GEOSCIENCES TESTING AND RESEARCH, INC.

55 Middlesex Street, Suite 225, N. Chelmsford, MA 01863
Ph: (978)251-9395, Fx: (978)251-9396

REFERENCE:

Yu Fu Chapter 2 of Interim Progress Report to the National Cooperative Highway Research Program (NCHRP) project: NCHRP 12-66 AASHTO LRFD Specifications for Serviceability in the Design of Bridge Foundations.

By:

Samuel G. Paikowsky, GTR, N. Chelmsford, MA
Yusuke Honjo, Gifu University, Japan
Susan Faraji, Faraji Consulting, Inc., Winchester, MA
Ikumaso Yoshida, TEPSCO, Tokyo, Japan
Lu Ye, Geotechnical Engineering Research Laboratory, UMass Lowell

With contributions by:
Mary Canniff, Geosciences Testing and Research, Inc. (GTR), N. Chelmsford, MA
Yu Fu and Roiy Guy of the Geotechnical Engineering Research Laboratory, UMass Lowell
Guy Levi, Zeidan Ashraf and Wisam Mualem of the Technion – Israel Institute of Technology
Junichi Hyodo and Ikumasa Yoshida of the Tokyo Electric Power Services Co., Ltd. (TEPSCO), Tokyo, Japan
Masahiro Shirato of the Public Works Research Institute (PWRI), Ibaraki-ken, Japan

CHAPTER 2
SETTLEMENT ANALYSIS METHODS
LITERATURE REVIEW

2.1 OVERVIEW

Foundation settlements are estimated using deformation analyses based on the results of laboratory testing and/or in-situ testing. The soil parameters used in the analyses represent its deformability and are chosen to reflect the loading history of the soil, the construction sequence, and the effect of soil layering.

Both total and differential settlements, including time dependent effects, need to be considered. The total settlement includes immediate (i.e., elastic) and time dependent response (i.e., consolidation and secondary components), which can be expressed in the following way:

$$S_t = S_e + S_c + S_s \tag{2.1}$$

where S_e = elastic settlement
S_c = consolidation settlement
S_s = secondary settlement

Other factors can affect settlement, e.g., embankment loading and lateral and/or eccentric loading, and for footings on granular soil, vibration loading from dynamic live loads or earthquake loads should also be considered where appropriate.

Immediate settlement is often referred to as elastic settlement because of the method of computation, and the result of instantaneous distribution of the load and strain across the soil mass that occurs as the soil is loaded. In a nearly saturated or saturated cohesive soil, the applied load is initially carried by the pore water pressure. As the pore water is migrating in the soil due to its higher head, the load is transferred to the soil skeleton. Consolidation settlement is therefore the gradual compression of the soil skeleton as the pore water is following in the soil. Secondary settlement occurs as a result of the plastic deformation or creep of the soil skeleton under a constant effective stress.

The methods proposed by the existing AASHTO specifications for shallow foundations on granular materials are presented in the following sections along with other prevailing methods including those recommended by the FHWA.

The methods used for the estimation of settlements of footings on cohessionless soils can be broadly categorized as: (1) methods based on elastic theory, (2) methods utilizing in-situ Standard Penetration Test (SPT), (3) methods utilizing in-situ Core Penetration Test (CPT) and (4) methods that make use of Plate Load Tests.

2.2 METHODS BASED ON ELASTIC THEORY

2.2.1 AASHTO – LRFD Bridge Design Specifications (1998, 2007)

The following section is based on AASHTO LRFD specifications (1998). Examination of the AASHTO specification (2007) revealed that it is identical to the presented material other than change of units and at times the correction method for NSPT with effective confining stresses.

The elastic settlement of footings on cohessionless soil may be estimated using the following:

$$S_e = \frac{[q_0(1-\nu^2)\sqrt{A}]}{E_s \beta_z} \qquad (2.2)$$

where
- S_e = settlement (FT)
- q_0 = load intensity (TSF)
- A = area of footing (SF)
- E_s = Young's modulus of soil taken as specified in Table 2.1 in lieu of laboratory test results (TSF)
- β_z = shape factor taken as specified in Table 2.2 (DIM)
- ν = Poisson's Ratio taken as specified in Table 2.1 in lieu of laboratory test results (DIM)

Unless E_s varies significantly with depth, E_s should be determined at a depth of about 1/2 or 2/3 B below the footing. B being the smaller footing dimension. If the soil modulus varies significantly with depth, a weighted average value of E_s (Eq. 2.14) maybe used. Depending on the footing length (L) to width (B) ratio, the influence depth below the footing varies from 2B to 4B, which will be presented in Chapter 4, Table 4.2. Further, the calculation of corrected SPT-N value (N_1) mentioned in the following Table 2.1 is also presented in detail in Chapter 4. Corrected SPT-N values are obtained by using the corrected proposed by Liao and Whitman (1986), which is presented in Figure 2.11.

Calculate the effective vertical stress, σ'_{vo}, at the midpoint of each layer and obtain corrected SPT blowcounts for overburden stress using the relation in Figure 2.11.

Table 2.1 Elastic Constants of Various Soils Modified after U.S. Department of the Navy (1982) and Bowels (1988) (AASHTO Table 10.6.2.2.3b-1)

Soil Type	Typical Range of Values	Poisson's Ratio, ν	Estimating E_s from N	
	Young's Modulus, (tsf)	(dim)	Soil Type	E_s (tsf)
clay:				
soft sensitive	25-150	0.4-0.5	Silts, sandy silts, slightly cohesive mixtures	$4N_1$
Medium stiff to stiff	150-500	(undrained)	Clean fine to medium sands and slightly silty sands	$7N_1$
Very stiff	500-1000		Coarse sands and sand with little gravel	$10N_1$
			Sandy gravel and gravels	$12N_1$
Loss Silt	150-600	0.1-0.3	Sandy gravel and gravels	$12N_1$
	20-200	0.3-0.35		
Fine Sand:			**Estimating E_s from S_u**	
Loose	80-120		Soft sensitive clay	$400S_u$-$1{,}000S_u$
Medium dense	120-200	0.25	Medium stiff to stiff clay	$1{,}500S_u$-$2{,}400S_u$
Dense	200-300		Very stiff clay	$3{,}000S_u$-$4{,}000S_u$
Sand:				
Loose	100-300	0.20-0.35		
Medium dense	300-500			
Dense	500-800	0.30-0.40		
Gravel:			**Estimating Es from q_c**	
Loose	300-800	0.2-0.35		
Medium dense	800-1,000		Sandy Soil	$4q_c$
Dense	1,000-2,000	0.3-0.4		

Note:
- N = Standard Penetration Test (SPT) resistance (blows per ft)
- N_1 = SPT corrected for depth
- S_u = undrained shear strength (TSF)
- q_c = cone penetration resistance (TSF)

Table 2.2 Elastic Shape and Rigidity Factor, Kulhawy (1983)
(AASHTO Table 10.6.2.2.3b-2)

L/B	Flexible, β_z (Average)	β_z Rigid
Circular	1.04	1.13
1	1.06	1.08
2	1.09	1.1
3	1.13	1.15
5	1.22	1.24
10	1.41	1.41

For calculation of footing area when loads are eccentric to the centroid of the footing, a reduced effective area, $B' \times L'$, within the confines of the physical footing shall be used. The design bearing pressure on the effective area shall be assumed to be uniform. The reduced effective area shall be concentric with the load.

The reduced dimensions for an eccentrically loaded rectangular footing may be taken as:

$$B' = B - 2e_B \qquad (2.3)$$

$$L' = L - 2e_L \qquad (2.4)$$

where e_B = eccentrically parallel to dimension B (FT)
e_L = eccentrically parallel to dimension L (FT)

Footing under eccentric loads shall be designed to ensure that:

- The factored bearing resistance is not less then the effects of factor loads, and
- For footings on soils, the eccentricity of the footing evaluated based on factored loads, is less than ¼ of the corresponding footing dimension, B or L.

The reduced dimensions for rectangular and circular footings are as shown in Figure 2.1.

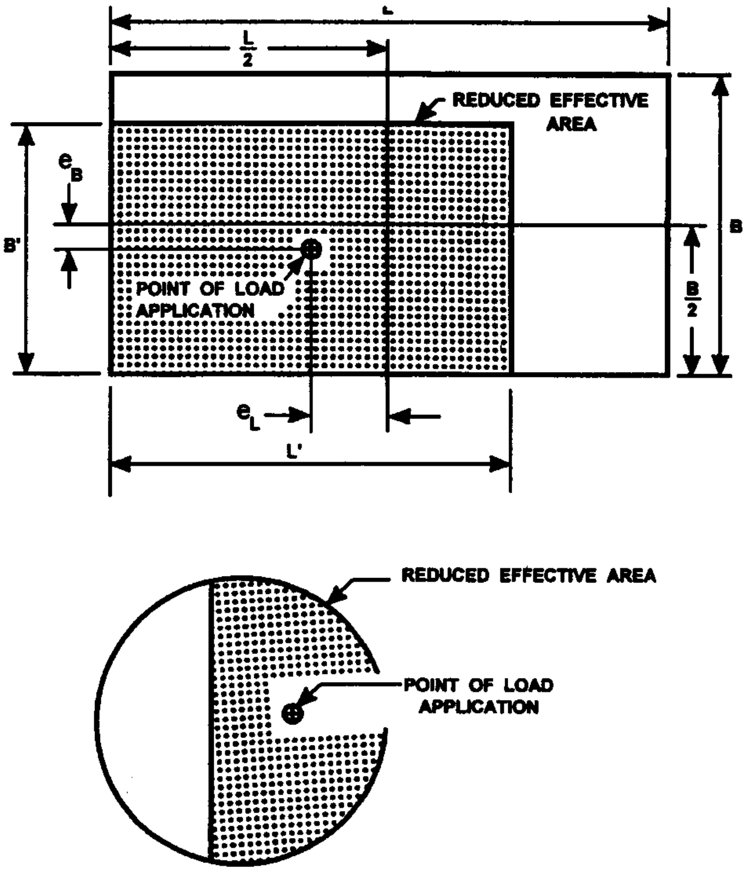


Figure 2.1 Reduced Footing Dimensions
(AASHTO C10.6.3.1.5, based on Meyerhof, 1965)

