9

Shallow Foundations

9–1 INTRODUCTION

The word *foundation* might be defined in general as "that which supports something." Many universities, for example, have an "athletic foundation," which supports in part the school's sports program. In the context of this book, *foundation* normally refers to something that supports a structure, such as a column or wall, along with the loads carried by the structure.

Foundations may be characterized as shallow or deep. *Shallow foundations* are located just below the lowest part of the superstructures they support; *deep foundations* extend considerably down into the earth. In the case of shallow foundations, the means of support is usually either a *footing*, which is often simply an enlargement of the base of the column or wall that it supports, or a *mat* or *raft foundation*, in which a number of columns are supported by a single slab. This chapter deals with shallow foundations—primarily footings. For deep foundations, the means of support is usually either a pier, drilled shaft, or group of piles. These are covered in Chapters 10 and 11.

An individual footing is shown in Figure 9–1a . For purposes of analysis, a footing such as this may be thought of as a simple flat plate or slab, usually square in plan, acted on by a concentrated load (the column) and a distributed load (soil pressure) (Figure 9–1b). The enlarged size of the footing (compared with the column it supports) gives an increased contact area between the footing and the soil; the increased area serves to reduce pressure on the soil to an allowable amount, thereby preventing excessive settlement or bearing failure of the foundation.

Footings may be classified in several ways. For example, the footing depicted in Figure 9–1a is an *individual footing*. Sometimes one large footing may support two or more columns, as shown in Figure 9–2a. This is known as a *combined footing*. A footing

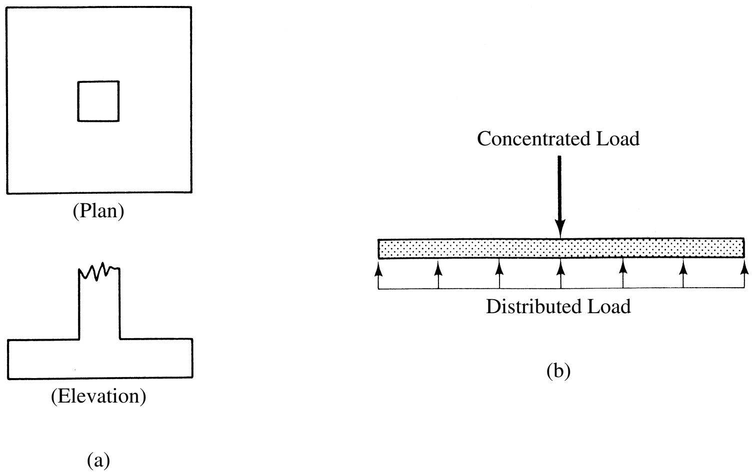


FIGURE 9–1 Individual footing.

extended in one direction to support a long structure such as a wall is called a *continuous footing*, or *wall footing* (Figure 9–2b). Two or more footings joined by a beam (called a *strap*) are called a *strap footing* (Figure 9–2c). A large slab supporting a number of columns not all of which are in a straight line is called a mat or raft foundation (Figure 9–2d).

Foundations must be designed to satisfy three general criteria:

1. They must be located properly (both vertical and horizontal orientation) so as not to be adversely affected by outside influences.
2. They must be safe from bearing capacity failure (collapse).
3. They must be safe from excessive settlement.

Specific procedures for designing footings are given in the remainder of this chapter. For initial orientation and future quick reference, the following steps are offered at this point:

1. Calculate the loads acting on the footing—Section 9–2.
2. Obtain soil profiles along with pertinent field and laboratory measurements and testing results—Chapter 3.
3. Determine the depth and location of the footing—Section 9–3.
4. Evaluate the bearing capacity of the supporting soil—Section 9–4.
5. Determine the size of the footing—Section 9–5.
6. Compute the footing's contact pressure and check its stability against sliding and overturning—Section 9–6.
7. Estimate the total and differential settlements—Chapter 7 and Section 9–7.
8. Design the footing structure—Section 9–8.

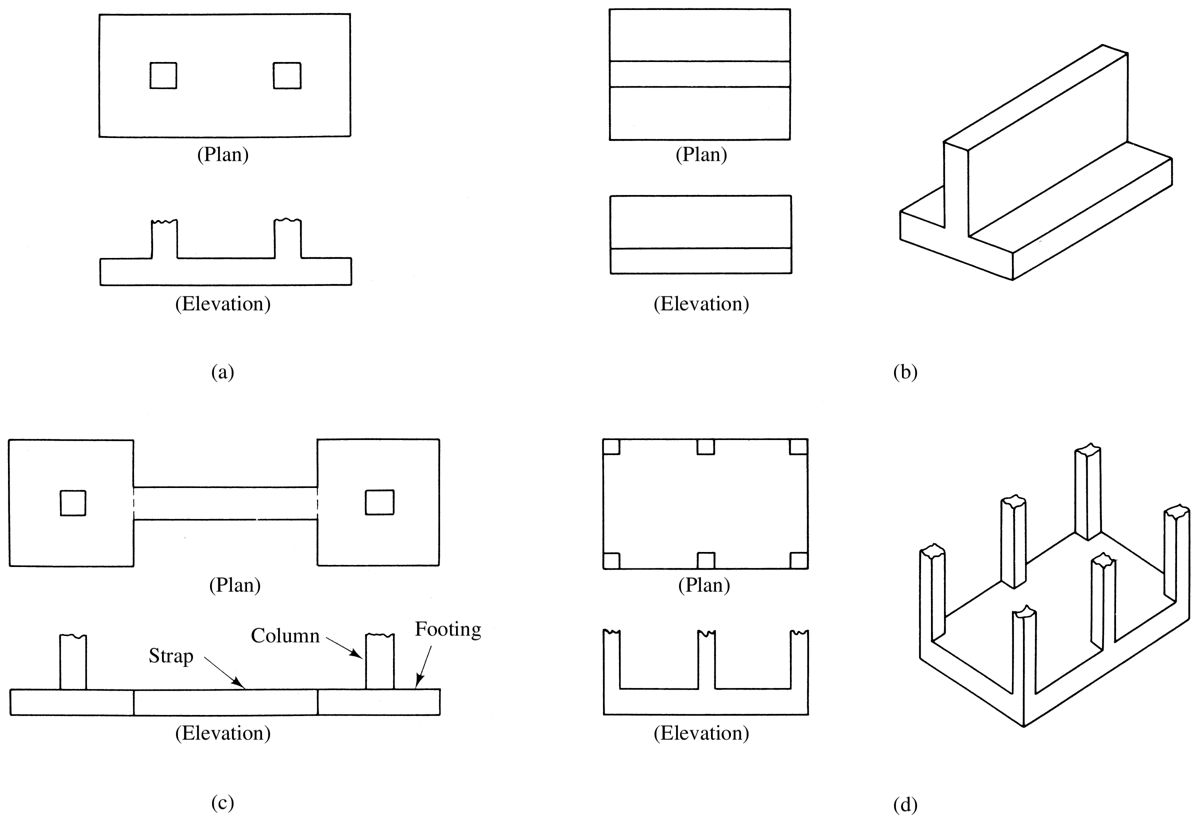


FIGURE 9–2 Classification of footings: (a) combined footing; (b) wall footing; (c) strap footing; (d) mat or raft foundation.

9–2 LOADS ON FOUNDATIONS

When one is designing any structure, whether it is a steel beam or column, a floor slab, a foundation, or whatever, it is of basic and utmost importance that an accurate estimation (computation) of all loads acting on the structure be made. In general, a structure may be subjected upon construction or sometime in the future to some or all of the following loads, forces, and pressures: (1) dead load, (2) live load, (3) wind load, (4) snow load, (5) earth pressure, (6) water pressure, and (7) earthquake forces. These are discussed in this section.

Dead Load

Dead load refers to the overall weight of a structure itself. It includes the weight of materials permanently attached to the structure (such as flooring) and fixed service equipment (such as air-conditioning equipment). Dead load can be calculated if sizes and types of structural material are known. This presents a problem, however, because a structure's weight is not known until its size is known, and its size cannot be known until it has been designed based (in part) on its weight. Normal procedure is to estimate dead load initially, use the estimated dead load (along with the live load, wind load, etc.) to size the structure, and then compare the sized structure's weight with the estimated dead load. If the sized structure's weight differs appreciably from the estimated dead load, the design procedure should be repeated, using a revised estimated weight.

Live Load

Live load refers to weights of applied bodies that are not permanent parts of a structure. These may be applied to the structure during part of its useful life (such as people, warehouse goods) or during its entire useful life (e.g., furniture). Because of the nature of live load, it is virtually impossible in most cases to calculate live load directly. Instead, live loads to be used in structural design are usually specified by local building codes. For example, a state building code might specify a minimum live loading of 100 lb/ft^2 for restaurants and 80 lb/ft^2 for office buildings.

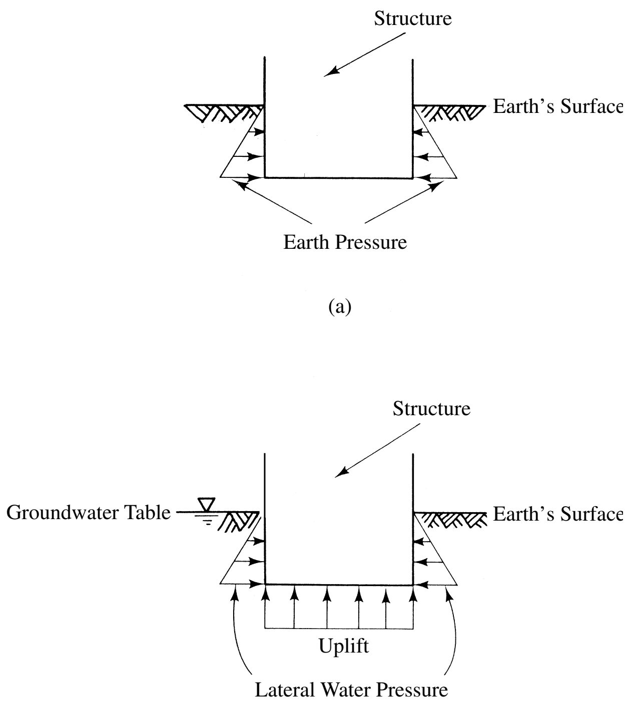
Wind Load

Wind load, which is not considered as live load, may act on all exposed surfaces of structures. In addition, overhanging parts of buildings may be subject to uplift pressure as a result of wind. Like design live loads, design wind loads are usually calculated based on building codes. For example, a building code might specify a design wind loading for a particular locality of 15 lb/ft^2 for buildings less than 30 ft tall and 40 lb/ft^2 for buildings taller than 1200 ft, with a sliding scale in between.

Snow Load

Snow load results from accumulation of snow on roofs and exterior flat surfaces. The unit weight of snow varies, but it averages about 6 lb/ft^3. Thus, an accumulation of several feet of snow over a large roof area results in a very heavy load. (Two feet of snow over a 50-ft by 50-ft roof would be about 15 tons.) Design snow loads are also usually based on building codes. A building code might specify a minimum snow loading of 30 lb/ft^2 for a specific locality.

FIGURE 9–3 (a) Earth pressure; (b) water pressure.



(b)

Earth Pressure

Earth pressure produces a lateral force that acts against the portion of substructure lying below ground or fill level (see Figure 9–3a). It is normally treated as dead load.

Water Pressure

Water pressure may produce a lateral force similar in nature to that produced by earth pressure. Water pressure may also produce a force that acts upward (hydrostatic uplift) on the bottom of a structure. These forces are illustrated in Figure 9–3b. Lateral water pressure is generally balanced, but hydrostatic uplift is not. It must be counteracted by the structure's dead load, or else some provision must be made to anchor the structure.

Earthquake Forces

Earthquake forces may act laterally, vertically, or torsionally on a structure in any direction. A building code should be consulted for the specification of earthquake forces to be used in design.

9–3 DEPTH AND LOCATION OF FOUNDATIONS

As related previously (Section 9–1), foundations must be located properly (both vertical and horizontal orientation) so as not to be adversely affected by outside influences. Outside influences include adjacent structures; water, including frost and groundwater; significant soil volume change; and underground defects (caves, for example). Additionally, foundation locations are dependent on applicable local building codes. Thus, the depth and location of foundations are dependent on the following factors:

1. Frost action.
2. Significant soil volume change.
3. Adjacent structures and property lines.
4. Groundwater.
5. Underground defects.
6. Building codes.

Frost Action

In areas where air temperature falls below the freezing point, moisture in the soil near the ground surface will freeze. When the temperature subsequently rises above the freezing point, any frozen moisture will melt. As soil moisture freezes and melts, it alternately expands and contracts. Repeated expansion and contraction of soil moisture beneath a footing may cause it to be lifted during cold weather and dropped during warmer weather. Such a sequence generally cannot be tolerated by the structure.

Frost action on footings is prevented by placing the foundation below the maximum depth of soil that can be penetrated by frost. Depth of frost penetration varies from 4 ft (1.2 m) or more in some northern states (Maine, Minnesota) to zero in parts of some southern states (Florida, Texas). Because frost penetration varies with location, local building codes often dictate minimum depths of footings.

Significant Soil Volume Change

Some soils, particularly certain clays having high plasticity, shrink and swell significantly upon drying and wetting, respectively. This volume change is greatest near the ground surface and decreases with increasing depth. The specific depth and volume change relationship for a particular soil is dependent on the type of soil and level of groundwater. Volume change is usually insignificant below a depth of 5 to 10 ft (1.5 to 3.0 m) and does not occur below the groundwater table. In general, soil beneath the center of a structure is more protected from sun and precipitation; therefore, moisture change and resulting soil movement are smallest there. On the other hand, soil beneath the edges of a structure is less protected, and moisture change and consequent soil movement are greatest there.

As in the case of frost action, significant soil volume change beneath a footing may cause alternate lifting and dropping of the footing. Possible means of avoidance include placing the footing below all strata that are subject to significant volume changes (those soils with plasticity indices greater than 30%), placing it below the zone of volume change, and placing it below any objects that could affect moisture content unduly (such as roots, steam lines, etc.).

Adjacent Structures and Property Lines

Adjacent structures and property lines often affect the horizontal location of a footing. Existing structures may be damaged by construction of new foundations nearby, as a result of vibration, shock resulting from blasting, undermining by excavation,

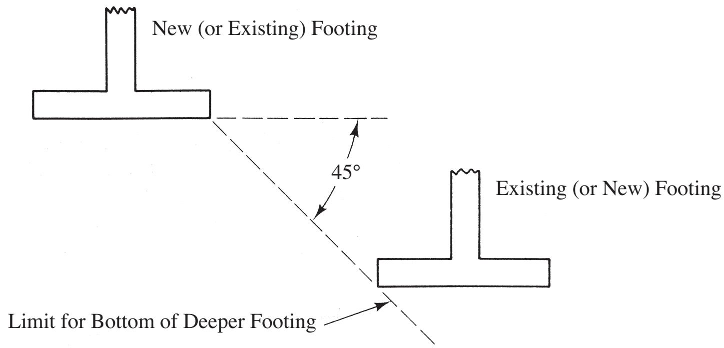


FIGURE 9–4 Empirical rule for the minimum spacing of footings to avoid interference between an old footing and a new footing.
Source: G. A. Leonards, ed. *Foundation Engineering*. McGraw-Hill Book Company, New York, 1962. Reprinted by permission.

or lowering of the water table. After new foundations have been constructed, the (new) load they place on the soil may cause settlement of previously existing structures as a result of new stress patterns in the surrounding soil.

Because damage to existing structures by new construction may result in liability problems, new structures should be located and designed very carefully. In general, the deeper the new foundation and the closer to the old structure, the greater will be the potential for damage to the old structure. Accordingly, old and new foundations should be separated as much as is practical. This is particularly true if the new foundation will be lower than the old one. A general rule is that a straight line drawn downward and outward at a 45° angle from the end of the bottom of any new (or existing) higher footing should not intersect any existing (or new) lower footing (see Figure 9–4).

Special care must be exercised in placing a footing at or near a property line. One reason is that, because a footing is wider than the structure it supports, it is possible for part of the footing to extend across a property line and encroach on adjacent land, although the structure supported by the footing does not do so (see Figure 9–5). Also, excavation for a footing at or near a property line may have a harmful effect (cave-in, for example) on adjacent land. Either of these cases could result in liability problems; hence, much care should be exercised when footings are required near property lines.

