Source: PRACTICAL FOUNDATION ENGINEERING HANDBOOK

SECTION 2B
SHALLOW FOUNDATIONS

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2B.1 INTRODUCTION

A foundation is a structure built to transfer the weight of a building to the material below. This transfer must occur such that the soil below does not rupture or compress to such magnitude that the integrity of the superstructure is threatened.

Foundations are generally classified as either deep or shallow. The depth of the bearing area of shallow foundations is generally no deeper than about the width of the bearing surface.

Deep foundations provide support for a structure by transferring the loads to competent soil and/or rock at some depth below the structure.

2B.2 SHALLOW FOUNDATIONS

Shallow foundations can be either footings or mats. They consist of reinforced concrete slabs formed directly on a prepared soil base. Footings may be either spread, combined, or continuous.

2B.2.1 Footings

Spread footings are footings that support one column or load. These footings are also called isolated or column footings. They typically are 3 to 8 to 10 ft (0.9 to 2.4 to 3 m) square. Their bearing surface depth is typically less than 2.5 times their length ($D_f < 2.5B$) (Figure 2B.1).

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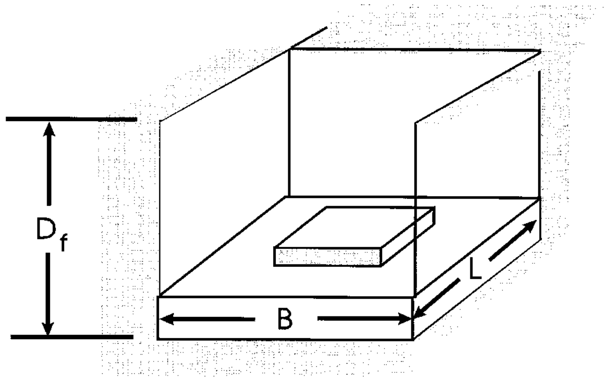

FIGURE 2B.1 Isolated shallow footing.

Combined footings are similar to spread footings but support two or more columns. The shape is more likely to be a rectangle or occasionally trapezoidal (Figure 2B.2). These footings are used where column spacing is nonuniform and for the support of exterior columns near property lines where there isn't enough room for a spread footing.

A continuous or strip footing is an elongated shallow foundation that typically supports a single row of columns or a wall or other type of strip loading. Continuous footings tie together columns in one direction at their base, and reduce construction costs through use of appropriate equipment for trenching (Figure 2B.3).

The advantages of shallow foundations lie primarily in their low cost and speed of construction.

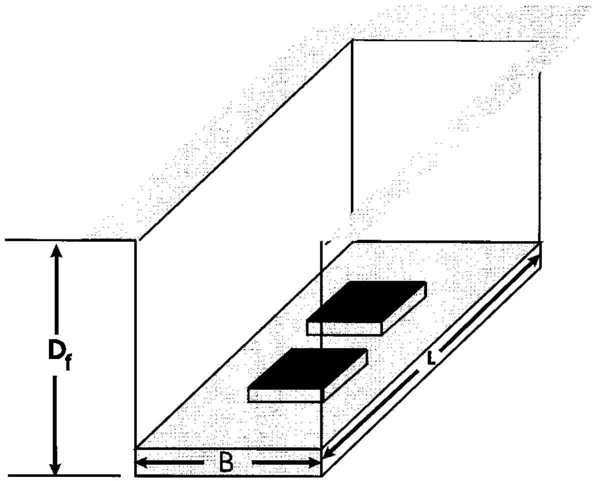

FIGURE 2B.2 Combined footing.

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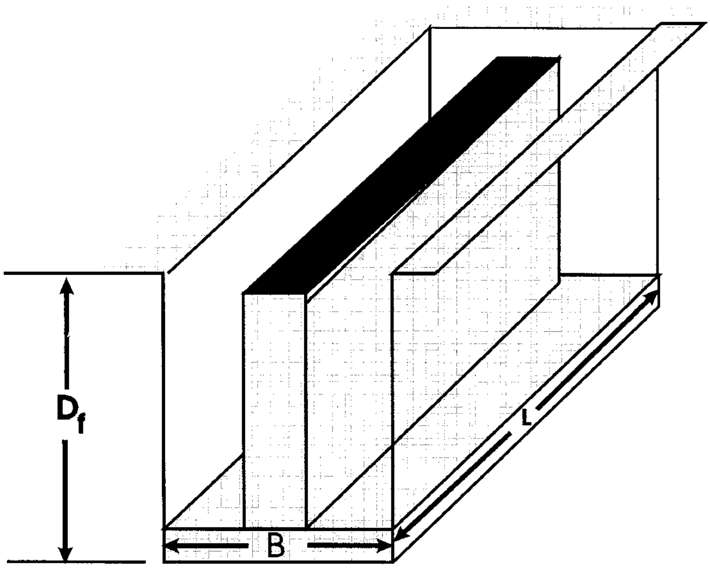

FIGURE 2B.3 Continuous footing.

2B.2.2 Mats

A mat (raft) foundation is a structural reinforced concrete slab that supports a number of columns distributed in both horizontal directions or supports uniform pressure, as from a tank. Rafts are used to bridge over soft spots if the spots are very localized, and to reduce the average pressure applied to the soil (Figure 2B.4).

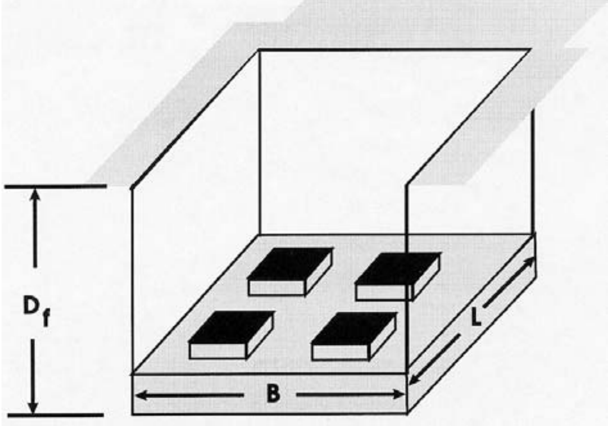

FIGURE 2B.4 Mat foundation.

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2B.3 DESIGN PARAMETERS

Design parameters for shallow foundations fall into two classes: structural design parameters and geotechnical design parameters.

2B.3.1 Structural Design Parameters

Structural design parameters that influence the design of the shallow foundation include the building type and use, loading (live, dead, and uplift), column spacing, presence or absence of a basement, allowable settlement, and applicable building codes.

2B.3.2 Geotechnical Design Parameters

Geotechnical factors that influence the design include the thickness and lateral extent of bearing strata, the depth of frost penetration, the depth of seasonal volume change, and the cut/fill requirements. The strength, compressibility, and shrink-swell potential of the bearing strata are the properties of concern.

In addition, the presence or absence of groundwater and its minimum and maximum elevations have an important impact on the design process.

2B.4 BEARING CAPACITY OF SHALLOW FOUNDATIONS

The design of a shallow foundation requires that the applied load does not exceed the load that would cause the soil strata beneath the foundation to rupture. The maximum load that can be applied to the foundation soil without rupture is called the bearing capacity.

2B.4.1 Development of General Bearing Capacity Equation

Simplifying the model of a shallow, continuous footing at impending failure, as in Figure 2B.5, allows the problem to be treated as an earth pressure problem. When the footing is loaded, the wedge of soil beneath the footing generates lateral pressures at the wedge boundaries. These pressures cause the adjacent wedges to displace, as shown in Figure 2B.5. The slip surfaces for the wedge beneath the footing develop slip lines at $\alpha = 45 + \phi/2$ with the horizontal. The adjacent wedge has slip line angles of $\rho = 45 - \phi/2$ with the horizontal.