2.2.2 Bowles (1987)

For a uniform load applied on a flexible foundation of dimension $L \times B$ at embedment depth D_f in a deep elastic layer, the immediate settlement can be evaluated in the following way:

$$S_e = q_0(\alpha B')\frac{1-\mu_s^{2}}{E_s} I_s I_f \qquad (2.5)$$

where q_0 = net applied pressure on the foundation

μ_s = Poisson's ratio of soil

E_s = average modulus of elasticity of the soil under the foundation, measured from depth z = 0 to about z = 4B

B' = B/2 for center of the foundation

= B for corner of the foundation

I_s = shape factor (Steinbrenner, 1934); use Table 2.3

$$I_s = F_1 + \frac{1-2\mu}{1-\mu} F_2 \qquad (2.6)$$

$$F_1 = \frac{1}{\pi}(A_0 + A_1) \tag{2.7}$$

$$F_2 = \frac{n'}{2\pi}\tan^{-1} A_2 \tag{2.8}$$

$$A_0 = m' \ln \frac{(1+\sqrt{m'^2+1})\sqrt{m'^2+n'^2}}{m'(1+\sqrt{m'^2+n'^2+1}} \tag{2.9}$$

$$A_1 = \ln \frac{(m'+\sqrt{m'^2+1})\sqrt{1+n'^2}}{m'+\sqrt{m'^2+n'^2+1}} \tag{2.10}$$

$$A_2 = \frac{m'}{n'\sqrt{m'^2+n'^2+1}} \tag{2.11}$$

I_f = depth factor (Fox, 1948); use Table 2.4

$$I_f = f(\frac{D_f}{B}, \mu_s, \frac{L}{B}) \tag{2.12}$$

$$=1 \text{ for } D_f = 0$$

α = a factor that depends on the location on the foundation where settlement is being calculated

for settlement under the center: α=4, m'=L/B, n'=H/ (B/2)

for settlement under the corner: α=1, m'=L/B, n'=H/B

Table 2.3 Values of F_1 and F_2 for Calculating Steinbrenner Influence Factors (After Bowles (1987))

	L/B																
H/B (1)	1.0 (2)	1.2 (3)	1.4 (4)	1.6 (5)	1.8 (6)	2.0 (7)	2.5 (8)	3.0 (9)	3.5 (10)	4.0 (11)	4.5 (12)	5.0 (13)	6.0 (14)	7.0 (15)	8.0 (16)	9.0 (17)	10.0 (18)
0.5																	
F_1	0.049	0.046	0.044	0.042	0.041	0.040	0.038	0.038	0.037	0.037	0.036	0.036	0.036	0.036	0.036	0.036	0.036
F_2	0.074	0.077	0.080	0.081	0.083	0.084	0.085	0.086	0.087	0.087	0.087	0.087	0.088	0.088	0.088	0.088	0.088
0.8																	
F_1	0.104	0.100	0.096	0.093	0.091	0.089	0.086	0.084	0.083	0.082	0.081	0.081	0.080	0.080	0.080	0.079	0.079
F_2	0.083	0.090	0.095	0.098	0.101	0.103	0.107	0.109	0.110	0.111	0.112	0.112	0.113	0.113	0.113	0.113	0.114
1.0																	
F_1	0.142	0.138	0.134	0.130	0.127	0.125	0.121	0.118	0.116	0.115	0.114	0.113	0.112	0.112	0.112	0.111	0.111
F_2	0.083	0.091	0.098	0.102	0.106	0.109	0.114	0.117	0.119	0.120	0.121	0.122	0.123	0.123	0.124	0.124	0.124
2.0																	
F_1	0.285	0.290	0.292	0.292	0.291	0.289	0.284	0.279	0.275	0.271	0.269	0.267	0.264	0.262	0.261	0.260	0.259
F_2	0.064	0.074	0.083	0.090	0.097	0.102	0.114	0.121	0.127	0.131	0.134	0.136	0.139	0.141	0.143	0.144	0.145
4.0																	
F_1	0.408	0.431	0.448	0.460	0.469	0.476	0.484	0.487	0.486	0.484	0.482	0.479	0.474	0.470	0.466	0.464	0.462
F_2	0.037	0.044	0.051	0.057	0.063	0.069	0.082	0.093	0.102	0.110	0.116	0.121	0.129	0.135	0.139	0.142	0.145
6.0																	
F_1	0.457	0.489	0.514	0.534	0.550	0.563	0.585	0.598	0.606	0.609	0.611	0.610	0.608	0.604	0.601	0.598	0.595
F_2	0.026	0.031	0.036	0.040	0.045	0.050	0.060	0.070	0.079	0.087	0.094	0.101	0.111	0.120	0.126	0.131	0.135
8.0																	
F_1	0.482	0.519	0.549	0.573	0.594	0.611	0.643	0.664	0.678	0.688	0.694	0.697	0.700	0.700	0.698	0.695	0.692
F_2	0.020	0.023	0.027	0.031	0.035	0.038	0.047	0.055	0.063	0.071	0.077	0.084	0.095	0.104	0.112	0.118	0.124
10.0																	
F_1	0.498	0.537	0.570	0.597	0.621	0.641	0.679	0.707	0.726	0.740	0.750	0.758	0.766	0.770	0.770	0.770	0.768
F_2	0.016	0.019	0.022	0.025	0.028	0.031	0.038	0.046	0.052	0.059	0.065	0.071	0.082	0.091	0.099	0.106	0.112
12.0																	
F_1	0.508	0.550	0.585	0.614	0.639	0.661	0.704	0.736	0.760	0.777	0.791	0.801	0.815	0.823	0.826	0.828	0.828
F_2	0.013	0.016	0.018	0.021	0.024	0.026	0.032	0.038	0.044	0.050	0.056	0.061	0.071	0.080	0.088	0.095	0.102
100.0																	
F_1	0.555	0.605	0.649	0.688	0.722	0.753	0.819	0.872	0.918	0.956	0.990	1.020	1.072	1.114	1.150	1.182	1.209
F_2	0.002	0.002	0.002	0.003	0.003	0.003	0.004	0.005	0.006	0.006	0.007	0.008	0.010	0.011	0.013	0.014	0.016
1,000.0																	
F_1	0.560	0.612	0.657	0.697	0.733	0.765	0.833	0.890	0.938	0.979	1.016	1.049	1.106	1.154	1.196	1.233	1.266
F_2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.002

The elastic settlement of a rigid foundation can be estimated by calculating as:

$$S_{e(rigid)} \approx 0.93 S_{e\,(Flexible\ footing)} \quad (2.13)$$

Table 2.4 Fox depth factor (Fox (1948), after Bowles (1987))

D_f/B	L/B 1.0	1.2	1.4	1.6	1.8	2.0	5.0
Poisson's Ratio = 0.00 = μ_s							
0.05	0.950	0.954	0.957	0.959	0.961	0.963	0.973
0.10	0.904	0.911	0.917	0.922	0.925	0.928	0.948
0.20	0.825	0.838	0.847	0.855	0.862	0.867	0.903
0.40	0.710	0.727	0.740	0.752	0.761	0.769	0.827
0.60	0.635	0.652	0.666	0.678	0.689	0.698	0.769
0.80	0.585	0.600	0.614	0.626	0.637	0.646	0.723
1.00	0.549	0.563	0.576	0.587	0.598	0.607	0.686
2.00	0.468	0.476	0.484	0.492	0.499	0.506	0.577
Poisson's Ratio = 0.10 = μ_s							
0.05	0.958	0.962	0.965	0.967	0.968	0.970	0.978
0.10	0.919	0.926	0.930	0.934	0.938	0.940	0.957
0.20	0.848	0.859	0.868	0.875	0.881	0.886	0.917
0.40	0.739	0.755	0.768	0.779	0.788	0.795	0.848
0.60	0.665	0.682	0.696	0.708	0.718	0.727	0.793
0.80	0.615	0.630	0.644	0.656	0.667	0.676	0.749
1.00	0.579	0.593	0.606	0.618	0.628	0.637	0.714
2.00	0.496	0.505	0.513	0.521	0.528	0.535	0.606
Poisson's Ratio = 0.30 = μ_s							
0.05	0.979	0.981	0.982	0.983	0.984	0.985	0.990
0.10	0.954	0.958	0.962	0.964	0.966	0.968	0.977
0.20	0.902	0.911	0.917	0.923	0.927	0.930	0.951
0.40	0.808	0.823	0.834	0.843	0.851	0.857	0.899
0.60	0.738	0.754	0.767	0.778	0.788	0.796	0.852
0.80	0.687	0.703	0.716	0.728	0.738	0.747	0.813
1.00	0.650	0.665	0.678	0.689	0.700	0.709	0.780
2.00	0.562	0.571	0.580	0.588	0.596	0.603	0.675
Poisson's Ratio = 0.40 = μ_s							
0.05	0.989	0.990	0.991	0.992	0.992	0.993	0.995
0.10	0.973	0.976	0.978	0.980	0.981	0.982	0.988
0.20	0.932	0.940	0.945	0.949	0.952	0.955	0.970
0.40	0.848	0.862	0.872	0.881	0.887	0.893	0.927
0.60	0.779	0.795	0.808	0.819	0.828	0.836	0.886
0.80	0.727	0.743	0.757	0.769	0.779	0.788	0.849
1.00	0.689	0.704	0.718	0.730	0.740	0.749	0.818
2.00	0.596	0.606	0.615	0.624	0.632	0.640	0.714
Poisson's Ratio = 0.50 = μ_s							
0.05	0.997	0.997	0.998	0.998	0.998	0.998	0.999
0.10	0.988	0.990	0.991	0.992	0.993	0.993	0.996
0.20	0.960	0.966	0.969	0.972	0.974	0.976	0.985
0.40	0.886	0.899	0.908	0.916	0.922	0.926	0.953
0.60	0.818	0.834	0.847	0.857	0.866	0.873	0.917
0.80	0.764	0.781	0.795	0.807	0.817	0.826	0.883
1.00	0.723	0.740	0.754	0.766	0.777	0.786	0.852
2.00	0.622	0.633	0.643	0.653	0.662	0.670	0.747

Due to the nonhomogeneous nature of soil deposits, the magnitude of E_s may vary with depth. For this reason, Bowels (1987) recommended using a weighted average of E_s in Eq. 2.5 which is calculated as:

$$E_s = \frac{\sum E_{s(i)} \Delta z}{\bar{z}} \tag{2.14}$$

where $E_{s(i)}$ = soil modulus of elasticity within a depth

$\bar{z}$ = H or 5B, whichever is smaller,
where H=depth to incompressible layer from below the footing base

E_s can also be directly evaluated from laboratory tests (triaxial) or the use of general values and/or empirical correlation, using Table 2.1 and/or Tables 2.5 or 2.6.