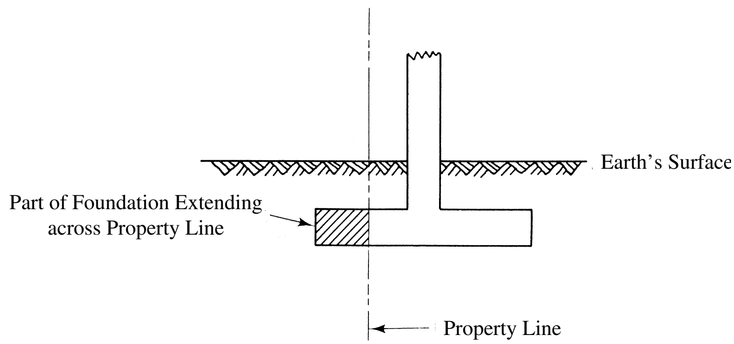


FIGURE 9–5 Sketch showing part of foundation extending across property line.

Groundwater

The presence of groundwater within soil immediately around a footing is undesirable for several reasons. First, footing construction below groundwater level is difficult and expensive. Generally, the area must be drained prior to construction. Second, groundwater around a footing can reduce the strength of soils by reducing their ability to carry foundation pressures. Third, groundwater around a footing may cause hydrostatic uplift problems; fourth, frost action may increase; and fifth, if groundwater reaches a structure's lowest floor, waterproofing problems are encountered. For these reasons, footings should be placed above the groundwater level whenever it is practical to do so.

Underground Defects

Footing location is also affected by the presence of underground defects, including faults, caves, and mines. In addition, human-made discontinuities such as sewer lines and underground cables and utilities must be considered when one is locating footings. Minor breaks in bedrock are seldom a problem unless they are active. Structures should never be built on or near tectonic faults that may slip. Certainly, foundations placed directly above a cave or mine should be avoided if at all possible. Human-made discontinuities are often encountered, and generally foundations should not be placed above them. When they are encountered where a footing is desired, either they or the footing should be relocated. As a matter of fact, a survey of underground utility lines should be made prior to excavation for a foundation in order to avoid damage to the utility lines (or even an explosion) during excavation.

9–4 BEARING CAPACITY ANALYSIS

The conventional method of designing foundations is based on the concept of bearing capacity. One meaning of the verb *to bear* is "to support or hold up." Generally, therefore, *bearing capacity* refers to the ability of a soil to support or hold up a foundation and structure. The *ultimate bearing capacity* of a soil refers to the loading per unit area that will just cause shear failure in the soil. It is given the symbol q_{ult}. The *allowable bearing capacity* (symbol q_a) refers to the loading per unit area that the soil is able to support without unsafe movement. It is the "design" bearing capacity. The allowable load is equal to allowable bearing capacity multiplied by area of contact between foundation and soil. The allowable bearing capacity is equal to the ultimate bearing capacity divided by the factor of safety. A factor of safety of 2.5 to 3 is commonly applied to the value of q_{ult}. Care must be taken to ensure that a footing design is safe with regard to (1) foundation failure (collapse) and (2) excessive settlement.

The basic principles governing bearing capacity theory as developed by Terzaghi (Terzaghi and Peck, 1967) can be better followed by referring to Figure 9–6. As load (Q) is applied, the footing undergoes a certain amount of settlement as it is pushed downward, and a wedge of soil directly below the footing's base moves downward with the footing. The soil's downward movement is resisted by shear resistance of the foundation soil along slip surfaces *cde* and *cfg* and by the weight of the soil in sliding wedges *acfg* and *bcde*. For each set of assumed slip surfaces, the corresponding load Q that would cause failure can be determined. The set of slip

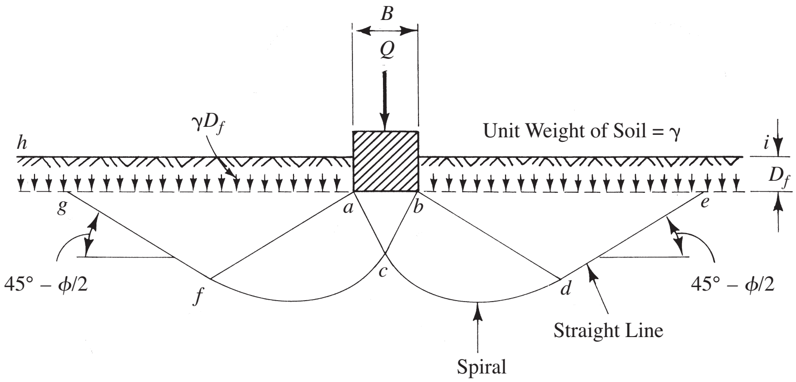


FIGURE 9–6 Plastic analysis of bearing capacity.

surfaces giving the least applied load Q (that would cause failure) is the most critical; hence, the soil's ultimate bearing capacity (q_{ult}) is equal to the least load divided by the footing's area.

The following equations for calculating ultimate bearing capacity were developed by Terzaghi (Terzaghi and Peck, 1967):

Continuous footings (width B):

$$q_{\text{ult}} = cN_c + \gamma_1 D_f N_q + 0.5\gamma_2 B N_\gamma \tag{9–1}$$

Circular footings (radius R):

$$q_{\text{ult}} = 1.2cN_c + \gamma_1 D_f N_q + 0.6\gamma_2 R N_\gamma \tag{9–2}$$

Square footings (width B):

$$q_{\text{ult}} = 1.2cN_c + \gamma_1 D_f N_q + 0.4\gamma_2 B N_\gamma \tag{9–3}$$

The terms in these equations are as follows:

q_{ult} = ultimate bearing capacity
c = cohesion of soil
N_c, N_q, N_γ = Terzaghi's bearing capacity factors
γ_1 = effective unit weight of soil above base of foundation
γ_2 = effective unit weight of soil below foundation
D_f = depth of footing, or distance from ground surface to base of footing
B = width of continuous or square footing
R = radius of a circular footing

The Terzaghi bearing capacity factors (N_c, N_q, N_γ) are functions of the soil's angle of internal friction (ϕ). The term in each equation containing N_c cites the influence of the soil's cohesion on its bearing capacity, the term containing N_q reflects the influence of surcharge, and that containing N_γ shows the influence of soil weight and foundation width or radius.

Values of the Terzaghi dimensionless bearing capacity factors for different values of ϕ can be obtained from Figure 9–7 or Table 9–1. The lines on Figure 9–7 representing N_q and N_c were drawn on the basis of the following equations (Reissner, 1924) for Eqs. (9–4) and (9–5) and (Meyerhof, 1955) for Eq. (9–6):

$$N_q = e^{\pi \tan \phi} \tan^2\left(45° + \frac{\phi}{2}\right) \quad \textbf{(9–4)}$$

$$N_c = \cot \phi \,(N_q - 1) \quad \textbf{(9–5)}$$

$$N_\gamma = (N_q - 1) \tan(1.4\phi) \quad \textbf{(9–6)}$$

Note: e is the base of natural logarithms—approximately 2.71828.

The line on Figure 9–7 representing N_γ was found by plotting values determined in studies by Meyerhof (1955).

Equations (9–1) through (9–3) are applicable for both cohesive and cohesionless soils. Dense sand and stiff clay produce what is called *general shear*, whereas loose sand and soft clay produce what is called *local shear* (see Figure 9–8). In the latter case (loose sand and soft clay), the term c (cohesion) in Eqs. (9–1) through (9–3) is replaced by c', which is equal to $\frac{2}{3}c$; in addition, the terms N_c, N_q, and N_γ are replaced by N'_c, N'_q, and N'_γ, where the latter are obtained from Figure 9–7 using

FIGURE 9–7 Chart showing relation between bearing capacity factors and ϕ [values of N_γ after Meyerhof (1955)]. *Source:* From K. Terzaghi, R. B. Peck, and G. Mesri, *Soil Mechanics in Engineering Practice*, 3rd ed., John Wiley & Sons, Inc., New York, 1996. Copyright © 1996, by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.

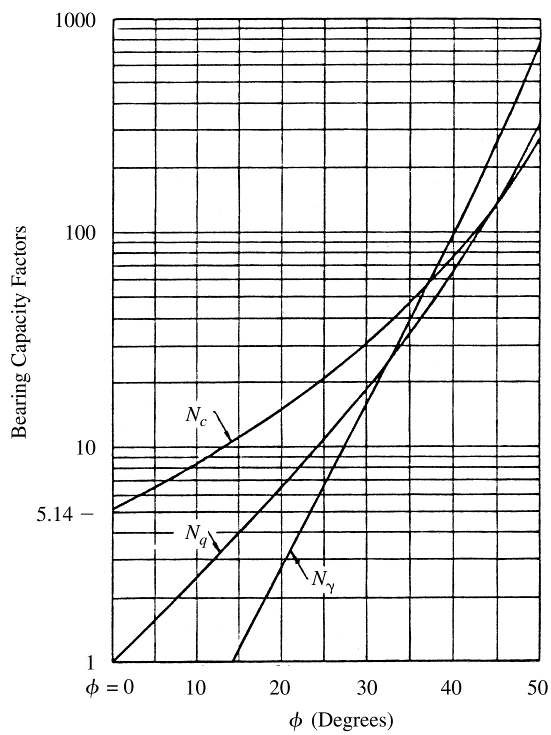


TABLE 9–1
Relation between Bearing Capacity Factors and ϕ

ϕ°	N_c	N_q	N_γ
0	5.14	1.00	0.00
2	5.63	1.20	0.01
4	6.19	1.43	0.04
6	6.81	1.72	0.11
8	7.53	2.06	0.21
10	8.34	2.47	0.37
12	9.28	2.97	0.60
14	10.37	3.59	0.92
16	11.63	4.34	1.37
18	13.10	5.26	2.00
20	14.83	6.40	2.87
22	16.88	7.82	4.07
24	19.32	9.60	5.72
26	22.25	11.85	8.00
28	25.80	14.72	11.19
30	30.14	18.40	15.67
32	35.49	23.18	22.02
34	42.16	29.44	31.15
36	50.59	37.75	44.43
38	61.35	48.93	64.08
40	75.32	64.20	93.69
42	93.71	85.38	139.32
44	118.37	115.31	211.41
46	152.10	158.51	329.74
48	199.27	222.31	526.47
50	266.89	319.07	873.89

FIGURE 9–8 Relation between load and settlement of a footing on dense sand or hard clay and loose sand or soft clay.

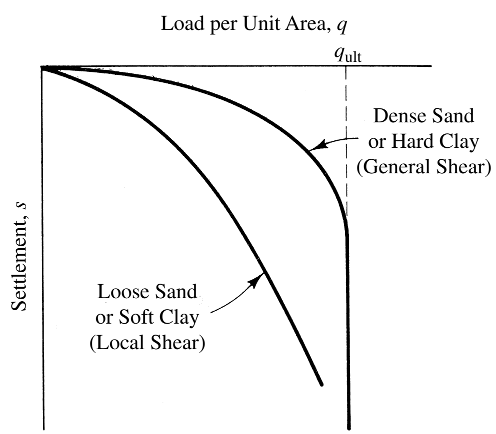
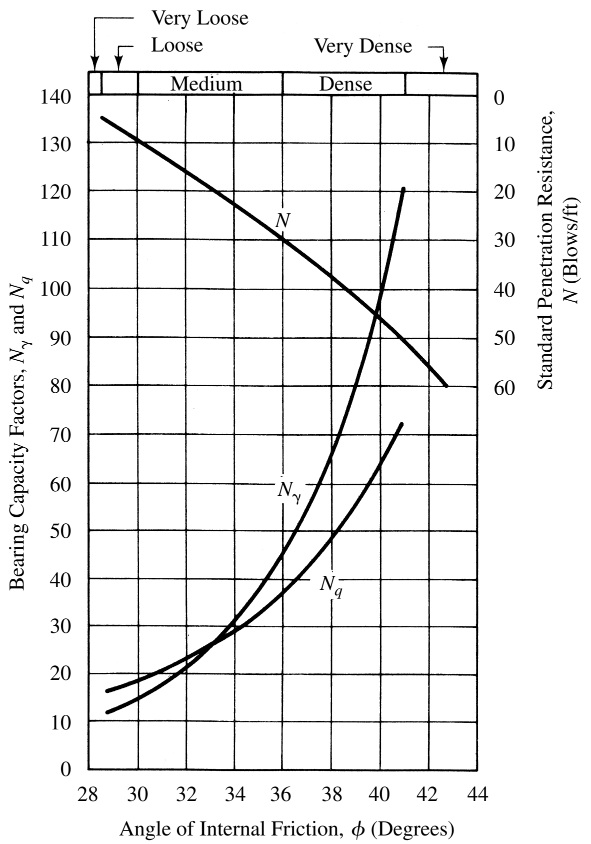


FIGURE 9–9 Curves showing the relationship between bearing capacity factors and ϕ, as determined by theory, and the rough empirical relationship between bearing capacity factors or ϕ and values of standard penetration resistance, *N*. *Source:* R. B. Peck, W. E. Hansen, T. H. Thornburn, *Foundation Engineering*, 2nd ed., John Wiley & Sons, Inc., as elsewhere New York, 1974. Copyright © 1974 by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.



a modified ϕ value (ϕ′) given by the following:

$$c' = \tfrac{2}{3}c \qquad \textbf{(9–7)}$$

$$\phi' = \arctan\left(\tfrac{2}{3}\tan\phi\right) \qquad \textbf{(9–8)}$$

Thus, for loose sand and soft clay, the terms c', N'_c, N'_q, and N'_γ are used in Eqs. (9–1) through (9–3) in place of the respective unprimed terms.

With cohesive soils, shear strength is most critical just after construction or as the load is first applied, at which time shear strength is assumed to consist of only cohesion. In this case, ϕ (angle of internal friction) is taken to be zero. There are several means of evaluating cohesion [*c* terms in Eqs. (9–1) through (9–3)]. One is to use the unconfined compression test for ordinary sensitive or insensitive normally consolidated clay. In this test, *c* is equal to half the unconfined compressive strength (i.e., $\frac{1}{2}q_u$) (see Chapter 8). For sensitive clay, a field vane test may be used to evaluate cohesion (see Chapter 3).

In the case of cohesionless soils, the c term in Eqs. (9–1) through (9–3) is zero. The value of ϕ may be determined by several methods. One is to use corrected standard penetration test (SPT) values (see Chapter 3) and the curves shown in Figure 9–9. One enters the graph at the upper right with a corrected SPT N-value, moves horizontally to the curve marked N, then vertically downward to the abscissa to read the value of ϕ. This value of ϕ can be used with the curves in Figure 9–7 to determine the values of N_q and N_γ. Or, the values of N_q and N_γ may be determined using Figure 9–9 by projecting vertically downward from the curve marked N to the curves marked N_q and N_γ, then projecting horizontally over to the ordinate to read the values of N_q and N_γ, respectively. It is not necessary to determine a value of N_c because c is zero for cohesionless soils; thus, the cN_c terms of Eqs. (9–1) through (9–3) are zero.

The four example problems that follow demonstrate the application of the Terzaghi bearing capacity formulas [i.e., Eqs. (9–1) through (9–3)]. Example 9–1 deals with a wall footing in stiff clay. Example 9–2 involves a square footing in a stiff cohesive soil. A circular footing on a mixed soil is covered in Example 9–3, and a square footing in a dense cohesionless soil is considered in Example 9–4.

EXAMPLE 9–1

Given

1. A strip of wall footing 3.5 ft wide is supported in a uniform deposit of stiff clay (see Figure 9–10).
2. Unconfined compressive strength of this soil (q_u) = 2.8 kips/ft^2.
3. Unit weight of the soil (γ) = 130 lb/ft^3.
4. Groundwater was not encountered during subsurface soil exploration.
5. Depth of wall footing (D_f) = 2 ft.

