SETTLEMENT CALCULATION REFERENCE

over a representative depth for the footing, as we did for the Meyerhof (1965) method. Equation (9.26) was based on the assumption that settlements predicted by the Terzaghi and Peck (1967) correlation would produce excessively conservative results (that is, excessively large settlements).

The groundwater correction factor C_{gw} is simply the ratio of the vertical effective stress at a depth of $B/2$ below the base of the footing when the water table is deep versus when it is within the zone of influence for the footing:

$$C_{gw} = \frac{\left(\sigma'_v\big|_{z_f=\frac{B}{2}}\right)_{\text{without WT}}}{\left(\sigma'_v\big|_{z_f=\frac{B}{2}}\right)_{\text{with WT}}} \tag{9.29}$$

The Peck and Bazaraa (1969) method should be corrected for embedment, although exactly what this correction should be was not elaborated on by the authors of the equations. Presumably, the same correction suggested by Meyerhof (1965) would be appropriate.

Another, more recent method for estimating settlements of footings in sand or gravel was proposed by Burland and Burbidge (1985):

$$\boxed{\frac{w}{L_R} = 0.10 f_s f_L f_t I_c \frac{q_b - \frac{2}{3}\sigma'_{vp}\big|_{z_f=0}}{p_A}\left(\frac{B}{L_R}\right)^{0.7}} \tag{9.30}$$

where f_s = shape factor, f_L = layer thickness factor, f_t = time factor, q_b = unit load at footing base (including both structural loads and the weight of the backfill and foundation element), $\sigma'_{vp}|_{z_f=0}$ = maximum previous vertical effective stress at footing base level, z_f = depth measured from foundation base level, B = footing width; I_c = compressibility index, L_R = reference length = 1 m = 3.281 ft = 39.37 in.; p_A = reference stress = 100 kPa ≈ 1 tsf.

The use of $\sigma'_{vp}|_{z_f=0}$ in Eqs. (9.24)–(9.26) and (9.30) allows for the fact that any measurable settlement will only develop after the vertical effective stress exceeds the maximum vertical effective stress ever experienced by the soil below the foundation base.

Burland and Burbidge (1985) proposed values of the compressibility index I_c as a function of SPT blow count:

$$I_c = \frac{1.71}{\bar{N}^{1.4}} \tag{9.31}$$

where $\bar{N}$ is the average blow count over the depth of influence z_{f0} below the footing base. The authors are not explicit as to whether to normalize blow counts, but we may assume $\bar{N}$ to be an energy-corrected, standardized blow count N_{60}. The depth of influence can be calculated from

$$\frac{z_{f0}}{L_R} = \left(\frac{B}{L_R}\right)^{0.79} \tag{9.32}$$

REF:
Salgado, R. 2008. The Engineering of Foundations. McGraw Hill Higher Education, Boston, MA. 2008.

The blow counts are corrected as follows:

1. For gravel or sandy gravel, increase measured blow counts by 25%;
2. For sands below the water table, any excess of the measured blow count over 15 blows should be halved (for example, 21 would really be 15 + 6/2 = 15 + 3 = 18).

The shape factor is given by

$$f_s = \left(\frac{1.25 \frac{L}{B}}{\frac{L}{B} + 0.25} \right)^2 \tag{9.33}$$

The sand layer thickness factor f_L is used to account for cases in which the depth of influence z_{f0} is larger than the actual thickness H of the sand layer. This means that, if a very hard layer (typically bedrock) exists at a depth smaller than the depth of influence, the settlement should be less; this result is obtained in the method by using f_L. This factor is given as

$$f_L = \begin{cases} \frac{H}{z_{f0}}\left(2 - \frac{H}{z_{f0}}\right) & \text{if } H \leq z_{f0} \\ 1 & \text{if } H > z_{f0} \end{cases} \tag{9.34}$$