Table 2.5 Elastic Parameters of Various Soils (Das, 2004)

Type of soil	Modulus of elasticity, E_s		Poisson's ratio, μ_s
	MN/m^2	Lb/in^2	
Loose sand	10.5-24.0	1500-3500	0.20-0.40
Medium dense sand	17.25-27.60	2500-4000	0.25-0.40
Dense sand	34.50-55.20	5000-8000	0.30-0.45
Silty sand	10.35-17.25	1500-2500	0.20-0.40
Sand and gravel	69.00-172.50	10,000-25,000	0.15-0.35
Soft clay	4.1-20.7	600-3000	
Medium clay	20.7-41.4	3000-6000	0.20-0.50
Stiff clay	41.47- 96.6	6000-14,000	

Table 2.6 Empirical Relationship of Modulus of Elasticity

Empirical Equation		**Reference**	**Note:**
E_s=766N	(E_s in kPa)	Schmertmann (1970)	N: standard penetration resistance
E_s=8N	(E_s E in tsf)		E_s:Modulus of Elasticity
E_s =2q_c	(CPT test)		q_c:cone resistance

2.2.3 Mayne and Poulos (1999)

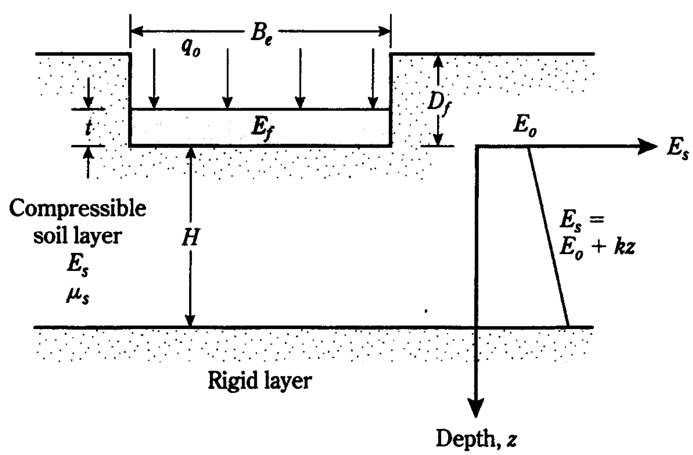


Figure 2.2 Improved equation for calculating elastic settlement: general parameters (Das, 2004)

Consider the foundation rigidity, embedment depth, and increase of E_s with depth, location of rigid layers within the zone of influence, there is an improved formula for calculating the elastic settlement of foundation.

The settlement below the center of the foundation:

$$S_e = \frac{q_0 B_e I_G I_F I_E}{E_0}(1-\mu_s^2) \quad (2.15)$$

and $$B_e = \sqrt{\frac{4BL}{\pi}} \text{ for rectangular foundation} \quad (2.16)$$

$$B_e = B \text{ for circular foundation} \quad (2.17)$$

$$I_F = \frac{\pi}{4} + \frac{1}{4.6 + 10\left(\frac{E_f}{E_0 + \frac{B_e}{2}k}\right)\left(\frac{2t}{B_e}\right)^3} \quad (2.18)$$

$$I_E = 1 - \frac{1}{3.5\exp(1.22\mu_s - 0.4)\left(\frac{B_e}{D_f} + 1.6\right)} \quad (2.19)$$

$$E_s = E_0 + kz \quad (2.20)$$

where
- B = width of rectangular foundation or diameter for circular foundation
- L = length of foundation
- I_G = influence factor for the variation of E_s with depth
  $I_G = f(B, H/B_e), \beta = E_0/KB_e$ see Figure 2.3.
- I_F = foundation rigidity correction factor, see Figure 2.4.
- I_E = foundation embedment correction factor, see Figure 2.4 and Figure 2.5
- E_s = modulus of elasticity
- E_0 = initial modulus of elasticity
- E_f = modulus of foundation material
- t = thickness of foundation

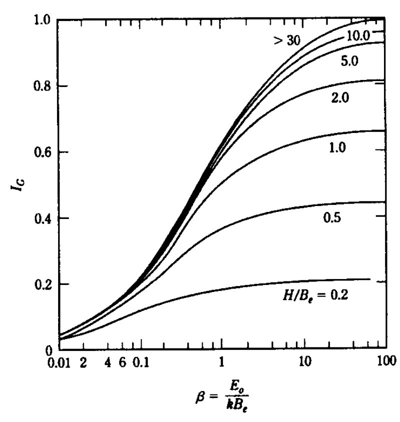


Figure 2.3 Variation of I_G with β

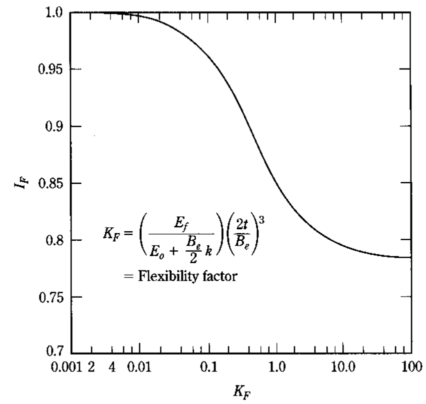


Figure 2.4 Variation of rigidity correction factor I_F with flexibility factor K_F

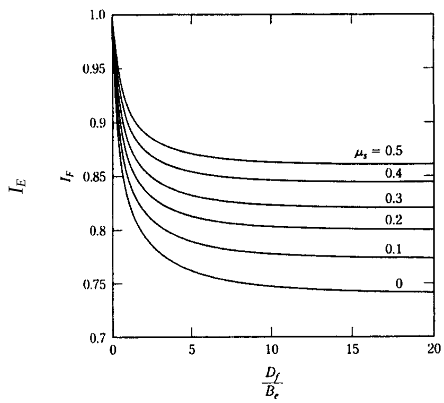


Figure 2.5 Variation of embedment correction factor I_E with D_f / B_e

2.2.4 Canadian foundation manual (1975, 1985, 1992)

The Canadian Foundation Manual (CFM) suggests settlement estimates of footings to be made by dividing the soil into layers, calculating the value of the applied stress at the midpoint of each layer and using an apparent modulus of elasticity of the soil layer which is can be obtained from Table 2.1, 2.5 or 2.6, to determine the settlement of each layer. The layer strain, E_z, is determined according to:

$$E_z = q_z / E_s \tag{2.21}$$

where q_z = applied stress at the midpoint of the layer
E_s = modulus of elasticity

The total settlement is obtained from:

$$s = \sum_i E_{z_i} h_{z_i} \quad \text{or} \quad s = \sum_i (q_{z_i} / E_{s_i}) h_{z_i} \tag{2.22}$$

where s = settlement
h_{z_i} = thickness of individual layer i

The CFM indicated that "for most practical applications, the stress distribution can be calculated according to the 2:1 method." According to the 2:1 distribution, for a footing of width B and length L, with an applied foundation stress of q_0, the corresponding stress at depth z (also shown in Figure 2.19) is:

$$q_z = [q_0 BL]/[(B+z)(L+z)] \qquad (2.23)$$

For a infinitely long (strip) footing, Eq. 2.23 becomes:

$$q_z = (q_0 B)/(B+z) \qquad (2.24)$$

For a more refined analysis, the CFM presents a form of the general elastic solution for calculating settlement as:

$$s = (q_0 B i_c)/E_s \qquad (2.25)$$

where:
- s = settlement
- q_0 = applied net footing stress
- B = footing width
- E_s = apparent modulus elasticity
- i_c = influence factor

The influence factor, i_c, as presented in the CFM, is taken from Kany (1959) and is shown in Figure 2.6 for different value of z/B and L/B and therefore, like other influence factors, takes into account the layer thickness and foundation geometry.