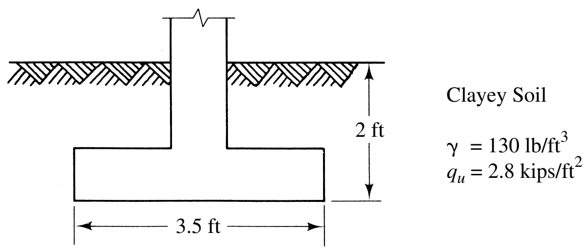
Required

1. Ultimate bearing capacity of this footing.
2. Allowable wall load, using a factor of safety of 3.

Solution

Because the supporting stratum is stiff clay, a general shear condition is evident in this case.

FIGURE 9–10



1. For a continuous wall footing,

$$q_{\text{ult}} = cN_c + \gamma_1 D_f N_q + 0.5\gamma_2 B N_\gamma \tag{9–1}$$

$$c = \frac{q_u}{2} = \frac{2.8 \text{ kips/ft}^2}{2} = 1.4 \text{ kips/ft}^2$$

$$\gamma_1 = \gamma_2 = 0.130 \text{ kip/ft}^3$$

$$D_f = 2 \text{ ft}$$

$$B = 3.5 \text{ ft}$$

If we use $c > 0$, $\phi = 0$ analysis for cohesive soil, when $\phi = 0$, Figure 9–7 gives

$$N_c = 5.14$$
$$N_q = 1.0$$
$$N_\gamma = 0$$

$$q_{\text{ult}} = (1.4 \text{ kips/ft}^2)(5.14) + (0.130 \text{ kip/ft}^3)(2 \text{ ft})(1.0) + (0.5)(0.130 \text{ kip/ft}^3)(3.5 \text{ ft})(0) = 7.46 \text{ kips/ft}^2$$

2. $q_a = (7.46 \text{ kips/ft}^2)/3 = 2.49 \text{ kips/ft}^2$

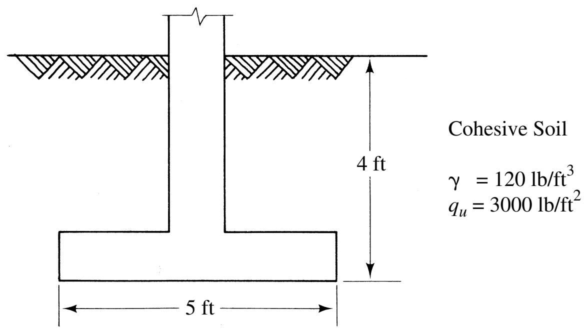
Allowable wall loading $= q_a \times B = (2.49 \text{ kips/ft}^2)(3.5 \text{ ft})$
$= 8.72$ kips/ft of wall length

EXAMPLE 9–2

Given

1. A square footing with 5-ft sides is located 4 ft below the ground surface (see Figure 9–11).
2. The groundwater table is at a great depth, and its effect can be ignored.

FIGURE 9–11



3. The subsoil consists of a thick deposit of stiff cohesive soil, with unconfined compressive strength (q_u) equal to 3000 lb/ft^2.
4. The unit weight (γ) of the soil is 120 lb/ft^3.

Required

Allowable bearing capacity, using a factor of safety of 3.0.

Solution

Because the supporting stratum is stiff clay, a general shear condition is evident in this case. For a square footing,

$$q_{\text{ult}} = 1.2cN_c + \gamma_1 D_f N_q + 0.4\gamma_2 BN_\gamma \tag{9–3}$$

$$c = \frac{q_u}{2} = \frac{3000 \text{ lb/ft}^2}{2} = 1500 \text{ lb/ft}^2$$

$$\gamma_1 = \gamma_2 = 120 \text{ lb/ft}^3$$

$$D_f = 4 \text{ ft}$$

$$B = 5 \text{ ft}$$

If we use $c > 0$, $\phi = 0$ analysis for cohesive soil, when $\phi = 0$, Figure 9–7 gives

$$N_c = 5.14$$

$$N_q = 1.0$$

$$N_\gamma = 0$$

$$q_{\text{ult}} = (1.2)(1500 \text{ lb/ft}^2)(5.14) + (120 \text{ lb/ft}^3)(4 \text{ ft})(1.0) + (0.4)(120 \text{ lb/ft}^3)(5 \text{ ft})(0)$$

$$= 9732 \text{ lb/ft}^2$$

$$q_a = \frac{9732 \text{ lb/ft}^2}{3} = 3244 \text{ lb/ft}^2$$

EXAMPLE 9–3

Given

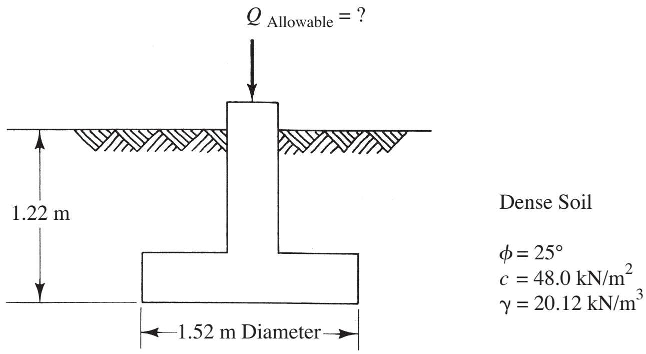
1. A circular footing with a 1.52-m diameter is to be constructed 1.22 m below the ground surface (see Figure 9–12).
2. The subsoil consists of a uniform deposit of dense soil having the following strength parameters:

$$\text{Angle of internal friction} = 25^\circ$$

$$\text{Cohesion} = 48.0 \text{ kN/m}^2$$

3. The groundwater table is at a great depth, and its effect can be ignored.

FIGURE 9–12



Required

The total allowable load (including column load, weight of footing, and weight of soil surcharge) that the footing can carry, using a factor of safety of 3.

Solution

Because the soil supporting the footing is dense soil, a general shear condition is evident. For a circular footing,

$$q_{\text{ult}} = 1.2cN_c + \gamma_1 D_f N_q + 0.6\gamma_2 R N_\gamma \qquad \textbf{(9–2)}$$

$$c = 48.0\ \text{kN/m}^2$$

$$\gamma_1 = \gamma_2 = 20.12\ \text{kN/m}^3$$

$$D_f = 1.22\ \text{m}$$

$$R = \frac{1.52\ \text{m}}{2} = 0.76\ \text{m}$$

From Figure 9–7, with $\phi = 25°$,

$$N_c = 21$$

$$N_q = 10$$

$$N_\gamma = 6$$

Therefore,

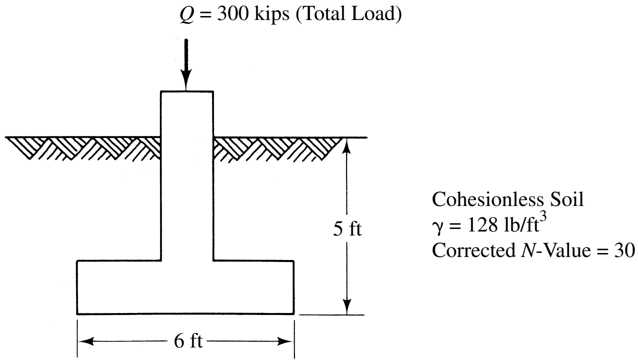
$$q_{\text{ult}} = (1.2)(48.0\ \text{kN/m}^2)(21) + (20.12\ \text{kN/m}^3)(1.22\ \text{m})(10) + (0.6)(20.12\ \text{kN/m}^3)(0.76\ \text{m})(6) = 1510\ \text{kN/m}^2$$

$$q_a = \frac{1510\ \text{kN/m}^2}{3} = 503\ \text{kN/m}^2$$

Therefore,

$$Q_{\text{allowable}} = A \times q_a = \frac{(\pi)(1.52\ \text{m})^2}{4}(503\ \text{kN/m}^2) = 913\ \text{kN}$$

FIGURE 9–13



EXAMPLE 9–4

Given

1. A column footing 6 ft by 6 ft is buried 5 ft below the ground surface in a dense cohesionless soil (see Figure 9–13).
2. The results of laboratory and field tests on the soil are as follows:
   a. Unit weight of soil (γ) = 128 lb/ft^3.
   b. Average corrected SPT N-value beneath the footing = 30.
   c. Groundwater was not encountered during subsurface soil exploration.
3. The footing is to carry a total load of 300 kips, including column load, weight of footing, and weight of soil surcharge.

Required

The factor of safety against bearing capacity failure.

Solution

Because the supporting stratum is dense cohesionless soil, a general shear condition is evident. Hence, the Terzaghi bearing capacity formula for a square footing is used, with $c = 0$, $\phi > 0$. For a square footing,

$$q_{\text{ult}} = 1.2cN_c + \gamma_1 D_f N_q + 0.4\gamma_2 B N_\gamma \tag{9–3}$$

$$c = 0 \quad \text{(cohesionless soil)}$$

$$\gamma_1 = \gamma_2 = 128 \text{ lb/ft}^3$$

$$D_f = 5 \text{ ft}$$

$$B = 6 \text{ ft}$$

From Figure 9–9, with the corrected N-value = 30, $\phi = 36°$. Then, from Table 9–1, with $\phi = 36°$, the following bearing capacity factors are obtained:

$$N_q = 37.75$$

$$N_\gamma = 44.43$$

$$q_{ult} = (1.2)(0)(N_c) + (128\text{ lb/ft}^3)(5\text{ ft})(37.75) + (0.4)(128\text{ lb/ft}^3)(6\text{ ft})(44.43)$$
$$= 37{,}800\text{ lb/ft}^2\text{, or }37.8\text{ kips/ft}^2$$

$$q_{actual} = \frac{Q}{A} = \frac{300\text{ kips}}{6\text{ ft} \times 6\text{ ft}} = 8.33\text{ kips/ft}^2$$

$$\text{Factor of safety against bearing capacity failure} = \frac{q_{ult}}{q_{actual}} = \frac{37.8\text{ kips/ft}^2}{8.33\text{ kips/ft}^2}$$
$$= 4.5 > 3.0 \quad \therefore \text{O.K.}$$

Effect of Water Table on Bearing Capacity

Heretofore in this discussion of bearing capacity, it has been assumed that the water table was well below the footings and thus did not affect the soil's bearing capacity. This is not always the case, however. Depending on where the water table is located, two terms in Eqs. (9–1) through (9–3)—the $\gamma_2 BN_\gamma$ (or $\gamma_2 RN_\gamma$) term and the $\gamma_1 D_f N_q$ term—may require modification.

If the water table is at or above the footing's base, the soil's submerged unit weight (unit weight of soil minus unit weight of water) should be used in the $\gamma_2 BN_\gamma$ (or $\gamma_2 RN_\gamma$) terms of Eqs. (9–1) through (9–3). If the water table is at distance B (note that B is the footing's width) or more below the footing's base (see Figure 9–14), the water table is assumed to have no effect, and the soil's full unit weight should be used. If the water table is below the base of the footing but less than distance B below the base, a linearly interpolated value of effective unit weight should be used in the $\gamma_2 BN_\gamma$ (or $\gamma_2 RN_\gamma$) terms. (That is, the soil's effective unit weight is

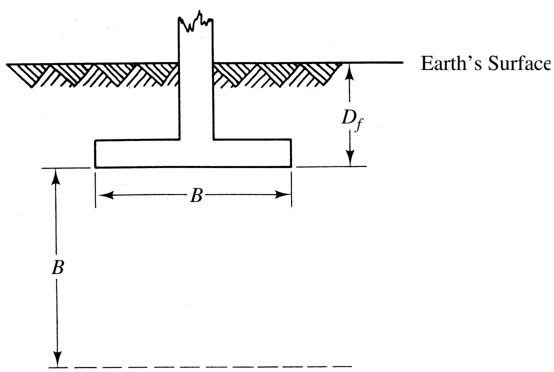


FIGURE 9–14 Sketch showing depth B (equal to footing width) below footing's base.

considered to vary linearly from the submerged unit weight at the footing's base to the full unit weight at distance B below the footing's base.)

If the water table is at the ground surface, the soil's submerged unit weight should be used in the $\gamma_1 D_f N_q$ terms of Eqs. (9–1) through (9–3). If the water table is at or below the footing's base, the soil's full unit weight should be used in these terms. If the water table is between the footing's base and the ground surface, a linearly interpolated value of effective unit weight should be used in the $\gamma_1 D_f N_q$ terms. (That is, the soil's effective unit weight is considered to vary linearly from submerged unit weight at the ground surface to the full unit weight at the footing's base.)

Example 9–5 deals with a square footing in soft, loose soil with the groundwater table located at the ground surface.

EXAMPLE 9–5

Given

1. A 7-ft by 7-ft square footing is located 6 ft below the ground surface (see Figure 9–15).
2. The groundwater table is located at the ground surface.
3. The subsoil consists of a uniform deposit of soft, loose soil. The laboratory test results are as follows:

$$\text{Angle of internal friction} = 20^\circ$$

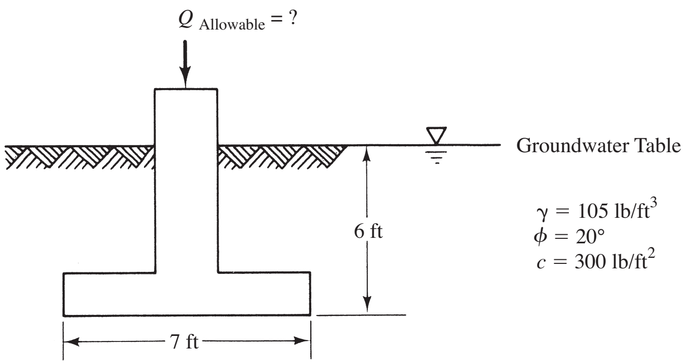
$$\text{Cohesion} = 300\ \text{lb/ft}^2$$

$$\text{Unit weight of soil} = 105\ \text{lb/ft}^3$$

Required

Allowable (design) load that can be imposed on this square footing, using a factor of safety of 3.

FIGURE 9–15



Solution

Because the footing is resting on soft, loose soil, Eq. (9–3) must be modified to reflect a local shear condition.

$$q_{ult} = 1.2c'N'_c + \gamma_1 D_f N'_q + 0.4\gamma_2 B N'_\gamma$$
$$c' = \tfrac{2}{3}c = \tfrac{2}{3} \times 300\ \text{lb/ft}^2 = 200\ \text{lb/ft}^2$$

From Eq. (9–8),

$$\phi' = \arctan(\tfrac{2}{3}\tan\phi) \qquad (9\text{–}8)$$

$$\phi' = \arctan(\tfrac{2}{3}\tan 20^\circ) = 13.6^\circ$$

with $\phi' = 13.6^\circ$, Figure 9–7 gives

$N'_c = 10$

$N'_q = 3$

$N'_\gamma = 1$

$B = 7$ ft

$D_f = 6$ ft

$\gamma_1 = \gamma_2 = 105\ \text{lb/ft}^3 - 62.4\ \text{lb/ft}^3 = 42.6\ \text{lb/ft}^3$ (with the water table at the ground surface, the soil's submerged unit weight must be used)

$$q_{ult} = (1.2)(200\ \text{lb/ft}^2)(10) + (42.6\ \text{lb/ft}^3)(6\ \text{ft})(3) + (0.4)(42.6\ \text{lb/ft}^3)(7\ \text{ft})(1)$$
$$= 3286\ \text{lb/ft}^2$$
$$q_a = \frac{3286\ \text{lb/ft}^2}{3} = 1095\ \text{lb/ft}^2$$
$$Q_{allowable} = q_a \times \text{Area of footing} = (1095\ \text{lb/ft}^2)(7\ \text{ft})(7\ \text{ft}) = 53{,}700\ \text{lb}$$
$$= 53.7\ \text{kips}$$

EXAMPLE 9–6

Given

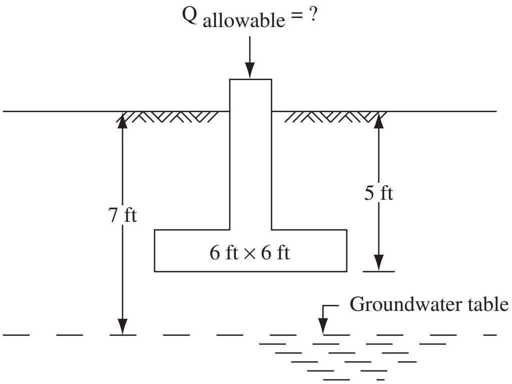
1. A 6-ft by 6-ft square footing is located 5 ft below the ground surface (see Figure 9–16).
2. The groundwater table is located 7 ft below the ground level.
3. The subsoil consists of a uniform deposit of medium dense sand. The field and laboratory test results are as follows:

$$\text{Unit weight of soil} = 102\ \text{lb/ft}^3$$
$$\text{Angle of internal friction} = 32^\circ$$

Required

Allowable (design) load that can be imposed on this square footing, using a factor of safety of 3.