If the effect of the soil above the base level of the footing is replaced with a surcharge γD_f then

$$\bar{q} = \gamma D_f \tag{2B.1}$$

A look at the stress block on the right allows computations of the total resisting earth pressure as force P_p from equation (2B.2).

$$P_p = \int_0^H \sigma_1 (dz) = \int_0^H \left[(\gamma z + \bar{q}) \tan^2\left(45 + \frac{\phi}{2}\right) + (2c)\tan\left(45 + \frac{\phi}{2}\right) \right] dz \tag{2B.2}$$

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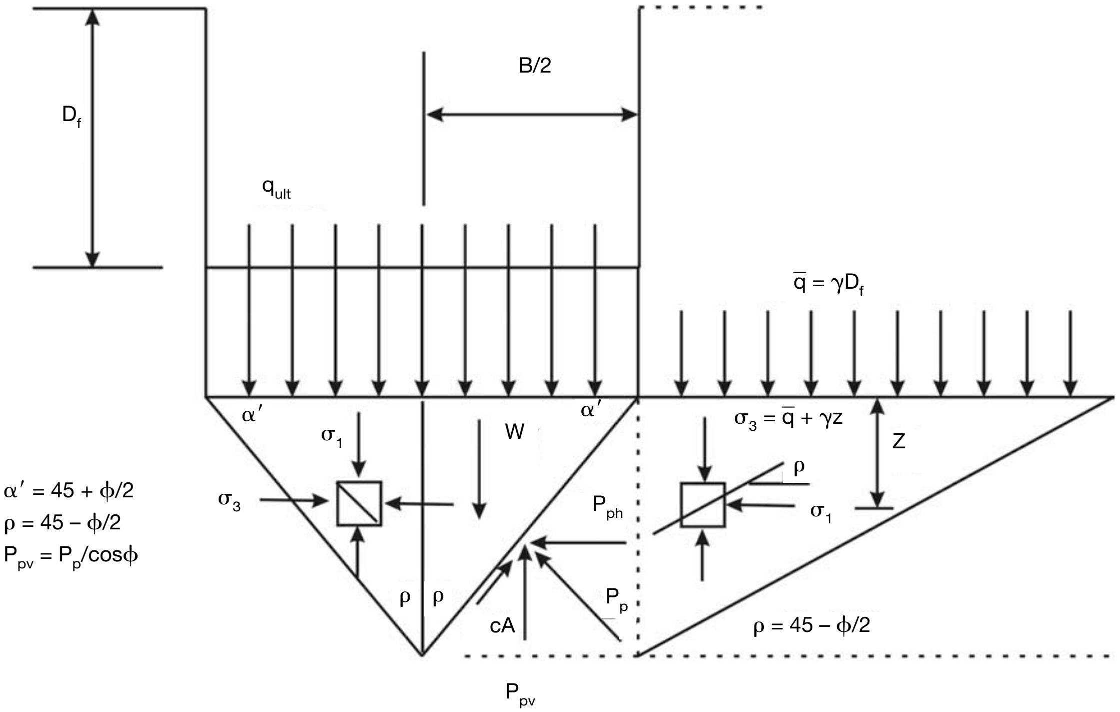

FIGURE 2B.5 Simplified bearing capacity diagram.

where σ_1 is

$$\sigma_1 = \sigma_3 N_\phi + 2cN_\phi \tag{2B.3}$$

where c is the cohesion and ϕ is the angle of internal friction.

Defining K_p as in equation (2B.4), the result of the integration is given in equation (2B.5).

$$K_p = N_\phi = \tan^2\left(45 + \frac{\phi}{2}\right) \tag{2B.4}$$

$$P_p = \frac{\gamma H^2}{2} K_p + \bar{q} H K_p + 2cH\sqrt{K_p} \tag{2B.5}$$

To find q_{ult}, the vertical forces are added to obtain

$$q_{\text{ult}}\left(\frac{B}{2}\right) + \gamma\left(\frac{B}{2}\right)\left(\frac{H}{2}\right) - (cA)\cos\rho - \frac{P_p}{(\sin\rho)(\cos\phi)} = 0 \tag{2B.6}$$

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since

$$H = \left(\frac{B}{2}\right)\tan\alpha; \quad W = \frac{1}{2}\gamma\left(\frac{B}{2}\right)\left(\frac{B}{2}\right)\tan\alpha$$

$$A = \frac{B}{2\cos\alpha}; K_p = \tan^2\left(45 + \frac{\phi}{2}\right) \tag{2B.7}$$

$$P_{p,v} = \frac{P_p}{\cos\phi}; K_a = \tan^2\left(45 + \frac{\phi}{2}\right)$$

then

$$q_{\text{ult}} = c\left[\frac{2K_p}{\cos\phi} + \sqrt{K_p}\right] + \bar{q}\frac{\sqrt{K_p}K_p}{\cos\phi} + \frac{\gamma B}{4}\left[\frac{K_p^2}{\cos\phi} - \sqrt{K_p}\right] \tag{2B.8}$$

This equation can be written as

$$q_{\text{ult}} = cN_c + \bar{q}N_q + \gamma BN_\gamma \tag{2B.9}$$

where

$$N_c = \left[\frac{2K_p}{\cos\phi} + \sqrt{K_p}\right]$$

$$N_q = \frac{\sqrt{K_p}K_p}{\cos\phi} \tag{2B.10}$$

$$N_\gamma = \frac{1}{4}\left[\frac{K_p^2}{\cos\phi} - \sqrt{K_p}\right]$$

This is a form of the *general bearing capacity equation*. The N factors are the *bearing capacity* factors.

N_c = nondimensional bearing capacity factor relating the influence of soil cohesion on bearing capacity (a function of ϕ of the soil).

N_q = nondimensional bearing capacity factor relating the influence of soil overburden on bearing capacity (a function of ϕ of the soil).

N_γ = nondimensional bearing capacity factor relating the influence of soil unit weight on bearing capacity (a function of ϕ of the soil).

Equation (2B.9) generally underestimates the capacities of footings.

Various investigators have studied the bearing capacity problem. Each has made assumptions as to the character of the failure surface, the effect of the footing depth and shape and other factors. Although almost all the investigators developed equations similar to equation (2B.9), they computed different values of the bearing capacity factors. Some researchers included modifications to account for the footing depth, shape and inclination of loading. Bearing capacity equations reported by several authors are given in Bowles[1].

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2B.4.2 Net Ultimate Bearing Capacity

The net ultimate bearing capacity is defined as the ultimate bearing capacity of the foundation in excess of the surcharge from the surrounding soil In most cases, the difference in the unit weight of soil and the unit weight of concrete is ignored. Consequently

$$q_{\text{net}} = q_{\text{ult}} - \gamma \cdot D_f - \overline{q} \tag{2B.11}$$

$$q_{\text{ult}} = cN_c + \overline{q}(N_q - 1) + \gamma BN_\gamma \tag{2B.12}$$

2B.4.3 Choice of Unit Weight, *c*

The choice of which soil parameters to use for a given design is primarily related to the soil's hydraulic conductivity in relation to the rate of foundation loading.

2B.4.3.1 Cohesive (Clay) Soils

For cohesive soils, the assumption is usually made that the loads are applied much more rapidly than the soil can drain. Consequently: $\phi = 0$ and $N_c = 5.14$; $N_q = 1.00$ and $N_\gamma = 0$.

$$q_{\text{ult}} = cN_c \tag{2B.13}$$

2B.4.3.2 Cohesionless (Drained)

For soils that will drain rapidly, the use of effective stress parameters is suggested, i.e., use ϕ and $c = 0$.

$$q_{\text{ult}} = \overline{q}(N_q - 1) + \gamma BN_\gamma \tag{2B.14}$$

2B.4.4 Influence of Water

The presence of a water table affects equation (2B.9), depending upon where the water table lies with respect to the bearing surface of the footing (Fig. 2B.6). Consider the following conditions.

2B.4.4.1 Case I: $D_w = D_f$

In this case, the maximum level of the water table is at the base of the footing, i.e., $D_w = D_f$. Therefore, the soil above the footing base is at its natural moisture content, whereas the soil below the footing base is submerged. Therefore, the unit weight in the N_γ term should be the submerged unit weight and the unit weight in the N_q term should be the total unit weight.

$$q_{\text{ult}} = cN_c + (\gamma_{\text{sat}} D_f)N_q + \gamma' BN_\gamma \tag{2B.15}$$

where γ'= submerged weight.

2B.4.4.2 Case II: $D_w = 0$

In this case, the water table is at the ground surface. Therefore, the unit weight in both terms is the submerged unit weight.

$$q_{\text{ult}} = cN_c + (\gamma' D_f)N_q + \gamma' BN_\gamma \tag{2B.16}$$

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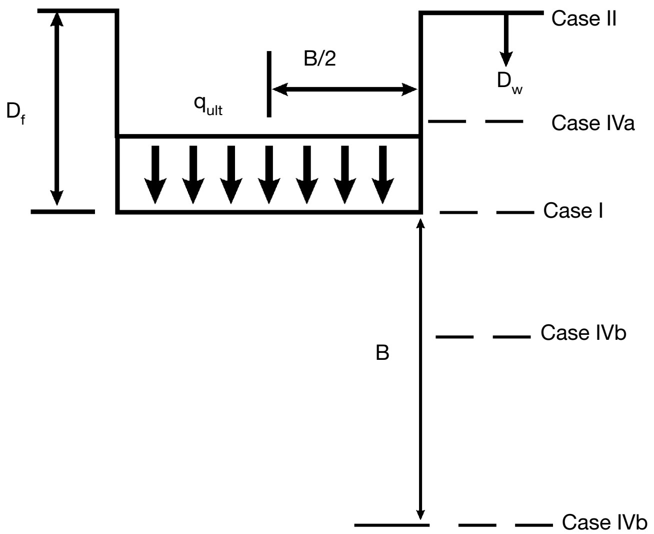

FIGURE 2B.6 Influence of water table on bearing capacity.

