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31

Shallow Foundations

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31.1 Introduction

A shallow foundation may be defined as one in which the foundation depth (D) is less than or on the order of its least width (B), as illustrated in Figure 31.1. Commonly used types of shallow foundations include spread footings, strap footings, combined footings, and mat or raft footings. Shallow foundations or footings provide their support entirely from their bases, whereas deep foundations derive the capacity from two parts, skin friction and base support, or one of these two. This chapter is primarily designated to the discussion of the bearing capacity and settlement of shallow foundations, although structural considerations for footing design are briefly addressed. Deep foundations for bridges are discussed in Chapter 32.

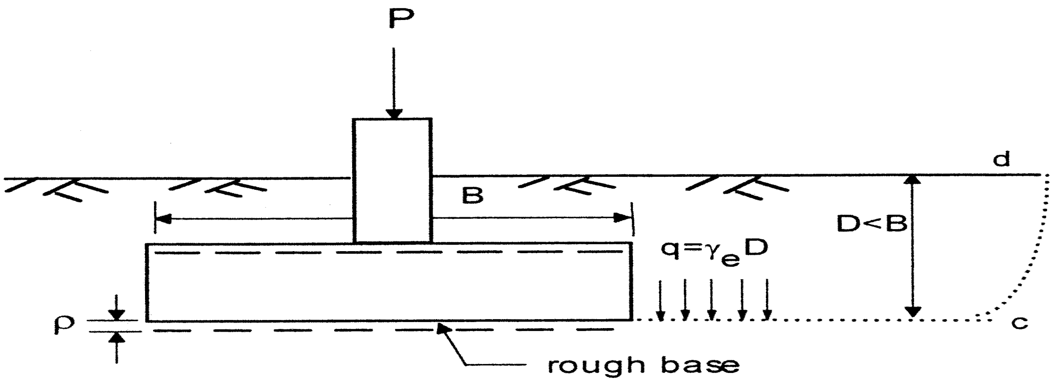


FIGURE 31.1 Definition sketch for shallow footings.

TABLE 31.1 Typical Values of Safety Factors Used in Foundation Design (after Barker et al. [9])

Failure Type	Failure Mode	Safety Factor	Remark
Shearing	Bearing capacity failure	2.0–3.0	The lower values are used when
	Overturning	2.0–2.5	uncertainty in design is small
	Overall stability	1.5–2.0	and consequences of failure are
	Sliding	1.5–2.0	minor; higher values are used
Seepage	Uplift	1.5–2.0	when uncertainty in design is
	Heave	1.5–2.0	large and consequences of failure
	Piping	2.0–3.0	are major

Source: Terzaghi, K. and Peck, R.B., *Soil Mechanics in Engineering Practice*, 2nd ed., John Wiley & Sons, New York, 1967. With permission.

31.2 Design Requirements

In general, any foundation design must meet three essential requirements: (1) providing adequate safety against structural failure of the foundation; (2) offering adequate bearing capacity of soil beneath the foundation with a specified safety against ultimate failure; and (3) achieving acceptable total or differential settlements under working loads. In addition, the overall stability of slopes in the vicinity of a footing must be regarded as part of the foundation design. For any project, it is usually necessary to investigate both the bearing capacity and the settlement of a footing. Whether footing design is controlled by the bearing capacity or the settlement limit rests on a number of factors such as soil condition, type of bridge, footing dimensions, and loads. Figure 31.2 illustrates the load–settlement relationship for a square footing subjected to a vertical load *P*. As indicated in the curve, the settlement *p* increases as load *P* increases. The ultimate load P_u is defined as a peak load (curves 1 and 2) or a load at which a constant rate of settlement (curve 3) is reached as shown in Figure 31.2. On the other hand, the ultimate load is the maximum load a foundation can support without shear failure and within an acceptable settlement. In practice, all foundations should be designed and built to ensure a certain safety against bearing capacity failure or excessive settlement. A safety factor (*SF*) can be defined as a ratio of the ultimate load P_u and allowable load P_u. Typical value of safety factors commonly used in shallow foundation design are given in Table 31.1.

31.3 Failure Modes of Shallow Foundations

Bearing capacity failure usually occurs in one of the three modes described as general shear, local shear, or punching shear failure. In general, which failure mode occurs for a shallow foundation depends on the relative compressibility of the soil, footing embedment, loading conditions, and drainage conditions. General shear failure has a well-defined rupture pattern consisting of three zones, I, II, and III, as shown in Figure 31.3a. Local shear failure generally consists of clearly defined rupture surfaces beneath the footing (zones I and II). However, the failure pattern on the sides of the footing (zone III) is not clearly defined. Punch shear failure has a poorly defined rupture pattern concentrated within zone I; it is usually associated with a large settlement and does not mobilize shear stresses in zones II and III as shown in Figure 31.3b and c. Ismael and Vesic [40] concluded that, with increasing overburden pressure (in cases of deep foundations), the failure mode changes from general shear to local or punch shear, regardless of soil compressibility. The further examination of load tests on footings by Vesic [68,69] and De Beer [29] suggested that the ultimate load occurs at the breakpoint of the load–settlement curve, as shown in Figure 31.2. Analyzing the modes of failure indicates that (1) it is possible to formulate a general bearing capacity equation for a loaded footing failing in the general shear mode, (2) it is very difficult to generalize the other two failure modes for shallow foundations because of their poorly defined rupture surfaces, and (3) it is of significance to know the magnitude of settlements of footings required to mobilize ultimate loads. In the following sections, theoretical and empirical methods for evaluating both bearing capacity and settlement for shallow foundations will be discussed.

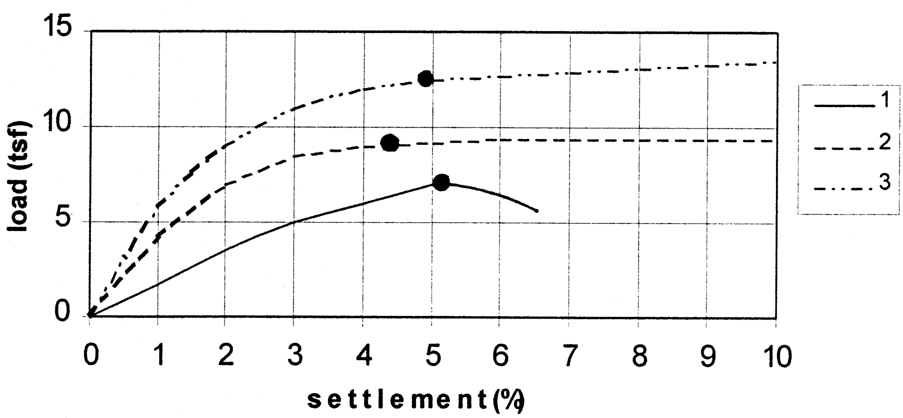


FIGURE 31.2 Load-settlement relationships of shallow footings.

31.4 Bearing Capacity for Shallow Foundations

31.4.1 Bearing Capacity Equation

The computation of ultimate bearing capacity for shallow foundations on soil can be considered as a solution to the problem of elastic–plastic equilibrium. However, what hinders us from finding closed analytical solutions rests on the difficulty in the selection of a mathematical model of soil constitutive relationships. Bearing capacity theory is still limited to solutions established for the rigid-plastic solid of the classic theory of plasticity [40,69]. Consequently, only approximate methods are currently available for the posed problem. One of them is the well-known Terzaghi's bearing capacity equation [19,63], which can be expressed as

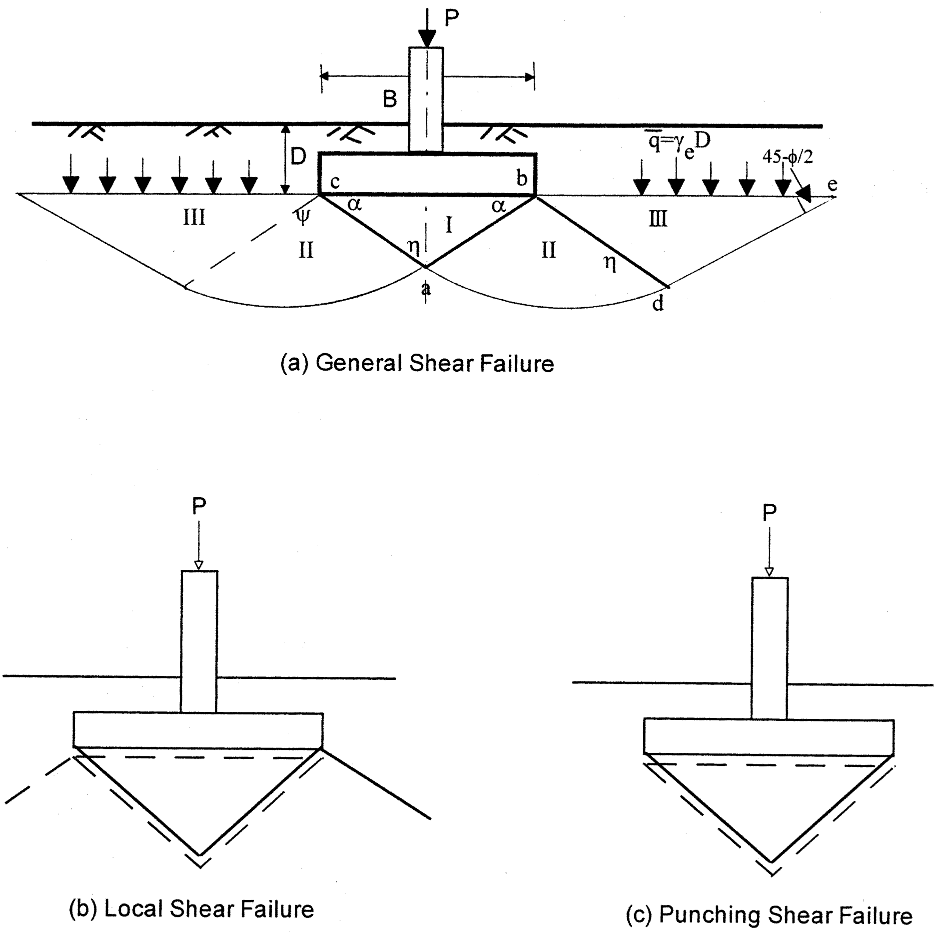


FIGURE 31.3 Three failure modes of bearing capacity.

$$q_{\text{ult}} = cN_c s_c + \overline{q}N_q + 0.5\gamma BN_\gamma s_\gamma \tag{31.1}$$

where q_{ult} is ultimate bearing capacity, c is soil cohesion, $\overline{q}$ is effective overburden pressure at base of footing ($= \gamma_1 D$), γ is effective unit weight of soil or rock, and B is minimum plan dimension of footing. N_c, N_q, and N_γ are bearing capacity factors defined as functions of friction angle of soil and their values are listed in Table 31.2. s_c and s_r are shape factors as shown in Table 31.3.

These three N factors are used to represent the influence of the cohesion (N_c), unit weight (N_γ), and overburden pressure (N_q) of the soil on bearing capacity. As shown in Figures 31.1 and 31.3(a), the assumptions used for Eq. (31.1) include

1. The footing base is rough and the soil beneath the base is incompressible, which implies that the wedge *abc* (zone I) is no longer an active Rankine zone but is in an elastic state. Consequently, zone I must move together with the footing base.
2. Zone II is an immediate zone lying on a log spiral arc *ad*.

TABLE 31.2 Bearing Capacity Factors for the Terzaghi Equation

ϕ (°)	N_c	N_q	N_γ	$K_{p\gamma}$
0	5.7[a]	1.0	0	10.8
5	7.3	1.6	0.5	12.2
10	9.6	2.7	1.2	14.7
15	12.9	4.4	2.5	18.6
20	17.7	7.4	5.0	25.0
25	25.1	12.7	9.7	35.0
30	37.2	22.5	19.7	52.0
34	52.6	36.5	36.0	—
35	57.8	41.4	42.4	82.0
40	95.7	81.3	100.4	141.0
45	172.3	173.3	297.5	298.0
48	258.3	287.9	780.1	—
50	347.5	415.1	1153.2	800.0

[a] $N_c = 1.5\pi + 1$ (Terzaghi [63], p. 127); values of N_γ for ϕ of 0, 34, and 48° are original Terzaghi values and used to backcompute $K_{p\gamma}$.

After Bowles, J.E., *Foundation Analysis and Design*, 5th ed., McGraw-Hill, New York, 1996. With permission.

TABLE 31.3 Shape Factors for the Terzaghi Equation

	Strip	Round	Square
s_c	1.0	1.3	1.3
s_γ	1.0	0.6	0.8

After Terzaghi [63].

3. Zone III is a passive Rankine zone in a plastic state bounded by a straight line *ed*.
4. The shear resistance along *bd* is neglected because the equation was intended for footings where $D < B$.

It is evident that Eq. (31.1) is only valid for the case of general shear failure because no soil compression is allowed before the failure occurs.