FIGURE 9–16



Solution

Because the footing is resting on medium dense sand, a general shear condition prevails. For a square footing,

$$q_{\text{ult}} = 1.2cN_c + \gamma_1 D_f N_q + 0.4\gamma_2 BN_\gamma \tag{9–3}$$

Because the groundwater table is below the footing's base in this case, the soil's full unit weight ($\gamma_1 = 102\ \text{lb/ft}^3$) should be used in the $\gamma_1 D_f N_q$ term of Eq. (9–3). However, because the groundwater table is below the footing's base but less than distance B ($B = 6$ ft) below the base, a linearly interpolated value of effective unit weight should be used in the $\gamma_2 BN_\gamma$ term of Eq. (9–3). Hence,

$$\begin{aligned}\gamma_2 &= (102\ \text{lb/ft}^3)(2\ \text{ft}/6\ \text{ft}) + (102\ \text{lb/ft}^3 - 62.4\ \text{lb/ft}^3)(4\ \text{ft}/6\ \text{ft})\\ &= 60.4\ \text{lb/ft}^3\end{aligned}$$

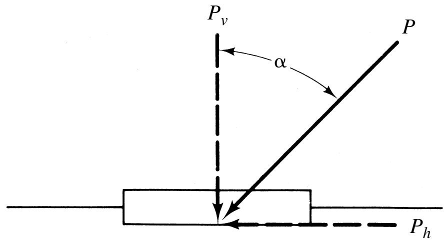
With $\phi = 32°$, Table 9–1 gives

$$\begin{aligned}N_c &= 35.49\\ N_q &= 23.18\\ N_\gamma &= 22.02\end{aligned}$$

Because this soil is medium dense sand, $c = 0$.

$$\begin{aligned}q_{\text{ult}} &= (1.2)(0)(35.49) + (102\ \text{lb/ft}^3)(5\ \text{ft})(23.18)\\ &\quad + (0.4)(60.4\ \text{lb/ft}^3)(6\ \text{ft})(22.02)\\ &= 15{,}014\ \text{lb/ft}^2\\ q_a &= \frac{15{,}014\ \text{lb/ft}^2}{3} = 5005\ \text{lb/ft}^2\\ Q_{\text{allowable}} &= q_a \times \text{Area of footing} = (5005\ \text{lb/ft}^2)(6\ \text{ft})(6\ \text{ft})\\ &= 180{,}200\ \text{lb}\\ &= 180.2\ \text{kips}\end{aligned}$$

FIGURE 9–17 Footing subjected to an inclined load.



Inclined Load

If a footing is subjected to an inclined load (see Figure 9–17), the inclined load can be resolved into vertical and horizontal components. The vertical component can then be used for bearing capacity analysis in the same manner as described previously. After the bearing capacity has been computed by the normal procedure, it must be corrected by an R_i factor, which can be obtained from Figure 9–18. The footing's stability with regard to the inclined load's horizontal component must be checked by calculating the factor of safety against sliding (see Section 9–6).

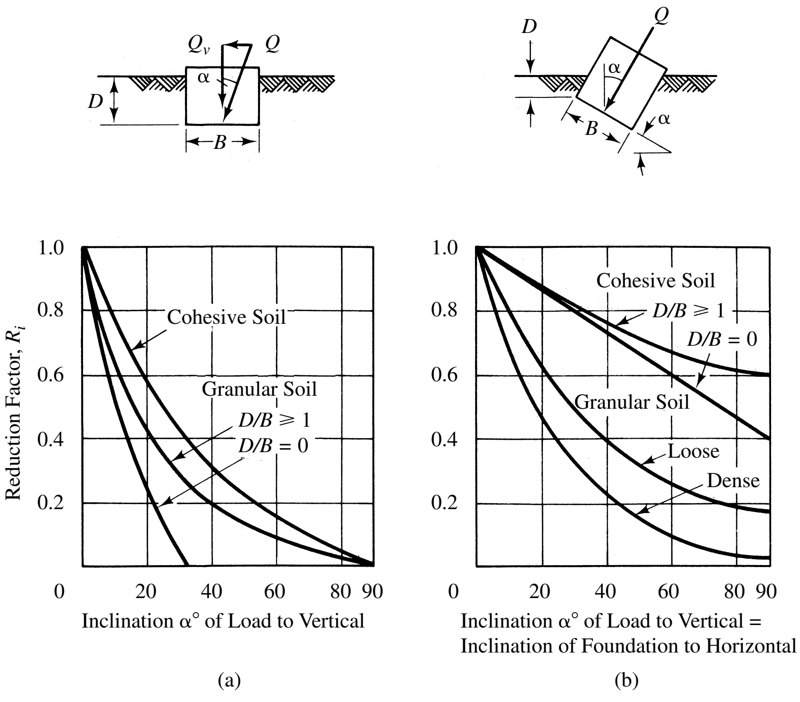
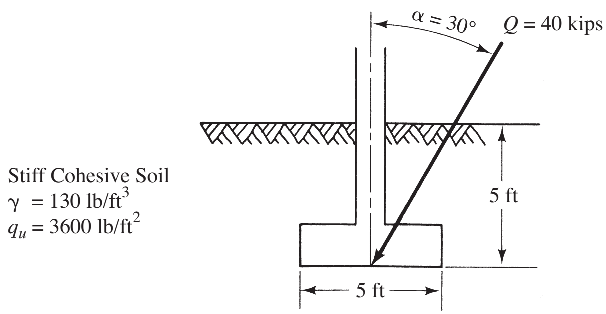


FIGURE 9–18 Inclined load reduction factors: (a) horizontal foundation; (b) inclined foundation (Meyerhof, 1953).
Source: Manual of Recommended Practice, Construction and Maintenance Section, Engineering Division, Association of American Railroads, Chicago, 1958. Reprinted by permission.

FIGURE 9–19



EXAMPLE 9–7

Given

A square footing (5 ft by 5 ft) is subjected to an inclined load as shown in Figure 9–19.

Required

The factor of safety against bearing capacity failure.

Solution

For a square footing,

$$q_{\text{ult}} = 1.2cN_c + \gamma_1 D_f N_q + 0.4\gamma_2 B N_\gamma \qquad \textbf{(9–3)}$$

$$c = \frac{q_u}{2} = \frac{3600\ \text{lb/ft}^2}{2} = 1800\ \text{lb/ft}^2$$

$$\gamma_1 = \gamma_2 = 130\ \text{lb/ft}^3$$

$$D_f = 5\ \text{ft}$$

$$B = 5\ \text{ft}$$

If we use $c > 0$, $\phi = 0$ analysis for cohesive soil, Figure 9–7 gives

$N_c = 5.14$

$N_q = 1.0$

$N_\gamma = 0$

$$q_{\text{ult}} = (1.2)(1800\ \text{lb/ft}^2)(5.14) + (130\ \text{lb/ft}^3)(5\ \text{ft})(1.0) + (0.4)(130\ \text{lb/ft}^3)(5\ \text{ft})(0)$$
$$= 11{,}800\ \text{lb/ft}^2 = 11.8\ \text{kips/ft}^2$$

From Figure 9–18, with $\alpha = 30°$ and cohesive soil, the reduction factor for the inclined load is 0.42.

$$\text{Corrected } q_{\text{ult}} \text{ for inclined load} = (0.42)(11.8\ \text{kips/ft}^2) = 4.96\ \text{kips/ft}^2$$

$$Q_v = Q\cos 30° = (40\ \text{kips})(\cos 30°) = 34.6\ \text{kips}$$

$$\text{Factor of safety} = \frac{Q_{\text{ult}}}{Q_v} = \frac{(4.96\ \text{kips/ft}^2)(5\ \text{ft} \times 5\ \text{ft})}{34.6\ \text{kips}} = 3.6$$

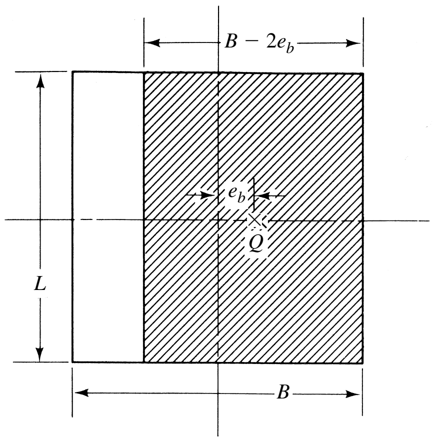


FIGURE 9–20 Useful width for determination of bearing capacity of eccentrically loaded footing on cohesive soil.

Eccentric Load

Design of a footing is somewhat more complicated if it must support an eccentric load. Eccentric loads result from loads applied somewhere other than the footing's centroid or from applied moments, such as those resulting at the base of a tall column from wind loads on the structure. Footings with eccentric loads may be analyzed for bearing capacity by two methods: (1) the concept of useful width and (2) application of reduction factors.

In the useful width method, only that part of the footing that is symmetrical with regard to the load is used to determine bearing capacity by the usual method, with the remainder of the footing being ignored. Thus, in Figure 9–20, with the (eccentric) load applied at the point indicated, the shaded area is symmetrical with regard to the load, and it is used to determine bearing capacity. That area is equal to $L \times (B - 2e_b)$ in this example.

Upon reflection, it can be observed that this method means mathematically that the bearing capacity decreases linearly as eccentricity (distance e_b in Figure 9–20) increases. This linear relationship has been confirmed in the case of cohesive soils. With cohesionless soils, however, a more nearly parabolic bearing capacity reduction has been determined (Meyerhof, 1953). The linear relationship for cohesive soils and the parabolic relationship for cohesionless soils are illustrated in Figure 9–21. Because the useful width method is based on a linear bearing capacity reduction, it is recommended that this method be used only with cohesive soils.

To use the reduction factors method, one first computes bearing capacity by the normal procedure, assuming that the load is applied at the centroid of the footing. The computed value of bearing capacity is then corrected for eccentricity by multiplying by a reduction factor (R_e) obtained from Figure 9–22.

Example 9–8 shows how bearing capacity can be calculated for an eccentric load in a cohesive soil by each of the two methods.

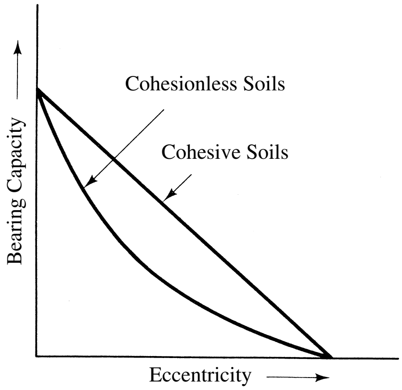


FIGURE 9–21 Relation between bearing capacity and eccentricity for cohesionless and cohesive soils.

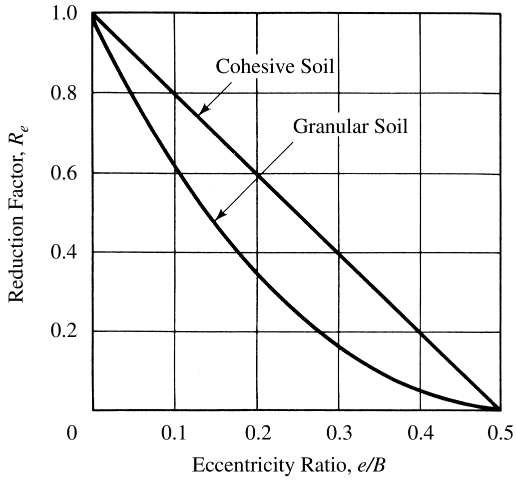


FIGURE 9–22 Eccentric load reduction factors.
Source: Manual of Recommended Practice, Construction and Maintenance Section, Engineering Division, Association of American Railroads, Chicago, 1958. Reprinted by permission.

EXAMPLE 9–8

Given

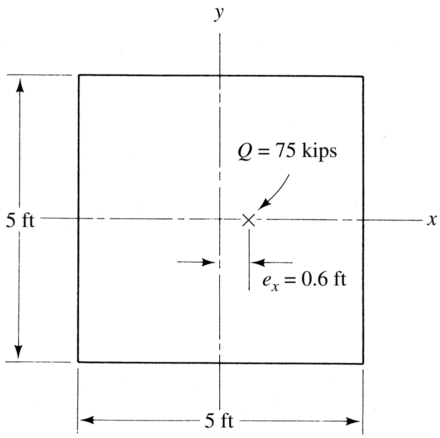
1. A 5-ft by 5-ft square footing is located 4 ft below the ground surface.
2. The footing is subjected to an eccentric load of 75 kips (see Figure 9–23).
3. The subsoil consists of a thick deposit of cohesive soil with $q_u = 4.0$ kips/ft^2 and $\gamma = 130$ lb/ft^3.
4. The water table is at a great depth, and its effect on bearing capacity can be ignored.

Required

The factor of safety against bearing capacity failure:

1. By the concept of useful width.
2. Using a reduction factor from Figure 9–22.

FIGURE 9–23



Solution

1. *The concept of useful width:* From Figure 9–24, the useful width is 3.8 ft.

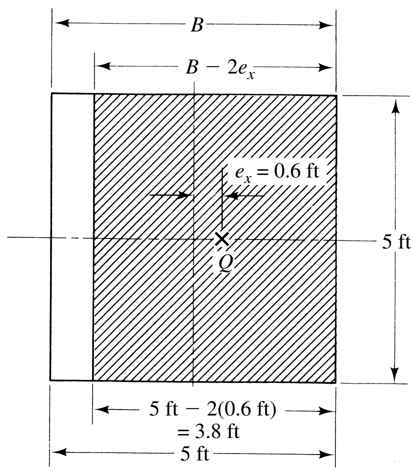
$$q_{\text{ult}} = 1.2cN_c + \gamma_1 D_f N_q + 0.4\gamma_2 B N_\gamma \tag{9–3}$$

$$c = \frac{q_u}{2} = \frac{4.0\ \text{kips/ft}^2}{2} = 2.0\ \text{kips/ft}^2$$

If we use $c > 0$, $\phi = 0$ analysis for cohesive soil, Figure 9–7 gives

$$N_c = 5.14$$
$$N_q = 1.0$$
$$N_\gamma = 0$$

FIGURE 9–24 .



$$\gamma_1 = \gamma_2 = 0.130 \text{ kip/ft}^3$$
$$B = \text{Useful width} = 3.8 \text{ ft}$$
$$q_{\text{ult}} = (1.2)(2.0 \text{ kips/ft}^2)(5.14) + (0.130 \text{ kip/ft}^3)(4 \text{ ft})(1.0) + (0.4)(0.130 \text{ kip/ft}^3)(3.8 \text{ ft})(0) = 12.9 \text{ kips/ft}^2$$

$$\text{Factor of safety} = \frac{12.9 \text{ kips/ft}^2}{\left(\dfrac{75 \text{ kips}}{3.8 \text{ ft} \times 5 \text{ ft}}\right)} = 3.27$$

2. *Using a reduction factor from Figure 9–22:*

$$\text{Eccentricity ratio} = \frac{e_x}{B} = \frac{0.6 \text{ ft}}{5 \text{ ft}} = 0.12$$

For cohesive soil, Figure 9–22 gives $R_e = 0.76$. In this case, q_{ult} is computed based on the actual width: $B = 5$ ft.

$$q_{\text{ult}} = 1.2cN_c + \gamma D_f N_q + 0.4\gamma B N_\gamma \quad \textbf{(9–3)}$$
$$q_{\text{ult}} = (1.2)(2.0 \text{ kips/ft}^2)(5.14) + (0.130 \text{ kip/ft}^3)(4 \text{ ft})(1.0) + (0.4)(0.130 \text{ kip/ft}^3)(5 \text{ ft})(0) = 12.9 \text{ kips/ft}^2$$

$$q_{\text{ult}} \text{ corrected for eccentricity} = q_{\text{ult}} \times R_e = (12.9 \text{ kips/ft}^2)(0.76) = 9.80 \text{ kips/ft}^2$$

$$\text{Factor of safety} = \frac{9.80 \text{ kips/ft}^2}{\left(\dfrac{75 \text{ kips}}{5 \text{ ft} \times 5 \text{ ft}}\right)} = 3.27$$

