil or in the fill. Excessive differential settlements may also result from consolidation in the underlying soft strata if the fill varies considerably in thickness and particularly if part of the structure is on an excavation or virgin soil and part is on fill. A poorly constructed fill will also undergo settlements with time, and there is no theory available that can be used to estimate the amount of or the length of time for the settlement to be completed.

The determination of the bearing capacity (and settlements) proceeds as with the virgin soil. If the fill is placed before exploration takes place, the usual exploration methods of Chap. 3 (standard penetration tests on recovered samples) are applicable. When the field exploration has already been performed, the bearing capacity of the fill may be determined by performing laboratory tests on specimens compacted to the proposed in situ density. Building code values, coupled with successful experience on soils of similar properties and density, may also be used as a guide.

5-15 STRUCTURAL TOLERANCE TO SETTLEMENT AND DIFFERENTIAL SETTLEMENTS

Theoretical settlements can be computed for various points such as corner, center, or beneath the lightest- and heaviest-loaded footings to obtain the total settlement and the differential settlement between adjacent points. If the entire structure moves vertically some amount or rotates as a plane rigid body, this movement will not generally cause structural or architectural distress. For example, if a structure settles 20 mm on one side and 100 mm on the other with a linear settlement variation between the two points, structural damage is not likely to develop, although there are aesthetic and public confidence considerations. The building will have settled 20 mm and tilted an amount $\zeta = (100 - 20)/L$. Local settlements below the tilt line between the two sides of the structure will be the cause of any building distress. These local settlements below either the settlement or tilt line are the differential settlements that the foundation designer must control, since they will determine the acceptability of the structure. The initial settlements that occur during construction (or shortly after) can usually

TABLE 5-7
Tolerable differential settlement of buildings, mm*
Recommended maximum values in parentheses

Criterion	Isolated foundations	Rafts
Angular distortion (cracking)	1/300	
Greatest differential settlement		
Clays	45 (35)	
Sands	32 (25)	
Maximum settlement		
Clays	75	75–125 (65–100)
Sands	50	50–75 (35–65)

*After MacDonald and Skempton (1955) but see also Wahls (1981).

be landscaped into concealment when the building is completed or later. A cracked wall or warped roof is much more difficult to conceal.

Differential settlement can be computed as the difference in settlement between two adjacent points. It may be estimated as three-fourths of the computed maximum total settlement; i.e., maximum total settlement = 40 mm; expected differential settlement, $\Delta h = \frac{3}{4}(40) = 30$ mm.

MacDonald and Skempton (1955) made a study of 98 buildings, mostly older structures of load-bearing wall, steel, and reinforced concrete construction to provide the data of Table 5-7. This study was substantiated by Grant et al. (1974) from a study of 95 additional buildings of more recent construction (some were constructed after 1950). Feld (1965) cited a rather large number of specific structures with given amounts of settlement and structural response, which might be of interest in considering a specific problem. Combining all sources, one can conclude [see Wahls (1981)] that

1. The values in Table 5-7 should be adequate most of the time. The values in brackets are recommended for design; others are the range of settlements found for satisfactory structural performance.
2. One must carefully look at the differential movement between two adjacent points in assessing what constitutes an acceptable slope.
3. Residual stresses in the structure may be important, as it has been observed that there is a range of tolerable differential settlements between similar buildings.
4. Construction materials that are more ductile—for example, steel—can tolerate larger movements than either concrete or load-bearing masonry walls.
5. Time interval during which settlement occurs can be important—long time spans allow the structure to adjust and better resist differential movement.

If computed differential settlements are kept within the values in parentheses in Table 5-7, statistically the structure should adequately resist that deformation. Values of acceptable slopes between two adjacent points from the U.S.S.R. building code are in Table 5-8.

One might use the following, a composite from several sources, as a guide in estimating differential settlement. Define L = column spacing and δ = differential displacement

TABLE 5-8

Permissible differential building slopes by the USSR code on both unfrozen and frozen ground

All values to be multiplied by L = length between two adjacent points under consideration. H = height of wall above foundation.*

Structure	On sand or hard clay	On plastic clay	Average max. settlement, mm
Crane runway	0.003	0.003	
Steel and concrete frames	0.002	0.002	100
End rows of brick-clad frame	0.0007	0.001	150
Where strain does not occur	0.005	0.005	
Multistory brick wall			25 $L/H \geq 2.5$
L/H to 3	0.0003	0.0004	100 $L/H \leq 1.5$
Multistory brick wall			
L/H over 5	0.0005	0.0007	
One-story mill buildings	0.001	0.001	
Smokestacks, water towers, ring foundations	0.004	0.004	300
Structures on permafrost			
Reinforced concrete	0.002–0.0015		150 at 40 mm/year†
Masonry, precast concrete	0.003–0.002		200 at 60 mm/year
Steel frames	0.004–0.0025		250 at 80 mm/year
Timber	0.007–0.005		400 at 129 mm/year

*From Mikhejev et al. (1961) and Polshin and Tokar (1957).