Meyerhof [45,48], Hansen [35], and Vesic [68,69] further extended Terzaghi's bearing capacity equation to account for footing shape (s_i), footing embedment depth (d_1), load inclination or eccentricity (i_i), sloping ground (g_i), and tilted base (b_i). Chen [26] reevaluated N factors in Terzaghi's equation using the limit analysis method. These efforts resulted in significant extensions of Terzaghi's bearing capacity equation. The general form of the bearing capacity equation [35,68,69] can be expressed as

$$q_{\text{ult}} = cN_c s_c d_c i_c g_c b_c + \bar{q} N_q s_q d_q b_q + 0.5\gamma B N_\gamma s_\gamma d_\gamma i_\gamma g_\gamma b_\gamma \tag{31.2}$$

when $\phi = 0$,

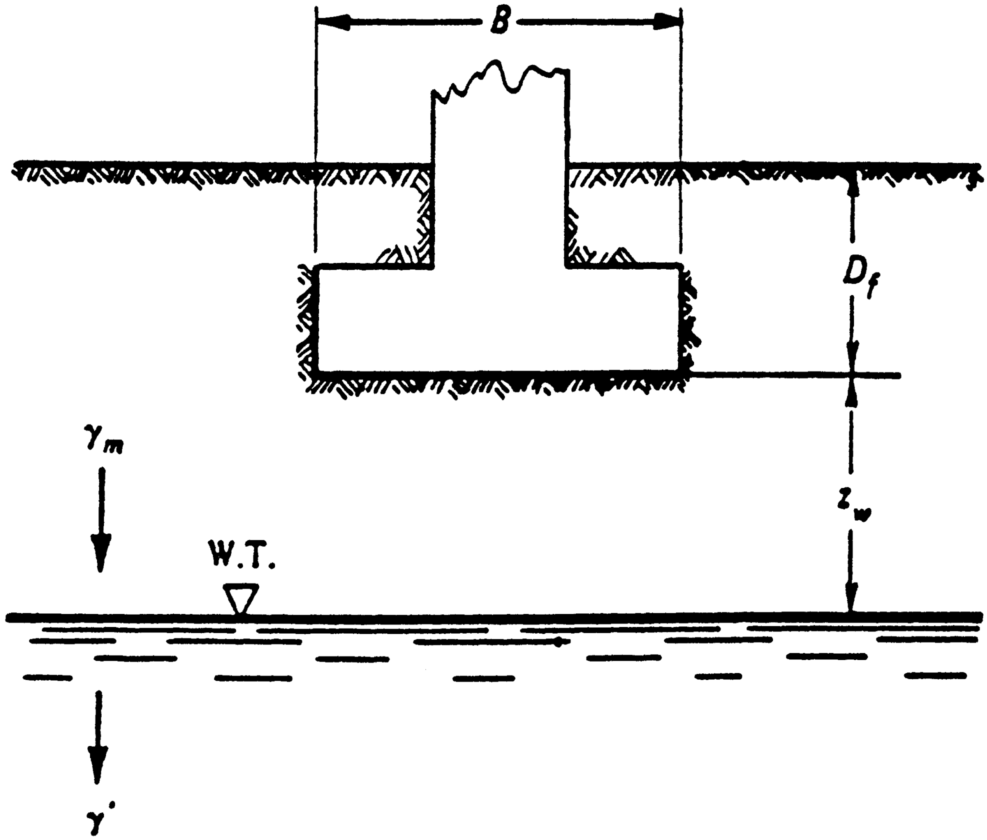


FIGURE 31.4 Influence of groundwater table on bearing capacity. (After AASHTO, 1997.)

$$q_{\text{ult}} = 5.14s_u\left(1 + s_c' + d_c' - i_c' - b_c' - g_c'\right) + \bar{q} \tag{31.3}$$

where s_u is undrained shear strength of cohesionless. Values of bearing capacity factors N_c, N_q, and N_γ can be found in Table 31.4. Values of other factors are shown in Table 31.5. As shown in Table 31.4, N_c and N_q are the same as proposed by Meyerhof [48], Hansen [35], Vesic [68], or Chen [26]. Nevertheless, there is a wide range of values for N_γ as suggested by different authors. Meyerhof [48] and Hansen [35] use the plain-strain value of ϕ, which may be up to 10% higher than those from the conventional triaxial test. Vesic [69] argued that a shear failure in soil under the footing is a process of progressive rupture at variable stress levels and an average mean normal stress should be used for bearing capacity computations. Another reason causing the N_γ value to be unsettled is how to evaluate the impact of the soil compressibility on bearing capacity computations. The value of N_γ still remains controversial because rigorous theoretical solutions are not available. In addition, comparisons of predicted solutions against model footing test results are inconclusive.

Soil Density

Bearing capacity equations are established based on the failure mode of general shearing. In order to use the bearing capacity equation to consider the other two modes of failure, Terzaghi [63] proposed a method to reduce strength characteristics c and ϕ as follows:

$$c^* = 0.67c \quad \text{(for soft to firm clay)} \tag{31.4}$$

TABLE 31.4 Bearing Capacity Factors for Eqs. (31.2) and (31.3)

ϕ	N_c	N_q	$N_{\gamma(M)}$	$N_{\gamma(H)}$	$N_{\gamma(V)}$	$N_{\gamma(C)}$	N_q/N_c	tan ϕ
0	5.14	1.00	0.00	0.00	0.00	0.00	0.19	0.00
1	5.38	1.09	0.00	0.00	0.07	0.07	0.20	0.02
2	5.63	1.20	0.01	0.01	0.15	0.16	0.21	0.03
3	5.90	1.31	0.02	0.02	0.24	0.25	0.22	0.05
4	6.18	1.43	0.04	0.05	0.34	0.35	0.23	0.07
5	6.49	1.57	0.07	0.07	0.45	0.47	0.24	0.09
6	6.81	1.72	0.11	0.11	0.57	0.60	0.25	0.11
7	7.16	1.88	0.15	0.16	0.71	0.74	0.26	0.12
8	7.53	2.06	0.21	0.22	0.86	0.91	0.27	0.14
9	7.92	2.25	0.28	0.30	1.03	1.10	0.28	0.16
10	8.34	2.47	0.37	0.39	1.22	1.31	0.30	0.18
11	8.80	2.71	0.47	0.50	1.44	1.56	0.31	0.19
12	9.28	2.97	0.60	0.63	1.69	1.84	0.32	0.21
13	9.81	3.26	0.74	0.78	1.97	2.16	0.33	0.23
14	10.37	3.59	0.92	0.97	2.29	2.52	0.35	0.25
15	10.98	3.94	1.13	1.18	2.65	2.94	0.36	0.27
16	11.63	4.34	1.37	1.43	3.06	3.42	0.37	0.29
17	12.34	4.77	1.66	1.73	3.53	3.98	0.39	0.31
18	13.10	5.26	2.00	2.08	4.07	4.61	0.40	0.32
19	13.93	5.80	2.40	2.48	4.68	5.35	0.42	0.34
20	14.83	6.40	2.87	2.95	5.39	6.20	0.43	0.36
21	15.81	7.07	3.42	3.50	6.20	7.18	0.45	0.38
22	16.88	7.82	4.07	4.13	7.13	8.32	0.46	0.40
23	18.05	8.66	4.82	4.88	8.20	9.64	0.48	0.42
24	19.32	9.60	5.72	5.75	9.44	11.17	0.50	0.45
25	20.72	10.66	6.77	6.76	10.88	12.96	0.51	0.47
26	22.25	11.85	8.00	7.94	12.54	15.05	0.53	0.49
27	23.94	13.20	9.46	9.32	14.47	17.49	0.55	0.51
28	25.80	14.72	11.19	10.94	16.72	20.35	0.57	0.53
29	27.86	16.44	13.24	12.84	19.34	23.71	0.59	0.55
30	30.14	18.40	15.67	15.07	22.40	27.66	0.61	0.58
31	32.67	20.63	18.56	17.69	25.99	32.33	0.63	0.60
32	35.49	23.18	22.02	20.79	30.21	37.85	0.65	0.62
33	38.64	26.09	26.17	24.44	35.19	44.40	0.68	0.65
34	42.16	29.44	31.15	28.77	41.06	52.18	0.70	0.67
35	46.12	33.30	37.15	33.92	48.03	61.47	0.72	0.70
36	50.59	37.75	44.43	40.05	56.31	72.59	0.75	0.73
37	55.63	42.92	53.27	47.38	66.19	85.95	0.77	0.75
38	61.35	48.93	64.07	56.17	78.02	102.05	0.80	0.78
39	67.87	55.96	77.33	66.75	92.25	121.53	0.82	0.81
40	75.31	64.19	93.69	79.54	109.41	145.19	0.85	0.84
41	83.86	73.90	113.98	95.05	130.21	174.06	0.88	0.87
42	93.71	85.37	139.32	113.95	155.54	209.43	0.91	0.90
43	105.11	99.01	171.14	137.10	186.53	253.00	0.94	0.93
44	118.37	115.31	211.41	165.58	224.63	306.92	0.97	0.97
45	133.87	134.97	262.74	200.81	271.74	374.02	1.01	1.00
46	152.10	158.50	328.73	244.64	330.33	458.02	1.04	1.04
47	173.64	187.20	414.32	299.52	403.65	563.81	1.08	1.07
48	199.26	222.30	526.44	368.66	495.99	697.93	1.12	1.11
49	229.92	265.49	674.91	456.40	613.13	869.17	1.15	1.15
50	266.88	319.05	873.84	568.56	762.85	1089.46	1.20	1.19

Note: N_c and N_q are same for all four methods; subscripts identify author for N_γ: M = Meyerhof [48]; H = Hansen [35]; V = Vesic [69]; C = Chen [26].

TABLE 31.5 Shape, Depth, Inclination, Ground, and Base Factors for Eq. (31.3)

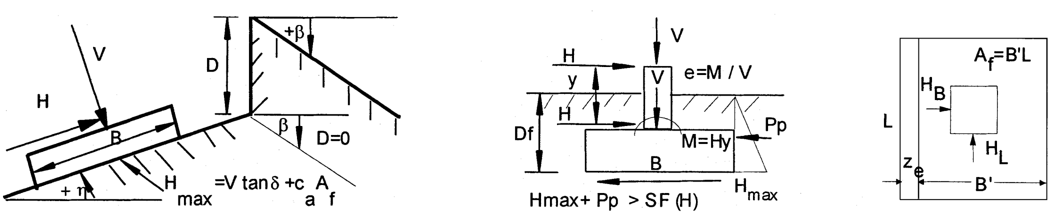
Shape Factors	Depth Factors
$s_c = 1.0 + \frac{N_q}{N_c}\frac{B}{L}$ $s_c = 1.0$ (for strip footing)	$d_c = 1.0 + 0.4k \begin{cases} k = \frac{D_f}{B} & \text{for } \frac{D_f}{B} \le 1 \\ k = \tan^{-1}\left(\frac{D_f}{B}\right) & \text{for } \frac{D_f}{B} > 1 \end{cases}$
$s_q = 1.0 + \frac{B}{L}\tan\phi$ (for all ϕ)	$d_q = 1 + 2\tan\phi\,(1-\sin\phi)^2 k$ (k defined above)
$s_\gamma = 1.0 - 0.4\frac{B}{L} \ge 0.6$	$d_\gamma = 1.00$ (for all ϕ)
Inclination Factors	**Ground Factors (base on slope)**
$i_c' = 1 - \frac{mHi}{A_f c_a N_c}$ $(\phi = 0)$	$g_c' = \frac{\beta}{5.14}$ β in radius $(\phi = 0)$
$i_c = i_q - \frac{1-i_q}{N_q - 1}$ $(\phi > 0)$	$g_c = i_q - \frac{1-i_q}{5.14\tan\phi}$ $(\phi > 0)$
$i_q = \left[1.0 - \frac{H_i}{V + A_f c_a \cot\phi}\right]^m$	$g_q = g_\gamma = (1.0 - \tan\beta)^2$ (for all ϕ)
	Base Factors (tilted base)
$i_\gamma = \left[1.0 - \frac{H_i}{V + A_f c_a \cot\phi}\right]^{m+1}$	$b_c' = g_c'$ $(\phi = 0)$
$m = m_B = \frac{2 + B/L}{1 + B/L}$ or	$b_c = 1 - \frac{2\beta}{5.14\tan\phi}$ $(\phi > 0)$
$m = m_L = \frac{2 + L/B}{1 + L/B}$	$b_q = b_\gamma = (1.0 - \eta\tan\phi)^2$ (for all ϕ)

Notes:
1. When $\gamma = 0$ (and β 'ne 0) use $N_\gamma = 2\sin(\pm\beta)$ in N_γ term
2. Compute $m = m_B$ when $H_i = H_B$ (H parallel to B) and $m = m_L$ when $H_i = H_L$ (H parallel to L); for both H_B and H_L use

$$m = \sqrt{m_B^2 + m_L^2}$$

3. $0 \le i_q, i_\gamma \le 1$
4. $\beta + \eta \le 90°;\ \beta \le \phi$

where
A_f = effective footing dimension as shown in Figure 31.6
D_f = depth from ground surface to base of footing
V = vertical load on footing
H_i = horizontal component of load on footing with $H_{max} \le V\tan\delta + c_a A_f$
c_a = adhesion to base $(0.6c \le c_a \le 1.0c)$
δ = friction angle between base and soil $(0.5\phi \le \delta \le \phi)$
β = slope of ground away from base with (+) downward
η = tilt angle of base from horizontal with (+) upward