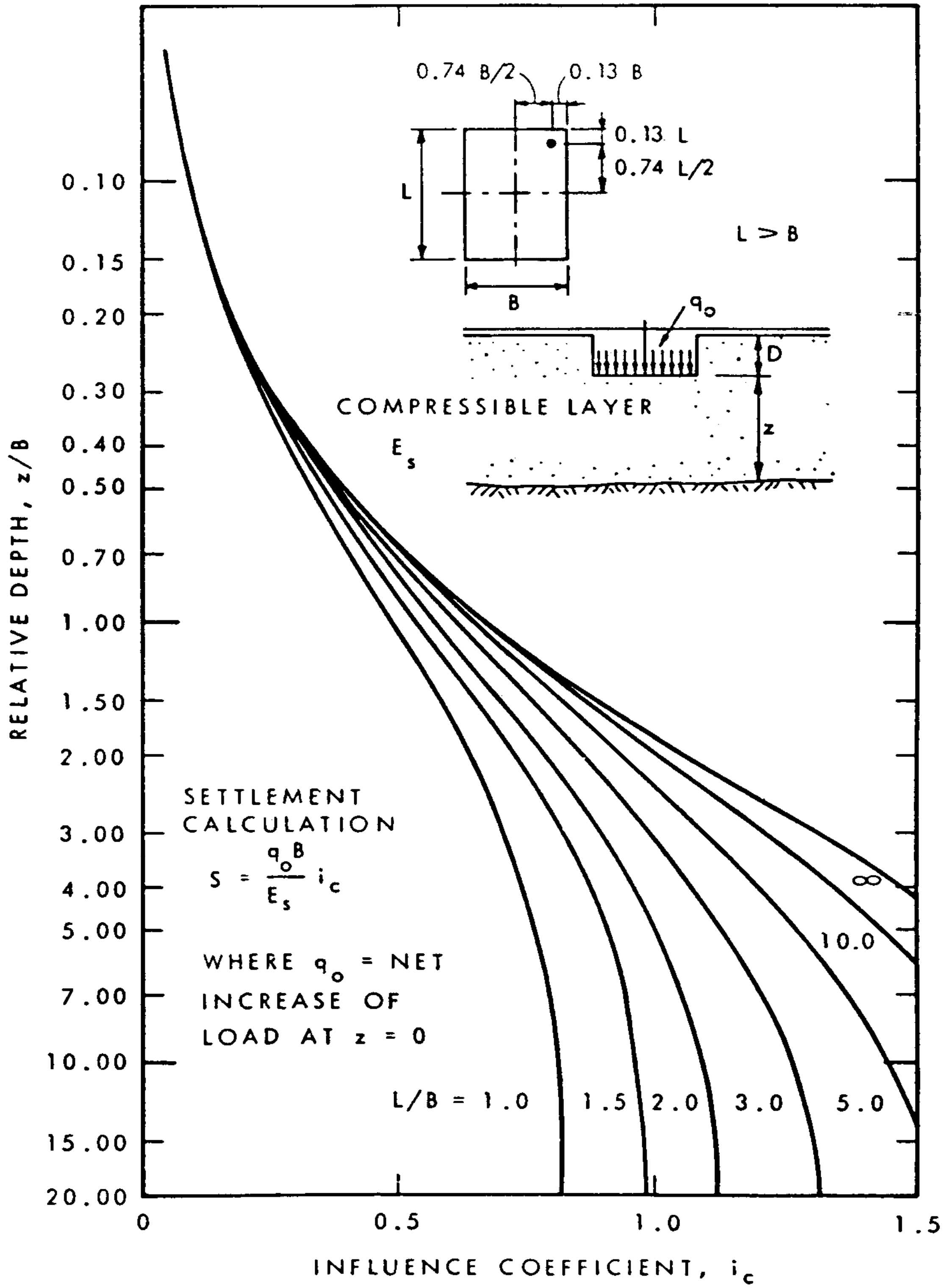


Figure 2.6 Chart of Influence Factor, i_c, after Kany (1959) (Canadian Foundation Manual, 1985)

2.3 METHODS UTILIZING IN-SITU STANDARD PENETRATION TEST (SPT)

2.3.1 Terzaghi and Peck (1948, 1967)

The Terzaghi and Peck settlement method is based on shallow foundation bearing capacity charts developed using the allowable bearing capacity equations presented by Meyerhof (1956, see section 2.2.2.2). The charts are used to obtain the allowable bearing capacity (assuming a F.S.=3) for different footings width and SPT blow counts values with the maximum settlement and differential settlement not exceeding 25mm (1 in.) and 19mm (3/4 in.) respectively at a given allowable contact stress. According to Terzaghi and Peck, square and continuous footings of the same width show similar settlement behavior for the same soil and loading intensity. The settlement is given as:

$$s = 8q / N(C_w C_d) \text{ for B} \leq \text{4ft} \quad (2.26)$$

$$s = (12q / N)[B/(B+1)]^2 C_w C_d \text{ for B>4ft} \quad (2.27)$$

$$s = (12q / N) C_w C_d \text{ for rafts} \quad (2.28)$$

These expressions can also be stated in general form as:

$$s = (3q / N)(2B / B+1)^2 C_w C_d \quad (2.29)$$

where s = settlement (in inches)
q = net footing stress (in tsf)
N = uncorrected (field) SPT blow count
B = footing width (in ft.)
C_w = water table correction
= 2-(W/2B) ≤ 2.0 for surface footings
= 2-0.5(D/B) ≤ 2.0 for fully submerged,
= embedded footing; W≤D
C_d = embedment correction
= 1-0.25(D/B)

where W = depth of water table (in ft.)
D = footing depth (in ft.)

The uncorrected SPT blow count data are used in calculating settlement. However, if the sand is dense, saturated and very fine or silty (e.g., abundant fines content), the blow count should be corrected according to:

$$N_c = 15 + 0.5(N - 15) \text{ for N > 15} \quad (2.30)$$

The correction for water table applied to cases where ground water is at or above the base of footing (complete submerged cased). For partial submergence (water located between D and D+B) a correction factor is given for surface footings (no embedment) only. In common practice, the water table correction is often omitted from the settlement estimates using this method since the method is generally considered to be overly conservative.

2.3.2 Meyerhof (1956, 1965)

Meyerhof (1956) suggested that the allowable bearing pressures for a footing on granular soil could be estimated based on the result of SPT blow count. The allowable pressure includes a minimum factor of safety of 3 against bearing capacity failure and may be less than the safe bearing pressure $(q_{ult}/3)$ if the settlement resulting from the safe bearing pressure is excessive. Assuming that the allowable bearing pressure causes 25mm (1 in.) of total settlement, Meyerhof (1956) proposed the following expression for dry and moist sands:

$$q_a = N/8 \text{ (for B} \leq \text{4ft)} \quad (2.31)$$

$$q_a = N(1+1/B)^2/12 \text{ (for B > 4ft)} \quad (2.32)$$

$$q_a = N/10 \text{ (approximately, for any B)} \quad (2.33)$$

where q_a = allowable bearing pressure (tsf)
N = uncorrected SPT blow count
B = footing width (ft)

In saturated very fine or silty sands, Meyerhof suggested using the equivalent N values if N > 15 as:

$$N_c = 15 + 0.5(N-15) \quad (2.34)$$

which is the same as Eq. 2.30 (in Terzaghi).

The settlement for any footing loaded to some stress level other than q_a (presumably less) could then be obtained by proportioning the settlement from 25mm (1 in.) as a proportion of the q/q_a ratio.

Since submergence increases the settlement, the allowable bearing capacity Eq.2.31, 2.32 and 2.33 should be reduced with position of the water table, when the water table is below 1.5B under the bottom of the footing, no effect is considered, any place above 1.5B, linear interpolation is used.

Meyerhof (1965) suggested a modification to his earlier (Meyerhof, 1956) expression. The allowable bearing capacity is increased by 50% given a settlement of 25mm (1 in.) accounting for the fact that the earlier method tended to be conservative. The expressions for the modified settlement then become:

$$s = 4q/N \text{ for B} \leq \text{4ft} \quad (2.35)$$

$$s = [6q/N][B/(B+1)]^2 \text{ for B > 4ft} \quad (2.36)$$

$$s = [6q/N] \text{ for rafts} \quad (2.37)$$

where s = settlement (in inches)
q = footing stress (in tsf)
N = uncorrected blow counts
B = footing width (in ft.)

No correction is applied to the SPT blow count value for overburden stress and since it is assumed that the presence of ground water is reflected in the blow count values, no additional correction is applied for the ground water table.

2.3.3 Alpan (1964)

An additional settlement method based primarily on the Terzaghi and Peck (1948) approach was presented by Alpan (1964). This method indirectly uses a corrected blow count to evaluate a modulus of subgrade reaction from a plate loading test.

The method assumes that the settlement response of a shallow footing resting on sands will be linear in the range of allowable bearing pressures (i.e., $q_{ult}/2.5$) and is given as:

$$s = s_0[2B/(B+1)]^2 mC_w \tag{2.38}$$

where
- s = settlement (in inches)
- s_0 = settlement of a 1 ft^2 plate (in inches)
- B = footing width (in ft.)
- m = shape correction factor (Table 2.7)
- C_w = water table correction factor
  $= 0.5(D/B) \leq 2.0$ for water located immediately below the footing

The settlement of the 1 ft^2 plate is given as:

$$s_0 = \alpha q B \tag{2.39}$$

where
- q = footing stress (in tsf)
- α = a constant (dependent upon the corrected blow count N_c)

The blow count value at the foundation level is first used to estimate the relative density of the sand, D_r, using the correction of Gibbs and Holtz (1957) which was put into a more convenient from by Coffman (1960) as shown in Figure 2.7. The correction factor, α, is shown in a graphical form in Figure 2.8. Note that two charts are suggested by Alpan (1964); one for corrected blow count values between 5 to 50; and another for corrected blow count values between 25 to 80.

Alpan suggested that the correction for ground water is to account for the reduced confining stress which would increase the settlement. A conservative approach would be to increase the settlement estimate by 100% if the foundation depth ratio (i.e., D/B) is small and only 50% as D/B approaches 1.

In very fine sand or silty sand, the SPT blow count value may be too high, leading to an overestimation of relative density and thus an underestimation of the settlement, Alpan suggested using the correction presented by Terzaghi and Peck for N values greater than 15 as:

$$N_c = 15 + 0.5(N - 15)\,;\ N > 15 \tag{2.40}$$

An additional shape correction was suggested by Alpan to account for foundation geometry. Shape corrections factors, m, are presented in Table 2.7

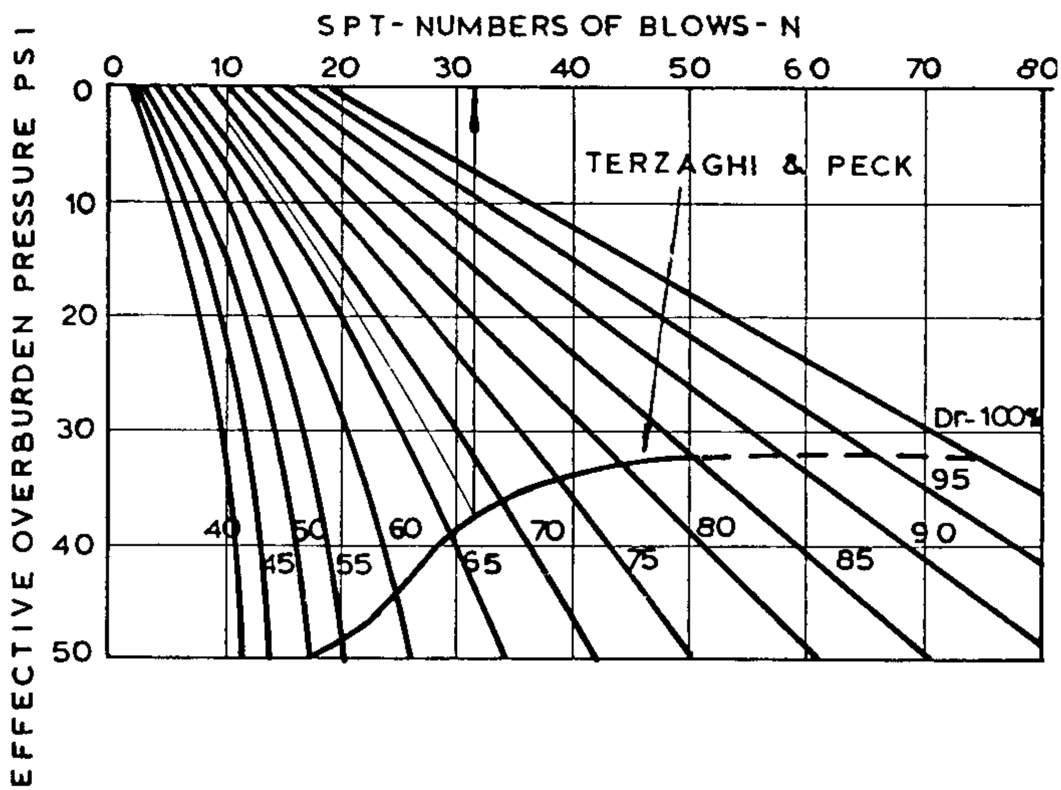


Figure 2.7 Interpretations of Gibbs and Holtz SPT Correction, Coffman (1960)