The time factor is based on observations of increasing settlement with time by Burland and Burbidge (1985). Some like to refer to this as a creep factor, but this factor is also intended to capture the effect, particularly on tall structures (such as tall chimneys, silos, high-rise buildings), of wind loads, which is not creep in the strict definition of the term. The repeated application of wind loads can always generate some additional settlement. Over time, these can add up to a nonnegligible amount. Burland and Burbidge (1985) proposed the following expression for it:

$$f_t = \left(1 + R_3 + R_t \log \frac{t}{3}\right) \tag{9.35}$$

where t is the time in years, R_3 = ratio of settlement developing over a period of 3 yr to the immediate settlement, and R_t = ratio of settlement developing over a log cycle of time to the immediate settlement. It is not necessarily easy to estimate the values of R_3 and R_t because precise measurements of immediate versus long-term settlement are not available. A design based on $f_t = 1$ is typically acceptable, although Burland and Burbidge give values of $R_t = 0.2$ and $R_3 = 0.3$.

EXAMPLE 9-4

A 6-ft square footing (see Fig. 9-7), which is backfilled, is embedded 2.5 ft into a sand. SPT N_{60} values for the sand are given in Table 9-3. Estimate the settlement of this footing caused by a column load of 182 tons. The sand is approximately 40 ft thick and has an average unit

REF:
Salgado, R., 2008. The Engineering of Foundations. McGraw Hill Higher Education, Boston, MA. 2008.

BEARING CAPACITY CALCULATION REFERENCE

Bearing Capacity

The bearing-capacity problem is really one of passive earth pressure. The shearing stresses at the contact face *BC* of Zone I are expressed by Coulomb's equation (Eq. 3.8) in which shear stress *s* is replaced by the total passive resistance and the normal stress σ_n is replaced by the normal component of the passive earth pressure per unit of area of the contact face. At the failure instant, the pressure on *BC* is equal to the resultant of the passive resistance plus cohesion along the failure surface *BC*. The total passive resistance includes the weight of the mass in Zones II and III, a cohesive resistance along the failure surface *AE*, and a surcharge q_s from the overburden soil.

Terzaghi's Expression

A closed form for this problem in plasticity has been found only for the case of weightless soil [Prandtl (1921)[37]]. The real nature of soil with weight can be considered approximately (with an error <20% for ϕ between 30 and 40°) by Terzaghi's (1943)[34] expression for drained loading conditions for a long rectangular footing (the Buisman-Terzaghi equation):

$$q_b = cN_c + qN_q + 0.5\gamma BN_\gamma \qquad (6.10)$$

where q_b = ultimate bearing capacity = Q_b/B
B = width of smaller footing side
c = cohesion
γ = effective soil unit weight
q = overburden stress at footing base = γD_f
N_c, N_q, N_γ = the *bearing-capacity factors*, dimensionless functions of ϕ

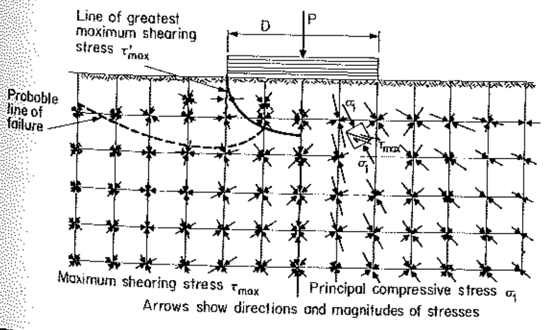

FIG. 6.29 Distribution of compressive and shearing stresses beneath a circular loaded area during general shear. [After Converse (1939)[35]; from Tschebotarioff (1973).[36] Reprinted with permission of McGraw-Hill Book Company.]

The three parts of Eq. 6.10 provide for soil cohesion, foundation depth (overburden pressure), and foundation width, with ϕ a factor in all three parts. Even a small amount of cohesion substantially increases q_b. The first term is a function of parameters ϕ and c, the second term is a function of the vertical confining effective stress ($\bar{\sigma}_v = \gamma D_f$) at the elevation of the footing base, and the third term is a function of the footing width on the average weight of the soil mass under the footing. All three terms also are a function of the assumed shape of the rupture surface and of ϕ. [For *deep foundations* (see Art. 7.3.3), the third term is very small in comparison to the second and is neglected.]