After Vesic [68,69].

$$\phi^* = \tan^{-1}(0.67 \tan \phi) \quad (\text{for loose sands with } \phi < 28°) \tag{31.5}$$

Vesic [69] suggested that a flat reduction of ϕ might be too conservative in the case of local and punching shear failure. He proposed the following equation for a reduction factor varying with relative density D_r:

$$\phi^* = \tan^{-1}\left(\left(0.67 + D_r - 0.75D_r^2\right) \tan \phi\right) \quad (\text{for } 0 < D_r < 0.67) \tag{31.6}$$

Groundwater Table

Ultimate bearing capacity should always be estimated by assuming the highest anticipated groundwater table. The effective unit weight γ_e shall be used in the qN_q and $0.5\gamma B$ terms. As illustrated in Figure 31.5, the weighted average unit weight for the $0.5\gamma B$ term can be determined as follows:

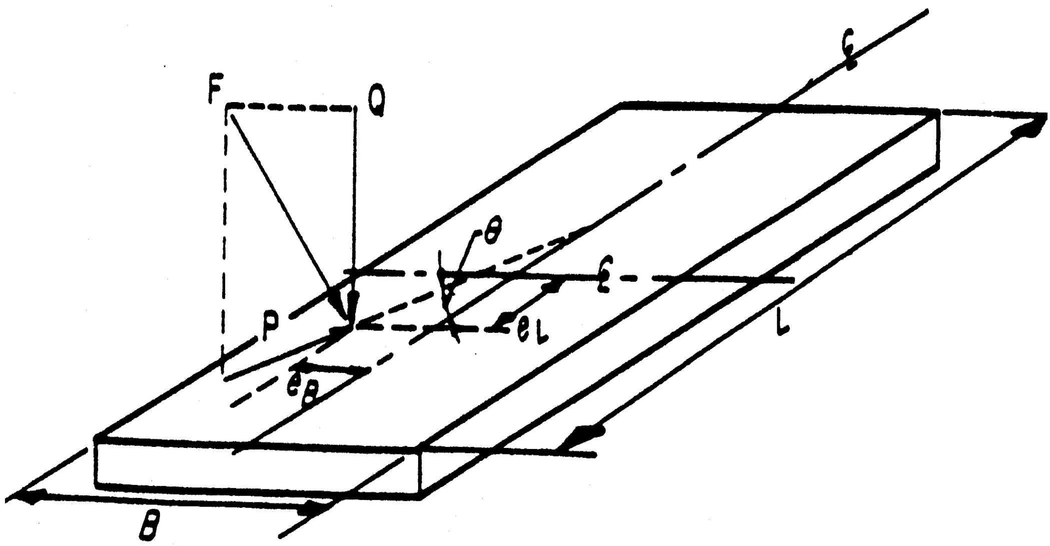


FIGURE 31.5 Definition sketch for loading and dimensions for footings subjected to eccentric or inclined loads. (After AASHTO, 1997.)

$$\gamma = \begin{cases} \gamma_{\text{avg}} & \text{for } d_w \geq B \\ \gamma' + (d_w/B)(\gamma_{\text{avg}} - \gamma') & \text{for } 0 < d_w < B \\ \gamma' & \text{for } d \leq 0 \end{cases} \tag{31.7}$$

Eccentric Load

For footings with eccentricity, effective footing dimensions can be determined as follows:

$$A_f = B'L' \tag{31.8}$$

where $L = L - 2e_L$ and $B = B - 2e_B$. Refer to Figure 31.5 for loading definitions and footing dimensions. For example, the actual distribution of contact pressure for a rigid footing with eccentric loading in the L direction (Figure 31.6) can be obtained as follows:

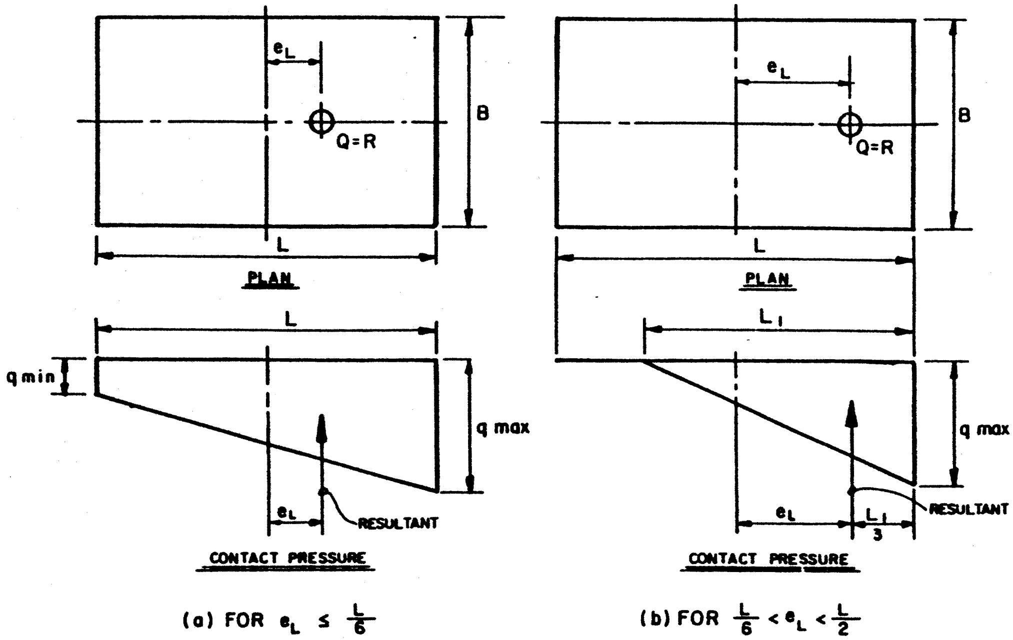


FIGURE 31.6 Contact pressure for footing loaded eccentrically about one axis. (After AASHTO 1997.)

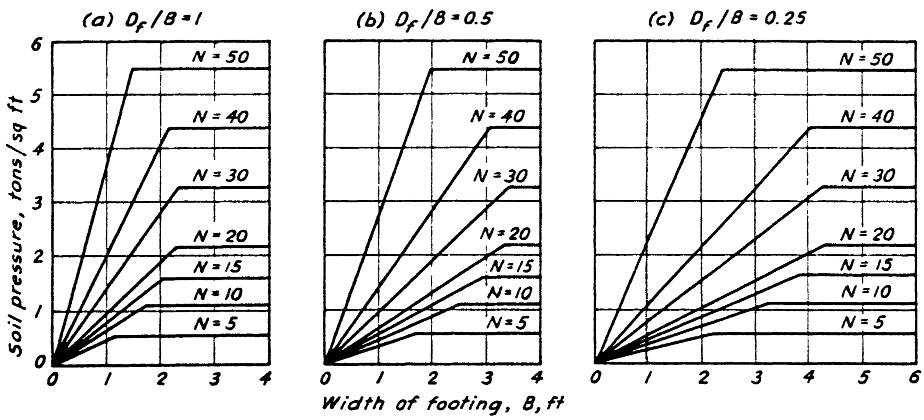


FIGURE 31.7 Design chart for proportioning shallow footings on sand. (a) Rectangular base; (b) round base. (After Peck et al. [53])

$$q_{\substack{\max\\\min}} = P\left[1 \pm 6e_L/L\right]/BL \quad \left(\text{for } e_L < L/6\right) \tag{31.9}$$

$$q_{\substack{\max\\\min}} = \begin{cases} 2P/\left[3B\left(L/2 - e_L\right)\right] \\ 0 \end{cases} \quad \left(\text{for } L/6 < e_L < L/2\right) \tag{31.10}$$

Contact pressure for footings with eccentric loading in the B direction may be determined using above equations by replacing terms L with B and terms B with L. For an eccentricity in both directions, reference is available in AASHTO [2,3].

31.4.2 Bearing Capacity on Sand from Standard Penetration Tests (SPT)

Terzaghi and Peck [64,65] proposed a method using SPT blow counts to estimate ultimate bearing capacity for footings on sand. Modified by Peck et al. [53], this method is presented in the form of the chart shown in Figure 31.7. For a given combination of footing width and SPT blow counts, the chart can be used to determine the ultimate bearing pressure associated with 25.4 mm (1.0 in.) settlement. The design chart applies to shallow footings ($D_f \leq B$) sitting on sand with water table at great depth. Similarly, Meyerhof [46] published the following formula for estimating ultimate bearing capacity using SPT blow counts:

$$q_{\text{ult}} = N'_{\text{avg}} \frac{B}{10}\left(C_{w1} + C_{w2}\frac{D_f}{B}\right)R_I \tag{31.11}$$

where R_I is a load inclination factor shown in Table 31.6 (R_I = 1.0 for vertical loads). C_{w1} and C_{w2} are correction factors whose values depend on the position of the water table:

TABLE 31.6 Load Inclination Factor (R_I)

For Square Footings			
	Load Inclination Factor (R_I)		
H/V	$D_f/B=0$	$D_f/B=1$	$D_f/B=3$
0.10	0.75	0.80	0.85
0.15	0.65	0.75	0.80
0.20	0.55	0.65	0.70
0.25	0.50	0.55	0.65
0.30	0.40	0.50	0.55
0.35	0.35	0.45	0.50
0.40	0.30	0.35	0.45
0.45	0.25	0.30	0.40
0.50	0.20	0.25	0.30
0.55	0.15	0.20	0.25
0.60	0.10	0.15	0.20

For Rectangular Footings						
	Load Inclination Factor (R_I)					
H/H	$D_f/B=0$	$D_f/B=1$	$D_f/B=5$	$D_f/B=0$	$D_f/B=1$	$D_f/B=5$
0.10	0.70	0.75	0.80	0.80	0.85	0.90
0.15	0.60	0.65	0.70	0.70	0.80	0.85
0.20	0.50	0.60	0.65	0.65	0.70	0.75
0.25	0.40	0.50	0.55	0.55	0.65	0.70
0.30	0.35	0.40	0.50	0.50	0.60	0.65
0.35	0.30	0.35	0.40	0.40	0.55	0.60
0.40	0.25	0.30	0.35	0.35	0.50	0.55
0.45	0.20	0.25	0.30	0.30	0.45	0.50
0.50	0.15	0.20	0.25	0.25	0.35	0.45
0.55	0.10	0.15	0.20	0.20	0.30	0.40
0.60	0.05	0.10	0.15	0.15	0.25	0.35

After Barker et al. [9].

$$\begin{cases} C_{w1} = C_{w2} = 0.5 & \text{for } D_w = 0 \\ C_{w1} = C_{w2} = 1.0 & \text{for } D_w \geq D_f = 1.5B \\ C_{w1} = 0.5 \text{ and } C_{w2} = 1.0 & \text{for } D_w = D_f \end{cases} \tag{31.12}$$

N'_{avg} is an average value of the SPT blow counts, which is determined within the range of depths from footing base to 1.5B below the footing. In very fine or silty saturated sand, the measured SPT blow count (N) is corrected for submergence effect as follows:

$$N' = 15 + 0.5(N - 15) \quad \text{for } N > 15 \tag{31.13}$$

31.4.3 Bearing Capacity from Cone Penetration Tests (CPT)

Meyerhof [46] proposed a relationship between ultimate bearing capacity and cone penetration resistance in sands:

$$q_{\text{ult}} = q_c \frac{B}{40}\left(C_{w1} + C_{w2}\frac{D_f}{B}\right)R_I \tag{31.14}$$

where q_c is the average value of cone penetration resistance measured at depths from footing base to 1.5B below the footing base. C_{w1}, C_{w2}, and R_1 are the same as those as defined in Eq. (31.11).

Schmertmann [57] recommended correlated values of ultimate bearing capacity to cone penetration resistance in clays as shown in Table 31.7.

TABLE 31.7 Correlation between Ultimate Bearing Capacity (q^{ult}) and Cone Penetration Resistance (q_c)

	qult (ton/ft²)	
q_c (kg/cm² or ton/ft²)	Strip Footings	Square Footings
10	5	9
20	8	12
30	11	16
40	13	19
50	15	22

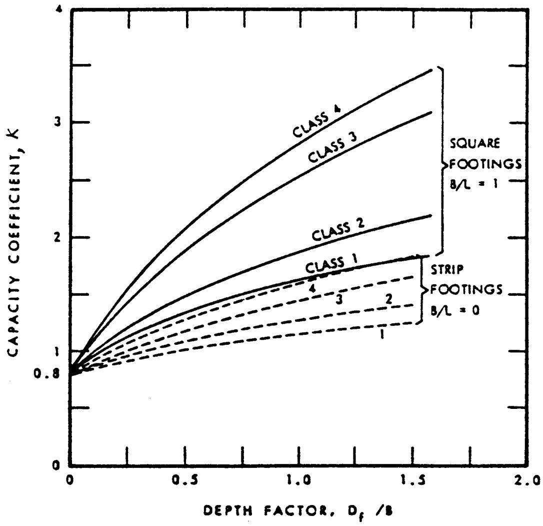
After Schmertmann [57] and Awkati, 1970.