2B.4.4.3 Case III: $D_w = D_f + B$

In this case, the maximum height of the water table is a distance B below the base of the footing, D_f. In this case, the unit weight in both terms is the total unit weight.

$$q_{\text{ult}} = cN_c + (\gamma)N_q + \gamma BN_\gamma \tag{2B.17}$$

2B.4.4.4 Case IV: Intermediate Cases

For cases where $0 < D_w < D_f$, the unit weight in N_γ term is the submerged unit weight. A weighted average is used for the unit weight in the N_q term:

$$\gamma = \gamma' + \left[\frac{D_w}{D_f}\right][\gamma - \gamma'] \tag{2B.18}$$

For cases where $D_f < D_w < [D_f + B]$, the unit weight in the N_γ term is

$$\gamma' + \left[\frac{D_w - D_f}{B}\right][\gamma - \gamma'] \tag{2B.19}$$

The unit weight in the N_q term is the total unit weight.

2B.4.4.5 Approximate Procedure

Peck, Hanson, and Thornburn[2] developed a procedure to adjust the bearing capacity to take into account the presence of the water table. In their procedure, first solve for q_{ult} for no water table within $D_w = D_f + B$. Then set $q = C_w q_{\text{ult}}$, where

$$C_w = 0.5 + 0.5\frac{D_w}{D_f + B} \tag{2B.20}$$

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2B.4.5 General Bearing Capacity Equation

Equation (2B.9) can be generalized to take various geometric and loading factors into account:

$$q_u = cN_c s_c d_c i_c + \gamma D_f N_q s_q d_q i_q + \tfrac{1}{2}\gamma B N_\gamma s_\gamma d_\gamma i_\gamma \tag{2B.21}$$

where

s_c, s_q, s_γ = shape factors
d_c, d_q, d_γ = depth factors
i_c, i_q, i_γ = load inclination factors

These factors are discussed below.

2B.4.5.1 Bearing Capacity Factors

Various investigators have published solutions to the bearing capacity equation. Each has provided equations for the bearing capacity factors. A selection of the most useful factors is presented below.

From Meyerhof[3]:

$$N_c = \tan^2\left(45 + \frac{\phi}{2}\right)e^{\pi \tan\phi} \tag{2B.22}$$

$$N_c = (N_q - 1)\cot\phi \tag{2B.23}$$

$$N_\gamma = (N_q - 1)\tan(1.4\phi) \tag{2B.24}$$

From Hansen[4]:

$$N_\gamma = 1.5(N_q - 1)\tan\phi \tag{2B.25}$$

From Vesic[5]:

$$N_\gamma = 2(N_q + 1)\tan\phi \tag{2B.26}$$

2B.4.5.2 Shape Factors

Although the derivation of the bearing capacity equation was in two dimensions, it is obvious that the problem is three dimensional. Therefore, the relationship between the width, B, and the length, L, on the bearing capacity is of great importance. In general, this relationship is given by application of shape factors applied to the appropriate component of the general bearing capacity equation. Listed below are shape factors reported by various authors.

$$\begin{aligned} s_c &= 1 + \left(\frac{B}{L}\right)\left(\frac{N_q}{N_c}\right)(\phi > 0) \\ s_{c'} &= 0.2\frac{B}{L}(\phi = 0) \\ s_c &= 1 + 0.2K_p\frac{B}{L} \end{aligned} \tag{2B.27}$$

$$s_c = 1.0 \text{ strip} = 1.3 \text{ round} = 1.3 \text{ square}$$

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$$s_q = 1 + \left(\frac{B}{L}\right) \tan \phi$$

$$s_q = 1 + 0.1K_p \frac{B}{L} (\phi > 10°) \tag{2B.28}$$

$$s_q = 1 (\phi = 0)$$

$$s_\gamma = 1 - 0.4\left(\frac{B}{L}\right)$$

$$s_\gamma = 1 + 0.1K_p \frac{B}{L} (\phi > 10°)$$

$$s_\gamma = 1 (\phi = 0) \tag{2B.29}$$

$$s_\gamma = 1 \text{ (strip)} = 0.6 \text{ (round)} = 0.8 \text{ (square)}$$

2B.4.5.3 Depth Factors

The depth of placement of the shallow footing below the surrounding ground surface also has a significant impact on the footing's bearing capacity. In particular, this depth influences the length of the failure surface available to resist movement. These factors are usually presented in a nondimensional format, similar to that of shape factors. A listing of the most useful depth factors is given below.

$$d_c = 1 + 0.4\left(\frac{D_f}{B}\right) \text{ for } \left(\frac{D_f}{B} \leq 1\right)$$

$$d_c = 1 + 0.4\left(\tan^{-1}\frac{D_f}{B}\right) \text{ for } \left(\frac{D_f}{B} > 1\right)$$

$$d_c = 0.4\frac{D}{B} \text{ or } 0.4\left(\tan^{-1}\frac{D}{B}\right) \text{ for } \phi = 0 \tag{2B.30}$$

$$d_c = 1 + 0.2\sqrt{K_p}\frac{D}{B}$$

$$d_q = 1 + 2 \tan \phi(1 - \sin \phi)^2\left(\frac{D_f}{B}\right) \text{ for } \left(\frac{D_f}{B} \leq 1\right)$$

$$d_q = 1 + 2 \tan \phi(1 - \sin \phi)^2\left(\tan^{-1}\frac{D_f}{B}\right) \text{ for } \left(\frac{D_f}{B} > 1\right) \tag{2B.31}$$

$$d_q = 1 + 0.1\sqrt{K_p}\frac{D_f}{B} \text{ for } (\phi > 10°)$$

$$d_q = 1 \text{ for } (\phi = 0)$$

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$$d_\gamma = 1 \text{ Hansen or Vesic}$$

$$d_\gamma = 1 + 0.1\sqrt{K_p}\frac{D_f}{B} \text{ for } (\phi > 10°) \text{ Meyerhof} \tag{2B.32}$$

$$d_\gamma = 1 \text{ for } (\phi = 0) \text{ Meyerof}$$

Note that the factor $\tan^{-1}[D_f/B]$ is in radians.

2B.4.5.4 Inclination Factors

Inclined loading significantly reduces the capacity of a shallow foundation. This reduction is determined by the use of inclination factors.

$$i_\gamma = \left(1 - \frac{\beta}{\phi}\right)^2$$

$$i_\gamma = \left(1 - \frac{0.7H}{V + A_f c_a \cot\phi}\right)^5 \text{ for } (\eta = 0)$$

$$i_\gamma = \left(1 - \frac{(0.7 - \eta°/450)H}{V + A_f c_a \cot\phi}\right)^5 \text{ for } (\eta > 0) \tag{2B.33}$$

$$i_\gamma = \left(1 - \frac{H}{V + A_f c_a \cot\phi}\right)^{m+1}$$

η = tilt angle from horizontal with (+) upward

$$i_c = i_q = \left(1 - \frac{\beta}{90°}\right)^2$$

$$i_c = i_q - \frac{1 - i_q}{N_q - 1}$$

$$i_c = 0.5 - 0.5\sqrt{1 - \frac{H}{A_f c_a}} \text{ for } (\phi = 0)$$

$$i_c = 1 - \sqrt{\frac{H}{A_f c_a}} \text{ for } (\phi = 0)$$

$$A_f = B' x L' \tag{2B.34}$$

c_a = adhesion

H = horizontal component of footing load

V = total load on footing

$$m = m_B = \frac{2 + B/L}{1 + B/L} \text{ for } (H \text{ parallel to } B)$$

$$m = m_L = \frac{2 + L/B}{1 + L/B} \text{ for } (H \text{ parallel to } L)$$

where β = inclination of the load on the foundation with respect to the vertical (Figure 2B.7).

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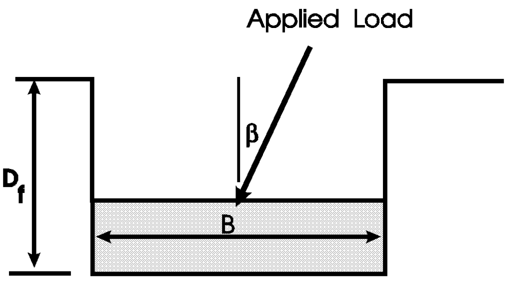

FIGURE 2B.7 Inclined loading.

2B.4.6 Eccentrically Loaded Foundations

Foundations are often subjected to moments in addition to the vertical load (Fig. 2B.8).

$$q_{\max} = \frac{Q}{BL} + \frac{6M}{B^2L}$$
$$q_{\min} = \frac{Q}{BL} - \frac{6M}{B^2L} \tag{2B.35}$$

where Q = total vertical load
M = moment on the foundation

$$e_x = \frac{M_x}{Q};\ e_y = \frac{M_y}{Q}$$

$$q_{\max} = \left(\frac{Q}{BL}\right)\left(1 + \frac{6e}{B}\right) \tag{2A.36}$$

$$q_{\min} = \left(\frac{Q}{BL}\right)\left(1 - \frac{6e}{B}\right)$$

2B.4.6.1 Meyerhof's Effective Area Method

When $e = B/6$, $q_{\min} = 0$. For $e > B/6$m $q_{\min}$ is negative, i.e., tension will develop. Since soil cannot take tension, there will be a separation between the foundation and the soil underlying it. Therefore

$$q_{\max} = \frac{4Q}{3L(B - 2e)} \tag{2B.37}$$

Determine the effective dimensions of the foundation as:

$$B' = \text{effective width} = B - 2e$$
$$L' = \text{effective length} = L.$$

If the eccentricity is in the direction of the length of the foundation, $L' = L - 2e$.