Footings on Slopes

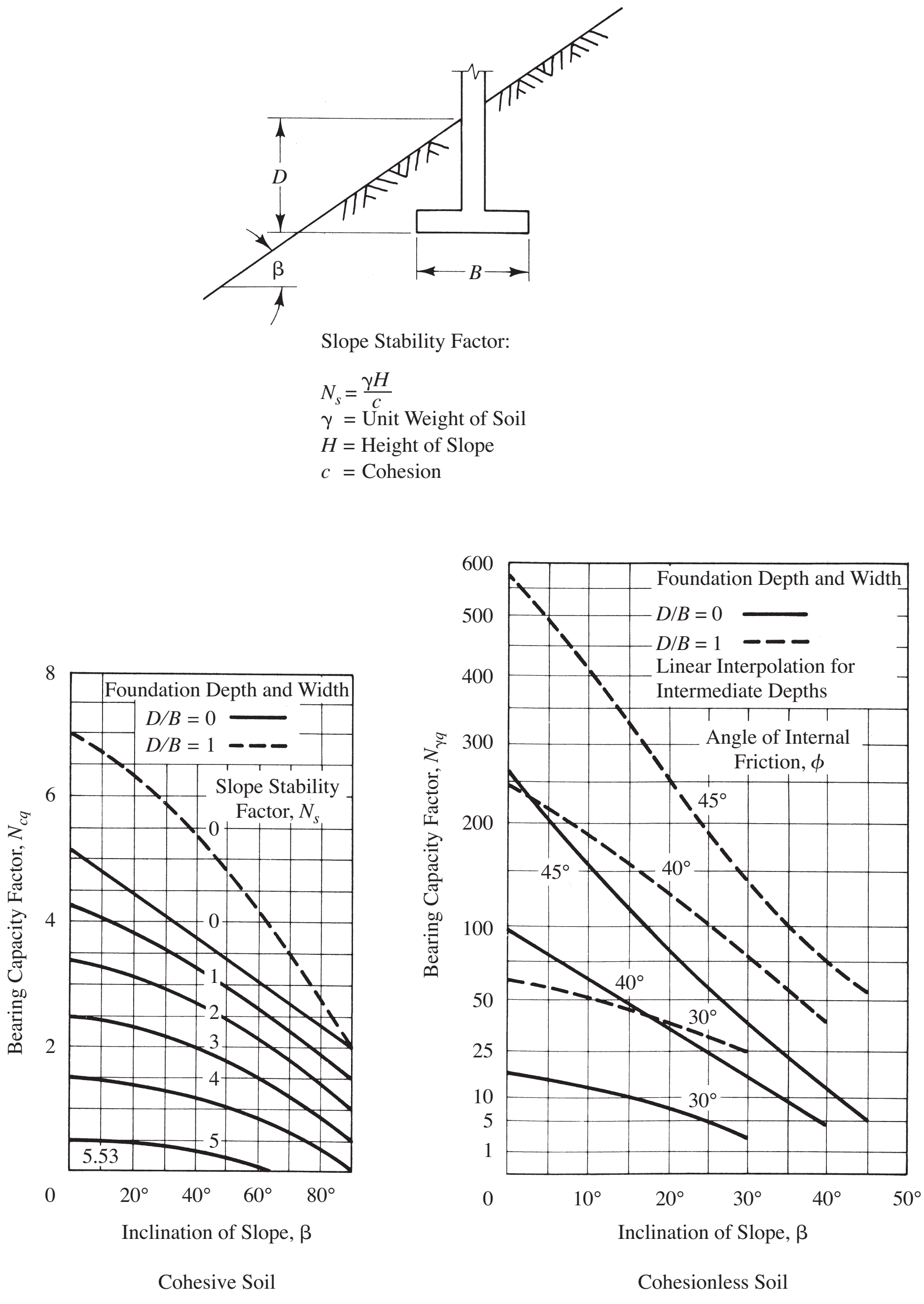
If footings are on slopes, their bearing capacities are less than if the footings were on level ground. In fact, bearing capacity of a footing is inversely proportional to ground slope.

Ultimate bearing capacity for continuous footings on slopes can be determined from the following equation (Meyerhof, 1957):

$$q_{\text{ult}} = cN_{cq} + \tfrac{1}{2}\gamma_2 B N_{\gamma q} \quad \textbf{(9–9)}$$

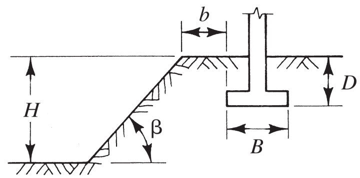
where N_{cq} and $N_{\gamma q}$ are the bearing capacity factors for footings on slopes, and the other terms are as defined previously for Eqs. (9–1) through (9–3). Bearing capacity factors for use in Eq. (9–9) can be determined from Figure 9–25.

For circular or square footings on slopes, it is assumed that the ratios of their bearing capacities on the slope to their bearing capacities on level ground are in the same proportions as the ratio of bearing capacities of continuous footings on slopes to the bearing capacities of the continuous footings on level ground. Hence, their ultimate bearing capacities can be evaluated by first computing q_{ult} by Eq. (9–9)



(a)

FIGURE 9–25 Bearing capacity factors for continuous footing on (a) face of slope and (b) top of slope.
Source: G. G. Meyerhof, "The Ultimate Bearing Capacity of Foundations on Slope," *Proc. 4th Int. Conf. Soil Mech. Found. Eng., London*, 1, 385–386 (1957).



Note:
If $B \leqslant H$:
(1) Obtain N_{cq} from Diagram Using the Curves for $N_s = 0$.
(2) Interpolate for Values of $0 < D/B < 1$.

If $B > H$:
(1) Obtain N_{cq} from Diagram Using the Curves for the Calculated Slope Stability Factor N_s.
(2) Interpolate for Values of $0 < D/B < 1$.

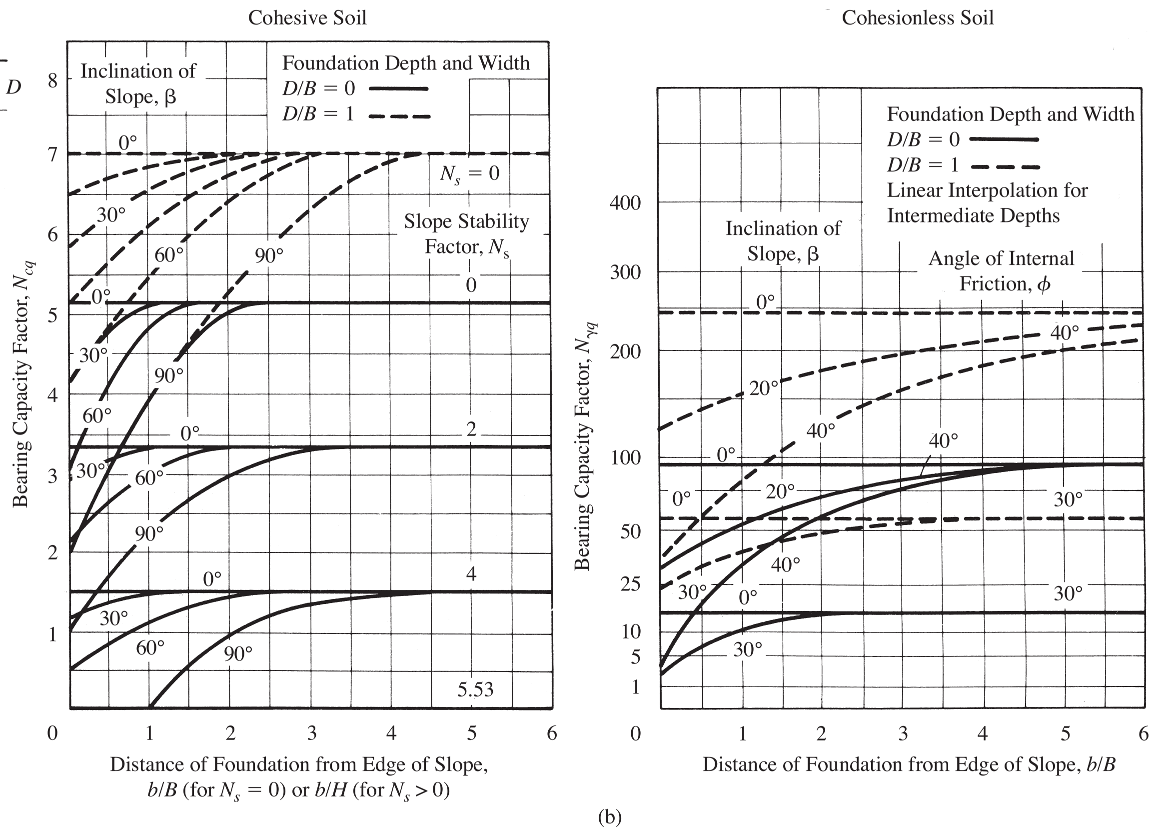


FIGURE 9–25 (*Continued*)

(i.e., as if the given footing on a slope were a continuous footing) and then multiplying that value by the ratio of q_{ult} computed from Eq. (9–2) or (9–3) (as if the given circular or square footing were on level ground) to q_{ult} determined from Eq. (9–1) (continuous footing on level ground). This may be expressed in equation form as follows (U.S. Department of the Navy, 1971):

$$(q_{ult})_{c\text{ or }s\text{ footing on slope}} = (q_{ult})_{\text{continuous footing on slope}}\left[\frac{(q_{ult})_{c\text{ or }s\text{ footing on slope}}}{(q_{ult})_{\text{continuous footing on level ground}}}\right] \quad \textbf{(9–10)}$$

Note: "*c* or *s*" footing denotes either circular or square footing. Examples 9–9 and 9–10 consider footings on slopes.

EXAMPLE 9–9

Given

A bearing wall for a building is to be located close to a slope as shown in Figure 9–26. The groundwater table is located at a great depth.

Required

Allowable bearing capacity, using a factor of safety of 3.

Solution

From Eq. (9–9),

$$q_{ult} = cN_{cq} + 1/2\,\gamma_2 BN_{\gamma q} \quad \textbf{(9–9)}$$

$$c = 0$$

$$\gamma_2 = 19.50\ \text{kN/m}^3$$

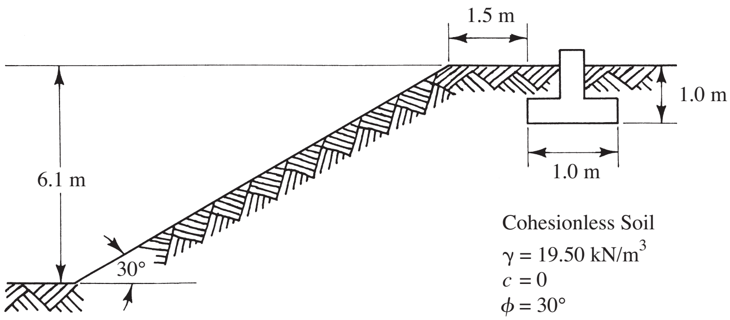
$$B = 1.0\ \text{m}$$

From Figure 9–25b, with $\phi = 30°$,

$$\beta = 30°$$

$$\frac{b}{B} = \frac{1.5\ \text{m}}{1.0\ \text{m}} = 1.5$$

FIGURE 9–26



$$\frac{D_f}{B} = \frac{1.0\text{ m}}{1.0\text{ m}} = 1.0 \text{ (use the dashed line)}$$
$$N_{\gamma q} = 40$$

Therefore,

$$q_{\text{ult}} = (0)(N_{cq}) + (1/2)(19.50\text{ kN/m}^3)(1.0\text{ m})(40) = 390\text{ kN/m}^2$$
$$q_a = \frac{390\text{ kN/m}^2}{3} = 130\text{ kN/m}^2$$

EXAMPLE 9–10

Given

Same conditions as Example 9–9, except that a 1.0-m by 1.0-m square footing is to be constructed on the slope.

Required

Allowable bearing capacity, using a factor of safety of 3.

Solution

From Eq. (9–10),

$$(q_{\text{ult}})_{\text{square footing on slope}} = (q_{\text{ult}})_{\text{continuous footing on slope}}\left[\frac{(q_{\text{ult}})_{\text{square footing on level ground}}}{(q_{\text{ult}})_{\text{continuous footing on level ground}}}\right] \quad \textbf{(9–10)}$$

From Example 9–9,

$$(q_{\text{ult}})_{\text{continuous footing on slope}} = 390\text{ kN/m}^2$$

From Eq. (9–3),

$$(q_{\text{ult}})_{\text{square footing on level ground}} = 1.2cN_c + \gamma_1 D_f N_q + 0.4\gamma_2 B N_\gamma \quad \textbf{(9–3)}$$

From Figure 9–7, with $\phi = 30°$,

$$N_c = 30$$
$$N_q = 18$$
$$N_\gamma = 16$$

$$(q_{\text{ult}})_{\text{square footing on level ground}} = (1.2)(0)(30) + (19.50\text{ kN/m}^3)(1.0\text{ m})(18) + (0.4)(19.50\text{ kN/m}^3)(1.0\text{ m})(16) = 475.8\text{ kN/m}^2$$

From Eq. (9–1),

$$(q_{\text{ult}})_{\text{continuous footing on level ground}} = cN_c + \gamma_1 D_f N_q + 0.5\gamma_2 B N_\gamma \quad \textbf{(9–1)}$$

$$(q_{\text{ult}})_{\text{continuous footing on level ground}} = (0)(30) + (19.50\text{ kN/m}^3)(1.0\text{ m})(18) + (0.5)(19.50\text{ kN/m}^3)(1.0\text{ m})(16)$$

$$= 507.0 \text{ kN/m}^2$$

Therefore, substituting into Eq. (9–10) yields the following:

$$(q_{\text{ult}})_{\text{square footing on slope}} = (390 \text{ kN/m}^2)\left(\frac{475.8 \text{ kN/m}^2}{507.0 \text{ kN/m}^2}\right) = 366 \text{ kN/m}^2$$

$$(q_a)_{\text{square footing on slope}} = \frac{366 \text{ kN/m}^2}{3} = 122 \text{ kN/m}^2$$

9–5 SIZE OF FOOTINGS

After the soil's allowable bearing capacity has been determined, the footing's required area can be determined by dividing the footing load by the allowable bearing capacity.

The following three examples illustrate the sizing of footings based on allowable bearing capacity.

EXAMPLE 9–11

Given

The footing shown in Figure 9–27 is to be constructed in a uniform deposit of stiff clay and must support a wall that imposes a loading of 152 kN/m of wall length.

Required

The width of the footing, using a factor of safety of 3.