31.4.4 Bearing Capacity from Pressure-Meter Tests (PMT)

Menard [44], Baguelin et al. [8], and Briaud [15,17] proposed using the limit pressure measured in PMT to estimate ultimate bearing capacity:

$$q_{\text{ult}} = r_0 + \kappa(p_1 - p_0) \tag{31.15}$$

where r_0 is the initial total vertical pressure at the foundation level, κ is the dimensionless bearing capacity coefficient from Figure 31.8, p_1 is limit pressure measured in PMT at depths from 1.5B above to 1.5B below foundation level, and p_0 is total horizontal pressure at the depth where the PMT is performed.



Soil Type	Consistency or Density	$(P_l - P_0)$ (tsf)	Class
clay	soft to very firm	<12	1
	stiff	8 – 40	2
sand and gravel	loose	4-8	2
	medium to dense	10-20	3
	very dense	30-60	4
silt	loose to medium	<7	1
	dense	12-30	2
rock	very low strength	10-30	2
	low strength	30-60	3
	medium to high strength	$60\text{-}100^{+}$	4

FIGURE 31.8 Values of empirical capacity coefficient, κ. (After Canadian Geotechnical Society [24].)

31.4.5 Bearing Capacity According to Building Codes

Recommendations for bearing capacity of shallow foundations are available in most building codes. Presumptive value of allowable bearing capacity for spread footings are intended for preliminary design when site-specific investigation is not justified. Presumptive bearing capacities usually do not reflect the size, shape, and depth of footing, local water table, or potential settlement. Therefore, footing design using such a procedure could be either overly conservative in some cases or unsafe in others [9]. Recommended practice is to use presumptive bearing capacity as shown in Table 31.8 for preliminary footing design and to finalize the design using reliable methods in the preceding discussion.

31.4.6 Predicted Bearing Capacity vs. Load Test Results

Obviously, the most reliable method of obtaining the ultimate bearing capacity is to conduct a full-scale footing load test at the project site. Details of the test procedure have been standardized as ASTM D1194 [5]. The load test is not usually performed since it is very costly and not practical for routine design. However, using load test results to compare with predicted bearing capacity is a vital tool to verify the accuracy and reliability of various prediction procedures. A comparison between the predicted bearing capacity and results of eight load tests conducted by Milovic [49] is summarized in Table 31.9.

Recently, load testing of five large-scale square footings (1 to 3 m) on sand was conducted on the Texas A&M University National Geotechnical Experimental Site [94]. One of the main objects of the test is to evaluate the various procedures used for estimating bearing capacities and settlements of shallow foundations. An international prediction event was organized by ASCE Geotechnical Engineering Division, which received a total of 31 predictions (16 from academics and 15 from consultants) from Israel, Australia, Japan, Canada, the United States, Hong Kong, Brazil, France, and Italy. Comparisons of predicted and measured values of bearing capacity using various procedures were summarized in Tables 31.10 through 31.12. From those comparisons, it can be argued that the most accurate settlement prediction methods are the Schmertmann-DMT (1986) and the Peck and Bazarra (1967) although they are on the unconservative side. The most conservative

TABLE 31.8 Presumptive Values of Allowable Bearing Capacity for Spread Foundations

Type of Bearing Material	Consistency in Place	q_{all} (ton/ft^2) Range	Recommended Value for Use
Massive crystalline igneous and metamorphic rock: granite, diorite, basalt, gneiss, thoroughly cemented conglomerate (sound condition allows minor cracks)	Hard sound rock	60–100	80
Foliated metamorphic rock: slate, schist (sound condition allows minor cracks)	Medium-hard sound rock	30–40	35
Sedimentary rock: hard cemented shales, siltstone, sandstone, limestone without cavities	Medium-hard sound rock	15–25	20
Weathered or broken bedrock of any kind except highly argillaceous rock (shale); RQD less than 25	Soft rock	8–12	10
Compaction shale or other highly argillaceous rock in sound condition	Soft rock	8–12	10
Well-graded mixture of fine and coarse-grained soil: glacial till, hardpan, boulder clay (GW-GC, GC, SC)	Very compact	8–12	10
Gravel, gravel–sand mixtures, boulder gravel mixtures (SW, SP)	Very compact	6–10	7
	Medium to compact	4–7	5
	Loose	2–5	3
Coarse to medium sand, sand with little gravel (SW, SP)	Very compact	4–6	4
	Medium to compact	2–4	3
	Loose	1–3	1.5
Fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC)	Very compact	3–5	3
	Medium to compact	2–4	2.5
	Loose	1–2	1.5
Homogeneous inorganic clay, sandy or silty clay (CL, CH)	Very stiff to hard	3–6	4
	Medium to stiff	1–3	2
	Soft	0.5–1	0.5
Inorganic silt, sandy or clayey silt, varved silt-clay-fine sand	Very stiff to hard	2–4	3
	Medium to stiff	1–3	1.5
	Soft	0.5–1	0.5

Notes:
1. Variations of allowable bearing pressure for size, depth, and arrangement of footings are given in Table 2 of NAFVAC [52].
2. Compacted fill, placed with control of moisture, density, and lift thickness, has allowable bearing pressure of equivalent natural soil.
3. Allowable bearing pressure on compressible fine-grained soils is generally limited by considerations of overall settlement of structure.
4. Allowable bearing pressure on organic soils or uncompacted fills is determined by investigation of individual case.
5. If tabulated recommended value for rock exceeds unconfined compressive strength of intact specimen, allowable pressure equals unconfined compressive strength.

After NAVFAC [52].

methods are Briaud [15] and Burland and Burbidge [20]. The most accurate bearing capacity prediction method was the $0.2q_c$ (CPT) method [16].

TABLE 31.9 Comparison of Computed Theoretical Bearing Capacities and Milovic and Muh's Experimental Values

	Test							
Bearing Capacity Method	1	2	3	4	5	6	7	8
	D = 0.0 m	0.5	0.5	0.5	0.4	0.5	0.0	0.3
	B = 0.5 m	0.5	0.5	1.0	0.71	0.71	0.71	0.71
	L = 2.0 m	2.0	2.0	1.0	0.71	0.71	0.71	0.71
	γ = 15.69 kN/m^3	16.38	17.06	17.06	17.65	17.65	17.06	17.06
	ϕ = 37°(38.5°)	35.5 (36.25)	38.5 (40.75)	38.5	22	25	20	20
	c = 6.37 kPa	3.92	7.8	7.8	12.75	14.7	9.8	9.8
Milovic (tests)					q_{ult} (kg/cm^2) 4.1	5.5	2.2	2.6
Muhs (tests)	q_{ult} (kg/cm^2) 10.8	12.2	24.2	33.0				
Terzaghi	9.4*	9.2	22.9	19.7	4.3*	6.5*	2.5	2.9*
Meyerhof	8.2*	10.3	26.4	28.4	4.8	7.6	2.3	3.0
Hansen	7.2	9.8	23.7*	23.4	5.0	8.0	2.2*	3.1
Vesic	8.1	10.4*	25.1	24.7	5.1	8.2	2.3	3.2
Balla	14.0	15.3	35.8	33.0*	6.0	9.2	2.6	3.8

[a] After Milovic (1965) but all methods recomputed by author and Vesic added.

Notes:
1. ϕ = triaxial value ϕ_{tr}; (plane strain value) = 1.5 ϕ_{tr} - 17.
2. * = best: Terzaghi = 4; Hansen = 2; Vesic = 1; and Balla = 1.
Source: Bowles, J.E., *Foundation Analysis and Design*, 5th ed., McGraw-Hill, New York, 1996. With permission.

TABLE 31.10 Comparison of Measured vs. Predicted Load Using Settlement Prediction Method

	Predicted Load (MN) @ s = 25 mm				
Prediction Methods	1.0 m Footing	1.5 m Footing	2.5 m Footing	3.0 m(n) Footing	3.0 m(s) Footing
Briaud [15]	0.904	1.314	2.413	2.817	2.817
Burland and Burbidge [20]	0.699	1.044	1.850	2.367	2.367
De Beer (1965)	1.140	0.803	0.617	0.597	0.597
Menard and Rousseau (1962)	0.247	0.394	0.644	1.017	1.017
Meyerhof CPT (1965)	0.288	0.446	0.738	0.918	0.918
Meyerhof — SPT (1965)	0.195	0.416	1.000	1.413	1.413
Peck and Bazarra (1967)	1.042	1.899	4.144	5.679	5.679
Peck, Hansen & Thornburn [53]	0.319	0.718	1.981	2.952	2.952
Schmertmann CPT (1970)	0.455	0.734	1.475	1.953	1.953
Schmertmann DMT (1970)	1.300	2.165	4.114	5.256	5.256
Schultze and Sherif (1973)	1.465	2.615	4.750	5.850	5.850
Terzaghi and Peck [65]	0.287	0.529	1.244	1.476	1.476
Measured Load @ s = 25mm	0.850	1.500	3.600	4.500	4.500

Source: FHWA, Publication No. FHWA-RD-97-068, 1997.

31.5 Stress Distribution Due to Footing Pressures

Elastic theory is often used to estimate the distribution of stress and settlement as well. Although soils are generally treated as elastic–plastic materials, the use of elastic theory for solving the problems is mainly due to the reasonable match between the boundary conditions for most footings and those of elastic solutions [37]. Another reason is the lack of availability of acceptable alternatives. Observation and experience have shown that this practice provides satisfactory solutions [14,37,54,59].

TABLE 31.11 Comparison of Measured vs. Predicted Load Using Bearing Capacity Prediction Method

	Predicted Bearing Capacity (MN)				
Prediction Methods	1.1 m Footing	1.5 m Footing	2.6 m Footing	3.0m(n) Footing	3.0m(s) Footing
Briaud — CPT [16]	1.394	1.287	1.389	1.513	1.513
Briaud — PMT [15]	0.872	0.779	0.781	0.783	0.783
Hansen [35]	0.772	0.814	0.769	0.730	0.730
Meyerhof [45,48]	0.832	0.991	1.058	1.034	1.034
Terzaghi [63]	0.619	0.740	0.829	0.826	0.826
Vesic [68,69]	0.825	0.896	0.885	0.855	0.855
Measured Load @ s = 150 mm					

Source: FHWA, Publication No. FHWA-RD-97-068, 1997.

TABLE 31.12 Best Prediction Method Determination

		Mean Predicted Load/ Mean Measured Load
	Settlement Prediction Method	
1	Briaud [15]	0.66
2	Burland & Burbidge [20]	0.62
3	De Beer [29]	0.24
4	Menard and Rousseau (1962)	0.21
5	Meyerhof CPT (1965)	0.21
6	Meyerhof SPT (1965)	0.28
7	Peck and Bazarra (1967)	1.19
8	Peck, et al. [53]	0.57
9	Schmertmann — CPT [56]	0.42
10	Schmertmann — DMT [56]	1.16
11	Shultze and Sherif (1973)	1.31
12	Terzaghi and Peck [65]	0.32
	Bearing Capacity Prediction Method	
1	Briaud — CPT [16]	1.08
2	Briaud — PMT [15]	0.61
3	Hansen [35]	0.58
4	Meyerhof [45,48]	0.76
5	Terzaghi [63]	0.59
6	Vesic [68,69]	0.66

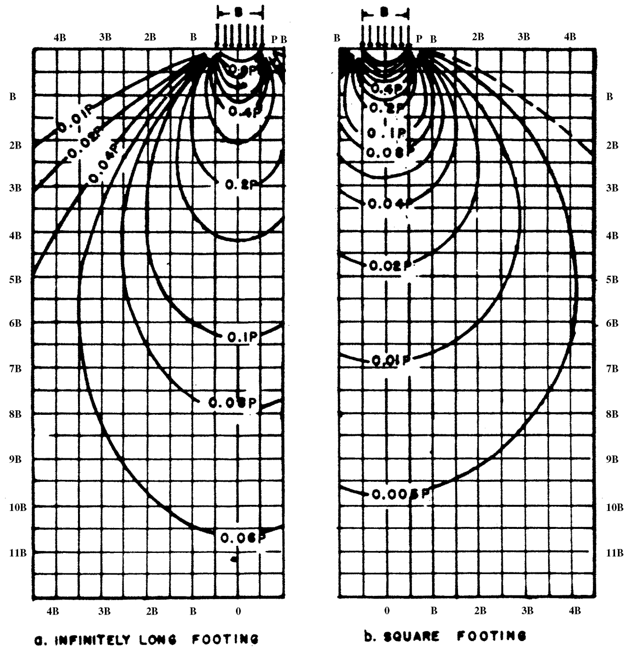
Source: FHWA, Publication No. FHWA-RD-97-068, 1997.