A number of values are available for the *bearing-capacity factors*, which are exclusively a function of ϕ and the failure surface geometry. The current trend of many practitioners is to use values developed by Prandtl (1921)[37] and Reissner (1924)[38] for N_c and N_q and those developed by Caquot and Kerisel (1953)[39] for N_γ, as given in Table 6.7.

Bearing-Capacity Analysis

Factors to Consider

- Drained versus undrained conditions and, for the drained case, general versus local shear
- Foundation shape and depth and base inclination
- Loading eccentricity and inclination
- Ground-slope condition
- Groundwater-table condition

Drained Loading Conditions

Drained loading conditions occur normally where loads are applied gradually as the structure is erected and parameters c and ϕ are mobilized. *General shear* prevails for relatively strong materials, and bearing capacity is commonly found for continuous footings by Eq. 6.10. For square and circular footings Terzaghi (1943)[34] proposed

$$q_b = \frac{Q}{A} = 1.2\,cN_c + qN_q + s_\gamma BN_\gamma \qquad (6.11)$$

where the shape factor s_γ = 0.4 for square footings and 0.6 for circular footings.

REF:
Hunt, Roy F., 1986. Geotechnical Engineering. McGraw Hill Higher Education, Boston, MA. 2008

SBN 0-07-031310-

TABLE 6.7
BEARING-CAPACITY FACTORS

ϕ	N_c	N_q	N_γ	N_q/N_c	$\tan\phi$
0	5.14	1.00	0.00	0.20	0.00
1	5.38	1.09	0.07	0.20	0.02
2	5.63	1.20	0.15	0.21	0.03
3	5.90	1.31	0.24	0.22	0.05
4	6.19	1.43	0.34	0.23	0.07
5	6.49	1.57	0.45	0.24	0.09
6	6.81	1.72	0.57	0.25	0.11
7	7.16	1.88	0.71	0.26	0.12
8	7.53	2.06	0.86	0.27	0.14
9	7.92	2.25	1.03	0.28	0.16
10	8.35	2.47	1.22	0.30	0.18
11	8.80	2.71	1.44	0.31	0.19
12	9.28	2.97	1.69	0.32	0.21
13	9.81	3.26	1.97	0.33	0.23
14	10.37	3.59	2.29	0.35	0.25
15	10.98	3.94	2.65	0.36	0.27
16	11.63	4.34	3.06	0.37	0.29
17	12.34	4.77	3.53	0.39	0.31
18	13.10	5.26	4.07	0.40	0.32
19	13.93	5.80	4.68	0.42	0.34
20	14.83	6.40	5.39	0.43	0.36
21	15.82	7.07	6.20	0.45	0.38
22	16.88	7.82	7.13	0.46	0.40
23	18.05	8.66	8.20	0.48	0.42
24	19.32	9.60	9.44	0.50	0.45
25	20.72	10.66	10.88	0.51	0.47
26	22.25	11.85	12.54	0.53	0.49
27	23.94	13.20	14.47	0.55	0.51
28	25.80	14.72	16.72	0.57	0.53
29	27.86	16.44	19.34	0.59	0.55
30	30.14	18.40	22.40	0.61	0.58
31	32.67	20.63	25.99	0.63	0.60
32	35.49	23.18	30.22	0.65	0.62
33	38.64	26.09	35.19	0.68	0.65
34	42.16	29.44	41.06	0.70	0.67
35	46.12	33.30	48.03	0.72	0.70
36	50.59	37.75	56.31	0.75	0.73
37	55.63	42.92	66.19	0.77	0.75
38	61.35	48.93	78.03	0.80	0.78
39	67.87	55.96	92.25	0.82	0.81
40	75.31	64.20	109.41	0.85	0.84
41	83.86	73.90	130.22	0.88	0.87
42	93.71	85.38	155.55	0.91	0.90
43	105.11	99.02	186.54	0.94	0.93
44	118.37	115.31	224.64	0.97	0.97
45	133.88	134.88	271.76	1.01	1.00
46	152.10	158.51	330.35	1.04	1.04
47	173.64	187.21	403.67	1.08	1.07
48	199.26	222.31	496.01	1.12	1.11
49	229.93	265.51	613.16	1.15	1.15
50	266.89	319.97	762.89	1.20	1.19