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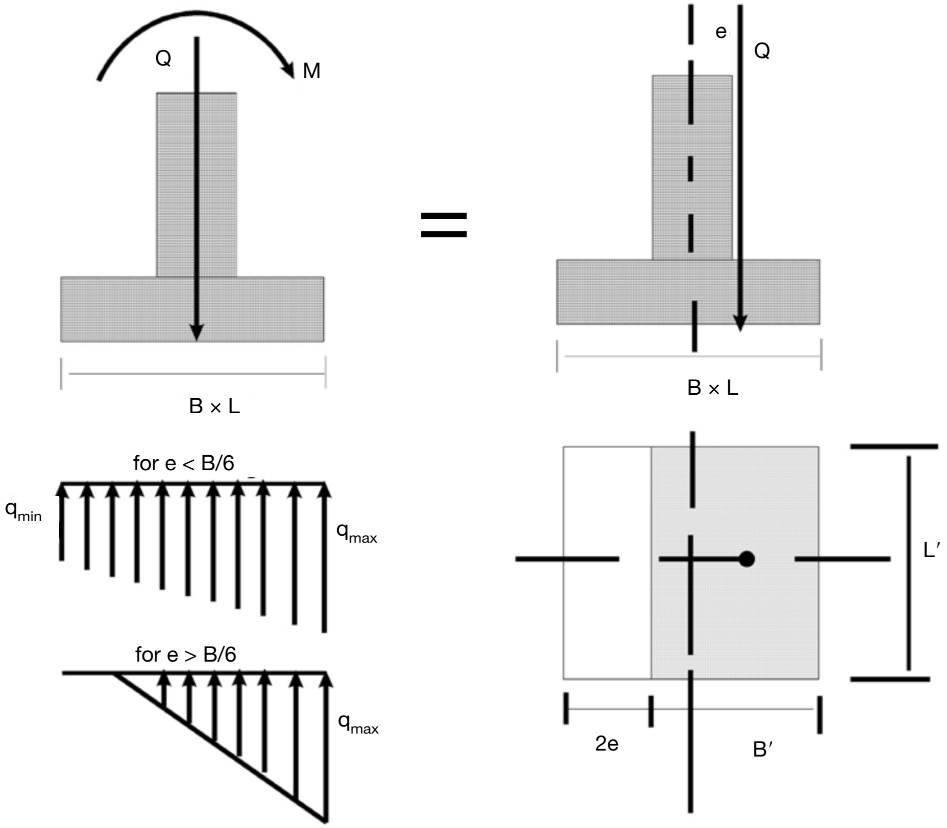

FIGURE 2B.8 Eccentrically loaded footing.

Use the general equation for bearing capacity except substitute B' for B. *Do not replace B with* B' for calculation of the depth factors. The total ultimate load that the foundation can sustain is:

$$Q_{\text{ult}} = q_u(B')(L') \tag{2A.38}$$

The factor of safety against bearing capacity failure is:

$$FS = \frac{Q_{\text{ult}}}{Q}$$

Bowles reduced the Meyerhof method such that

$$q_{\text{ult}} = q_{\text{ult}} R_e$$

$$R_e = 1 - 2\frac{e}{B} \text{ (cohesive soil)} \tag{2B.39}$$

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$$R_e = 1 - \left(\frac{e}{B}\right)^{1/2} \left(\text{cohesionless soil and } 0 < \frac{e}{B} < 0.3\right)$$

where q_{ult} = ultimate bearing capacity for concentric loading.

As you can see, eccentricity decreases the bearing capacity of a footing. It is advantageous to place the foundation columns off center, as shown below. This, in effect, produces a centrally loaded foundation with uniformly distributed pressure.

2B.5 FACTOR OF SAFETY

Because of the inherent uncertainty of the bearing capacity analysis, the usual practice is to reduce the applied stress from the foundation load by some arbitrary factor. This reduction is usually presented as a factor of safety. The reduced bearing capacity is known as the allowable bearing capacity, or allowable foundation stress q_{allow}:

$$q_{\text{allow}} = \frac{q_{\text{ult}}}{F.S.} \tag{2B.40}$$

where F.S. is the factor of safety. In general, the factor of safety runs from 2 to 3.

2B.6 BEARING CAPACITY OF MAT

The ultimate bearing capacity of mat foundations is calculated using equation (2B.41).

$$q_u = cN_c s_c d_c i_c + \gamma D_f N_q s_q d_q i_q + \frac{1}{2}\gamma B\, N_\gamma s_\gamma d_\gamma i_\gamma \tag{2B.41}$$

However, the great advantage of mat foundations is that by excavating below the ground surface for the placement of the mat, the net allowable applied load from the structure is increased. If Q is the total of the dead and live loads applied to the base of the mat, then

$$q_{\text{(net applied)}} = \frac{Q}{A} - \gamma D_f \tag{2B.42}$$

where A is the area of the mat.

It is possible to place the footing at such a depth D_{crit} such that the net applied load is zero, i.e., let

$$\begin{gathered} q_{\text{(net applied)}} = 0 = \frac{Q}{A} - \gamma D_{\text{(critical)}} \\ D_{\text{(critical)}} = \frac{Q}{A\gamma} \end{gathered} \tag{2B.43}$$

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For this condition, the factor of safety is:

$$F.S. = \frac{q_{\text{ult}}}{q_{(\text{net allowable})}} = \frac{q_{\text{ult}}}{0} = \infty \tag{2B.44}$$

2B.7 SETTLEMENT OF SHALLOW FOUNDATIONS

Although the analysis and design of foundations usually begins with the study of the bearing capacity of the foundation–soil system, in general, the settlement of the foundation controls the design.

2B.7.1 Allowable Settlements

The amount of settlement that a foundation can tolerate is called the allowable settlement. the magnitude of this settlement depends upon it mode.

2B.7.1.1 Uniform Settlement
A structure that has undergone uniform settlement is one where all points within the structure have moved vertically the same amount (Figure 2B.9a). This type of settlement does not result in struc-

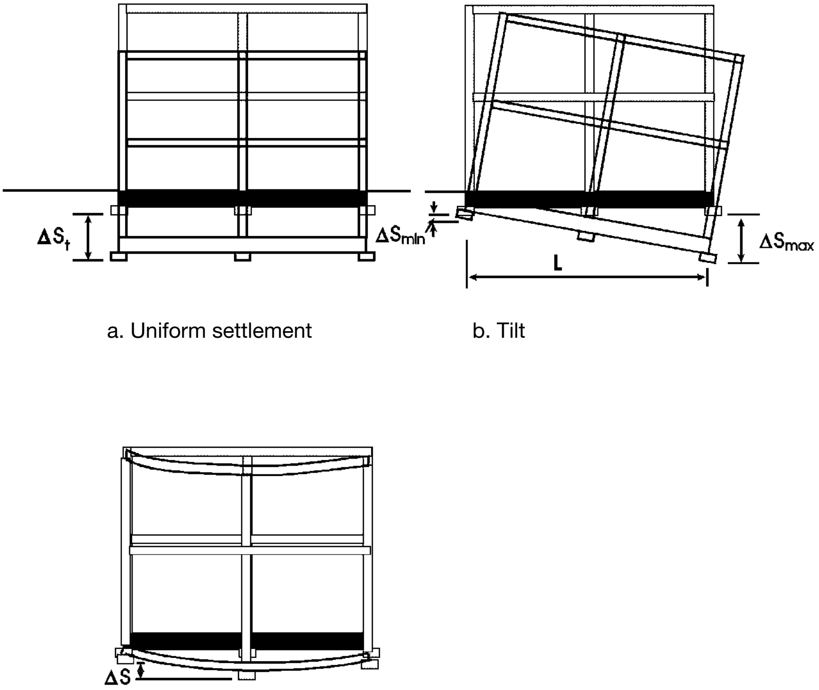

FIGURE 2B.9 Types of foundation settlement.

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tural damage if it is constant across whole structure. However, there will be problems with appurtenances such as with pipes, entrance-ways, etc.

2B.7.1.2 Tilt

Tilt is usually measured by its angular distortion (Figure 2B.9):

$$\text{angular distortion} = \frac{s_{max} - s_{min}}{L} \tag{2B.45}$$

The amount of tilt that a structure can tolerate is a function of many factors, including the size and type of construction. The Leaning Tower of Pisa is currently at about 10% tilt and is still standing. However, the Campenella in the Plaza San Marcos in Venice collapsed when it reach a 0.8% tilt. Tilt is visible at about 1/250 or 0.4%

2B.7.1.3 Differential (Distortion) Settlement

If s_{max} is the maximum total settlement anywhere in the structure then Δs_{max} is the maximum difference in total deformations between adjacent foundations. This is called differential settlement. Distortion is then defined as: $\Delta s_{max}/L$.