31.5.1 Semi-infinite, Elastic Foundations

Bossinesq equations based on elastic theory are the most commonly used methods for obtaining subsurface stresses produced by surface loads on semi-infinite, elastic, isotropic, homogenous, weightless foundations. Formulas and plots of Bossinesq equations for common design problems are available in NAVFAC [52]. Figure 31.9 shows the isobars of pressure bulbs for square and continuous footings. For other geometry, refer to Poulos and Davis [55].

31.5.2 Layered Systems

Westergaard [70], Burmister [21-23], Sowers and Vesic [62], Poulos and Davis [55], and Perloff [54] discussed the solutions to stress distributions for layered soil strata. The reality of interlayer shear is very complicated due to *in situ* nonlinearity and material inhomogeneity [37,54]. Either zero (frictionless) or with perfect fixity is assumed for the interlayer shear to obtain possible



SQUARE FOOTING

GIVEN

FOOTING SIZE = 20'X 20'

UNIT PRESSURE P = 2 TSF

FIND

PROFILE OF STRESS INCREASE BENEATH CENTER OF FOOTING DUE TO APPLIED LOAD

B = 20' P = 2 TSF

z (FT)	z/B	σ_z TSF		
10	0.5	0.70 X 2	=	1.4
20	1	0.38 X 2	=	0.76
30	1.5	0.19 X 2	=	0.38
40	2.0	0.12 X 2	=	0.24
50	2.5	0.07 X 2	=	0.14
60	3.0	0.05 X 2	=	0.10

FIGURE 31.9 Pressure bulbs based on the Bossinesq equation for square and long footings. (After NAVFAC 7.01, 1986].)

solutions. The Westergaard method assumed that the soil being loaded is constrained by closed spaced horizontal layers that prevent horizontal displacement [52]. Figures 31.10 through 31.12 by the Westergaard method can be used for calculating vertical stresses in soils consisting of alternative layers of soft (loose) and stiff (dense) materials.

31.5.3 Simplified Method (2:1 Method)

Assuming a loaded area increasing systemically with depth, a commonly used approach for computing the stress distribution beneath a square or rectangle footing is to use the 2:1 slope method as shown in Figure 31.13. Sometimes a 60° distribution angle (1.73–to–1 slope) may be assumed. The pressure increase Δq at a depth z beneath the loaded area due to base load P is

$$\Delta q = \begin{cases} P/(B+z)(L+z) & \text{(for a rectangle footing)} \\ P/(B+z)^2 & \text{(for a square footing)} \end{cases} \tag{31.16}$$

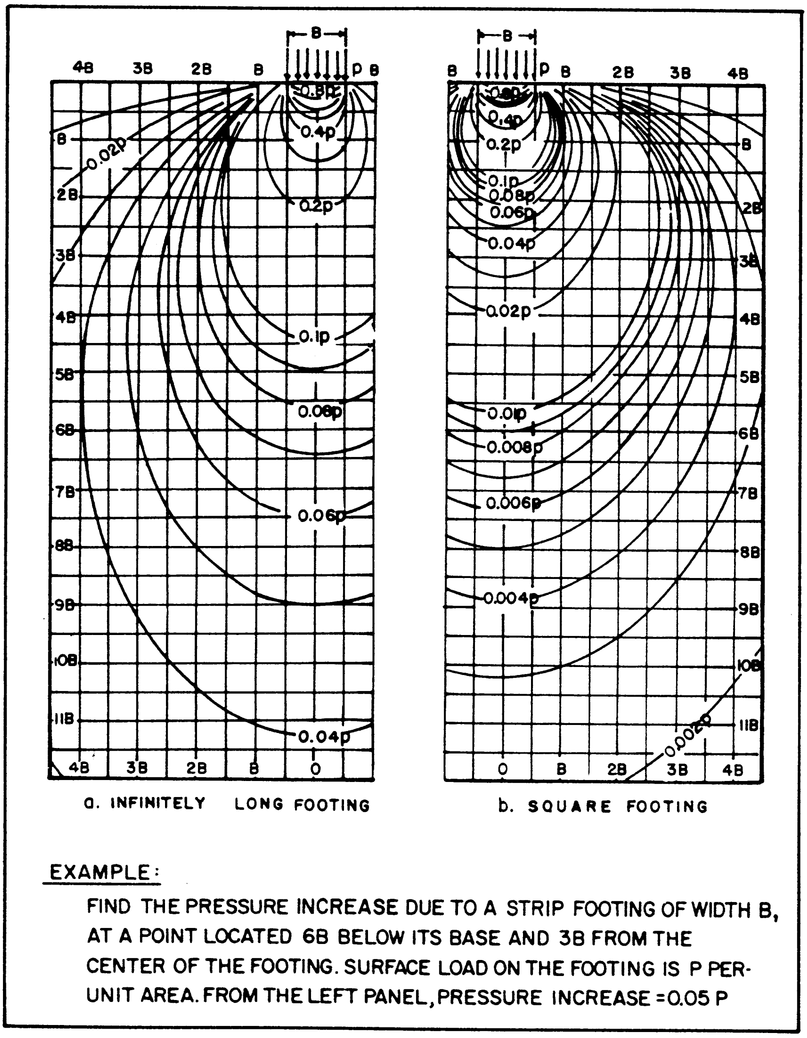


FIGURE 31.10 Vertical stress contours for square and strip footings [Westerqaard Case]. (After NAVFAC 7.01, 1986.)

where symbols are referred to Figure 31.14. The solutions by this method compare very well with those of more theoretical equations from depth z from B to about $4B$ but should not be used for depth z from 0 to B [14]. A comparison between the approximate distribution of stress calculated by a theoretical method and the 2:1 method is illustrated in Figure 31.15.

31.6 Settlement of Shallow Foundations

The load applied on a footing changes the stress state of the soil below the footing. This stress change may produce a time-dependent accumulation of elastic compression, distortion, or consolidation of the soil beneath the footing. This is often termed *foundation settlement*. True elastic deformation consists of a very small portion of the settlement while the major components of the settlement are due to a change of void ratio, particle rearrangement, or crushing. Therefore, very little of the settlement will be recovered even if the applied load is removed. The irrecoverable

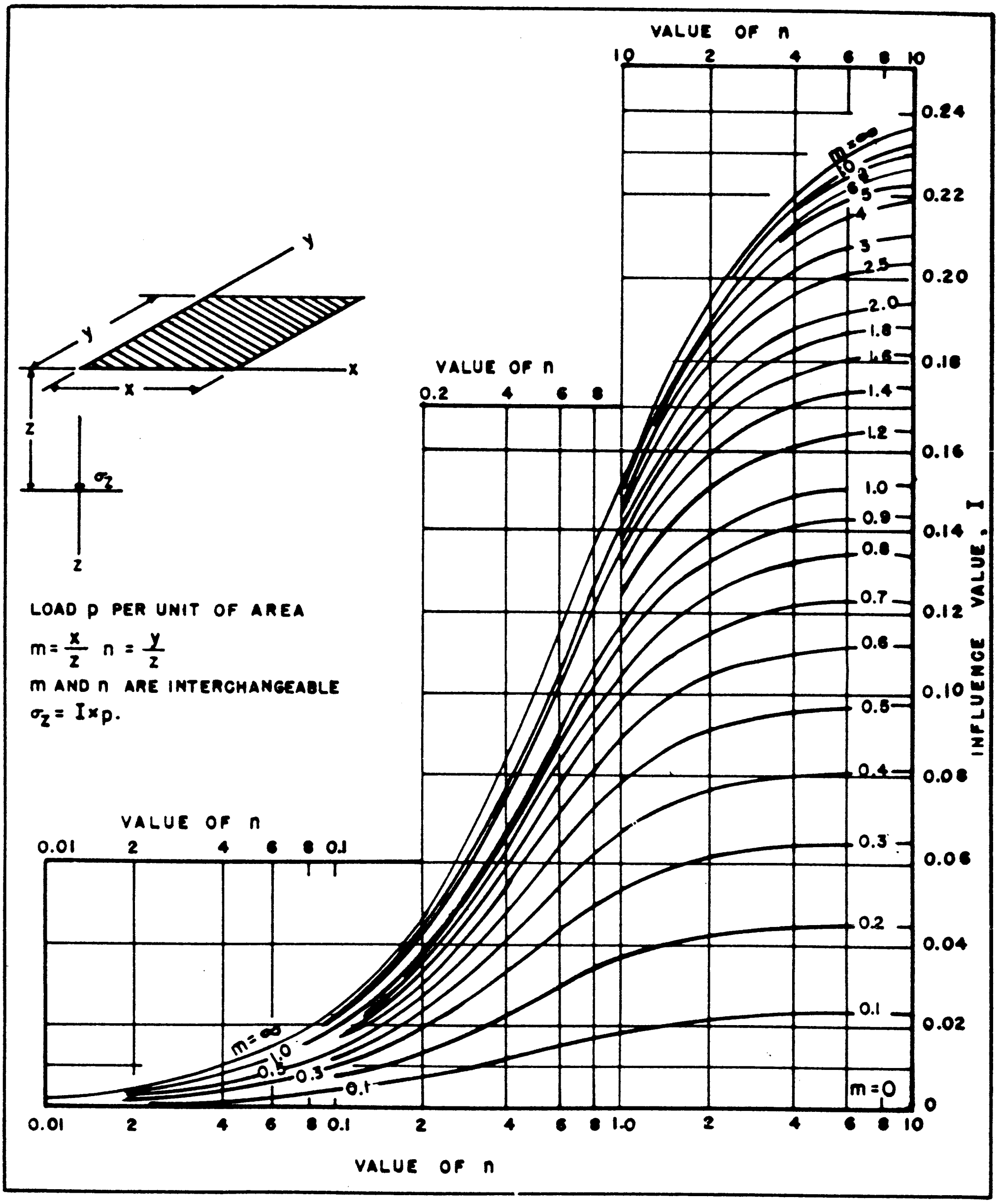


FIGURE 31.11 Influence value for vertical stress beneath a corner of a uniformly loaded rectangular area (Westerqaard Case). (After NAVFAC [52].)

deformation of soil reflects its inherent elastic–plastic stress–strain relationship. The reliability of settlement estimated is influenced principally by soil properties, layering, stress history, and the actual stress profile under the applied load [14,66]. The total settlement may be expressed as

$$s = s_i + s_c + s_s \tag{31.17}$$

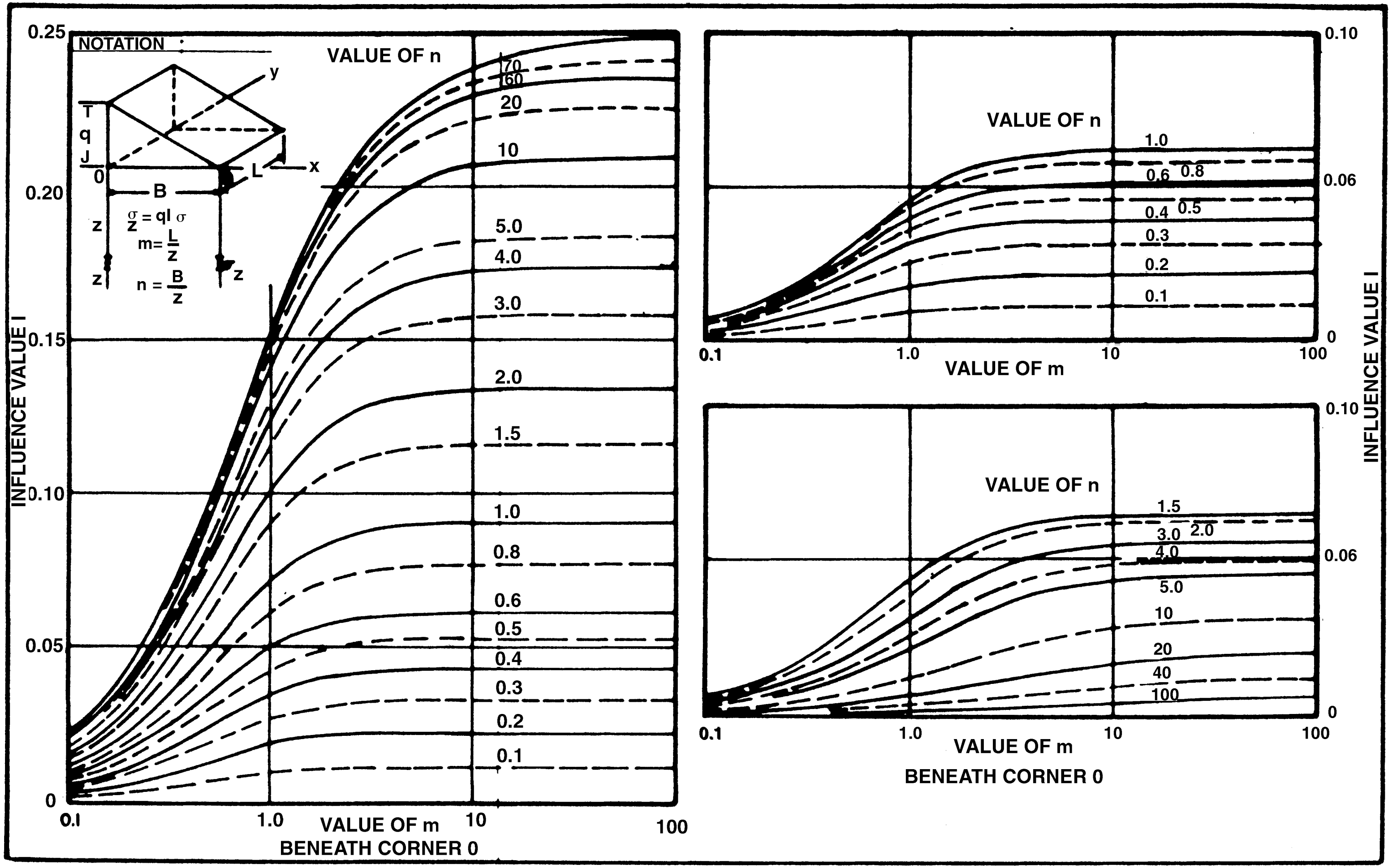


FIGURE 31.12 Influence value for vertical stress beneath triangular load (Westerqaard Case). (After NAVFAC [52].)