Local shear applies to weaker materials and Terzaghi accounted for soil compressibility by reducing the ultimate strength parameters c_g for ϕ_g (general shear) as follows:

- Soft to firm clays:

$$c_{\text{local}} = \tfrac{2}{3} c_g \qquad (6.12)$$

- Loose sands:

$$\tan\phi_{\text{local}} = \tfrac{2}{3} \tan\phi_g \qquad (6.13)$$

This results in an abrupt transition from general to local shear but is considered to be suitable for cases with overburden pressure although conservative for surface to near-surface loadings [Ismael and Vesić (1981)[33]]. See, for example, case 2 in Fig. 6.33.

Sands (c = 0)

In sands, where $c = 0$ (see for example, case 3 in Fig. 6.33), Eq. 6.10 for the gross ultimate bearing capacity becomes

$$q_b = \gamma D_f N_q + 0.5\gamma B N_\gamma \qquad (6.14)$$

The *net* ultimate bearing capacity $q_b' = q_b - \gamma D_f$, or the gross capacity minus overburden pressure at footing level. In terms of q_b', Eq. 6.14 can be expressed in the form [Peck et al. (1974)[17]] for the allowable bearing value as

$$q_a = \frac{q_b'}{FS} = \left[\gamma(N_q - 1)\frac{D_f}{B} + 0.5\gamma N_\gamma\right]\frac{B}{FS} \qquad (6.15)$$

where FS = safety factor. For a particular value of D_f/B and a given sand deposit, the expression within the brackets is a constant. Thus, the relation between footing width B and the net allowable soil pressure q_a for a given value of FS can be plotted as in Fig. 6.18 as a family of straight lines radiating from the origin. In Fig. 6.18, FS = 2. Each line corresponds to a different value for ϕ as determined roughly from the N value of the SPT as given in Fig. 6.30. Note that the values for N_γ given in Fig. 6.30 are lower than those given in Table 6.7.

Bearing-Capacity Equation: General Form

In recent years Eq. 6.10 has been expanded by various investigators to provide for the various factors that influence q_b as follows:

$$q_b = cN_c s_c b_c i_c g_c + qN_q s_q b_q i_q g_q + 0.5\gamma B N_\gamma s_\gamma b_\gamma i_\gamma g_\gamma \qquad (6.16)$$

REF:
Hunt, Roy F., 1986. Geotechnical Engineering. McGraw Hill Higher Education, Boston, MA. 1986.

SETTLEMENT CALCULATION REFERENCE

Z = 300
R = 76.625

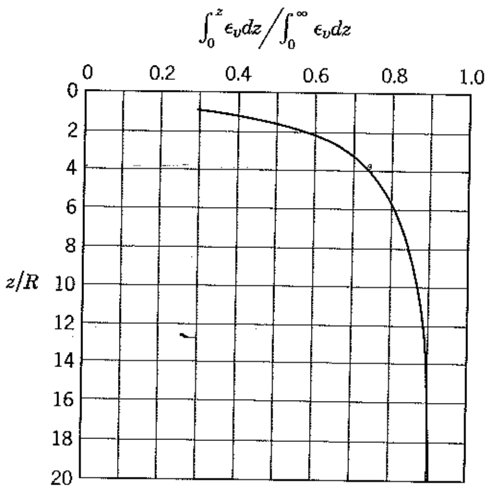

Fig. 14.21 Effect upon I_ρ of considering only a limited depth of strains.

▶ Example 14.13 method 3

Given. The tank loading and subsoil shown in Example 8.9.

$$E = 2000 \text{ kips/ft}^2$$
$$\mu = 0.45$$

Find. The settlement at the center of the tank for the condition of homogeneous, isotropic soil of infinite depth.