Field evidence indicates that architectural damage occurs when $\Delta s_{max}/L$ = 1/300 and structural damage occurs when $\Delta s_{max}/L$ = 1/150.

2B.7.1.4 Maximum Allowable Settlement

In general, foundations are limited to a specified amount of settlement. This settlement is called the design or maximum allowable settlement. For isolated foundations that support individual columns or small groups of columns on clay:

$$\frac{\Delta s_{max}}{L} = \frac{1}{1200} s_{max} \tag{2B.46}$$

Since $\Delta s_{max}/L$ = 1/300, s_{max} = 4 in (10 cm).

For isolated foundations that support individual columns or small groups of columns on sand:

$$\frac{\Delta s_{max}}{L} = \frac{1}{600} s_{max} \tag{2B.47}$$

Since $\Delta s_{max}/L$ = 1/300, s_{max} = 2 inches (5 cm). Therefore we must design for total settlements of isolated foundations less than 2–4 in (5–10 cm).

2B.7.1.5 Allowable Settlement of Mat (Raft) Foundations

There are few data available that document allowable settlement for raft foundations. Therefore only minor problems probably occur.

2B.7.2 Elastic (Immediate) Settlement

Immediate settlements occur as the load is applied to the soil. The soil particle matrix distorts and the soil voids are compressed. If the soil voids contain air, or if the permeability of the soil matrix is high, then the volume of the voids decreases, thereby contributing to the settlement. Since sands and gravels are highly conductive, almost all of the settlement of foundations on sands and gravels can be classified as immediate. Clays, on the other hand, have very low hydraulic conductivity; hence, if they are saturated, the immediate deformation is usually quite small and is limited to structural distortion of the soil fabric.

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2B.7.2.1 Key Variables in Elastic Settlements

The magnitude of settlement is inversely proportional to the strength of the soil. Factors affecting the soil strength are relative density, embedment of the foundation, and the effect of groundwater. Relative density is measured in the field using the standard penetration test, the cone penetrometer, or other devices.

The relationship between settlement and footing width was described by Terzaghi and Peck[6]. For the same load on the same soil, the settlement s is related to the square of the footing width B through the settlement of a 1 ft (0.3 m) square plate s_1 by:

$$s = s_1\left[\frac{2B}{B+1}\right]^2 \tag{2B.48}$$

The magnitude of settlement is also directly proportional to the magnitude of the applied load up to the allowable bearing pressure, with all else constant.

2B.7.2.2 Settlement Models

Many techniques are presented in the literature for predicting the settlement of shallow foundations on sand. Depending upon which method is used, this calculation can be a very simple one or it can be moderately complex, and the resulting predictions can differ greatly. A recent publication by the Corps of Engineers, Waterways Experiment Station[7] reported on fifteen methods. Most of the methods can be placed within one of two categories: some are modeled after the Terzaghi and Peck[6] (1948) bearing capacity and settlement–footing width relationship, and others are modeled after elasticity methods. A few methods combine some aspects of both. The backgrounds for both the Terzaghi–Peck-based settlement methods and elastic-based settlement methods are given below.

2B.7.2.2.1 Terzaghi–Peck-based Settlement. Terzaghi and Peck[6] developed the well-known design chart, Figure 2B.10, for estimating allowable bearing pressures for shallow foundations on sand using standard penetration blow count and footing width. These design curves correspond to a maximum footing settlement of 1 in (2.5 cm) and total differential settlement of ¾ in

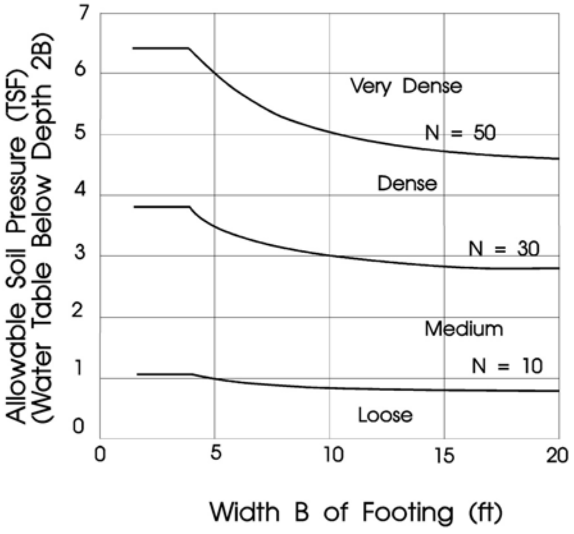

FIGURE 2B.10 Terzaghi and Peck design chart for q_{allow}.

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(1.9 cm). Data were interpreted conservatively in the development of this chart. History has proven that these values are very conservative. Modification to these values for less conservatism have been made by many. A general expression for this relations is

$$s = C\left[\frac{q}{N}\right]\left[\frac{B}{B+1}\right]^2 \tag{2B.49}$$

where s = settlement
q = net applied load
B = footing width
N = blowcount
C = empirical constant determined by observation and or experimentation

The Terzaghi–Peck chart gives $C = 8$ for footings less than 4 ft (1.2 m), and $C = 12$ for footings greater than 4 ft (1.2 m) in width.

2B.7.2.2.2 Elastic Soil Settlement. Soil is often treated as an elastic medium, linear or nonlinear, to which the elastic theory assumptions and principles of stress and strain are applied. Settlement computations of this form use the elastic properties of Poisson's ratio and Young's modulus to represent the soil. A general expression for the elastic settlement relation is:

$$s = \frac{qBI\mu}{E} \tag{2B.50}$$

where μ = Poissons ratio
E = elastic modulus
I = influence factor based on footing shape depth and extent of elastic region

One main difference between the Terzaghi–Peck model and the elastic model is the relationship between footing width and settlement. The elastic theory models a linear relation between settlement and footing width, while Terzaghi and Peck's work shows this to be a nonlinear relation. Elastic theory settlement methods can account for this nonlinear relationship through an appropriate use of the elastic or compressibility modulus.

2B.7.2.2.3 Standard Penetration Test. The relative density of the soil is the major factor that controls the settlement of foundations on cohesionless soil. The standard penetration test (SPT) is the most widely used test in the United States for indirectly determining the relative density of a sand. A description of the test can be found in Bowles[1]. With all other factors being the same, the larger the blow count (N), the less the settlement. However, a number of factors significantly effect the blow count. The overburden pressure dramatically impacts the SPT value. In a homogeneous deposit where relative density and friction angle are constant with depth, blow counts increase with depth due to the increasing confining pressures with depth. Therefore, each measured SPT value should be corrected for the influence of its corresponding overburden pressure. There are many techniques available to correct the SPT value for overburden pressure. In general, all of the techniques have the form of

$$N_c = C_N N \tag{2B.51}$$

where N_c = corrected SPT value
C_N = correction factor based on overburden pressure

Blow count correction factors for overburden developed by various authors are given in Table 2B.1. Each of the equations normalizes N to a standard reference overburden pressure. Typically, this is 1 tsf (96 kN/m2). However, Peck and Bazara[12] normalize to 0.75 tsf (72 kN/m^2) and Teng[10]

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TABLE 2B.1 Overburden Correction Factors

Reference	Equation for C_N	Units for p_o'
Skempton[8]	fine to medium sand: $2/(1 + p_o')$ coarse, dense sand: $3/(2 + p_o')$ overconsolidated: $1.7/(0.7 + p_o')$	p_o' = effective overburden pressure
		tsf
Peck, Hanson & Thornburn[2]	fine sand: $0.77 \log(20/p_o')$	tsf
Bazaraa[9]	$p_o' < 1.5$ ksf: $4/(1 + 2p_o')$ $p_o' > 1.5$ ksf: $4/(3.25 + 0.5p_o')$	ksf
Teng[10]	$50/(p_o' + 10)$	psi
Liao and Whitman[11]	$(1/p_o')^{0.5}$	tsf

uses 40 psi (276 kN/m^2). Some procedures for computing settlement do no advocate correcting the blow count for overburden but use the blow count values as obtained in the field. Most experiments and theories show that this correction is necessary.

Variations in the borehole diameter, rod length, and hammer type can affect the measured blow counts for identical sands at the same overburden and relative density values. The blow count is directly related to the driving energy of test equipment:

$$E_\epsilon = \frac{1}{2} m v^2 = \frac{1}{2}\frac{W}{g} v^2$$

$$v = (2gh)^{1/2} \tag{2B.52}$$

$$E_\epsilon = \frac{1}{2}\frac{W}{g}(2gh) = Wh$$

where W = weight or mass of hammer
h = height of fall
v = velocity of hammer

The energy ratio E_r is defined as:

$$E_r = \frac{\text{actual hammer energy to sampler, } E_a}{\text{input energy, } E_{in}} \times 100 \tag{2B.53}$$

Bowles suggests that the energy should be adjusted to a standard energy ratio of 70 (E_{70}) and that the equation for the standard corrected blow count be given as

$$N_{70} = C_N \times N \times \eta_1 \times \eta_2 \times \eta_3 \times \eta_4 \tag{2B.54}$$

where the η factors can be found in Table 2B.2. Each of the factors correct the field blow counts for differences in hammers, rod length, sampler differences, and borehole diameter differences.