†Not to exceed this rate per year.

between any two adjacent columns. Use $\delta = 0.75\delta_{max}$ if you only have estimates of settlements at the columns (or edges and center of the structure).

Construction and/or material	Maximum δ/L
Masonry (center sag)	1/250–1/700
(edge sag)	1/500–1/1000
Masonry and steel	1/500
Steel with metal siding	1/250
Tall structures	< 1/300 (so tilt not noticeable)
Storage tanks (center-to-edge)	< 1/300

Although the values in Table 5-8 *may appear dated,* an examination by the author of several current (as of 1995) building codes (BOCA, National, Uniform, etc.) reveals no guidance on tolerable, or allowable, building distortions.

5-16 GENERAL COMMENTS ON SETTLEMENTS

It is a rare event when footings all settle the amount computed by the designer. This is true for footings on sand, on slopes, or on sand and clay where there is a combination of immediate and long-term consolidation settlements.

Soil is too heterogeneous to make settlement predictions with any great accuracy. What is hoped is to design site footings with a 95 to 98 percent reliability such that any given footing settlement is within about ±20 percent of some amount considered tolerable for that structure. It is preferable that settlements all be within less than 20 percent.

Using simple statistics and assuming the work has been reasonably done, if there are 20 to 25 footings in an area, the average settlement will probably be within about ±20 percent (taken as the standard deviation) of the computed value, but there will be at least one whose settlement is about twice as large as the smallest settlement, thus establishing the extremes.

For this ±20 percent settlement range to occur it is necessary to use representative soil properties from the given site. Statistics may be employed to obtain the most probable value. There are a number of statistical procedures given in the literature, but most use symbols and terminology not familiar to engineers, causing them to underutilize these methods. The statistical methods of simple averaging or weighted averaging are easy to apply but are somewhat time-consuming.

Finally, field construction methods may be significant in the settlement outcome. For example, most footings require some soil excavation. If the soil is freestanding, the footing perimeter is often excavated slightly larger so that mechanical excavation equipment can be used, but the excavated pit walls serve as forms. If the soil is not free standing, excess perimeter excavation is required so that the footing forms can be set. In either case the soil beneath the footing must be recompacted. Depending on the compactor and amount of compaction, the soil state can be changed significantly (increase in density, apparent overconsolidation, stiffness, etc.). These state changes can substantially reduce the settlements, particularly on sand. On the other hand, if there is no compaction before the base is placed, the settlements can greatly exceed the computed values.

PROBLEMS

Problems 5-1 to 5-3 are to be assigned by the instructor from the following table by key number, which provides the thickness of the strata in the soil profile given in Fig. P5-1, in feet or meters.

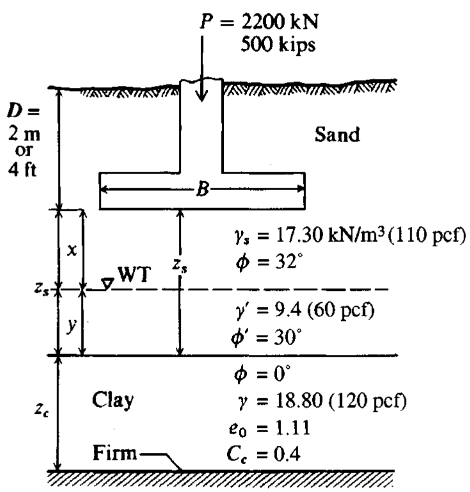

Figure P5-1

B = 2.5 × 2.5 m or 10 × 10

Key Number	z_s	x	y	z_c
1	1.5	1.5	0	1.5
2	2.5	2.5	0	2
3	4	3	1.2	1.2
4*	5	3	2	6
5	4.6	3	1.5	3
6	2	1	1	3
7*	10	6	4	15
8	2	2	0	5
9*	2	2	0	10
10	1	1	0	4

*Dimension in ft.

5-1. Referring to Fig. P5-1, compute the average increase in stress Δq for the clay stratum for the assigned key number from table of strata thickness by (*a*) Boussinesq method; (*b*) Westergaard method (and use $\mu = 0.45$ for saturated clay); (*c*) by 2 : 1 method [for this use Eq. (5-2*b*)].

Partial answer:

Problems	(*a*)	(*b*)	(*c*)
5-1(1)	144.2	202.1	100.0 kPa
5-1(2)	75.4	143.3	62.9
5-1(9)	2.81	3.25	1.89 ksf
5-1(10)	117.2	199.0	83.8

5-2. Compute the consolidation settlement using the Δq obtained from Prob. 5-1. Comment on any differences in the computed settlement.