Solution.

$$\rho_{℄} = \Delta q_s \frac{R}{E} 2(1 - \mu^2) \qquad \text{Eq. 14.15}$$

$$\left.\begin{aligned} \Delta q_s &= 5.50 \text{ kips/ft}^2 \\ R &= \frac{D}{2} = \frac{153\frac{1}{4}\text{ ft}}{2} \end{aligned}\right\} \quad \text{given in Example 8.9}$$

$$\rho_{℄} = \frac{5.50 \text{ kips/ft}^2 \times \frac{153\frac{1}{4}\text{ ft}}{2} \times 2(1 - 0.45^2)}{2000 \text{ kips/ft}^2}$$

$$= \underline{\underline{0.346}} \text{ ft} = \underline{\underline{4}} \text{ in.}$$

0.336

method 4

Settlement may be estimated by multiplying an average strain times the depth of the bulb of stresses. The following tabulation shows several ways in which this might be done.

Assumed Depth of Bulb	Average Strain	Settlement (in.)
$3R$ = 230 ft	Use strain at depth of $3R/2$: $\epsilon_v = 0.00106$	3.0
$4R$ = 306 ft	Use strain at depth of $2R$: $\epsilon_v = 0.00076$	2.8

The first method, using a bulb of depth $3R$, gives a closer estimate to the actual result of 4 in. ◀

loaded area the horizontal strains at the surface must be tensile, and this can happen only if the horizontal stress increments are tensile. Circular tension cracks are often observed around heavy loads resting on the surface of grounds. This general pattern of horizontal strains is somewhat similar to that in a fixed-end beam carrying a concentrated load at midspan.

Equation 14.14 may also be used when the elastic body is of limited depth. However, a different value of I_ρ must be used. Figure 14.20 gives values of I_ρ for two cases of an elastic stratum of limited depth. As would be expected, decreasing the depth of the elastic body decreases the settlement. When the elastic body is thin in comparison to the dimensions of the load, points outside of the loaded area may heave instead of settling. Burmister (1956) has compiled charts and tables that are especially useful when dealing with settlements of strata of limited thickness.

Elastic Theory for Settlement under Other Uniform Loads

The settlement at the corner of a rectangular area carrying a uniform stress Δq_s may be calculated from

$$\boxed{\rho = \Delta q_s \frac{B(1 - \mu^2)}{E} I_\rho} \qquad (14.16)$$

where

B = the width (least dimension) of the rectangle
L = the length (greatest dimension) of the rectangle
I_ρ = an influence coefficient given by Fig. 14.22

Settlements for points other than the corner of a rectangular area, and for any shape of loaded area that can be divided into rectangles, can be obtained using the method of superposition, as explained in Chapter 8 in connection with computing stresses (see Example 8.3). In particular, the settlement at the center of a square loaded area is

$$\rho = \Delta q_s \frac{B}{E} 1.12(1 - \mu^2) \qquad (14.17)$$

As L/B becomes very great (i.e., for a strip footing), I_ρ gradually increases beyond all bounds. Thus a strip footing resting on an elastic body of infinite depth would experience an infinite settlement. In real problems, of course, soil strata are not infinite in depth and strip footings are not infinite in length. For a rectangular loaded area on an elastic stratum of thickness D overlying a rigid body, the approximate settlement at the corner of the area may be computed using Eq. 14.16 and

$$I_\rho = (1 - \mu^2)F_1 + (1 - \mu - 2\mu^2)F_2 \qquad (14.18)$$

where the functions F_1 and F_2 may be read from Fig. 14.23. Burmister (1956) also presents useful charts for such problems.

REF:
Lambe T.W. and R.V. Whitman, 1969. Soil Mechanics. John Wiley & Sons, Inc. New York, NY. 1969.