A footing placed below the ground surface will settle less than a footing at the surface. The

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TABLE 2B.2 Blow Count Adjustment Factors, η_i

Hammer adjustment factor, η_1

Average energy ratio E_r

Donut		Safety	
R–P	Trip	R-P	Trip/Auto
5	—	70–80	80–100

R–P = Rope-pulley or cathead: $\eta_1 = E_r/E_{rb}$.
For U.S. trip/auto $w/E_r = 80$: $\eta_1 = 80/70 = 1.14$.

Rod Length correction factor, η_2

Length > 10 m	$\eta_2 = 1.00$
6–10 m	$= 0.80$
4–6 m	$= 0.85$
0–4 m	$= 0.75$

Sampler correction factor, η_3

Without liner	$\eta_3 = 1.00$
With liner:	
Dense sand, clay	$= 0.80$
Loose sand	$= 0.90$

Borehole diameter correction factor, η_4

Hole diameter:	
60–120 mm	$\eta_4 = 1.00$
150 mm	$= 1.05$
200 mm	$= 1.15$

depth correction factor reduces the calculated settlement to account for the increase in bearing capacity achieved by embedment. The embedment correction equations by the various authors are given in Table 2B.3.

2B.7.2.3 Settlement Computing Methods

Several of the more prominent methods of computing elastic settlements are presented in the following paragraphs. Other methods can be extracted from published literature.

TABLE 2B.3 Embedment Correction Factors

Reference	Equation for Embedment Correction Factor, C_D
Terzaghi and Peck[13] (1967)	$C_D = 1 - 0.25(D_f/B)$
Schultz and Sherif[14] (1973)	$C_D = 1/[1 + 0.4(D_f/B)]$
D'Appoplonia, et al.[14]	$C_D = 0.729 - 0.484 \log (D_f/B) - 0.224[\log(D_f/B)]^2$
Bowles[1] (1977)	$C_D = 1/[1 + 0.33(D_f/B)]$
Teng[10] (1962)	$C_D = 1/[1 + (D_f/B)]$
Bazaraa[9] (1969)	$C_D = 1 - 0.4[\gamma D_f/q]^{0.5}$
Schmertmann[16]	$C_D = 1 - 0.5[\gamma D_f/(q - D_f]$

Terms: D_f = foundation depth, B = foundation width, q = loading pressure.

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2B.7.2.3.1 Terzaghi and Peck[6,13]. This method is based on the bearing capacity charts given in Figure 2B.10. The equations shown below are given by Meyerhof (1956)[3]. The chart is used to determine the allowable bearing capacity for a range of footing widths and SPT blow count values with maximum settlement not to exceed 1 in (2.5 cm) and differential settlement not to exceed 3/4 in (1.9 cm). Their settlement expression is:

$$s = \frac{8q}{N}(C_W C_D) \qquad \text{for } B \leq 4 \text{ ft (1.2m)}$$

$$s = \frac{12q}{N}\left[\frac{B}{B+1}\right]^2 (C_W C_D) \qquad \text{for } B \geq 4 \text{ ft (1.2m)} \tag{2B.55}$$

$$s = \frac{12q}{N}(C_W C_D) \qquad \text{for rafts}$$

Their correction factors for water are:

$$C_W = 2 - \left[\frac{D_W}{2B}\right] \leq 2.0 \text{ (for surface footings)}$$

$$C_W = 2 - 0.5\left[\frac{D_W}{2B}\right] \leq 2.0 \text{ (for sumerged, embedded footing; } D_W \leq D_f) \tag{2B.56}$$

And for depth:

$$C_D = 1 - 0.25\left[\frac{D_f}{B}\right]$$

For blow count use the measured SPT blow count value. If the sand is saturated, dense, and very fine or silty, correct the blow count by:

$$N_c = 15 + 0.5(N - 15) \qquad \text{for } N > 15 \tag{2B.57}$$

2B.7.2.3.2 Teng[10]. Teng's method for computing settlement is an interpretation of the Terzaghi and Peck bearing capacity chart. Teng includes corrections for depth of embedment, the presence of water, and the blow count. The settlement expression is:

$$\Delta s = \frac{q_o}{720(N_c - 3)}\left[\frac{mH}{B+1}\right]^2 \frac{1}{(C_W)(C_D)} \tag{2B.58}$$

where: q_o = net pressure in psf

The correction factor for water is

$$C_W = 0.5 + 0.5\left[\frac{D_W - D_f}{B}\right] \geq 0.5 \qquad \text{for water at and below } D_f \tag{2B.59}$$

For depth:

$$C_D = 1 + \left[\frac{D_f}{B}\right] \leq 2.0$$

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For blow count:

$$N_c = N\left[\frac{50}{p_o' + 10}\right]$$

where p_o' = effective overburden at median blowcount depth abut $D_f + B/2$, in psi ($\leq$ 40 psi, 276kPa)

2B.7.2.3.3 Peck, Hanson, and Thornburn[2]. This method is based on Terzaghi and Peck settlement method.

$$\Delta s = \frac{q}{0.11 N_c C_w} \quad \text{for intermediate width footings } (> 2 \text{ ft, } 0.6\text{m})$$

$$\Delta s = \frac{q}{0.22 N_c C_w} \quad \text{for rafts} \tag{2B.60}$$

where q is in tsf.

The correction factor for water is

$$C_W = 0.5 + 0.5\left[\frac{D_W}{D_f + B}\right] \quad \text{for water from 0 to } D_f + B \tag{2B.61}$$

For blow count:

$$N_c = NC_n$$

$$C_n = 0.77\log\left[\frac{20}{p'}\right] \tag{2B.62}$$

where p' = effective overburden pressure for the measured blow count at $D_f + (B/2)$ in tsf (0.25 tsf = 24 kPa).

2B.7.2.3.4 Bowles[17,18]. Bowles' settlement method is based on the Terzaghi and Peck method, but is modified to produce results that are not as conservative. His equations are:

$$\Delta s = \frac{2.5 q_o}{N}\left[\frac{C_W}{C_D}\right] \quad \text{for } B \leq 4 \text{ ft}$$

$$\Delta s = \frac{4 q_o}{N}\left[\frac{B}{B+1}\right]^2\left[\frac{C_W}{C_D}\right] \quad \text{for } B \geq 4 \text{ ft} \tag{2B.63}$$

$$\Delta s = \frac{4 q_o}{N}\left[\frac{C_W}{C_D}\right] \quad \text{for mats}$$

where q is in kips/sf, N is measured in the field, and the settlement is in inches.

The correction factor for water is:

$$C_W = 2 - \left[\frac{D_W}{D_f + B}\right] \leq 2.0 \text{ and } \geq 1.0 \tag{2B.64}$$

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The correction factor for depth is:

$$C_D = 1 + 0.33\left[\frac{D_f}{B}\right] \leq 1.33 \tag{2B.65}$$

Therefore, the settlement can be computed from:

$$\Delta s = \frac{2.5q_o}{N}\left[\frac{C_W}{C_D}\right] \qquad \text{for } B \leq 4 \text{ ft}$$

$$\Delta s = \frac{4q_o}{N}\left[\frac{B}{B+1}\right]^2\left[\frac{C_W}{C_D}\right] \qquad \text{for } B \geq 4 \text{ ft} \tag{2B.66}$$

$$\Delta s = \frac{4q_o}{N}\left[\frac{C_W}{C_D}\right] \qquad \text{for mats}$$

2B.7.2.3.5 Elastic Theory. Settlement computed by elastic theory uses elastic parameters to model a homogeneous, linearly elastic medium. The elastic modulus of a soil depends upon confinement and is assumed in elastic theory to be constant with depth. For uniform saturated cohesive soils, this assumption is usually valid. For cohesionless soils, elastic methods can be inappropriate because the modulus often increases with depth. However, the immediate settlement of sand is often considered to be elastic within a small strain range.

The equations are based on the theory of elasticity and are for settlement at the surface of a semi-infinite, homogeneous half-space. The equations are

$$\Delta s = \frac{q_o B'}{E_s}(1-\mu^2)\left[I_1 + \frac{(1+2\mu)}{(1-\mu)}I_2\right]I_F \tag{2B.67}$$

$$I_1 = \frac{1}{\pi}\left[M \ln\frac{(1+\sqrt{M^2+1})\sqrt{M^2+N^2}}{M(1+\sqrt{M^2+N^2+1})} + \ln\frac{(M+\sqrt{M^2+1})\sqrt{1+N^2}}{M+\sqrt{M^2+N^2+1}}\right] \tag{2B.68}$$

$$I_2 = \frac{N}{2\pi}\tan^{-1}\left(\frac{M}{N\sqrt{M^2+N^2+1}}\right)(\tan^{-1} \text{ in rad}) \tag{2B.69}$$

where $M = (L'/B')$; $H = (H/B')$

For the center influence factor,

$$B' = \frac{B}{2}; \quad L' = \frac{L}{2}$$

For the corner influence factor, $B' = B5$; $L' = L$.