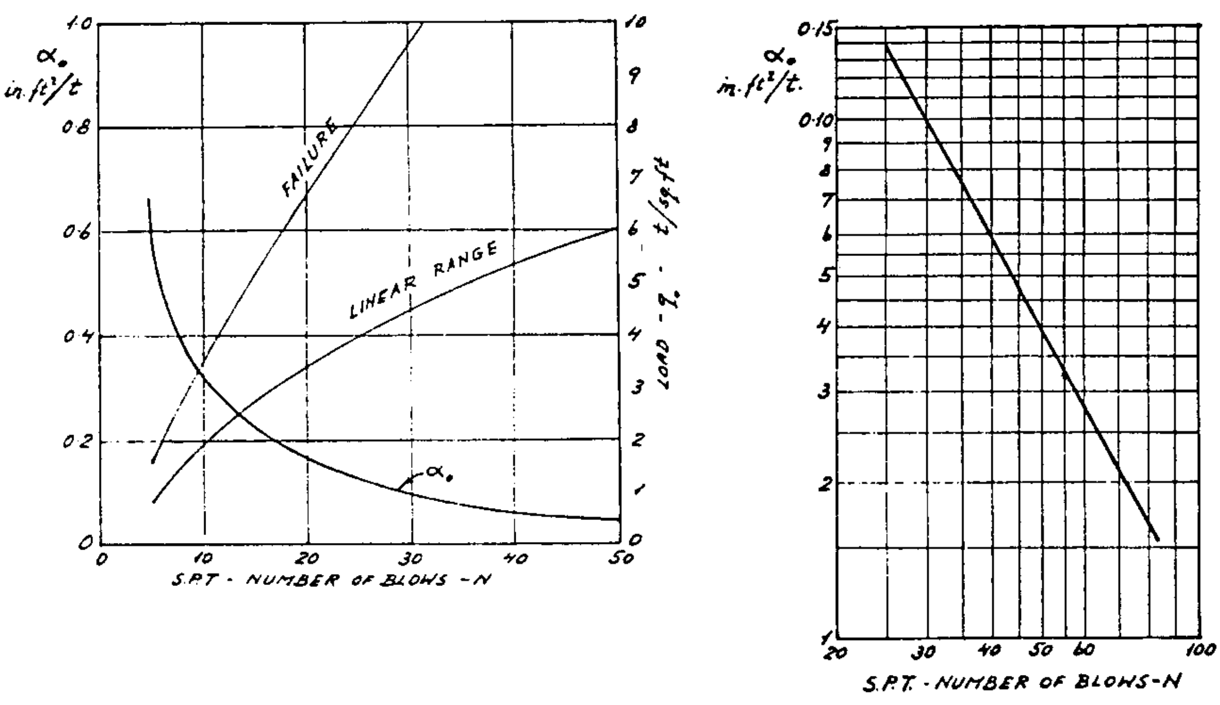


Figure 2.8 Alpan (1964) Correction Factors

Table 2.7 Foundation Shape Factor, Alpan (1964)

	Circle	Rectangle with L/B					
		1	1.5	2	3	5	10
m	1.0	1.0	1.21	1.37	1.60	1.94	2.36

2.3.4 D'Appolonia et al. (1968)

Settlement Prediction method from Standard Penetration Test data which was presented by *D'Appolonia et al (1968)* is:

$$S = PBM_v(1-v^2)I \qquad (2.41)$$

where
- S = settlement
- P = bearing pressure
- B = footing width
- M_v = coefficient of compressibility from oedometer test
- ν = Poisson's ratio (ν=0 for no lateral strain)
- I = influence factor (I=1.4 for L/B=1.6)

2.3.5 D'Appolonia et al. (1970)

In the closure to their 1968 ASCE article, D'Appolonia et al. (1970) suggested an alternative method for predicting settlement which is based more or less on an elastic solution. The method requires an estimate of the modulus of compressibility of the soil, M, which is obtained from SPT blow count. The settlement is calculated from the general elastic solution equation:

$$s = (qBI)/M \qquad (2.42)$$

where:
- s = settlement (in ft.)
- q = footing stress (in tsf)
- B = footing width (in ft)
- I = influence factor
- M = modulus of compressibility (in tsf)

The influence factor I in Eq. 2.42 is the product of two factors, μ_0 and μ_1, which account for the geometry and the depth of the footing and the depth to an incompressible layer. The factors μ_0 and μ_1 were developed by Janbu et al. (1956), modified by Christian & Carrier (1978), see Figure 2.9.

The blow count value is taken as the average uncorrected value obtained between the base of the footing and a depth of B below the footing. No other correction factor is applied. The soil modulus of compressibility is obtained from the SPT blow count as:

$$M = 196 + 7.9(N) \text{ (in tsf) for NC sand} \qquad (2.43)$$

$$M = 416 + 10.9(N) \quad \text{for OC sand} \qquad (2.44)$$

Figure 2.10 present the original correlations proposed by D'Appolonia et al. (1970).

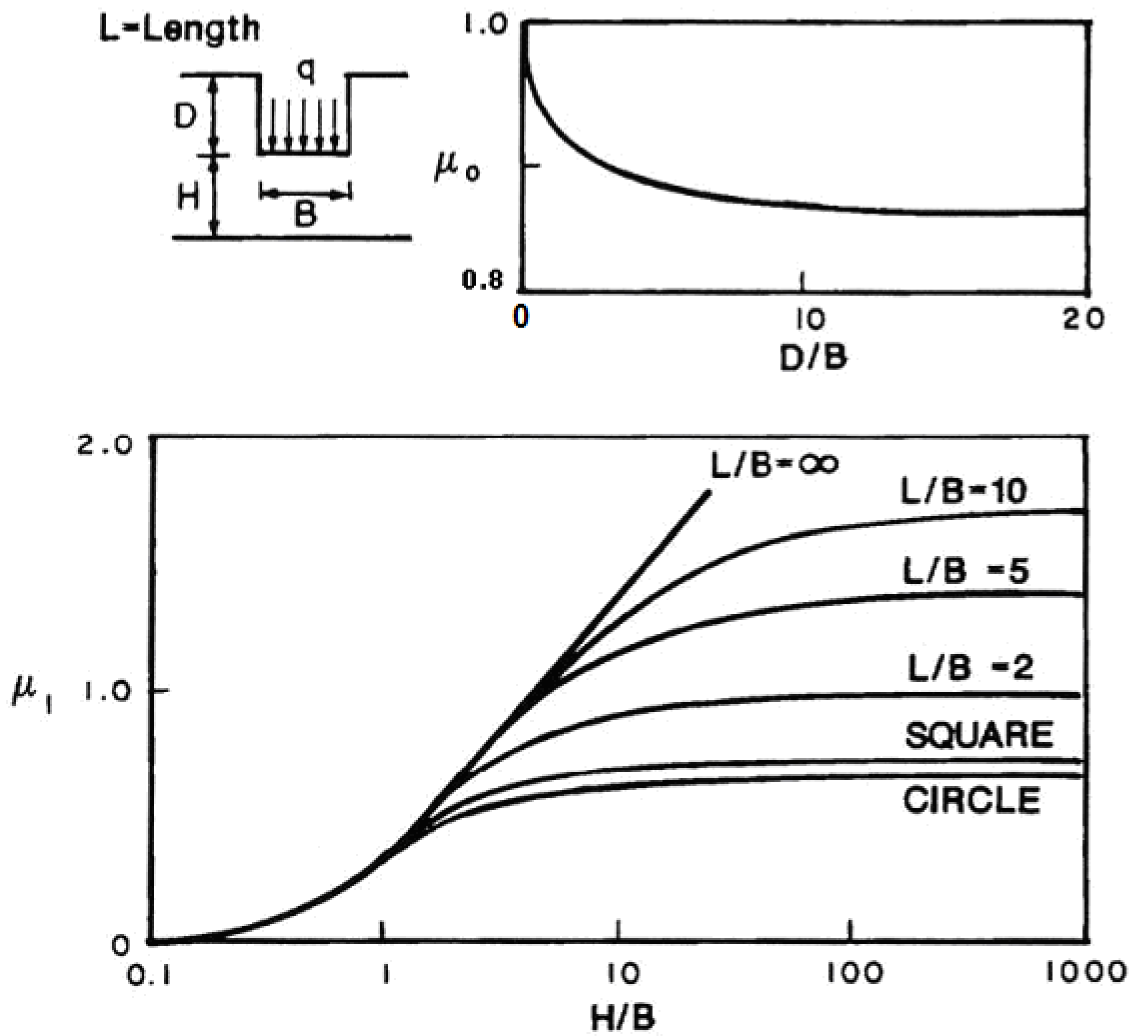
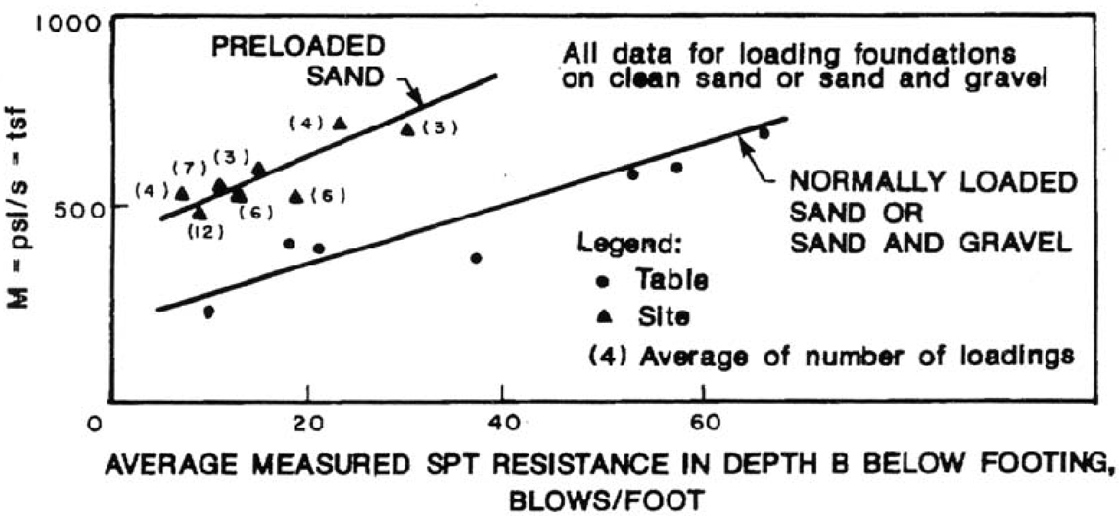