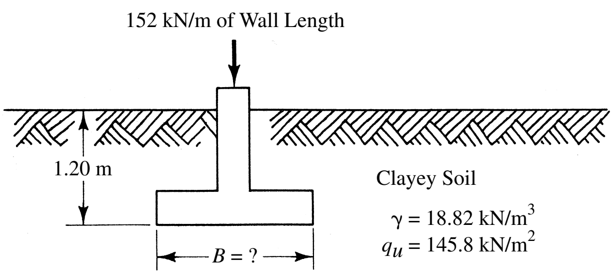
Solution

From Eq. (9–1),

$$q_{\text{ult}} = cN_c + \gamma_1 D_f N_q + 0.5\gamma_2 B N_\gamma \qquad \textbf{(9–1)}$$

$$c = \frac{q_u}{2} = \frac{145.8 \text{ kN/m}^2}{2} = 72.9 \text{ kN/m}^2$$

FIGURE 9–27



If we use $c > 0$, $\phi = 0$ analysis for cohesive soil, when $\phi = 0$, Figure 9–7 gives

$$N_c = 5.14$$
$$N_q = 1.0$$
$$N_\gamma = 0$$
$$q_{\text{ult}} = (72.9\ \text{kN/m}^2)(5.14) + (18.82\ \text{kN/m}^3) \times (1.20\ \text{m})(1.0) + (0.5)(18.82\ \text{kN/m}^3)(B)(0)$$
$$= 397.3\ \text{kN/m}^2$$
$$q_a = \frac{397.3\ \text{kN/m}^2}{3} = 132.4\ \text{kN/m}^2$$
$$\text{Required width of wall} = \frac{152.0\ \text{kN/m}}{132.4\ \text{kN/m}^2} = 1.15\ \text{m}$$

EXAMPLE 9–12

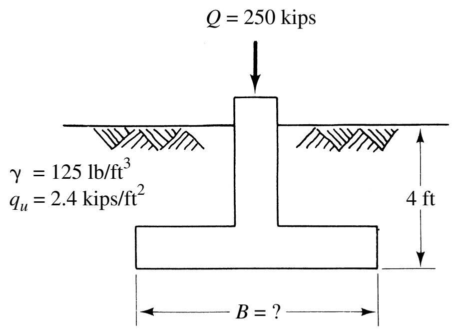
Given

1. A square footing rests on a uniform thick deposit of stiff clay with an unconfined compressive strength (q_u) of 2.4 kips/ft^2.
2. The footing is located 4 ft below the ground surface and is to carry a total load of 250 kips (see Figure 9–28).
3. The clay's unit weight is 125 lb/ft^3.
4. Groundwater is at a great depth.

Required

The necessary square footing dimension, using a factor of safety of 3. Also, find the necessary diameter of a circular footing, using a factor of safety of 3, if the footing is located 5 ft below the ground surface and is to carry a total load of 300 kips, and if $q_u = 2.6$ kips/ft^2.

FIGURE 9–28



Solution

Because the supporting stratum is stiff clay, a condition of general shear governs this case.

$$q_{\text{ult}} = 1.2cN_c + \gamma_1 D_f N_q + 0.4\gamma_2 B N_\gamma \quad \text{(9–3)}$$

$$c = \frac{q_u}{2} = \frac{2.4 \text{ kips/ft}^2}{2} = 1.2 \text{ kips/ft}^2$$

Assuming $\phi = 0$, from Figure 9–7,

$$N_c = 5.14$$
$$N_q = 1.0$$
$$N_\gamma = 0$$
$$\gamma_1 = \gamma_2 = 0.125 \text{ kip/ft}^3$$
$$D_f = 4 \text{ ft}$$
$$q_{\text{ult}} = (1.2)(1.2 \text{ kips/ft}^2)(5.14) + (0.125 \text{ kip/ft}^3)(4 \text{ ft})(1.0) + (0.4)(0.125 \text{ kip/ft}^3)(B)(0) = 7.90 \text{ kips/ft}^2$$
$$q_a = \frac{7.90 \text{ kips/ft}^2}{3} = 2.63 \text{ kips/ft}^2$$

$$\text{Required footing area} = \frac{250 \text{ kips}}{2.63 \text{ kips/ft}^2} = 95.1 \text{ ft}^2$$

Therefore,

$$B^2 = 95.1 \text{ ft}^2$$
$$B = 9.75 \text{ ft}$$

A 10-ft by 10-ft square footing would probably be specified.

For a circular footing,

$$q_{\text{ult}} = 1.2cN_c + \gamma_1 D_f N_q + 0.6\gamma_2 R N_\gamma \quad \text{(9–2)}$$

$$c = \frac{q_u}{2} = \frac{2.6 \text{ kips/ft}^2}{2} = 1.3 \text{ kips/ft}^2$$

Assuming $\phi = 0$, from Figure 9–7,

$$N_c = 5.14$$
$$N_q = 1.0$$
$$N_\gamma = 0$$
$$\gamma_1 = \gamma_2 = 0.125 \text{ kip/ft}^3$$
$$D_f = 5 \text{ ft}$$

$$q_{\text{ult}} = (1.2)(1.3\ \text{kips/ft}^2)(5.14) + (0.125\ \text{kip/ft}^3)(5\ \text{ft})(1.0) + (0.6)(0.125\ \text{kip/ft}^3)(R)(0) = 8.64\ \text{kips/ft}^2$$

$$q_a = \frac{8.64\ \text{kips/ft}^2}{3} = 2.88\ \text{kips/ft}^2$$

$$\text{Required footing area} = \frac{300\ \text{kips}}{2.88\ \text{kips/ft}^2} = 104.2\ \text{ft}^2$$

Therefore,

$$\pi D^2/4 = 104.2\ \text{ft}^2$$
$$D = 11.5\ \text{ft}$$

EXAMPLE 9–13

Given

1. A uniform soil deposit has the following properties:

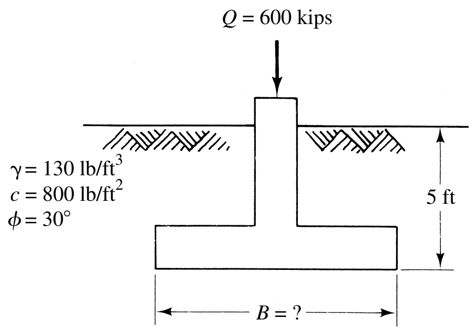
$$\gamma = 130\ \text{lb/ft}^3$$
$$\phi = 30°$$
$$c = 800\ \text{lb/ft}^2$$

2. A proposed footing to be located 5 ft below the ground surface must carry a total load of 600 kips (see Figure 9–29).
3. The groundwater table is at a great depth, and its effect can be ignored.

Required

Determine the required dimension of a square footing to carry the proposed total load of 600 kips, using a general shear condition and a factor of safety of 3.

FIGURE 9–29



Solution

$$q_{\text{ult}} = 1.2cN_c + \gamma_1 D_f N_q + 0.4\gamma_2 B N_\gamma \qquad (9\text{–}3)$$

$$c = 800 \text{ lb/ft}^2$$

$$\gamma_1 = \gamma_2 = 130 \text{ lb/ft}^3$$

$$D_f = 5 \text{ ft}$$

$$\phi = 30°$$

From Figure 9–7,

$$N_c = 30$$

$$N_q = 18$$

$$N_\gamma = 16$$

First Trial

Assume that $B = 10$ ft.

$$q_{\text{ult}} = (1.2)(800 \text{ lb/ft}^2)(30) + (130 \text{ lb/ft}^3)(5 \text{ ft})(18) + (0.4)(130 \text{ lb/ft}^3)(10 \text{ ft})(16)$$

$$= 48{,}820 \text{ lb/ft}^2$$

$$q_a = \frac{48{,}820 \text{ lb/ft}^2}{3} = 16{,}270 \text{ lb/ft}^2$$

$$\text{Required footing area} = \frac{600{,}000 \text{ lb}}{16{,}270 \text{ lb/ft}^2} = 36.9 \text{ ft}^2$$

$$B^2 = 36.9 \text{ ft}^2$$

$$B = 6.07 \text{ ft}$$

Second Trial

Assume that $B = 6$ ft.

$$q_{\text{ult}} = (1.2)(800 \text{ lb/ft}^2)(30) + (130 \text{ lb/ft}^3)(5 \text{ ft})(18) + (0.4)(130 \text{ lb/ft}^3)(6 \text{ ft})(16)$$

$$= 45{,}492 \text{ lb/ft}^2$$

$$q_a = \frac{45{,}492 \text{ lb/ft}^2}{3} = 15{,}164 \text{ lb/ft}^2$$

$$\text{Required footing area} = \frac{600{,}000 \text{ lb}}{15{,}164 \text{ lb/ft}^2} = 39.6 \text{ ft}^2$$

$$B^2 = 39.6 \text{ ft}^2$$

$$B = 6.29 \text{ ft}$$

A 6.5-ft by 6.5-ft square footing would probably be specified.

A footing sized in the manner just described and illustrated should be checked for settlement (see Chapter 7). If settlement is excessive (see Section 9–7), the size of the footing should be revised.

9–6 CONTACT PRESSURE

The pressure acting between a footing's base and the soil below is referred to as *contact pressure.* A knowledge of contact pressure and associated shear and moment distribution is important in footing design.

The pressure distribution beneath a footing varies depending on footing shape, rigidity, and depth as well as type of soil. In general, a rigid footing resting on cohesive soil will exhibit a pressure distribution that is concave upward, as depicted in Figure 9–30a. On cohesionless soil, the footing will normally have a pressure distribution that is concave downward (Figure 9–30b). It is common in practice to assume and use a uniform pressure distribution, as shown in Figure 9–30c. Hence, the pressure distributions in this book will be based entirely on uniform distributions.

Contact pressure can be computed by using the flexural formula:

$$q = \frac{Q}{A} \pm \frac{M_x y}{I_x} \pm \frac{M_y x}{I_y} \qquad \textbf{(9–11)}$$

where q = contact pressure
Q = total axial vertical load
A = area of footing
M_x, M_y = total moment about respective x and y axes
I_x, I_y = moment of inertia about respective x and y axes
x, y = distance from centroid to the point at which the contact pressure is computed along respective x and y axes

In the special case where moments about both x and y axes are zero, contact pressure is simply equal to the total vertical load divided by the footing's area. In theory, contact pressure in this special case is uniform; in practice, however, it tends to vary

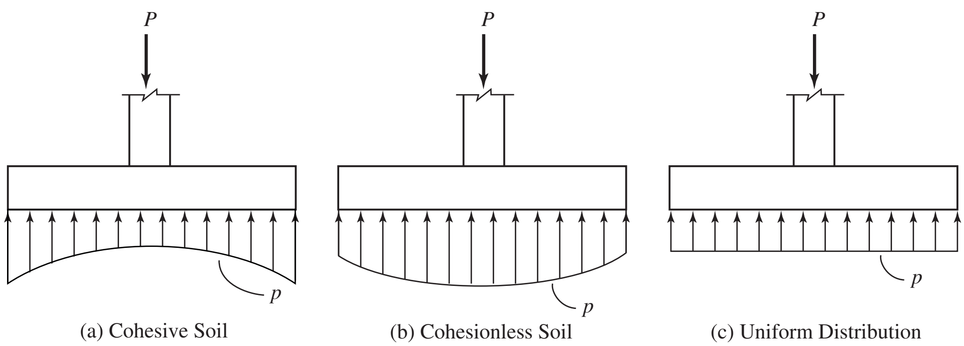


FIGURE 9–30 Pressure distributions beneath rigid footings.

somewhat because of distortion settlement. It is generally assumed to be uniform, however, for design purposes.

Use of the flexural formula to determine contact pressure is illustrated by the following examples. Example 9–14 illustrates computation of contact pressure when no moment is applied to either the x or the y axis. Examples 9–15 and 9–16 illustrate the computation when moment is applied to one axis.

EXAMPLE 9–14

Given

1. A 5-ft by 5-ft square footing as shown in Figure 9–31.
2. Centric column load on the footing = 50 kips.
3. Unit weight of soil = 120 lb/ft^3.
4. Unit weight of concrete = 150 lb/ft^3.
5. Cohesive soil with unconfined compressive strength = 3000 lb/ft^2.

Required

1. Soil contact pressure.
2. Factor of safety against bearing capacity failure.

Solution

1. *Soil contact pressure:*

$$q = \frac{Q}{A} \pm \frac{M_x y}{I_x} \pm \frac{M_y x}{I_y} \tag{9–11}$$

Because the column load is imposed on the centroid of the footing, $M_x = 0$ and $M_y = 0$.

Q = Total axial vertical load on the footing's base

Q = Column load + Weight of footing's base pad + Weight of footing's pedestal + Weight of backfill soil

FIGURE 9–31

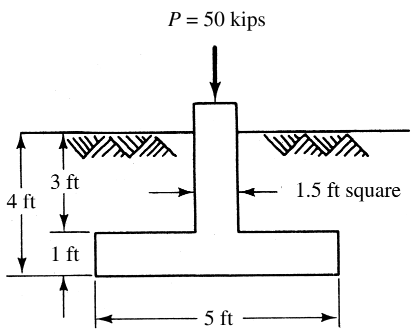
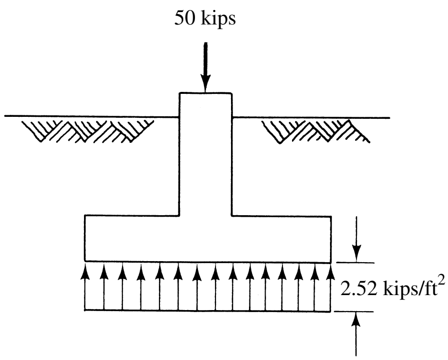


FIGURE 9–32



$$\text{Column load} = 50 \text{ kips (given)}$$

$$\text{Weight of footing's base} = (5 \text{ ft})(5 \text{ ft})(1 \text{ ft})(0.150 \text{ kip/ft}^3) = 3.75 \text{ kips}$$

$$\text{Weight of footing's pedestal} = (1.5 \text{ ft})(1.5 \text{ ft})(3 \text{ ft})(0.150 \text{ kip/ft}^3) = 1.01 \text{ kips}$$

$$\text{Weight of backfill soil} = [(5 \text{ ft})(5 \text{ ft}) - (1.5 \text{ ft})(1.5 \text{ ft})](3 \text{ ft}) \times (0.120 \text{ kip/ft}^3) = 8.19 \text{ kips}$$

$$Q = 50 \text{ kips} + 3.75 \text{ kips} + 1.01 \text{ kips} + 8.19 \text{ kips} = 62.95 \text{ kips}$$

$$A = (5 \text{ ft})(5 \text{ ft}) = 25 \text{ ft}^2$$

$$q = \frac{62.95 \text{ kips}}{25 \text{ ft}^2} = 2.52 \text{ kips/ft}^2$$

Thus, soil contact pressure $= 2.52$ kips/ft² (see Figure 9–32).

2. *Factor of safety against bearing capacity failure:* From Eq. (9–3),

$$q_{\text{ult}} = 1.2cN_c + \gamma_1 D_f N_q + 0.4\gamma_2 B N_\gamma \tag{9–3}$$

$$c = \frac{q_u}{2} = \frac{3000 \text{ lb/ft}^2}{2} = 1500 \text{ lb/ft}^2 = 1.50 \text{ kips/ft}^2$$

From Figure 9–7, if we use $c > 0$, $\phi = 0$ analysis,

$N_c = 5.14$
$N_q = 1.0$
$N_\gamma = 0$
$D_f = 4$ ft

$$q_{ult} = (1.2)(1.50 \text{ kips/ft}^2)(5.14) + (0.120 \text{ kip/ft}^3)(4 \text{ ft})(1.0) + (0.4)(0.120 \text{ kip/ft}^3)(B)(0)$$
$$= 9.73 \text{ kips/ft}^2$$

$$\text{Factor of safety} = \frac{9.73 \text{ kips/ft}^2}{2.52 \text{ kips/ft}^2} = 3.86$$

EXAMPLE 9–15

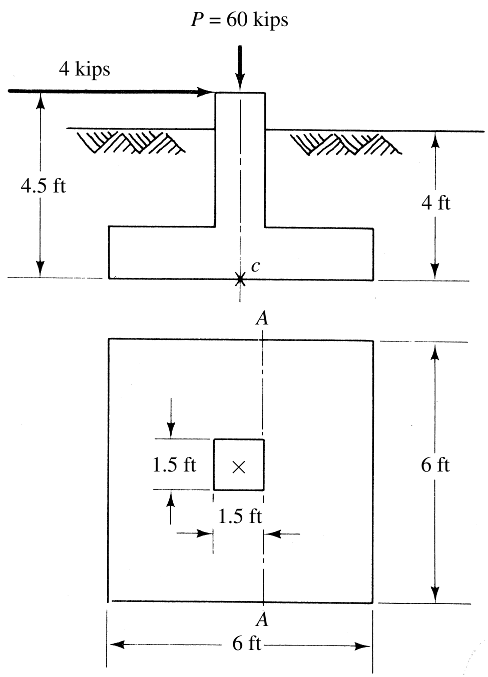
Given

1. A 6-ft by 6-ft square column footing as shown in Figure 9–33.
2. The column's base is hinged.
3. Load on the footing from the column (P) = 60 kips.
   Weight of concrete footing including pedestal and base pad (W_1) = 9.3 kips.
   Weight of backfill soil (W_2) = 11.2 kips.
4. Horizontal load acting on the base of the column = 4 kips.
5. Allowable bearing capacity of the supporting soil = 3.0 kips/ft^2.