The correction factors are from Das[19] (Figure 2B.11). The influence factor I_s is defined as

$$I_s = I_1 + \frac{1-2\mu}{1-\mu}I_2 \tag{2B.70}$$

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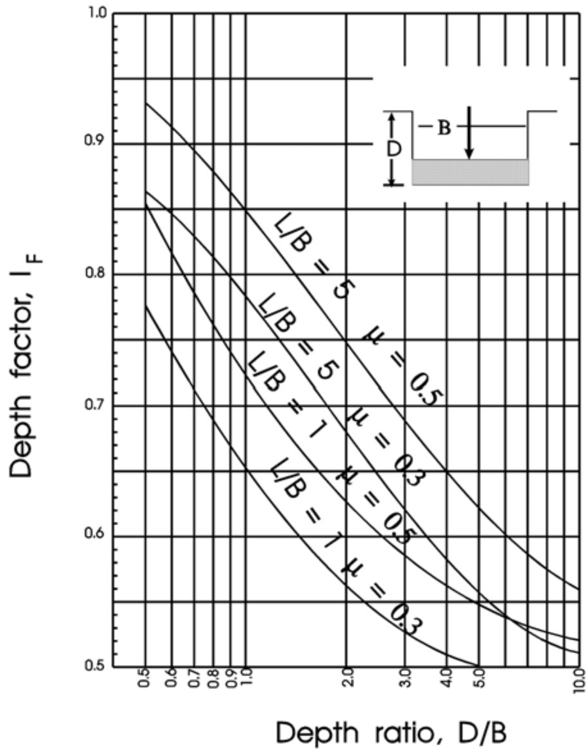

FIGURE 2B.11 Influence factors for footing at depth *D*. Use actual footing width and depth dimensions (Das[19]).

Therefore, Equation (2B.67) can be written as:

$$\Delta s = q_o B' \frac{1 - \mu^2}{E_s} I_s I_F \tag{2B.71}$$

For rigid footings, the value of I_s should be reduced by 7%, i.e., $I_{sr} = 0.93\ I_s$. Poisson's ratio, μ, can be determined from Table 2B.4.

Bowles suggests the following procedure:

1. Make the best estimate of q_o.
2. Convert round footings to an equivalent square.
3. Determine the point where the settlement is to be computed and divide the base so the point is at the corner or common corner of the contributing rectangles.
4. Note that the stratum depth actually causing settlement is not at H/B but is either:
   a. Depth $z = 5B$ (B = least total lateral dimension of base), or
   b. Depth to where a hard stratum is encountered. Take Hard as that where E_s in the hard layer is about $10E_s$ of adjacent layer. Table 2B.5 can be used for approximate values. Table 2B.6 gives equations for E_s as functions of cone or standard penetration test values.

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TABLE 2B.4 Range of values for Poisson's Ratio*

Soil	Poisson's Ratio
Loose Sand	0.2–0.4
Medium Sand	0.25–0.4
Dense Sand	0.3–0.45
Silty Sand	0.2–0.4
Soft Clay	0.15–0.25
Medium Clay	0.2–0.5

*After Das.[19]

TABLE 2B.5 Range of Elastic Modulus, E_s*

Soil	E_s psi(kPa)
Soft Clay	250–500(1725–3450)
Hard Clay	850–2,000(5860–13,800)
Loose Sand	1,500–4,000(10,350–27,600)
Dense Sand	5,000–10,000(34,400–69,000)

*After Das.[19]

TABLE 2B.6 Equations for E_s from SPT and CPT

SOIL	SPT (kPa)	CPT (units of q_c)
Sand	$E_s = 500(N + 15)$	$E_s = (2 \text{ to } 4)q_c$
	$E_s = 18{,}000 + 750N$	$E_s = 2(1 + D_r^2)q_c$
	$E_s = (15{,}000 \text{ to } 22{,}000)\ln N$	
Clayey sand	$E_s = 320(N + 15)$	$E_s = (3 \text{ to } 6)q_c$
Silty sand	$E_s = 300(N + 6)$	$E_s = (1 \text{ to } 2)q_c$
Gravelly sand	$E_s = 1{,}200(N + 6)$	
Soft clay		$E_s = (6 \text{ to } 8)q_c$

5. Compute H/B' ratio. For a depth $H = z = 5B$ and for the center of the base we have $H/B' = 5B/0.5B = 10$. For a corner $5B/B = 5$.
6. Obtain I_1 and I_2 with the best estimate for μ and compute I_s.
7. Determine I_F from Figure 2B.11.
8. Obtain the weighted average E_s in the depth $a = H$ using

$$E_{s(av)} = \frac{H_1E_{s1} + H_2E_{s2} + \cdots + H_nE_{sn}}{H} \tag{2B.72}$$

2B.7.2.3.6 Das[19] Elastic Settlement of Foundations on Saturated Clay (μ = 0.5). Das computes the settlement of a foundation on saturated clay using:

$$S_e = A_1A_2\frac{q_oB}{E_s} \tag{2B-73}$$

The coefficients A_1 and A_2 are found in Figure 2B.12 and Figure 2B.13, respectively.

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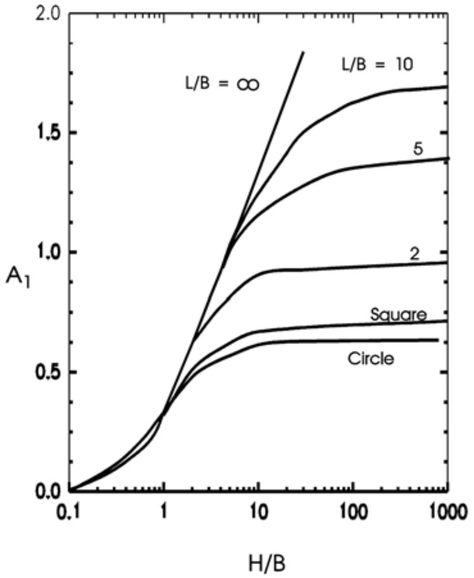

FIGURE 2B.12 Values of A_1.

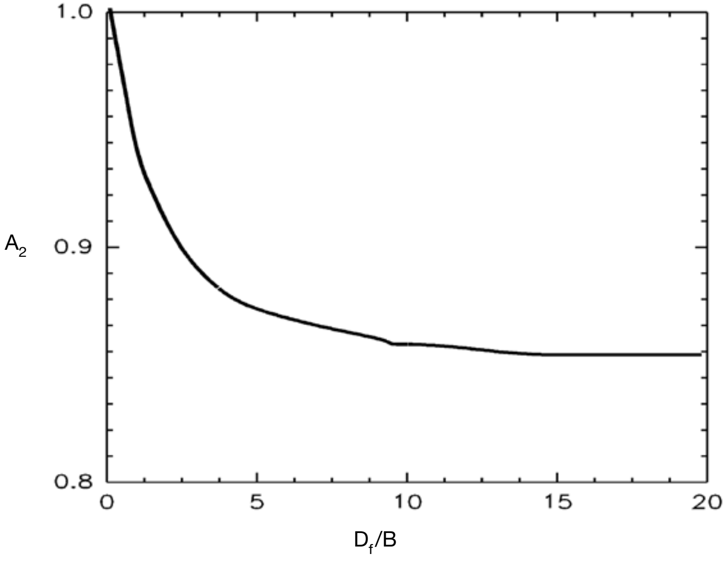

FIGURE 2B.13 Values of A_2.

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2B.7.2.3.7 Schmertmann[16]. Schmertmann proposes calculating total settlement by subdividing the compressible stratum and summing the settlements of each sublayer. The sublayer boundaries are defined by changes in the SPT or cone penetrometer (CPT) profile. His equation is

$$\Delta s - C_1 C_2 \Delta q \Sigma \frac{I_z \Delta z}{E_s} \tag{2B.74}$$

The correction factor for embedment is

$$C_1 = 1 - 0.5 \frac{\overline{q}}{q_o - \overline{q}} \tag{2B.75}$$

where $\overline{q}$ = surchage = γD_f.

$$C_2 = 1 + 0.2 \log \frac{t}{0.1} \tag{2B.76}$$

where t = time (> 0.1 years).

The variation of the strain influence factor is given in Figure 2B.14. Note that for square or circular foundations,

$$\begin{gathered} I_z = 0.1 \text{ at } z = 0 \\ I_z = 0.5 \text{ at } z = z_1 = 0.5B \\ I_z = 0 \text{ at } z = z_2 = 2B \end{gathered} \tag{2B.77}$$

Similarly, for foundations with $L/B > 10$,

$$\begin{gathered} I_z = 0.2 \text{ at } z = 0 \\ I_z = 0.5 \text{ at } z = z_1 = B \\ I_z = 0 \text{ at } z = z_2 = 4B \end{gathered} \tag{2B.78}$$

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

This profile is used to determine the elastic modulus as it changes with depth.