Figure 2.9 Correction Factors for Embedment and Layer Thickness (Christian & Carrier, 1978)



Note: In the above Figure, "Table" refers to a tabulation of load versus settlement data from seven cases histories, including six bridge footings, by D'Appolonia et al., (1970), and "Site" refers to the load versus settlement data obtained by D'Appolonia at al., (1968) at a large steel mill site in north Indiana.

Figure 2.10 Modulus of Compressibility (D'Appolonia, 1970)

2.3.6 Burland and Burbidge (1985)

Burland and Burbidge (1985) proposed a method of calculating the elastic settlement of sand using the field standard penetration number, N_{60}. The method can be summarized as follows:

1. Determine the variation of the Standard Penetration number with depth

   Obtain the field penetration number (N_{60}) with depth at the location of the foundation. The following adjustments of N_{60} may be necessary, depending on the field conditions: For gravel or sandy gravel:

   $$N_{60(a)} \approx 1.25 N_{60} \qquad (2.45)$$

   For fine sand or silty sand below the groundwater table and N_{60} >15,

   $$N_{60(a)} \approx 15 + 0.5(N_{60} - 15) \qquad (2.46)$$

   where $N_{60(a)}$ = adjusted N_{60} value

2. Determine the depth of stress influence (z')
   In determining the depth of stress influence, the following three cases may arise:

   Case I. If N_{60}[or $N_{60(a)}$] is approximately constant with depth, calculate z' from

   $$\frac{z'}{B_R} = 1.4\left(\frac{B}{B_R}\right)^{0.75} \qquad (2.47)$$

where B_R = reference width = 1 ft (if B is in ft)
= 0.3m (if B is in m)
B = width of the actual foundation

Case II. If N_{60}[or $N_{60(a)}$] is decreasing with depth, use Eq. 2.47 calculate z'.

Case III. If N_{60}[or $N_{60(a)}$] is decreasing with depth, calculate $z' = 2B$ and z'=distance from the bottom of the foundation to the bottom of the soft soil layer (z''). Use $z' = 2B$ or $z' = z''$ (whichever is smaller).

3. Calculation of Elastic Settlement S_e

The elastic settlement of the foundation, S_e, can be calculated from

$$\frac{S_e}{B_R} = \alpha_1 \alpha_2 \alpha_3 \left[\frac{1.25\left(\frac{L}{B}\right)}{0.25 + \left(\frac{L}{B}\right)} \right]^2 \left(\frac{B}{B_R}\right)\left(\frac{q'}{p_a}\right) \qquad (2.48)$$

where α_1 = a constant
α_2 = compressibility index
α_3 = correction for the depth of influence
p_a = atmospheric pressure = 100kN/m^2 (≈200lb/ft^2)
L = length of the foundation

For normal consolidated sand:

$$\alpha_1 = 0.14 \qquad (2.49)$$

$$\text{and } \alpha_2 = \frac{1.71}{[\overline{N}_{60} or \overline{N}_{60(a)}]^{1.4}} \qquad (2.50)$$

where N_{60} or $N_{60(a)}$ = average value of N_{60} or $N_{60(a)}$ in the depth of stress influence

$$\alpha_3 = \frac{z''}{z'}\left(2 - \frac{z''}{z'}\right) \le 1 \qquad (2.51)$$

$$\text{and } q' = q_0 \qquad (2.52)$$

in which q_0 = net applied stress at the level of the foundation (i.e., the stress at the level of the foundation minus the overburden pressure)

For overconsolidated sand with $q_0 \le \sigma'_c$, where the σ'_c being the preconsolidation pressure:

$$\alpha_1 = 0.047 \qquad (2.53)$$

$$\text{and } \alpha_2 = \frac{0.57}{[\overline{N}_{60} or \overline{N}_{60(a)}]^{1.4}} \quad (2.54)$$

For α_3, use Eq. 2.51

$$q' = q_0 \quad (2.55)$$

For *overconsolidated* sand with $q_0 > \sigma'_c$,

$$\alpha_1 = 0.14 \quad (2.56)$$

For α_2, use Eq. 2.54, and for α_3, use Eq. 2.51. Finally, use

$$q' = q_0 - 0.67\sigma'_c \quad (2.57)$$

2.3.7 Hough (1959)

Basic equation:

$$s = \sum_0^z \left(\frac{1}{C}\right)\Delta z \log\left(\frac{\overline{\sigma}_{v0} + \Delta\overline{\sigma}_v}{\overline{\sigma}_{v0}}\right) \quad (2.58)$$

where C = bearing capacity index = $\frac{1+e_0}{C_c}$; given in Figure 2.12

e_0 = initial void ratio

C_c = virgin compression index

Δz = layer thickness

$\overline{\sigma}_{v0}$ = initial effective overburden pressure at mid-height of layer

$\Delta\overline{\sigma}_v$ = change in effective vertical stress at layer mid-height

The total settlement by the Hough method is calculated as follows:

- **a.** Calculate the effective vertical stress, σ'_{vo}, at the midpoint of each layer and obtain corrected SPT blowcounts for overburden stress using the relation in Figure 2.11.
- **b.** Determine bearing capacity index (C') from Figure 2.12 using corrected SPT blowcounts, N', determined in Step a.
- **c.** Subdivide subsurface soil profile into approximately 3-m (10-ft) layers based on stratigraphy to a depth of about three times the footing width.
- **d.** Calculate the average bearing capacity index for that layer.
- **e.** Calculate the increase in stress at the midpoint of each layer, $\Delta\sigma'_v$, using 2:1 method (Figure 2.13).
- **f.** Calculate the settlement in each layer, Δz, under the applied load using the following formula:

$$\Delta H = \frac{1}{C'}\Delta z \log\left(\frac{\overline{\sigma}_{v0} + \Delta\overline{\sigma}_v}{\overline{\sigma}_{v0}}\right) \quad (2.59)$$

- **g.** Sum the incremental settlement to determine the total settlement.

Calculate the increase in stress at the midpoint of each layer using the 2:1 method (Chen and McCarron, 1991) as shown in Figure 2.13. This distribution can be computed as a function of applied stress according to:

$$\frac{\Delta\overline{\sigma}_v}{q} = \frac{B \times L}{(B+Z)(L+Z)} \qquad (2.60)$$

where:

	$\Delta\overline{\sigma}_v$	= change in vertical stress at depth Z below the footing bearing elevation
	q	= stress applied by the footing at the bearing elevation
	Z	= depth below footing bearing elevation to point of Interest, usually the midpoint of a soil layer or sublayer where a settlement computation is to be made
	B	= width of footing
	L	= length of footing

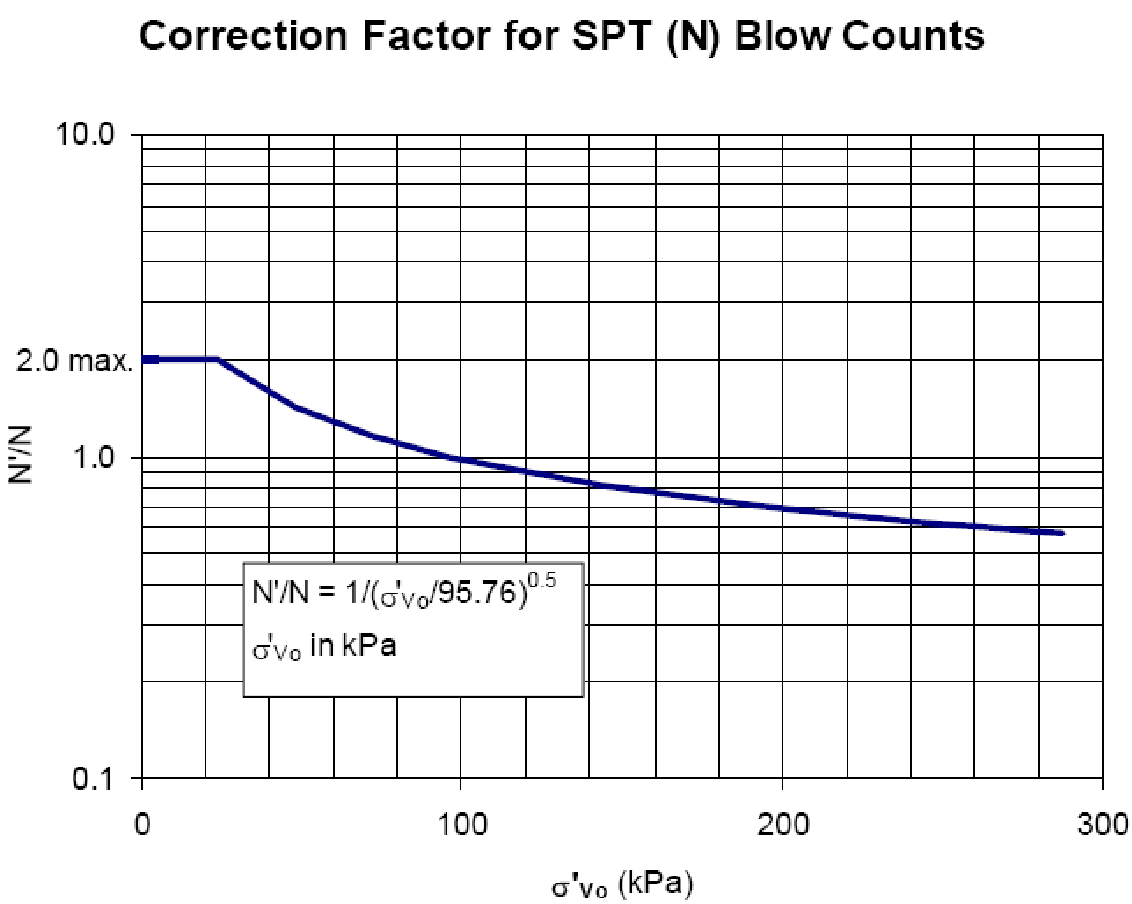


Figure 2.11 Corrected SPT (N) versus Overburden Pressure (after Liao & Whitman , 1986)