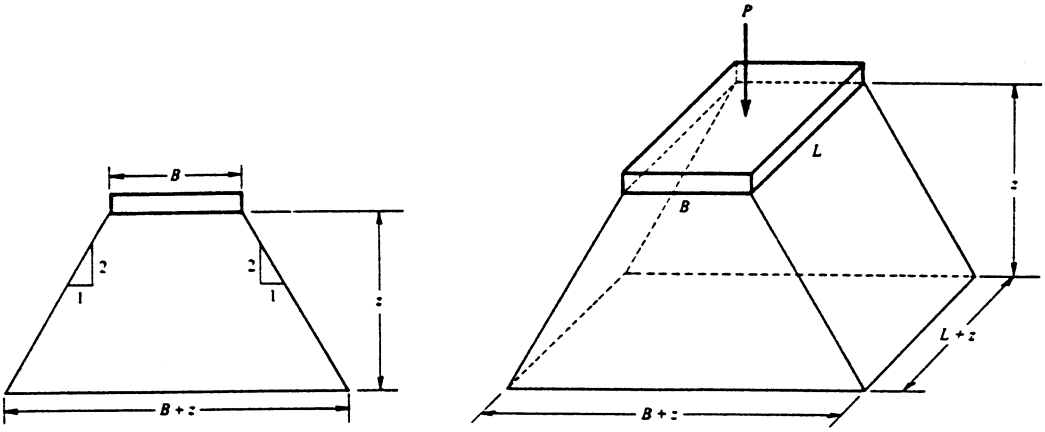


FIGURE 31.14 Approximate distribution of vertical stress due to surface load. (After Perloff [54].)

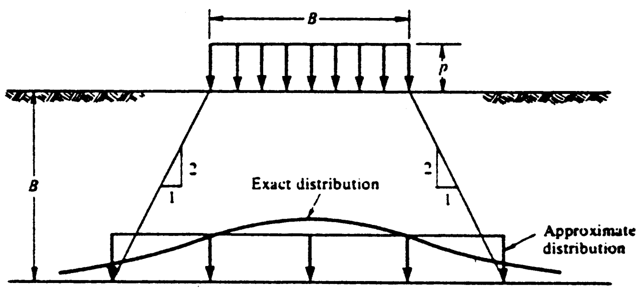


FIGURE 31.15 Relationship between vertical stress below a square uniformly loaded area as determined by approximate and exact methods. (After Perloff [54].)

where s is the total settlement, s_i is the immediate or distortion settlement, s_c is the primary consolidation settlement, and s_s is the secondary settlement. The time-settlement history of a shallow foundation is illustrated in Figure 31.15. Generally speaking, immediate settlement is not elastic. However, it is often referred to as elastic settlement because the elastic theory is usually used for computation. The immediate settlement component controls in cohesionless soils and unsaturated cohesive soils, while consolidation compression dictates in cohesive soils with a degree of saturation above 80% [3].

31.6.1 Immediate Settlement by Elastic Methods

Based on elastic theory, Steinbrenner [61] suggested that immediate settlements of footings on sands and clay could be estimated in terms of Young's modulus E of soils. A modified procedure developed by Bowles [14] may be used for computing settlements of footings with flexible bases on the half-space. The settlement equation can be expressed as follows

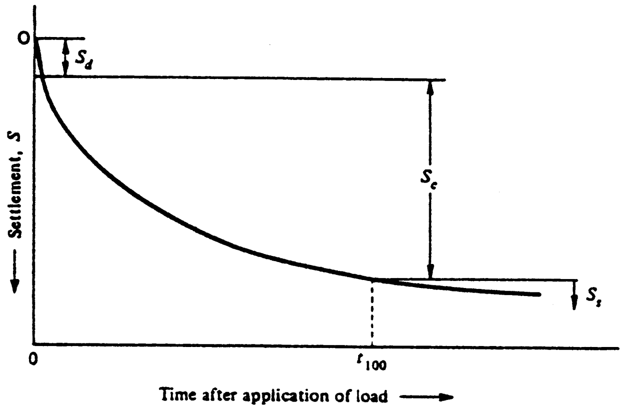


FIGURE 31.15 Schematic time–settlement history of typical point on a foundation. (After Perloff [54].)

$$s_i = q_0 B' \left(1 - \mu^2\right) m I_s I_F / E_s \tag{31.18}$$

$$I_s = n\,(I_1 + (1 - 2\mu)\, I_2 / (1 - \mu) \tag{31.19}$$

where q_0 is contact pressure, μ and E_s are weighted average values of Poisson's ratio and Young's modulus for compressive strata, B is the least-lateral dimension of contribution base area (convert round bases to equivalent square bases; $B = 0.5B$ for center and $B = B$ for corner I_i; $L' = 0.5L$ for center and $L' = L$ for corner I_i), I_i are influence factors depending on dimension of footings, base embedment depth, thickness of soil stratum, and Poisson's ratio (I_1 and I_2 are given in Table 31.13 and I_F is given in Figure 31.16; $M = L'/B'$ and $N = H/B'$), H is the stratum depth causing settlement (see discussion below), m is number of corners contributing to settlement (m = 4 at the footing center; m = 2 at a side; and m = 1 at a corner), and n equals 1.0 for flexible footings and 0.93 for rigid footings.

This equation applies to soil strata consisting of either cohesionless soils of any water content or unsaturated cohesive soils, which may be either organic or inorganic. Highly organic soils (both E_s and μ are subject to significant changes by high organic content) will be dictated by secondary or creep compression rather than immediate settlement; therefore, the applicability of the above equation is limited.

Suggestions were made by Bowles [14] to use the equations appropriately as follows: 1. Make the best estimate of base contact pressure q_0; 2. Identify the settlement point to be calculated and divide the base (as used in the Newmark stress method) so the point is at the corner or common corner of one or up to four contributing areas; 3. Determine the stratum depth causing settlement which does not approach to infinite rather at either the depth $z = 5B$ or depth to where a hard stratum is encountered (where F_s in the hard layer is about $10E_s$ of the adjacent upper layer); and 4. Calculate the weighted average E_s as follows:

$$E_{s,\,\text{avg}} = \sum_n^1 H_i E_{si} \Big/ \sum_n^1 H_i \tag{31.20}$$

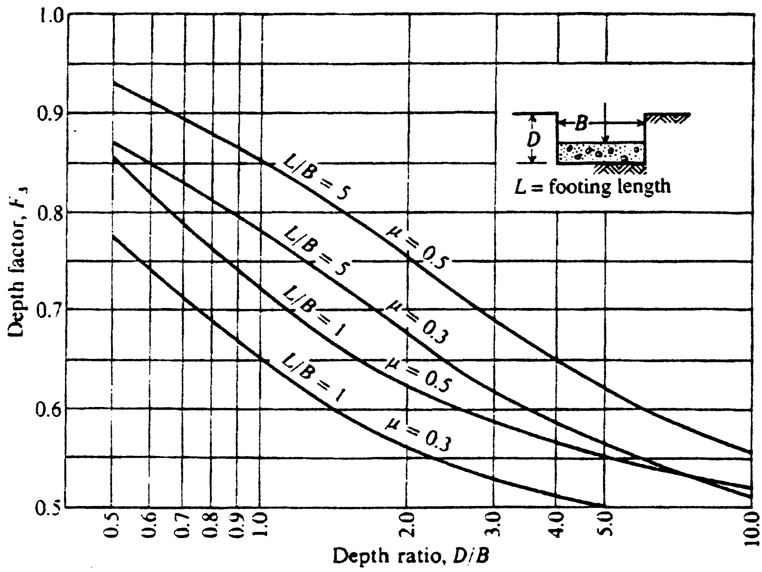


FIGURE 31.16 Influence factor I_F for footing at a depth D (use actual footing width and depth dimension for this D/B ratio). (After Bowles [14].)

31.6.2 Settlement of Shallow Foundations on Sand

SPT Method

D'Appolonio et al. [28] developed the following equation to estimate settlements of footings on sand using SPT data:

$$s = \mu_0 \mu_1 \, pB/M \tag{31.21}$$

where μ_0 and μ_1 are settlement influence factors dependent on footing geometry, depth of embedment, and depth to the relative incompressible layer (Figure 31.17), p is average applied pressure under service load and M is modulus of compressibility. The correlation between M and average SPT blow count is given in Figure 31.18.

Barker et al. [9] discussed the commonly used procedure for estimating settlement of footing on sand using SPT blow count developed by Terzaghi and Peck [64,65] and Bazaraa [10].

CPT Method

Schmertmann [56,57] developed a procedure for estimating footing settlements on sand using CPT data. This CPT method uses cone penetration resistance, q_c, as a measure of the *in situ* stiffness (compressibility) soils. Schmertmann's method is expressed as follows

$$s = C_1 C_2 \Delta p \Sigma \left(I_Z / E_s\right)_i \Delta z_i \tag{31.22}$$

$$C_1 = 1 - 0.5\left(\frac{\sigma'_{v0}}{\Delta p}\right) \geq 0.5 \tag{31.23}$$

$$C_2 = 1 + 0.2 \log\left(t_{yr}/0.1\right) \tag{31.24}$$

TABLE 31.13 Values of I_2 and I_2 to Compute Influence Factors as Used in Eq. (31.21)