If E_s is constant over $2B$ below the footing base, the simplified expression is

$$\Delta s = C_1 C_2 q_o \frac{0.6B}{E_s} \tag{2B.79}$$

2B.7.2.4 Proportioning Footings for Equal Settlement

For clay soils, the usual method is to use the following equation:

$$\frac{\Delta s_1}{\Delta s_2} = \frac{B_{2'}}{B_{1'}} \tag{2B.80}$$

for constant contact pressure. This has proven to work reasonably well.

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SHALLOW FOUNDATIONS

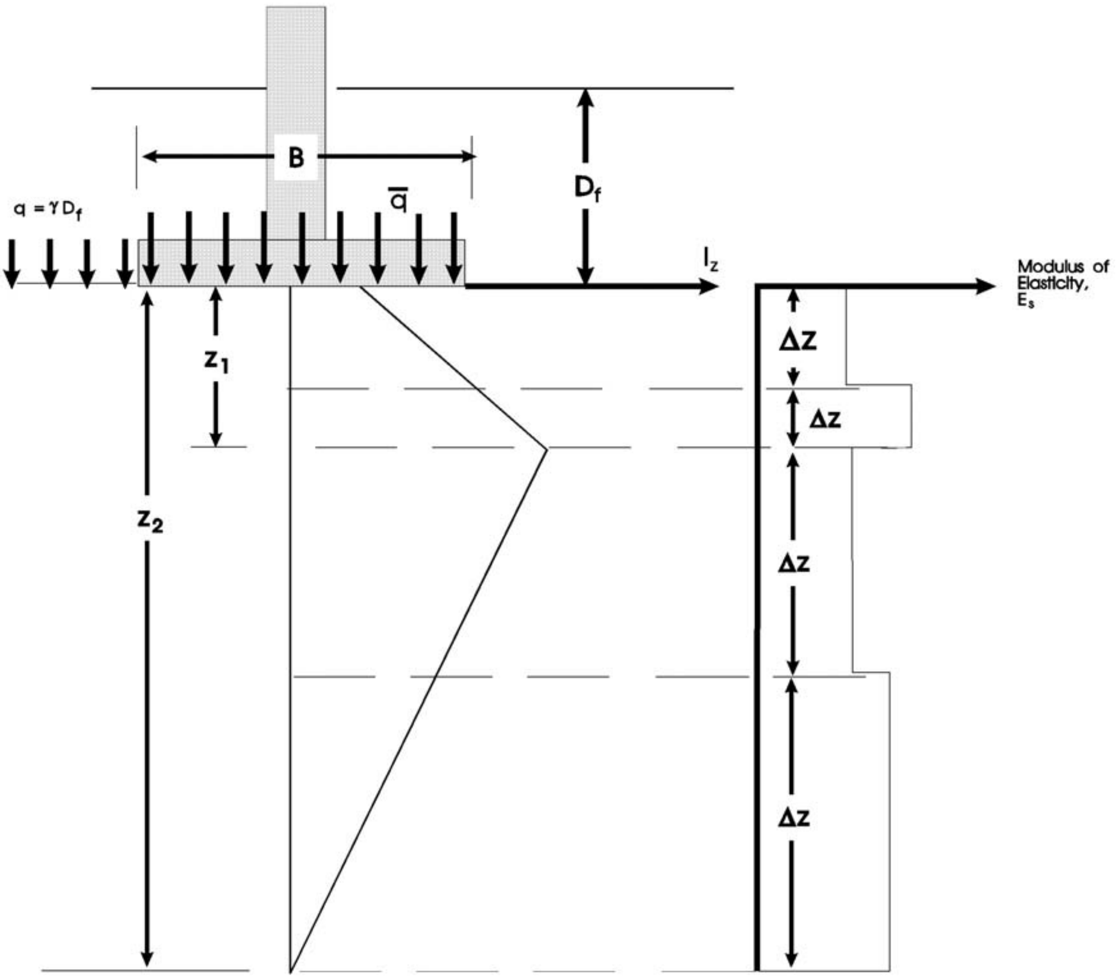

FIGURE 2B.14 Elastic settlement by the strain influence factor.

For sand soils, Bowles recommends

$$\frac{\Delta s_1}{\Delta s_2} = \frac{B_{2'}}{B_{1'}} \frac{I_{s2}}{I_{s1}} \frac{I_{f2}}{I_{f1}} \frac{E_{s'2}}{E_{s'1}} \tag{2B.81}$$

for constant contact pressure.

2B.7.3 Consolidation Settlement of Foundations on Clays

The settlement of a shallow foundation located on cohesive soils is governed by consolidation theory (Section 2A.8).

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In order to compute the consolidation settlement of a foundation on clay, the following steps must be done.

1. Determine if the soil is normally consolidated or overconsolidated.
2. Determine the thickness, H, and existing void ratio, e_0, of the consolidating soil layer.
3. Compute the average existing effective stress acting on the consolidating soil layer, p_0.
4. Compute the average increase in stress Δp on the consolidating layer due to the addition of the foundation load.
5. Compute the consolidation settlement using:

For normally consolidated soils:

$$\Delta s = \frac{H}{1 + e_0} C_c \log\left[\frac{\overline{p}_0 + \Delta p}{\overline{p}_0}\right] \tag{2B.82}$$

For overconsolidated soils:

$$\Delta s = \frac{H}{1 + e_0} C_r \log\left[\frac{\overline{p}_0 + \Delta p}{\overline{p}_0}\right] \tag{2B.83}$$

2B.8 ELASTIC SETTLEMENT OF ECCENTRICALLY LOADED FOUNDATIONS

Whitman and Richart[20] developed a procedure to estimate the settlement of a shallow foundation under eccentric loading. If Q (applied total load) and the eccentricity e are known, then determine the ultimate load Q_{ulte} that the foundation can sustain using the methods for eccentrically loaded foundations previously presented. Determine the factor of safety for the eccentrically loaded foundation as

$$FS = Q_{ulte}/Q = F_1 \tag{2B.84}$$

Next, determine the ultimate load $Q_{ulte=0}$ for the same foundation with $e = 0$:

$$Q_{ulte=0}/F_1 = Q_{e=0} \tag{2B.85}$$

$Q_{e=0}$ is the allowable load for the foundation with a factor of safety $FS = F_1$ for a central loading condition.

For the load $Q_{e=0}$ on the foundation, estimate the settlement by using the techniques presented previously. This settlement is equal to $S_{e=0}$. Calculate S_{e1}, S_{e2}, and t using:

$$S_{e1} = S_{e=0}\left[1 - 2\left(\frac{e}{B}\right)\right]^2 \tag{2B.86}$$

Next, solve for t as

$$t = \tan^{-1}\left[CS_e\left(\frac{e/B}{\sqrt{BL}}\right)\right] \tag{2B.87}$$

where $C = \beta_1\beta_2$

$\beta_1\beta_2$ = factors that depend on the L/B ratio (Fig.2B.15)

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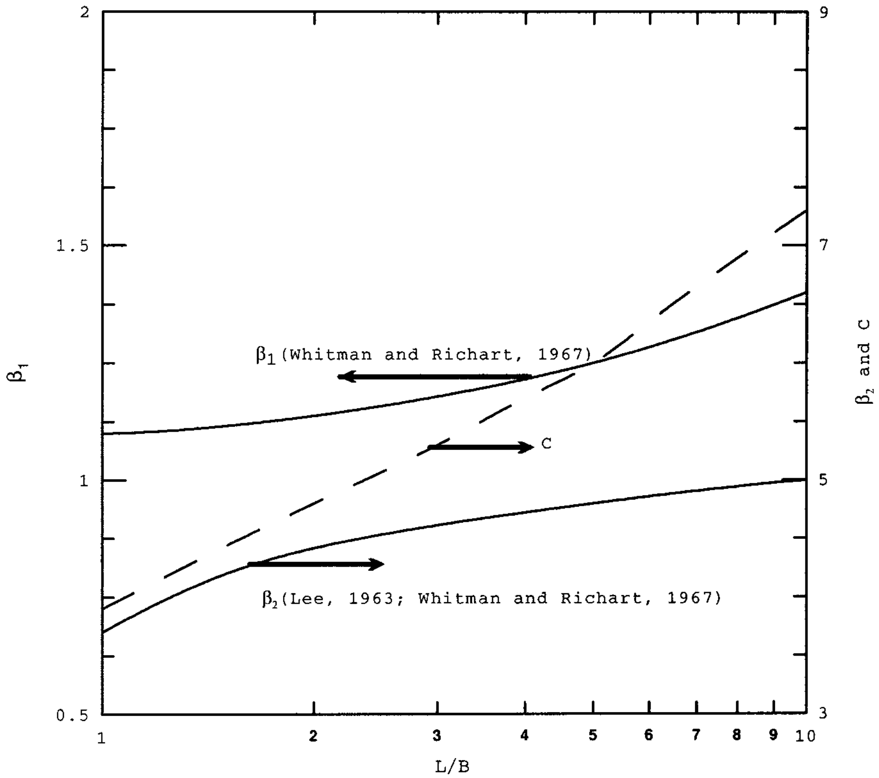

FIGURE 2B.15 Settlement factors for eccentrically loaded foundations.

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