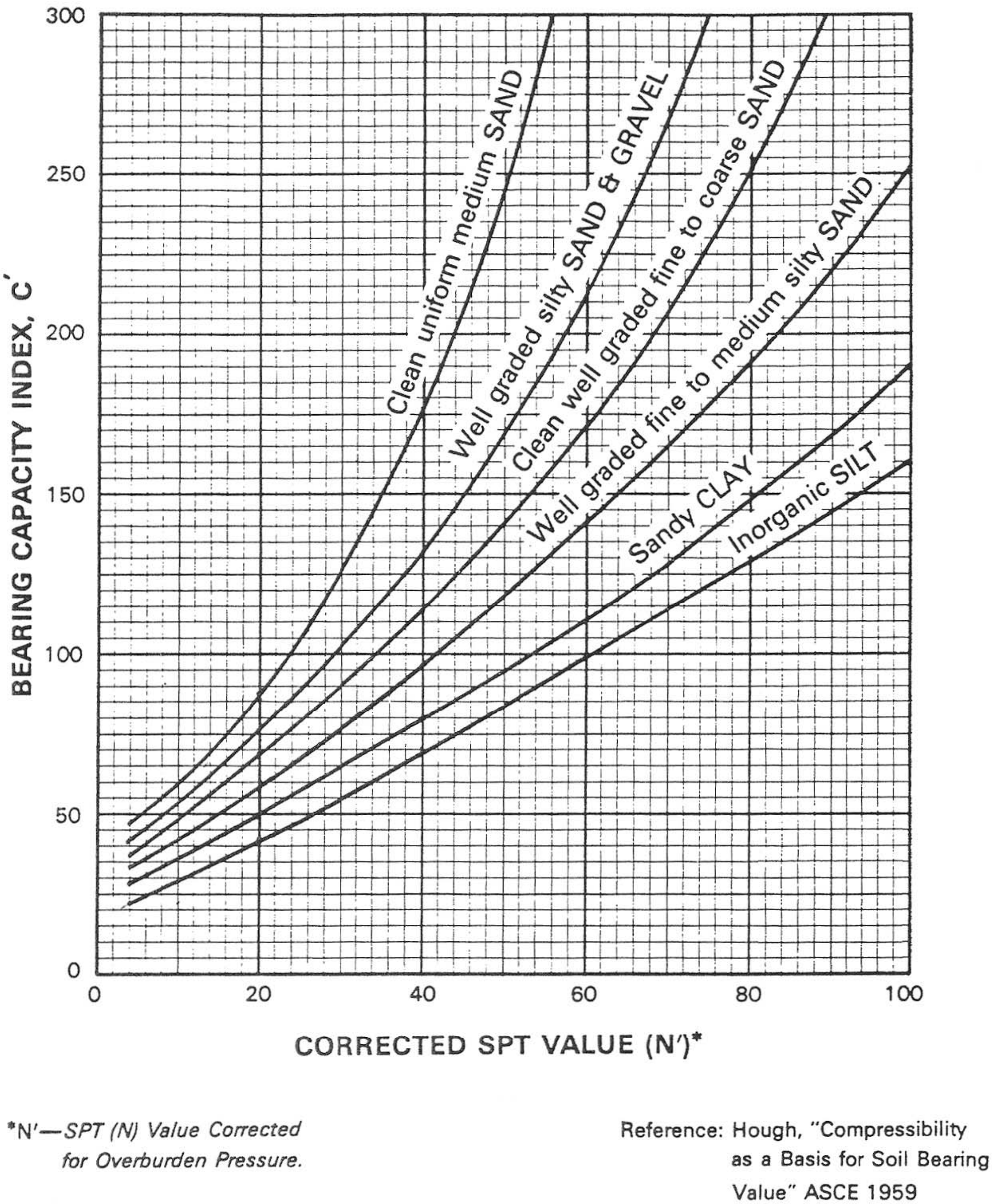


Figure 2.12 Bearing Capacity Index versus Corrected SPT
(Cheney & Chassie, 2000, modified from Hough, 1959)

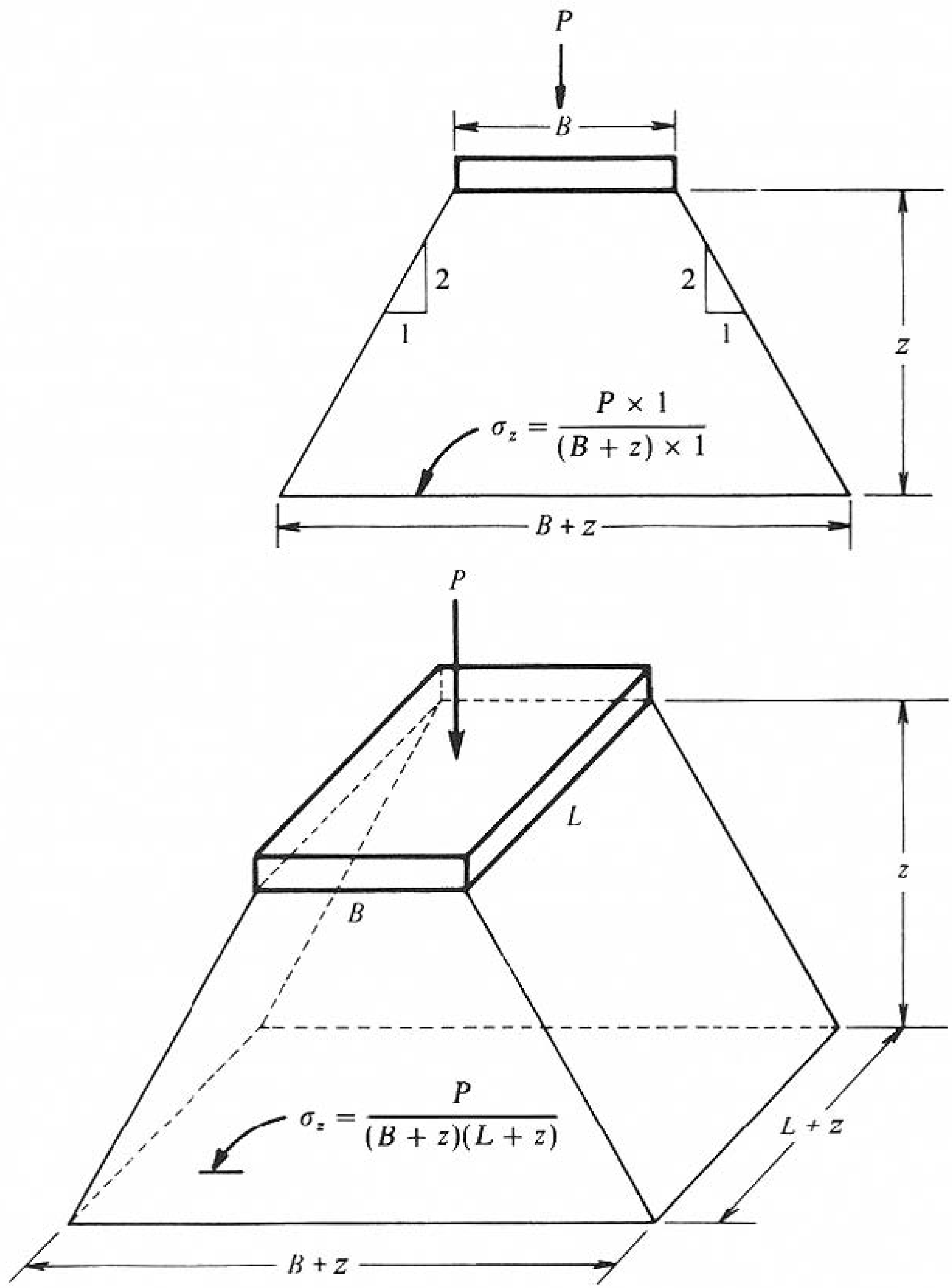


Figure 2.13 Distribution of Vertical Stress by 2:1 Method (after Chen and McCarron, 1991)

2.4 METHODS UTILIZING IN-SITU CONE PENETRATION TEST (CPT)

2.4.1 Meyerhof (1956, 1965, 1974)

A simple and rapid method for estimating settlement of footings on sand from CPT tip resistance was proposed by Meyerhof (1956, 1965 and 1974) using the following relations:

$$s = (qB)/(2q_c) \qquad (2.61)$$

where
s = settlement (in ft.)
q = net foundation stress (in tsf)
B = footing width (in ft.)
q_c = average cone tip resistance over a depth equal to B below the footing (in tsf)

Meyerhof (1974) used the results of 20 case histories to check the accuracy of the method and found that the mean ratio of calculated to measured settlements was about 1.25 over a settlement range from 7.6 to 84 mm (0.3 to 3.3 in).

2.4.2 Schmertmann (1970)

Schmertmann (1970) proposed a method for calculating to settlements of shallow foundations on sands by subdividing the compressible zone beneath the footing into individual layers and then summing the settlement of each sublayer. The method relies heavily on an assumed vertical strain distribution which develops beneath the footing. As presented originally by Schmertmann (1970), this method is often refereed to as "2B-0.6" method. An approximate strain influence diagram given in Figure 2.15 to calculate settlement over a zone of influence equal to 2B below the footing, with in a maximum value of the influence factor of 0.6 at B/2 below footing base, irrespective of the footing shape. Eq. 2.62 is used to calculate settlement.