Partial answers: (mm or in.)

Problems	(*a*)	(*b*)	(*c*)	p'_o
5-1(1)	141.4	171.3	112.5	67.3
5-1(2)	103.0	160.5	89.7	86.8
5-1(4)	6.91	8.02	5.82	1.06 ksf
5-1(6)	122.0	140.4	154.9	74.8
5-1(10)	259.5	362.7	443.7	69.9

5-3. What size footing in Prob. 5-1 (assign only key numbers 1 through 4) is required to limit the consolidation settlement to not over 1.5 in. or 40 mm?

Partial answer:

Problem		*B*
5-1(1)	(*b*)*	≅ 9 m
5-1(2)	(*b*)*	≅ 8.5 m
5-1(4)	(*w*)	≅ 40 ft

*(*b*) = Boussinesq; (*w*) = Westergaard

5-4. What footing load can be used for Prob. 5-1(1), using the Boussinesq pressure profile, to limit the 2.5 × 2.5 m square base to a settlement of 40 mm. The current load is 2200 kN. What load is the maximum allowable using the 2 : 1 method?

Partial answer: 2 : 1: $Q \simeq 570$ kN [by trial $Q_{\text{Boussinesq}} = 450$ kN (40.7 mm)]

5-5. If it will take a $B = 9$ m square (very large) base to carry 2200 kN, what might be an alternative solution to carry the 2200-kN column load?

5-6. Verify the centerline stress ratios of Fig. 5-4 using Eq. (5-5) (Boussinesq equation). Note along the center line $r = 0$ and $z = D/B$.

5-7. Assume in Example 5-14 that instead of 1.5 tsf, $p'_c = 1.0$ tsf and recompute the expected consolidation settlement ΔH_c. Next assume the given $p'_c = 1.5$ tsf and $C_c = 0.40$ instead of 0.32 and compute the settlement. Compare the two settlement values and see if you can draw any conclusions as to the relative effect of error in p'_c versus error in C_c.

5-8. Using either Method 3 or Method 4 of Sec. 2-10.3 compare p'_c to your best construction of Casagrande's Method 2. For both methods make an enlargement of Fig. E5-14*a* on a copy (or other) machine so you can pick off the data points with some confidence. Use the enlarged plot directly for the Casagrande construction. Comment on the preconsolidation pressure p'_c obtained by these two methods compared with that used by the author.

5-9. Using the Tan and Inove data set on Fig. 2-23, verify select additional plot points and replot the data on a sheet of graph paper and compute the expected settlement ΔH at the end of 2 years.

5-10. Referring to Sec. 5-12.2, what would be the secondary compression settlement and about how long would it take if instead of 100 minutes for t_{100} in the laboratory the plot of ΔH versus $\log t$ gives $t_{100} = 10$ minutes? For C_α use 0.032 and then compute a second value using the equation given in Table 2-5 with $I_P \approx 56$ (obtained from Fig. E5-14*b*). Average the two values for C_α for this problem. Can you draw any conclusions between the computations of Sec. 5-12.2 and here?

5-11. Rework Example 5-5 for $z = 5$ ft.

5-12. Rework Example 5-8 if the moment is resisted by $B = 2$ m.

5-13. Rework Example 5-9 if column loads are expected in the range of 900 to 1800 kN.

5-14. Referring to Example 5-12, if B increases to 6 m, what should the contact pressure q_o be to hold $\Delta H = \text{constant} = 16.8$ mm?

5-15. The allowable bearing pressure on a 30-ft thick (below base of footing) medium dense sand (take $\phi = 36°$, $\gamma = 112$ pcf) is 3 ksf. Column A has design load of 430 kips and Column B has 190 kips. What size footings would you use and what might one expect for differential settlement? By using Table 5-7, is this differential settlement satisfactory?

5-16. Two CU triaxial tests were performed on a light brown silty clay obtained from a depth of 5 m and the test data shown following. Footings are to be placed 1.8 m below ground surface on this material, which extends to a depth of approximately 7.3 m. The water table is at 9.3 m in a medium dense sand underlying this clay. Footing loads are 1000 to 1500 kN. What do you recommend for bearing capacity and what do you estimate for total and differential settlements? Is the soil in the CU tests saturated?

ϵ	**Test No. 1** $\sigma_3 = 70$ $\Delta\sigma_1$	**Test No. 3** $\sigma_3 = 140$ kPa $\Delta\sigma_1$, kPa
0	0	0
0.010	26	17
0.014	39	39
0.02	93	93
0.03	134	131
0.04	142	150
0.05	168	197
0.07	185	221
0.09	205	233
0.12	235	234
0.14	239	245
0.16	241	259
0.19	265	244
0.21	266	228

5-17. Verify the assigned case from Table 5-3 for predicted settlement and make any appropriate comments. Use the author's procedure for the verification process.