N	M = 1.0	1.1	1.2	1.3	1.4	1.5	1.6	1.7	1.8	1.9	2.0
0.2	I_1 = 0.009	0.008	0.008	0.008	0.008	0.008	0.007	0.007	0.007	0.007	0.007
	I_2 = 0.041	0.042	0.042	0.042	0.042	0.042	0.043	0.043	0.043	0.043	0.043
0.4	0.033	0.032	0.031	0.030	0.029	0.028	0.028	0.027	0.027	0.027	0.027
	0.066	0.068	0.069	0.070	0.070	0.071	0.071	0.072	0.072	0.073	0.073
0.6	0.066	0.064	0.063	0.061	0.060	0.059	0.058	0.057	0.056	0.056	0.055
	0.079	0.081	0.083	0.085	0.087	0.088	0.089	0.090	0.091	0.091	0.092
0.8	0.104	0.102	0.100	0.098	0.096	0.095	0.093	0.092	0.091	0.090	0.089
	0.083	0.087	0.090	0.093	0.095	0.097	0.098	0.100	0.101	0.102	0.103
1.0	0.142	0.140	0.138	0.136	0.134	0.132	0.130	0.129	0.127	0.126	0.125
	0.083	0.088	0.091	0.095	0.098	0.100	0.102	0.104	0.106	0.108	0.109
1.5	0.224	0.224	0.224	0.223	0.222	0.220	0.219	0.217	0.216	0.214	0.213
	0.075	0.080	0.084	0.089	0.093	0.096	0.099	0.102	0.105	0.108	0.110
2.0	0.285	0.288	0.290	0.292	0.292	0.292	0.292	0.292	0.291	0.290	0.289
	0.064	0.069	0.074	0.078	0.083	0.086	0.090	0.094	0.097	0.100	0.102
3.0	0.363	0.372	0.379	0.384	0.389	0.393	0.396	0.398	0.400	0.401	0.402
	0.048	0.052	0.056	0.060	0.064	0.068	0.071	0.075	0.078	0.081	0.084
4.0	0.408	0.421	0.431	0.440	0.448	0.455	0.460	0.465	0.469	0.473	0.476
	0.037	0.041	0.044	0.048	0.051	0.054	0.057	0.060	0.063	0.066	0.069
5.0	0.437	0.452	0.465	0.477	0.487	0.496	0.503	0.510	0.516	0.522	0.526
	0.031	0.034	0.036	0.039	0.042	0.045	0.048	0.050	0.053	0.055	0.058
6.0	0.457	0.474	0.489	0.502	0.514	0.524	0.534	0.542	0.550	0.557	0.563
	0.026	0.028	0.031	0.033	0.036	0.038	0.040	0.043	0.045	0.047	0.050
7.0	0.471	0.490	0.506	0.520	0.533	0.545	0.556	0.566	0.575	0.583	0.590
	0.022	0.024	0.027	0.029	0.031	0.033	0.035	0.037	0.039	0.041	0.043
8.0	0.482	0.502	0.519	0.534	0.549	0.561	0.573	0.584	0.594	0.602	0.611
	0.020	0.022	0.023	0.025	0.027	0.029	0.031	0.033	0.035	0.036	0.038
9.0	0.491	0.511	0.529	0.545	0.560	0.574	0.587	0.598	0.609	0.618	0.627
	0.017	0.019	0.021	0.023	0.024	0.026	0.028	0.029	0.031	0.033	0.034
10.0	0.498	0.519	0.537	0.554	0.570	0.584	0.597	0.610	0.621	0.631	0.641
	0.016	0.017	0.019	0.020	0.022	0.023	0.025	0.027	0.028	0.030	0.031
20.0	0.529	0.553	0.575	0.595	0.614	0.631	0.647	0.662	0.677	0.690	0.702
	0.008	0.009	0.010	0.010	0.011	0.012	0.013	0.013	0.014	0.015	0.016
500	0.560	0.587	0.612	0.635	0.656	0.677	0.696	0.714	0.731	0.748	0.763
	0.000	0.000	0.000	0.000	0.000	0.000	0.001	0.001	0.001	0.001	0.001
0.2	I_1 = 0.007	0.006	0.006	0.006	0.006	0.006	0.006	0.006	0.006	0.006	0.006
	I_2 = 0.043	0.044	0.044	0.044	0.044	0.044	0.044	0.044	0.044	0.044	0.044
0.4	0.026	0.024	0.024	0.024	0.024	0.024	0.024	0.024	0.024	0.024	0.024
	0.074	0.075	0.075	0.075	0.076	0.076	0.076	0.076	0.076	0.076	0.076
0.6	0.053	0.051	0.050	0.050	0.050	0.049	0.049	0.049	0.049	0.049	0.049
	0.094	0.097	0.097	0.098	0.098	0.098	0.098	0.098	0.098	0.098	0.098
0.8	0.086	0.082	0.081	0.080	0.080	0.080	0.079	0.079	0.079	0.079	0.079
	0.107	0.111	0.112	0.113	0.113	0.113	0.113	0.114	0.114	0.014	0.014
1.0	0.121	0.115	0.113	0.112	0.112	0.112	0.111	0.111	0.110	0.110	0.110
	0.114	0.120	0.122	0.123	0.123	0.124	0.124	0.124	0.125	0.125	0.125
1.5	0.207	0.197	0.194	0.192	0.191	0.190	0.190	0.189	0.188	0.188	0.188
	0.118	0.130	0.134	0.136	0.137	0.138	0.138	0.139	0.140	0.140	0.140
2.0	0.284	0.271	0.267	0.264	0.262	0.261	0.260	0.259	0.257	0.256	0.256
	0.114	0.131	0.136	0.139	0.141	0.143	0.144	0.145	0.147	0.147	0.148
3.0	0.402	0.392	0.386	0.382	0.378	0.376	0.374	0.373	0.368	0.367	0.367
	0.097	0.122	0.131	0.137	0.141	0.144	0.145	0.147	0.152	0.153	0.154
4.0	0.484	0.484	0.479	0.474	0.470	0.466	0.464	0.462	0.453	0.451	0.451
	0.082	0.110	0.121	0.129	0.135	0.139	0.142	0.145	0.154	0.155	0.156
5.0	0.553	0.554	0.552	0.548	0.543	0.540	0.536	0.534	0.522	0.519	0.519
	0.070	0.098	0.111	0.120	0.128	0.133	0.137	0.140	0.154	0.156	0.157
6.0	0.585	0.609	0.610	0.608	0.604	0.601	0.598	0.595	0.579	0.576	0.575
	0.060	0.087	0.101	0.111	0.120	0.126	0.131	0.135	0.153	0.157	0.157

TABLE 31.13 (continued) Values of I_1 and I_2 to Compute Influence Factors as Used in Eq. (31.21)

N	M = 1.0	1.1	1.2	1.3	1.4	1.5	1.6	1.7	1.8	1.9	2.0
7.0	0.618	0.653	0.658	0.658	0.656	0.653	0.650	0.647	0.628	0.624	0.623
	0.053	0.078	0.092	0.103	0.112	0.119	0.125	0.129	0.152	0.157	0.158
8.0	0.643	0.688	0.697	0.700	0.700	0.698	0.695	0.692	0.672	0.666	0.665
	0.047	0.071	0.084	0.095	0.104	0.112	0.118	0.124	0.151	0.156	0.158
9.0	0.663	0.716	0.730	0.736	0.737	0.736	0.735	0.732	0.710	0.704	0.702
	0.042	0.064	0.077	0.088	0.097	0.105	0.112	0.118	0.149	0.156	0.158
10.0	0.679	0.740	0.758	0.766	0.770	0.770	0.770	0.768	0.745	0.738	0.735
	0.038	0.059	0.071	0.082	0.091	0.099	0.106	0.122	0.147	0.156	0.158
20.0	0.756	0.856	0.896	0.925	0.945	0.959	0.969	0.977	0.982	0.965	0.957
	0.020	0.031	0.039	0.046	0.053	0.059	0.065	0.071	0.124	0.148	0.156
500.0	0.832	0.977	1.046	1.102	1.150	1.191	1.227	1.259	2.532	1.721	1.879
	0.001	0.001	0.002	0.002	0.002	0.002	0.003	0.003	0.008	0.016	0.031

Source: Bowles, J.E., *Foundation Analysis and Design*, 5th ed., McGraw-Hill, New York, 1996. With permission.

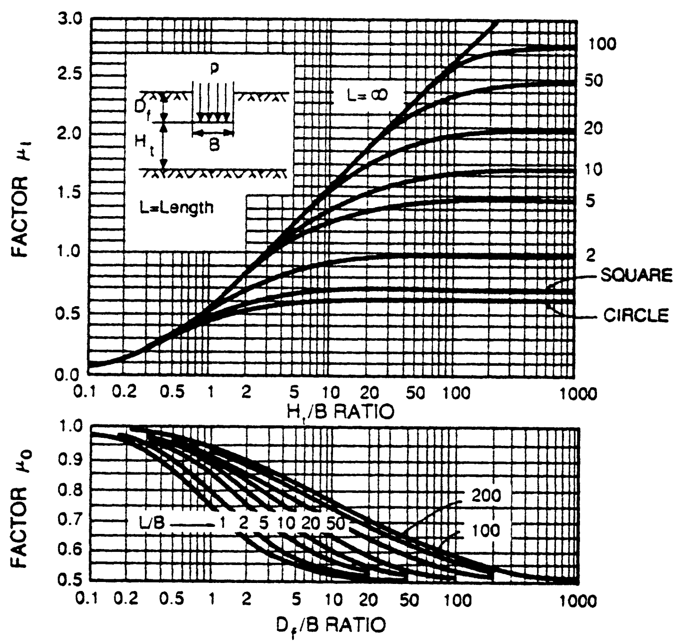


FIGURE 31.17 Settlement influence factors μ_0 and μ_1 for the D'Appolonia et al. procedure. (After D'Appolonia et al [28].)

$$E_s = \begin{cases} 2.5q_c & \text{for square footings (axisymmetric conditions)} \\ 3.5q_c & \text{for continuous footings with } L/B \geq 10 \text{ (plan strain conditions)} \\ \left[2.5 + (L/B - 1)/9\right]q_c & \text{for footings with } 1 \geq L/B \geq 10 \end{cases} \quad (31.25)$$

where $\Delta p = \sigma'_{vf} - \sigma'_{v0}$ is net load pressure at foundation level, σ'_{v0} is initial effective *in situ* overburden stress at the bottom of footings, σ'_{vf} is final effective *in situ* overburden stress at the

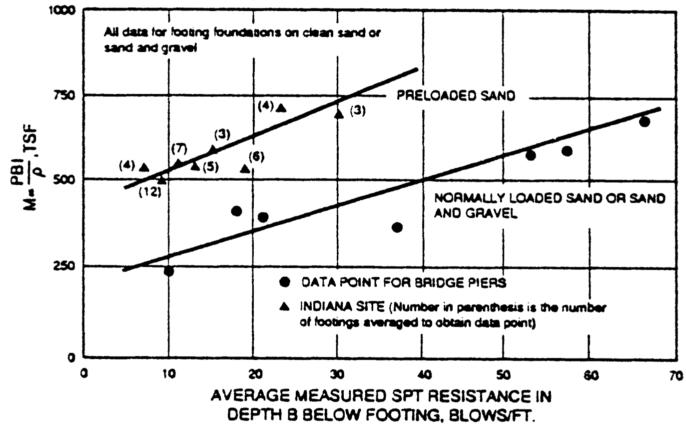


FIGURE 31.18 Correlation between modulus of compressibility and average value SPT blow count. (After D'Appolonia et al [28].)

bottom of footings, I_z is strain influence factor as defined in Figure 31.19 and Table 31.14, E_s is the appropriate Young's modulus at the middle of the ith layer of thickness Δz_1, C_1 is pressure correction factor, C_2 is time rate factor (equal to 1 for immediate settlement calculation or if the lateral pressure is less than the creep pressure determined from pressure-meter tests), q_c is cone penetration resistance, in pressure units, and Δz is layer thickness.

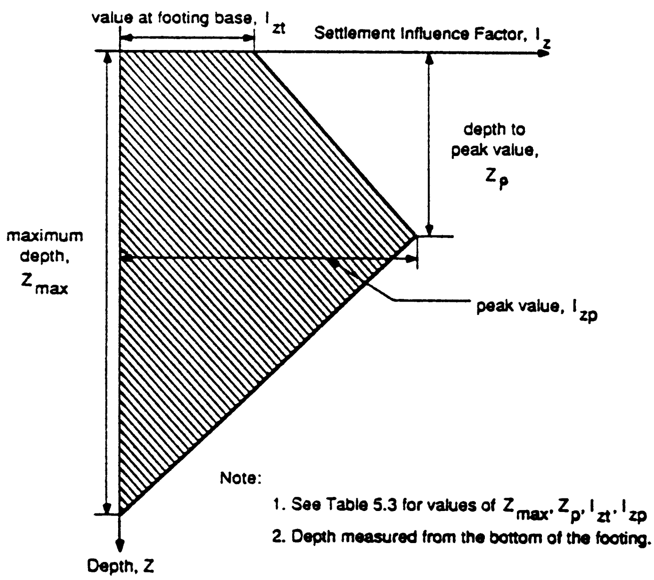


FIGURE 31.19 Variation of Schmertmann's improved settlement influence factors with depth. (After Schmertmann et al [58].)

Recent studies by Tan and Duncan [62] have compared measured settlements with settlements predicted using various procedures for footings on sand. These studies conclude that methods predicting settlements close to the average of measured settlement are likely to underestimate

TABLE 31.14 Coefficients to Define the Dimensions of Schmertmann's Improved Settlement Influence Factor Diagram in Figure 31.20

				Peak Value of Stress Influence Factor I_{zp}			
L/B	Max. Depth of Influence z_{max}/B	Depth to Peak Value z_p/B	Value of I_z at Top I_{zt}	$\frac{\Delta p}{\sigma'_{vp}} = 1$	$\frac{\Delta p}{\sigma'_{vp}} = 2$	$\frac{\Delta p}{\sigma'_{vp}} = 4$	$\frac{\Delta p}{\sigma'_{vp}} = 10$
1	2.00	0.50	0.10	0.60	0.64	0.70	0.82
2	2.20	0.55	0.11	0.60	0.64	0.70	0.82
4	2.65	0.65	0.13	0.60	0.64	0.70	0.82
8	3.55	0.90	0.18	0.60	0.64	0.70	0.82
≥10	4.00	1.00	0.20	0.60	0.64	0.70	0.82

Note: σ'_{vp} is the initial vertical pressure at depth of peak influence.

After Schmertmann et al. [57].

settlements half the time and to overestimate them half the time. The conservative methods (notably Terzaghi and Peck's) tend to overestimate settlements more than half the time and to underestimate them less often. On the other hand, there is a trade-off between accuracy and reliability. A relatively accurate method such as the D'Appolonia et al. method calculates settlements that are about equal to the average value of actual settlements, but it underestimates settlements half the time (a reliability of 50%). To ensure that the calculated settlements equal or exceed the measured settlements about 90% of the time (a reliability of 90%), an adjustment factor of two should be applied to the settlements predicted by the D'Appolonia et al. method. Table 31.15 shows values of the adjustment factor for 50 and 90% reliability in settlement predicted using Terzaghi and Peck, D'Appolonia et al., and Schmertmann methods.