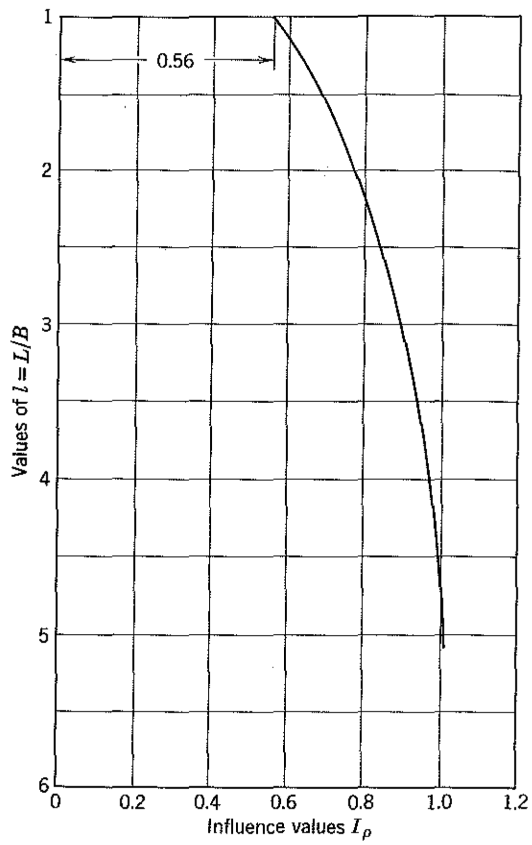

Fig. 14.22 Influence coefficient for settlement of rectangular loaded area (From Terzaghi, 1943).

Solutions have also been obtained for many other types of loading conditions, including loading by shear stresses. Scott (1963) gives a useful summary. With computer techniques, numerical values applicable to specific cases can be obtained.

Elastic Theory for Settlement under Rigid Footings

The condition of a uniformly distributed load occurs in practical problems, such as steel tanks for the storage of fluids. In many practical problems, however, the structural member (such as a footing) in contact with the soil will be quite rigid, and the settlement is more or less uniform over the area of contact between the footing and the soil. Since a uniform stress causes a dish-shaped pattern of settlement, in order to produce a uniform settlement the contact stresses must increase on the outside of the loaded area and decrease near the center line. The curves in Fig. 14.24 marked $K_r = \infty$ show the theoretical distribution of contact stress for the case of a truly rigid foundation. At the edge of the loaded area the contact stress theoretically is infinite.

A change in the distribution of stress over the contact area means a change in the relationship between load and settlement. For a circular rigid loaded area this becomes

$$\rho = \Delta q_s \frac{R}{E} \frac{\pi}{2} (1 - \mu^2) \qquad (14.19)$$

where Δq_s = average stress over the loaded area. Comparing Eq. 14.19 with Eq. 14.15, we see that the settlement of a rigid footing is 21% less than the center line settlement under a uniform load. Whitman and Richart (1967) present load-settlement relationships for rigid rectangular footings with various types of loading.

In some problems the structural member in contact with the soil cannot be considered perfectly flexible or perfectly rigid. Figure 14.24 can be used to estimate the contact stresses for intermediate conditions.

14.9 THEORETICAL PROCEDURES FOR USE WITH SOILS

As was discussed in Chapters 10 and 12, a mass of soil does not behave as an elastic, homogeneous, and isotropic material. Nonelasticity influences (*a*) the distribution of stress increments caused by these loads, and (*b*) the strains resulting from these stress increments. At present there are no theoretical procedures that consider both these difficulties, although such procedures are under development. Fortunately, experience has shown that useful predictions of settlement can be made by using the distribution of stress increments predicted by elastic theory, but employing special procedures to determine the resulting strains.

Stress Path Method

As applied to estimating the settlements, the stress path method consists of the following four steps:

1. Select one or more points within the soil under the proposed structure.
2. Estimate for each point the stress path for the loading to be imposed by the structure.
3. Perform laboratory tests which follow the estimated stress paths.
4. Use the strains measured in these tests to estimate the settlement of the proposed structure.

This same general approach, which is a powerful aid to understanding and solving deformation and stability problems, has already been used in Chapter 13.

Example 14.14 illustrates the application of this approach to the tank foundation of Example 8.9. Stress paths for selected points have already been given in Example 8.9. Figure 10.23 presents stress-strain results from triaxial tests following the stress paths for points A, B, D, and G. The vertical and horizontal strains as measured in these tests have been plotted in Example 14.14. Integration of these strains over a depth of 300

REF:
Lambe T.W. and R.V. Whitman, 1969. Soil Mechanics. John Wiley & Sons, Inc. New York, NY. 1969.