Required

1. Contact pressure and soil pressure diagram.
2. Shear and moment at section A–A (Figure 9–33).

FIGURE 9–33



3. Factor of safety against sliding if the coefficient of friction between the footing base and the supporting soil is 0.40.
4. Factor of safety against overturning.

Solution

1. *Contact pressure and soil pressure diagram:*

$$q = \frac{Q}{A} \pm \frac{M_x y}{I_x} \pm \frac{M_y x}{I_y} \quad \textbf{(9–11)}$$

$$Q = P + W_1 + W_2 = 60 \text{ kips} + 9.3 \text{ kips} + 11.2 \text{ kips} = 80.5 \text{ kips}$$

$$A = 6 \text{ ft} \times 6 \text{ ft} = 36 \text{ ft}^2$$

$$M_y = 4 \text{ kips} \times 4.5 \text{ ft}$$

$$= 18 \text{ ft-kips (take moment at point } C\text{; see Figure 9–33)}$$

$$x = \frac{6 \text{ ft}}{2} = 3 \text{ ft}$$

$$I_y = \frac{(6 \text{ ft})(6 \text{ ft})^3}{12} = 108 \text{ ft}^4$$

$$M_x = 0$$

$$\frac{M_x y}{I_x} = 0$$

$$q = \frac{80.5 \text{ kips}}{36 \text{ ft}^2} \pm \frac{(18 \text{ ft-kips})(3 \text{ ft})}{108 \text{ ft}^4} = 2.24 \text{ kips/ft}^2 \pm 0.50 \text{ kip/ft}^2$$

$$q_{\text{right}} = 2.24 \text{ kips/ft}^2 + 0.50 \text{ kip/ft}^2 = 2.74 \text{ kips/ft}^2 < 3.0 \text{ kips/ft}^2 \quad \therefore \text{ O.K.}$$

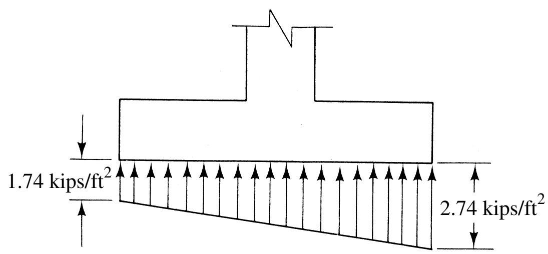
$$q_{\text{left}} = 2.24 \text{ kips/ft}^2 - 0.50 \text{ kip/ft}^2 = 1.74 \text{ kips/ft}^2 < 3.0 \text{ kips/ft}^2 \quad \therefore \text{ O.K.}$$

The pressure diagram is shown in Figure 9–34.

2. *Shear and moment at section A–A:* From Figure 9–35, ΔFDG and ΔEDH are similar triangles. Therefore,

$$\frac{DE}{DF} = \frac{EH}{FG}$$

FIGURE 9–34



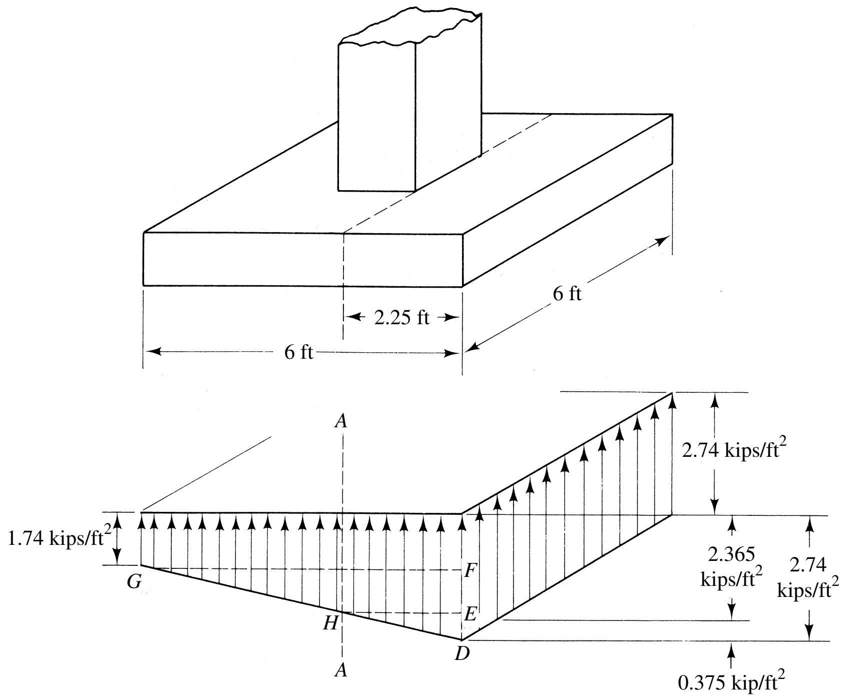


FIGURE 9–35

$$DF = 2.74\ \text{kips/ft}^2 - 1.74\ \text{kips/ft}^2 = 1.0\ \text{kip/ft}^2$$

$$EH = \frac{6\ \text{ft}}{2} - \frac{1.5\ \text{ft}}{2} = 2.25\ \text{ft} \quad \text{(see Figures 9–33 and 9–35)}$$

$$FG = 6\ \text{ft}$$

$$\frac{DE}{1.0\ \text{kip/ft}^2} = \frac{2.25\ \text{ft}}{6\ \text{ft}}$$

$$DE = 0.375\ \text{kip/ft}^2$$

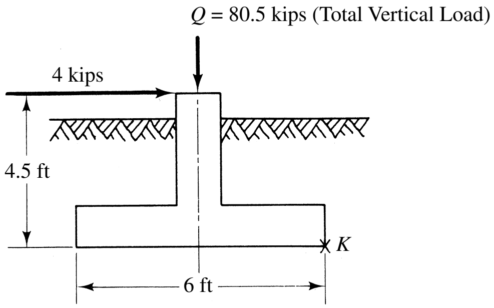
$$\text{Shear at } A\text{–}A = (2.25\ \text{ft})(2.365\ \text{kips/ft}^2)(6\ \text{ft}) + (1/2)(2.25\ \text{ft}) \times (0.375\ \text{kip/ft}^2)(6\ \text{ft})$$

$$= 31.93\ \text{kips} + 2.53\ \text{kips} = 34.46\ \text{kips}$$

$$\text{Moment at } A\text{–}A = (31.93\ \text{kips})\left(\frac{2.25\ \text{ft}}{2}\right) + (2.53\ \text{kips})(\tfrac{2}{3} \times 2.25\ \text{ft})$$

$$= 39.7\ \text{ft-kips}$$

FIGURE 9–36



3. *Factor of safety against sliding:*
   Factor of safety against sliding

$$= \frac{\text{Total vertical load} \times \text{Coefficient of friction between base and soil}}{\sum \text{Horizontal forces}}$$

$$= \frac{(60\text{ kips} + 9.3\text{ kips} + 11.2\text{ kips})(0.40)}{4\text{ kips}} = 8.05$$

4. *Factor of safety against overturning:* See Figure 9–36. By taking moments at point K, one can compute the factor of safety against overturning as follows:

$$\text{Factor of safety} = \frac{\text{Moment to resist turning}}{\text{Turning moment}} = \frac{(80.5\text{ kips})(6\text{ ft}/2)}{(4\text{ kips})(4.5\text{ ft})} = 13.4$$

EXAMPLE 9–16

Given

1. A 7.5-ft by 10-ft rectangular column footing as shown in Figure 9–37.
2. The column's base is fixed into the foundation.
3. Load on the footing from the column (P) = 50 kips.
   Weight of the concrete footing and weight of the backfill soil (W) = 25 kips.
   Horizontal load acting on the column's base (H) = 3 kips.
   Moment acting on the foundation (M) = 30 ft-kips.
4. Allowable bearing capacity of the soil = 2 kips/ft^2.

Required

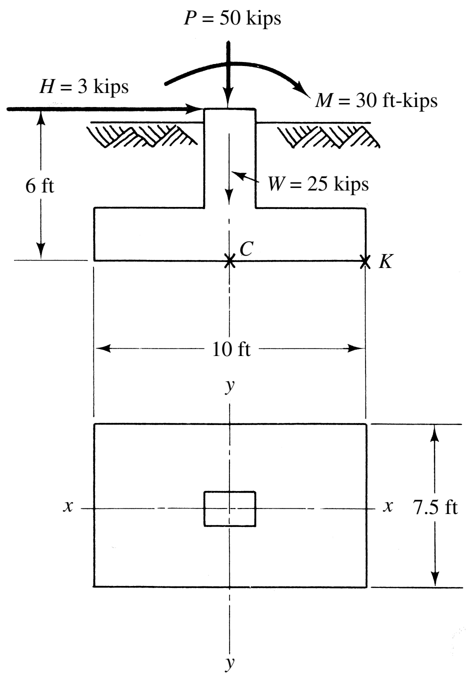
1. Contact pressure and soil pressure diagram.
2. Factor of safety against overturning.

Solution

1. *Contact pressure and soil pressure diagram:*

$$q = \frac{Q}{A} \pm \frac{M_x y}{I_x} \pm \frac{M_y x}{I_y} \tag{9–11}$$

FIGURE 9–37



$$Q = 50 \text{ kips} + 25 \text{ kips} = 75 \text{ kips}$$

$$A = 7.5 \text{ ft} \times 10 \text{ ft} = 75 \text{ ft}^2$$

$$M_y = (3 \text{ kips})(6 \text{ ft}) + 30 \text{ ft-kips}$$

$$= 48 \text{ ft-kips} \quad \text{(take moments at point } C\text{; see Figure 9–37)}$$

$$x = \frac{10 \text{ ft}}{2} = 5 \text{ ft}$$

$$I_y = \frac{(7.5 \text{ ft})(10 \text{ ft})^3}{12} = 625 \text{ ft}^4$$

$$M_x = 0$$

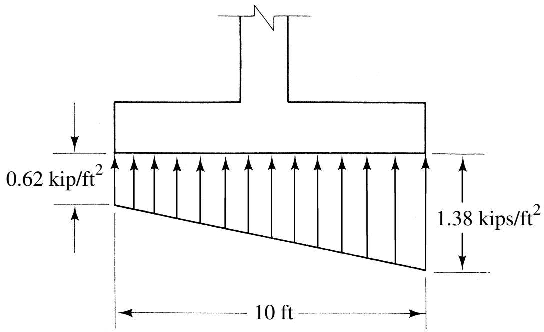
$$q = \frac{75 \text{ kips}}{75 \text{ ft}^2} \pm \frac{(48 \text{ ft-kips})(5 \text{ ft})}{625 \text{ ft}^4} = 1.00 \text{ kip/ft}^2 \pm 0.38 \text{ kip/ft}^2$$

$$q_{\text{right}} = 1.38 \text{ kips/ft}^2 < 2 \text{ kips/ft}^2 \quad \therefore \text{ O.K.}$$

$$q_{\text{left}} = 0.62 \text{ kip/ft}^2 < 2 \text{ kips/ft}^2 \quad \therefore \text{ O.K.}$$

The pressure diagram is shown in Figure 9–38.

FIGURE 9–38



2. *Factor of safety against overturning:* By taking moments at point K (Figure 9–37), one finds that

$$\text{Factor of safety} = \frac{\text{Moment to resist turning}}{\text{Turning moment}}$$

$$= \frac{(50\text{ kips} + 25\text{ kips})(10\text{ ft}/2)}{(3\text{ kips})(6\text{ ft}) + (30\text{ ft-kips})} = 7.8$$

Under certain conditions, such as very large applied moments, Eq. (9–11) may give a negative value for the contact pressure. This implies tension between the footing and the soil. Soil cannot furnish any tensile resistance; hence, the flexural formula is not applicable in this situation. Instead, contact pressure may be calculated according to the basic equations of statics in the following manner.

Referring to Figure 9–39, by summing all forces in the vertical direction and all moments about point C and setting both sums equal to zero, one obtains the following two equations:

$$\sum V = 0 \uparrow +$$

$$\left(\frac{q}{2}\right)(d)(L) - P - W = 0 \qquad \textbf{(9-12)}$$

$$\sum M_c = 0 + \curvearrowright$$

$$M + (H)(S) - \left(\frac{q}{2}\right)(d)(L)\left(x - \frac{d}{3}\right) = 0 \qquad \textbf{(9–13)}$$

Because all terms in Eqs. (9–12) and (9–13) are known except q and d, the two equations may be solved simultaneously to determine q and d. With q and d both known, the soil pressure diagram may be drawn. This technique is illustrated by Example 9–17.

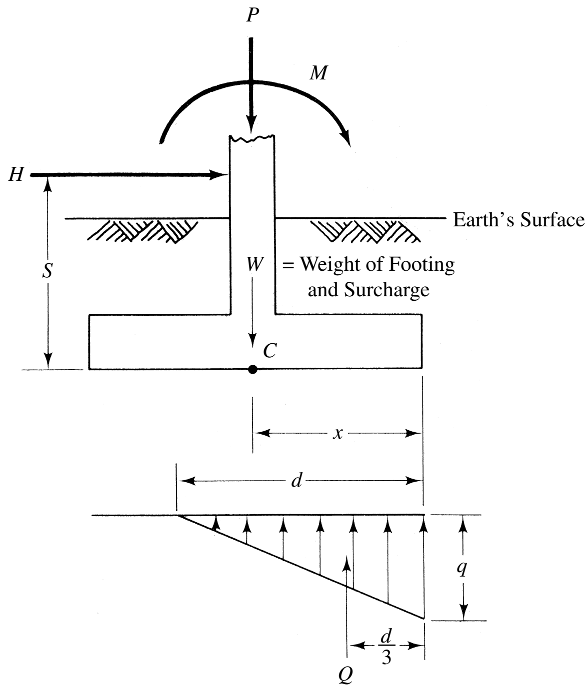


FIGURE 9–39 Footing contact pressure when resultant force on footing is outside middle third of base of footing.

EXAMPLE 9–17

Given

A rectangular footing 5 ft by 7.5 ft loaded as shown in Figure 9–40.

Required

Compute contact pressure and draw the soil pressure diagram.