TABLE 31.15 Value of Adjustment Factor for 50 and 90% Reliability in Displacement Estimates

		Adjustment Factor	
Method	Soil Type	For 50% Reliability	For 90% Reliability
Terzaghi and Peck [65]	Sand	0.45	1.05
Schmertmann	Sand	0.60	1.25
D'Appolonia et al. [28]	Sand	1.00	2.00

TABLE 31.16 Some Empirical Equations for C_c and C_α

Compression Index	Source	Comment
$C_c = 0.009(LL - 10)$	Terzaghi and Peck [65]	$S_t \leq 5$, $LL < 100$
$C_c = 0.2343e_0$	Nagaraj and Murthy [51]	
$C_c = 0.5G_s(PI/100)$	Worth and Wood [71]	Modified cam clay model
$C_c = 0PI/74$	EPRI (1990)	
$C_c = 0.37(e_0 + 0.003w_L + 0.0004w_N - 0.34)$	Azzouz et al. [7]	Statistical analysis
Recompression Index	**Source**	
$C_r = 0.0463w_LG_s$	Nagaraj and Murthy [50]	

31.6.3 Settlement of Shallow Foundations on Clay

Immediate Settlement

Immediate settlement of shallow foundations on clay can be estimated using the approach described in Section 31.6.1.

Consolidation Settlement

Consolidation settlement is time dependent and may be estimated using one-dimensional consolidation theory [43,53,66]. The consolidation settlement can be calculated as follows

$$s_c \begin{cases} \dfrac{H_c}{1+e_0}\left[C_r \log\left(\dfrac{\sigma'_p}{\sigma''_{vo}}\right) + C_c \log\left(\dfrac{\sigma'_{vf}}{\sigma'_p}\right)\right] & \left(\text{for OC soils, i.e., } \sigma'_p > \sigma'_{v0}\right) \\ \dfrac{H_c}{1+e_0} C_c \log\left(\dfrac{\sigma'_{vf}}{\sigma'_p}\right) & \left(\text{for NC soils, i.e., } \sigma'_p = \sigma'_{v0}\right) \end{cases} \tag{31.26}$$

where H_c is height of compressible layer, e_0 is void ratio at initial vertical effective stress, C_γ is recompression index (see Table 31.16), C_c is compression index (see Table 31.16), σ'_p is maximum past vertical effective stress, σ'_{v0} is initial vertical effective stress, σ'_{vf} is final vertical effective stress. Highly compressible cohesive soils are rarely chosen to place footings for bridges where tolerable amount of settlement is relatively small. Preloading or surcharging to produce more rapid consolidation has been extensively used for foundations on compressible soils [54]. Alternative foundation systems would be appropriate if large consolidation settlement is expected to occur.

TABLE 31.17 Secondary Compression Index

C_α/C_c	Material
0.02 ± 0.01	Granular soils including rockfill
0.03 ± 0.01	Shale and mudstone
0.04 ± 0.01	Inorganic clays and silts
0.05 ± 0.01	Organic clays and silts
0.06 ± 0.01	Peat and muskeg

Source: Terzaghi, I. et al., *Soil Mechanics in Engineering Practice*, 3rd ed., John Wiley & Sons, New York, 1996. With permission.

Secondary Settlement

Settlements of footings on cohesive soils continuing beyond primary consolidation are called secondary settlement. Secondary settlement develops at a slow and continually decreasing rate and may be estimated as follows:

$$s_s = C_\alpha H_t \log \frac{t_{sc}}{t_p} \tag{31.27}$$

where C_α is coefficient of secondary settlement (Table 31.17), H_t is total thickness of layers undergoing secondary settlement, t_{sc} is time for which secondary settlement is calculated (in years), and t_p is time for primary settlement (>1 year).

31.6.4 Tolerable Settlement

Tolerable movement criteria for foundation settlement should be established consistent with the function and type of structure, anticipated service life, and consequences of unacceptable movements on structure performance as outlined by AASHTO [3]. The criteria adopted by AASHTO considering the angular distortion (δ/l) between adjacent footings is as follows:

$$\frac{\delta}{l} \le \begin{cases} 0.008 & \text{for simple-span bridge} \\ 0.004 & \text{for continuous-span bridge} \end{cases} \tag{31.28}$$

where δ is differential settlement of adjacent footings and l is center–center spacing between adjacent footings. These (δ/l) limits are not applicable to rigid frame structures, which shall be designed for anticipated differential settlement using special analysis.

31.7 Shallow Foundations on Rock

Wyllie [72] outlines the following examinations which are necessary for designing shallow foundations on rock:

1. The bearing capacity of the rock to ensure that there will be no crushing or creep of material within the loaded zone;
2. Settlement of the foundation which will result from elastic strain of the rock, and possibly inelastic compression of weak seams within the volume of rock compressed by the applied load;
3. Sliding and shear failure of blocks of rock formed by intersecting fractures within the foundation.

This condition usually occurs where the foundation is located on a steep slope and the orientation of the fractures is such that the blocks can slide out of the free face.

31.7.1 Bearing Capacity According to Building Codes

It is common to use allowable bearing capacity for various rock types listed in building codes for footing design. As provided in Table 31.18, the bearing capacities have been developed based on rock strength from case histories and include a substantial factor of safety to minimize settlement.

31.7.2 Bearing Capacity of Fractured Rock

Various empirical procedures for estimating allowable bearing capacity of foundations on fractured rock are available in the literature. Peck et al. [53] suggested an empirical procedure for estimating allowable bearing pressures of foundations on jointed rock based on the RQD index. The predicted bearing capacities by this method shall be used with the assumption that the foundation settlement does not exceed 12.7 mm (0.5 in.) [53]. Carter and Kulhawy [25] proposed an empirical approach for estimating ultimate bearing capacity of fractured rock. Their method is based on the unconfined compressive strength of the intact rock core sample and rock mass quality.

Wyllie [72] detailed an analytical procedure for computing bearing capacity of fractured rock mass using Hoek–Brown strength criterion. Details of rational methods for the topic can also be found in Kulhawy and Goodman [42] and Goodman [32].

TABLE 31.18 Presumptive Bearing Pressures (tsf) for Foundations on Rock after Putnam, 1981

Code	Year[1]	Bedrock[2]	Sound Foliated Rock	Sound Sedimentary Rock	Soft Rock[3]	Soft Shale	Broken Shale
Baltimore	1962	100	35		10		
BOCA	1970	100	40	25	10	4	(4)
Boston	1970	100	50	10	10		1.5
Chicago	1970	100	100				(4)
Cleveland	1951/1969			25			
Dallas	1968	$0.2q_u$	$2q_u$	0.2qu	$0.2q_u$	$0.2q_u$	$0.2q_u$
Detroit	1956	100	100	9600	12	12	
Indiana	1967	$0.2q_u$	$2q_u$	0.2qu	$0.2q_u$	$0.2q_u$	$0.2q_u$
Kansas	1961/1969	$0.2q_u$	$2q_u$	0.2qu	$0.2q_u$	$0.2q_u$	$0.2q_u$
Los Angeles	1970	10	4	3	1	1	1
New York City	1970	60	60	60	8		
New York State		100	40	15			
Ohio	1970	100	40	15	10	4	
Philadelphia	1969	50	15	10–15	8		
Pittsburgh	1959/1969	25	25	25	8	8	
Richmond	1968	100	40	25	10	4	1.5
St. Louis	1960/1970	100	40	25	10	1.5	1.5
San Francisco	1969	3–5	3–5	3–5			
UBC	1970	$0.2q_u$	$2q_u$	$0.2q_u$	$0.2q_u$	$0.2q_u$	$0.2q_u$
NBC Canada	1970			100			
New South Wales, Australia	1974		33	13	4.5		

Notes:
1. Year of code or original year and date of revision.
2. Massive crystalline bedrock.
3. Soft and broken rock, not including shale.
4. Allowable bearing pressure to be determined by appropriate city official.
5. q_u = unconfined compressive strength.

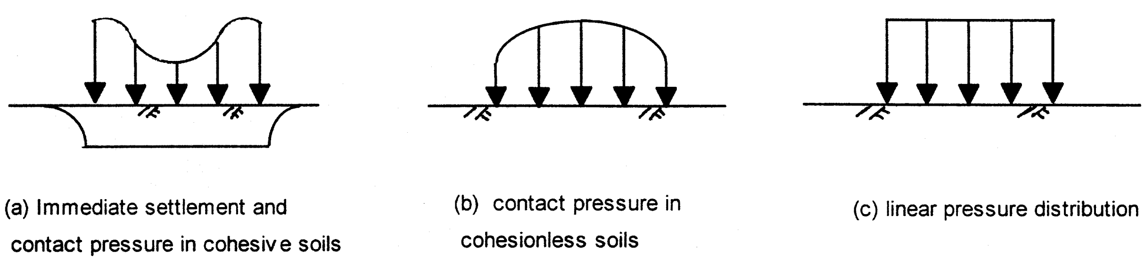


FIGURE 31.20 Contact pressure distribution for a rigid footing. (a) On cohesionless soils; (b) on cohesive soils; (c) usual assumed linear distribution.

31.7.3 Settlements of Foundations on Rock

Wyllie [72] summarizes settlements of foundations on rock as following three different types: 1. Elastic settlements result from a combination of strain of the intact rock, slight closure and movement of fractures and compression of any minor clay seams (less than a few millimeters). Elastic theory can be used to calculate this type of settlement. Detailed information can be found in Wyllie [72], Kulhawy, and AASHTO [3]. 2. Settlements result from the movement of blocks of rock due to shearing of fracture surfaces. This occurs when foundations are sitting at the top of a steep slope and unstable blocks of rocks are formed in the face. The stability of foundations on rock is influenced by the geologic characterization of rock blocks. The information required on structural geology consists of the orientation, length and spacing of fractures, and their surface and infilling materials. Procedures have been developed for identifying and analyzing the stability of sliding blocks [72], stability of wedge blocks [36], stability of toppling blocks [33], or three-dimensional stability of rock blocks [34]. 3. Time-dependent settlement occurs when foundations found on rock mass that consists of substantial seams of clay or other compressible materials. This type of settlement can be estimated using the procedures described in Section 31.6.3. Also time-dependent settlement can occur if foundations found on ductile rocks, such as salt where strains develop continuously at any stress level, or on brittle rocks when the applied stress exceeds the yield stress.

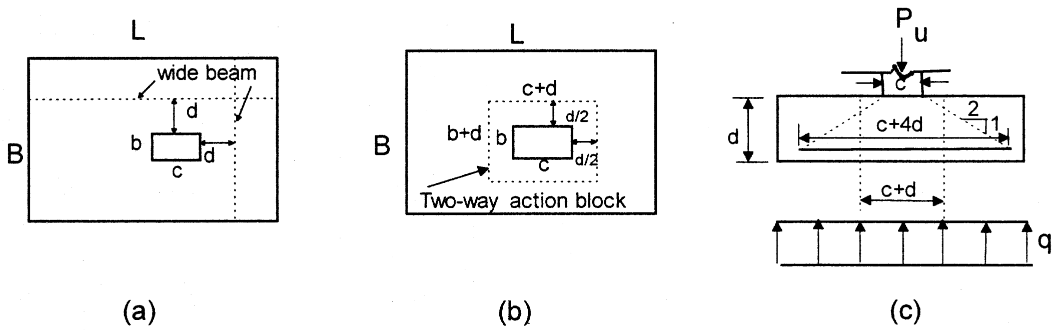


FIGURE 31.21 (a) Section for wide-beam shear; (b) section for diagonal-tension shear; (c) method of computing area for allowable column bearing stress.

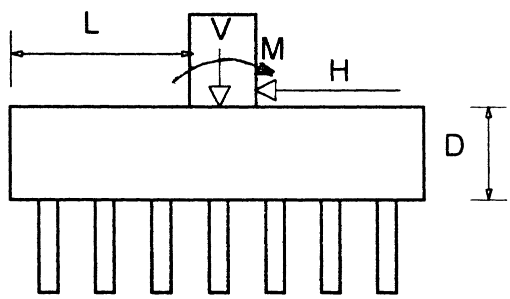


FIGURE 31.22 Illustration of the length-to-thickness ratio of cantilever of a footing or pile cap.

31.8 Structural Design of Spread Footings

The plan dimensions (*B* and *L*) of a spread footing are controlled by the allowable soil pressure beneath the footing. The pressure distribution beneath footings is influenced by the interaction of the footing rigidity with the soil type, stress–state, and time response to stress as shown in Figure 31.20 (a) (b). However, it is common practice to use the linear pressure distribution beneath rigid footings as shown in Figure 31.20 (c). The depth (*D*) for spread footings is usually controlled by shear stresses. Two-way action shear always controls the depth for centrally loaded square footings. However, wide-beam shear may control the depth for rectangular footings when the L/B ratio is greater than about 1.2 and may control for other L/B ratios when there is overturning or eccentric loading (Figure 31.21a). In addition, footing depth should be designed to satisfy diagonal (punching) shear requirement (Figure 31.21b). Recent studies by Duan and McBride [30] indicate that when the length-to-thickness ratio of cantilever (L/D as defined in Figure 31.22) of a footing (or pile-cap) is greater than 2.2, a nonlinear distribution of reaction should be used for footing or pile-cap design. The specifications and procedures for footing design can be found in AASHTO [2], ACI [4], or Bowles [12, 13].

Acknowledgment

I would like to take this opportunity to thank Bruce Kutter, who reviewed the early version of the chapter and provided many thoughtful suggestions. Advice and support from Prof. Kutter are greatly appreciated.

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