Solution

By the flexural formula,

$$q = \frac{Q}{A} \pm \frac{M_x y}{I_x} \pm \frac{M_y x}{I_y} \tag{9–11}$$

$$Q = 50\text{ kips} + 20\text{ kips} = 70\text{ kips}$$

$$A = 5\text{ ft} \times 7.5\text{ ft} = 37.5\text{ ft}^2$$

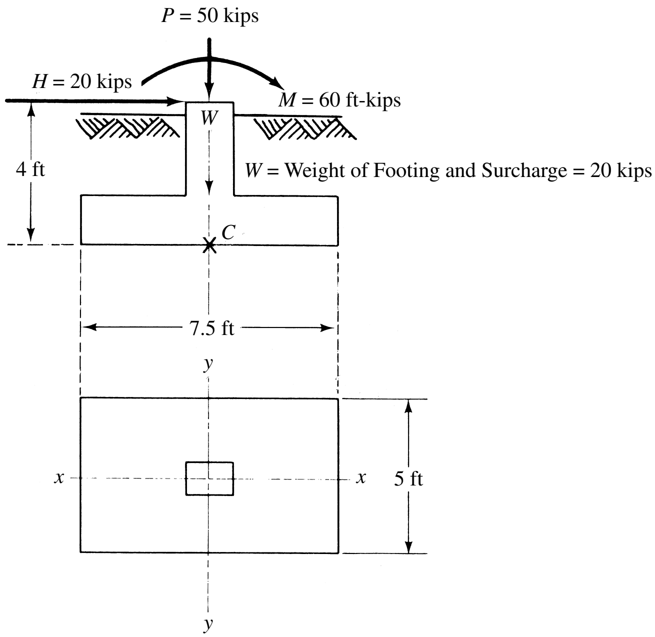
$$M_x = 0$$

$$M_y = (4\text{ ft})(20\text{ kips}) + 60\text{ ft-kips} = 140\text{ ft-kips}$$

(take moments at point *C*; see Figure 9–40)

$$x = \frac{7.5\text{ ft}}{2} = 3.75\text{ ft}$$

FIGURE 9–40



$$I_y = \frac{(5\text{ ft})(7.5\text{ ft})^3}{12} = 176\ \text{ft}^4$$

$$q = \frac{70\text{ kips}}{37.5\text{ ft}^2} \pm \frac{(140\text{ ft-kips})(3.75\text{ ft})}{176\text{ ft}^4} = 1.87\text{ kips/ft}^2 \pm 2.98\text{ kips/ft}^2$$

$$q_{\text{right}} = +4.85\text{ kips/ft}^2$$

$$q_{\text{left}} = -1.11\text{ kips/ft}^2$$

Because q_{left} has a negative value, the flexural formula is not applicable in this case. Solve this problem by $\sum V = 0$ and $\sum M_c = 0$ [i.e., Eqs. (9–12) and (9–13)]. Referring to Figures 9–40 and 9–41, one finds that

$$\left(\frac{q}{2}\right)(d)(L) - P - W = 0 \qquad \textbf{(9–12)}$$

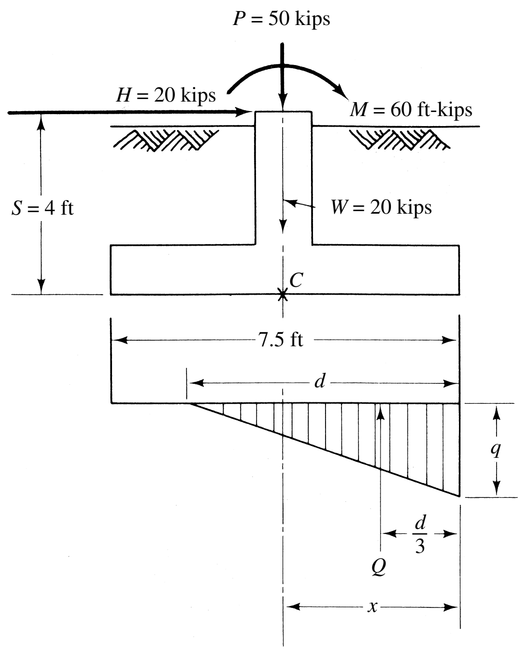
$$\left(\frac{qd}{2}\right)(5\text{ ft}) = 70\text{ kips} \qquad \textbf{(A)}$$

$$M + (H)(S) - \left(\frac{q}{2}\right)(d)(L)\left(x - \frac{d}{3}\right) = 0 \qquad \textbf{(9–13)}$$

$$60\text{ ft-kips} + (20\text{ kips})(4\text{ ft}) - (70\text{ kips})\left(\frac{7.5\text{ ft}}{2} - \frac{d}{3}\right) = 0 \qquad \textbf{(B)}$$

[Note that $(qd/2)(L) = 70$ kips, from Eq. (A).] From Eq. (B),

FIGURE 9–41



$$60 \text{ ft-kips} + 80 \text{ ft-kips} - 262.5 \text{ ft-kips} + \frac{70 \text{ kips}}{3} d = 0$$

$$d = 5.25 \text{ ft}$$

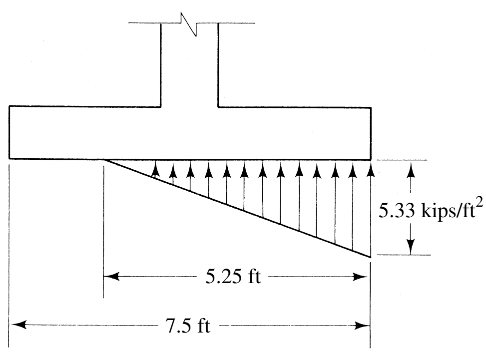
Substitute $d = 5.25$ ft into Eq. (A):

$$\left(\frac{q}{2}\right)(5.25 \text{ ft})(5 \text{ ft}) = 70 \text{ kips}$$

$$q = 5.33 \text{ kips/ft}^2$$

The pressure diagram is shown in Figure 9–42.

FIGURE 9–42



9–7 TOTAL AND DIFFERENTIAL SETTLEMENT

Previous material in this chapter dealt primarily with bearing capacity analysis and prevention of bearing capacity failure of footings. Footings may also fail as a result of excessive settlement; thus, after the size of the footing has been determined by bearing capacity analysis, footing settlement should be calculated and the design revised if the calculated settlement is considered to be excessive.

Calculation of settlement has already been covered (Chapter 7). Maximum permissible settlement depends primarily on the nature of the superstructure. Some suggested maximum permissible settlement values were given in Table 7–10.

9–8 STRUCTURAL DESIGN OF FOOTINGS

As was noted in Section 9–5, the required base area of a footing may be determined by dividing the column load by the allowable bearing capacity. Determining the thickness and shape of the footing and amount and location of reinforcing steel and performing other details of the actual structural design of footings are, however, ultimately the responsibility of a structural engineer.

In general, a geotechnical engineer furnishes the contact pressure diagram and the shear and moment at a section (in the footing) at the face of the column, pedestal, or wall. This was demonstrated in Example 9–15 when the contact pressure diagram and the shear and moment at section *A–A* were determined. From this information, the structural engineer can do the actual structural design of the footing.

9–9 PROBLEMS

9–1. A strip of wall footing 3 ft wide is located 3.5 ft below the ground surface. Supporting soil has a unit weight of 125 lb/ft^3. The results of laboratory tests on the soil samples indicate that the supporting soil's cohesion and angle of internal friction are 1200 lb/ft^2 and 25°, respectively. Groundwater was not encountered during subsurface soil exploration. Determine the allowable bearing capacity, using a factor of safety of 3.

9–2. A square footing with a size of 10 ft by 10 ft is located 8 ft below the ground surface. The subsoil consists of a thick deposit of stiff cohesive soil with an unconfined compressive strength equal to 3600 lb/ft^2. The soil's unit weight is 128 lb/ft^3. Compute the ultimate bearing capacity.

9–3. A circular footing with a 1.22-m diameter is to be constructed 1.07 m below the ground surface. The subsoil consists of a uniform deposit of dense soil having a unit weight of 21.33 kN/m^3, an angle of internal friction of 20°, and a cohesion of 57.6 kN/m^2. The groundwater table is at a great depth, and its effect can be ignored. Determine the safe total load (including column load and weight of footing and soil surcharge), using a factor of safety of 3.

9–4. A footing 8 ft by 8 ft is buried 6 ft below the ground surface in a dense cohesionless soil. The results of laboratory and field tests on the supporting soil indicate that the soil's unit weight is 130 lb/ft^3, and the average corrected SPT N-value beneath the footing is 37. Compute the allowable (design) load that can be imposed onto this footing, using a factor of safety of 3.

9–5. A square footing with a size of 8 ft by 8 ft is to carry a total load of 40 kips. The depth of the footing is 5 ft below the ground surface, and groundwater is located at the ground surface. The subsoil consists of a uniform deposit of soft clay, the cohesion of which is 500 lb/ft^2. The soil's unit weight is 110 lb/ft^3. Compute the factor of safety against bearing capacity failure.

9–6. A square footing 0.3 m by 0.3 m is placed on the surface of a dense cohesionless sand (unit weight = 18.2 kN/m^3) and subjected to a load test. If the footing fails at a load of 13.8 kN, what is the value of ϕ for the sand?

9–7. A load test is performed on a 0.3-m by 0.3-m square footing on a dense cohesionless sand (unit weight = 18.0 kN/m^3). The footing's base is located 0.6 m below the ground surface. If the footing fails at a load of 82 kN, what is the failure load per unit area of the base of a square footing 2.0 m by 2.0 m loaded with its base at the same depth in the same materials?

9–8. A square footing 2 m by 2 m is to be constructed 1.22 m below the ground surface, as shown in Figure 9–43. The groundwater table is located 1.82 m below the ground surface. The subsoil consists of a uniform, medium dense, cohesionless soil with the following properties:

$$\text{Unit weight of soil} = 18.53 \text{ kN/m}^3$$
$$\text{Angle of internal friction} = 32°$$
$$\text{Cohesion} = 0$$

Determine the foundation soil's allowable bearing capacity if a factor of safety of 3 is used.

9–9. A square footing is to be constructed on a uniform thick deposit of clay with an unconfined compressive strength of 3 kips/ft^2. The footing will be located 5 ft below the ground surface and is designed to carry a total load of 300 kips. The unit weight of the supporting soil is 128 lb/ft^3. No groundwater was encountered during soil exploration. Considering general shear, determine the square footing dimension, using a factor of safety of 3.

FIGURE 9–43

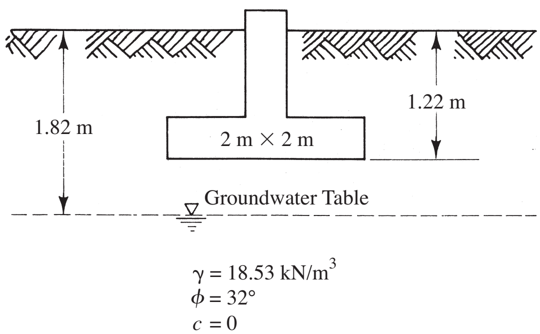
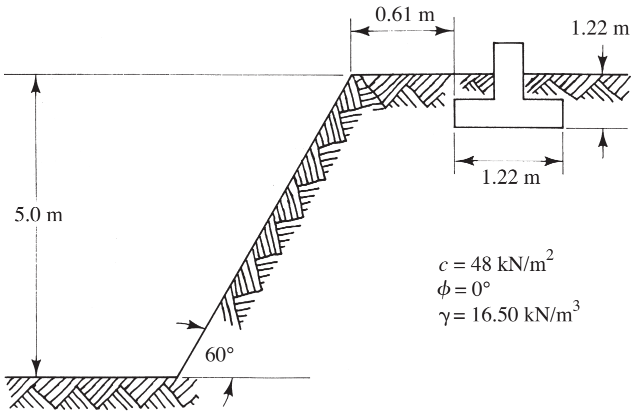


FIGURE 9–44



9–10. A proposed square footing carrying a total load of 500 kips is to be constructed on a uniform thick deposit of dense cohesionless soil. The soil's unit weight is 135 lb/ft^3, and its angle of internal friction is 38°. The depth of the footing is to be 5 ft. Determine the dimension of this proposed footing, using a factor of safety of 3.

9–11. A bearing wall for a building is to be located close to a slope as shown in Figure 9–44. The groundwater table is at a great depth. Determine the foundation soil's allowable bearing capacity for the wall if a factor of safety of 3 is used.

9–12. Solve Problem 9–11 if the proposed footing is to be a 1.22-m by 1.22-m square footing (instead of a wall).

9–13. A wall footing is to be constructed on a uniform deposit of stiff clay, as shown in Figure 9–45. The footing is to support a wall that imposes 130 kN/m of

FIGURE 9–45 Clayey soil.

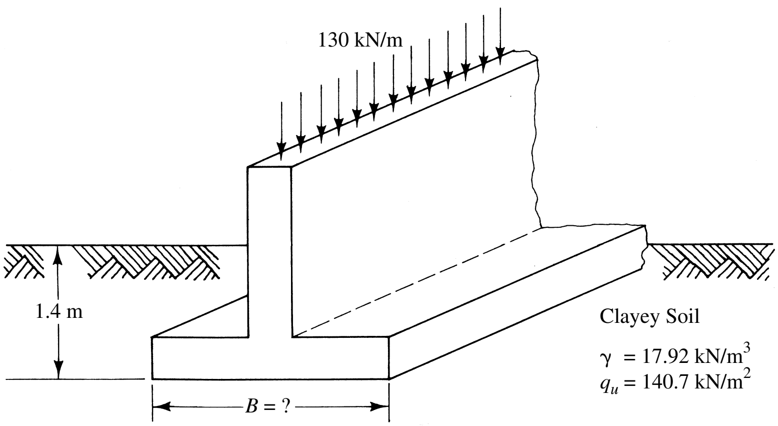
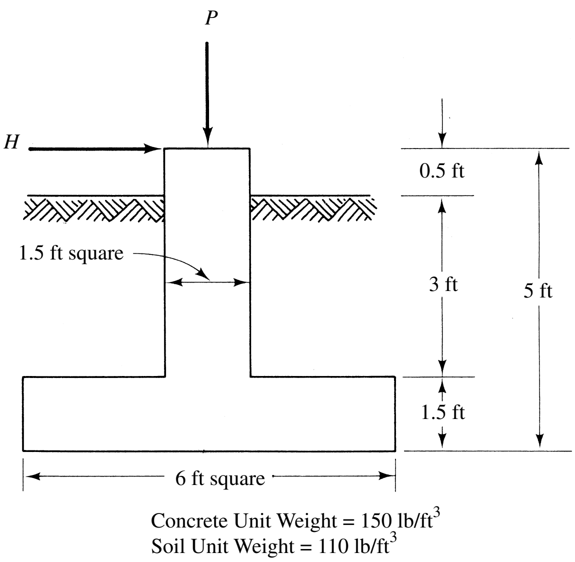


FIGURE 9–46



wall length. Determine the required width of the footing if a factor of safety of 3 is used.

9–14. Compute and draw soil pressure diagrams for the footing shown in Figure 9–46 for the following loads:

1. $P = 70$ kips and $H = 20$ kips
2. $P = 70$ kips and $H = 10$ kips

9–15. Considering general shear, compute the safety factor against a bearing capacity failure for each of the two loadings in Problem 9–14 if the bearing soil is as follows:

1. Cohesionless

   $\phi = 30°$
   $\gamma = 110 \text{ lb/ft}^3$
   $c = 0$

2. Cohesive

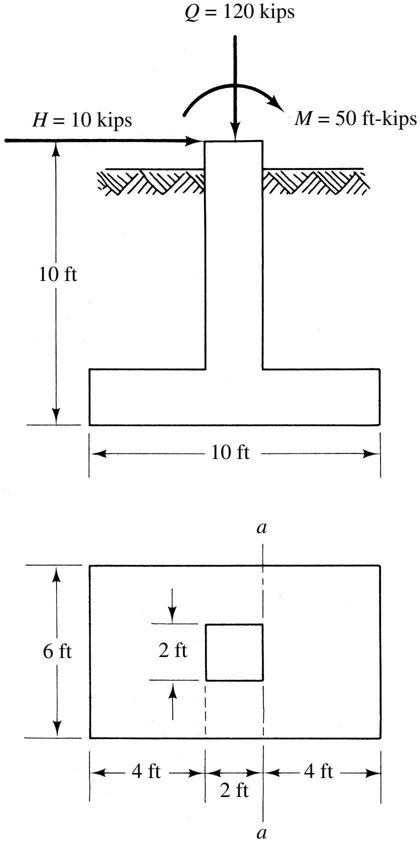
   $\phi = 0°$
   $\gamma = 110 \text{ lb/ft}^3$
   $c = 3000 \text{ lb/ft}^2$

In each case, groundwater is 10 ft below the base of the footing.

9–16. Same as Problem 9–15, except that groundwater is located at the ground surface.

9–17. For the footing shown in Figure 9–47, the vertical load, including the column load, surcharge weight, and weight of the footing, is 120 kips. The horizontal

FIGURE 9–47



load is 10 kips, and a moment of 50 ft-kips (clockwise) is also imposed on the foundation.

1. Compute the soil contact pressure and draw the soil contact pressure diagram.
2. Compute the shear on section *a–a* (Figure 9–47).
3. Compute the moment on section *a–a* (Figure 9–47).
4. Compute the factor of safety against overturning.
5. Compute the factor of safety against sliding if the coefficient of friction between the soil and the base of the footing is 0.60.
6. Compute the factor of safety against bearing capacity failure if the ultimate bearing capacity of the soil supporting the footing is 5.4 tons/ft^2.

9–18. A 6-ft by 6-ft square footing is buried 5 ft below the ground surface. The footing is subjected to an eccentric load of 200 kips. The eccentricity of the 200-kip load (e_x) is 0.8 ft. The supporting soil has values of $\phi = 38°$, $c = 0$, and $\gamma = 135$ lb/ft^3. Calculate the factor of safety against bearing capacity failure using a reduction factor from Figure 9–22.