2.4.3 Schmertmann et al. (1978)

The settlement of granular sand can also be evaluated by the use of a semi empirical strain influence factor proposed by Schmertmann et al. (1978).

$$S_e = C_1 C_2 (\bar{q} - q) \sum_{i=1}^{n} \frac{I_{zi}}{E_{si}} \Delta z_i \qquad (2.62)$$

where
I_{zi} = strain influence factor for layer i
C_1 = correction factor for the depth of foundation embedment
$C_1 = 1 - 0.5[q/(\bar{q} - q)]$
C_2 = correction factor to account for creep in soil
$C_2 = 1 + 0.2\log$ (time in years/0.1)
$\bar{q}$ = stress at the level of the foundation
q = initial effective overburden pressure at the foundation level
E_{si} = soil modulus for layer i; recommended using a weighted average of E_s (Eq. 2.14)
Δz_i = thickness of layer of constant E_{si}
i = layer i

n = total number of layers

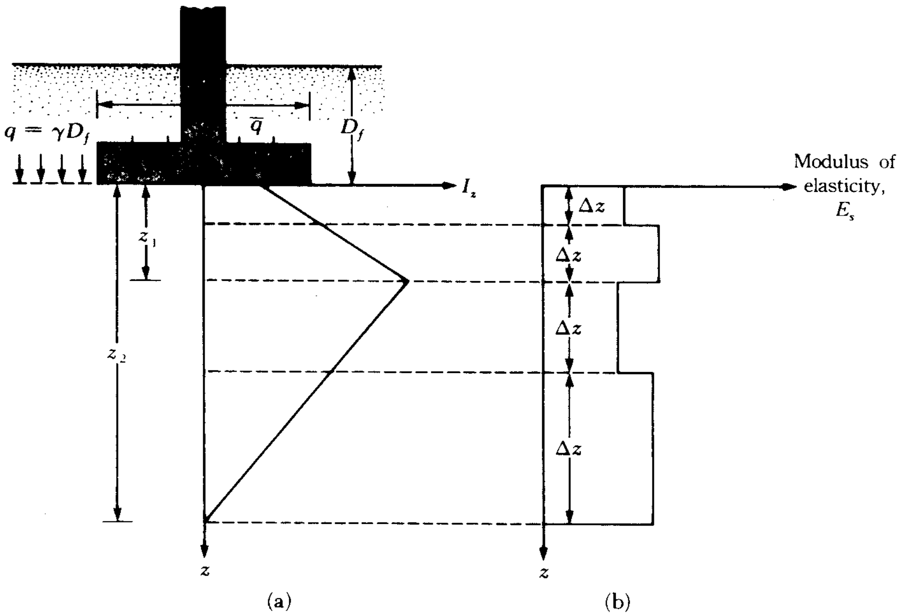


Figure 2.14 Depth of maximum strain influence factor z_1, depth of influence zone z_2 and thickness of soil layers Δz description

The variation of the strain influence factor width below the foundation is shown in Figure 2.14a, Table 2.8 shows the strain influence factor at different depth, (B is the width of the foundation, L is the length of the foundation). Figure 2.15 shows the calculation of elastic settlement by using the strain influence factor chart.

Table 2.8 Variation of I_z by different depth z

	z	I_z (Schmertmann, 1970)	I_z (Schmertmann, 1978)	I_z (Schmertmann (Das, 2004))
square or circular foundations	0	0	0.1	0.1
	$z=z_1=0.5B$	0.6	$0.5+0.1[\Delta q/\sigma'_{vp}]^{0.5}$	0.5
	$z=z_2=2B$	0	0	0
foundations with L/B≥10	0	0	0.2	0.2
	$z=z_1=0.5B$	0.6	---	---
	$z=z_1=B$	---	$0.5+0.1[\Delta q/\sigma'_{vp}]^{0.5}$	0.5
	$z=z_2=2B$	0	---	---
	$z=z_2=4B$	---	0	0

Note: Δq = net applied footing stress

σ'_{vp} =initial vertical effective stress at maximum I_z for each loading case (i.e., 0.5B for axisymmetric and B for plane strain)

Schmertmann (Das, 2004) lists the influence factors as given in Das (2004)

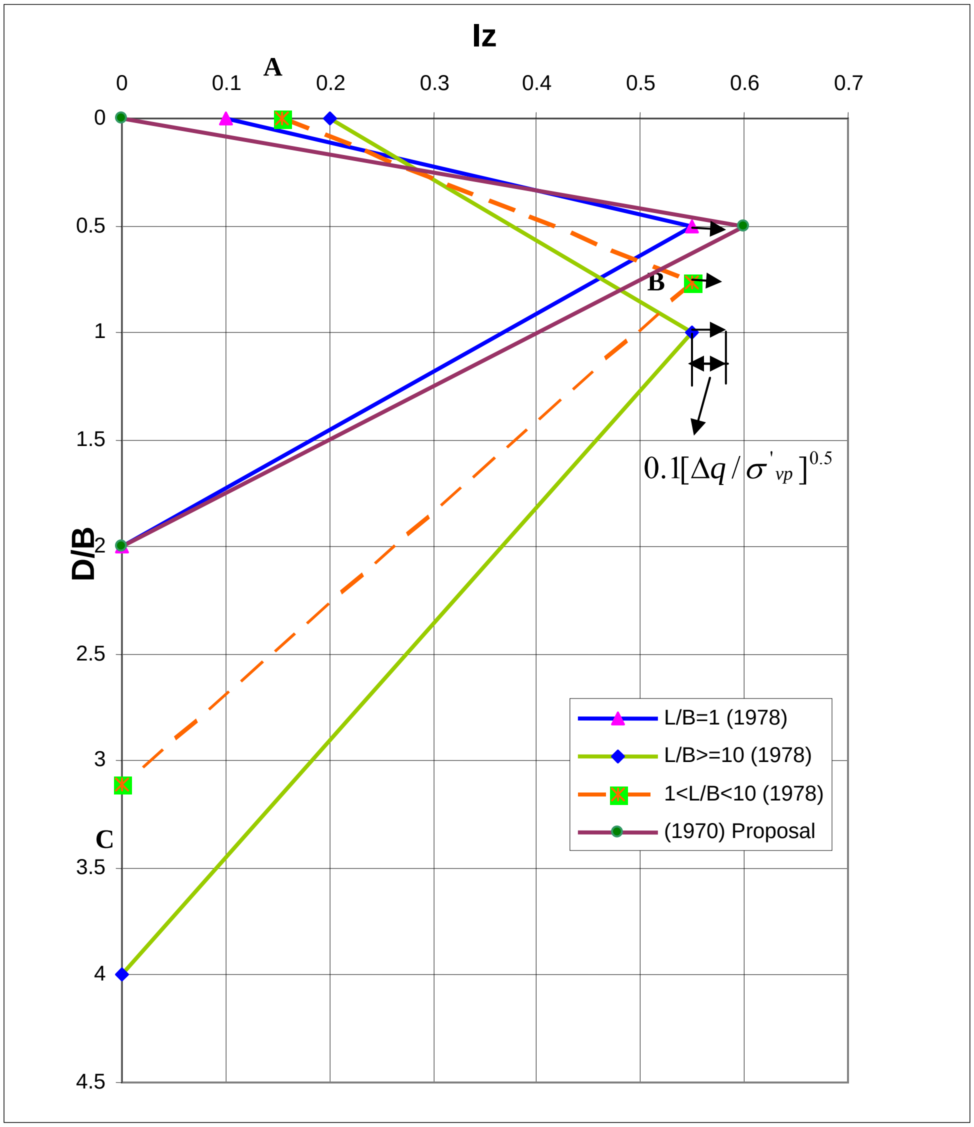


Figure 2.15 Variation of strain influence factor Iz by different depth Z Chart (Schmertmann, 1970, 1978)

The use of Eq. 2.62 requires the evaluation of the modulus of elasticity with depth (Figure 2.14). This evaluation can be made by using the standard penetration test numbers or the cone penetration resistances. The soil is divided into several layers to a depth of $z = z_2$, and the elastic deformation of each layer is estimated. The sum of the deformation of all layers equals the immediate settlement S_e.

These equivalent Young's Modulus E_s (tsf) estimated using Dutch cone bearing capacity (q_c) (kg/cm^2) are calculated as follow:

For footing length to width ratio (L/B):

L/B=1 $\quad E_s = 2.5q_c \quad$ (2.63)

L/B=10 $\quad E_s = 3.5q_c \quad$ (2.64)

$1<L/B<10$ interpolate between $2.5q_c$ and $3.5q_c$ (2.65)

If only SPT results are available to engineer, the SPT blow count value needs to be converted to CPT cone tip resistance value by using the q_c/N ratio, which is as given in Table 2.9 for different soil types.

Table 2.9 Empirical Relationship of Modulus of Elasticity

Empirical Equation		Reference	Note:
$E_s=2q_c$ (E_s in tsf)		Schmertmann (1970)	N: stand penetration resistance
$E_s=2.5q_c$ (for axisymmetric cases, E_s in tsf))		Schmertmann et al. (1978)	E_s:modulus of elasticity
$E_s=3.5q_c$ (for plain strain cases, E_s in tsf))			q_c:cone resistance (kg/cm^2)
Empirical Value			
Soil Type	**q_c/N**		
Silts, sandy silts, slightly cohesion silt-sand mixtures	2	Schmertmann (1970)	
Clean, fine to medium, sand & slightly silty sands	3.5		
Coarse sands & sands with little gravel	5		
Sandy gravels with gravel	8		

2.5 METHODS UTILIZING PLATE LOAD TEST RESULTS

2.5.1 Terzaghi and Peck (1948, 1967)

Terzaghi and Peck (1948, 1967) proposed a relationship between the settlement of a footing of width B (ft.) s_B and the observed settlement of a 0.305m (1 ft.) plate loaded to the same stress level s_1:

$$(s_B)/(s_1) = ((2B)/(B+1))^2 \qquad (2.66)$$

For large footings on the order of B > 8ft, the ratio tends to arrive to a maximum value of about 4 as shown in Figure 2.16.

Bejerrum and Eggestad (1963) demonstrated that there could be considerable scatter in the settlement ratio observed from different cases and those settlement ratios much larger than 4 could occur. They suggested the settlement ratio being dependent on the density with loose sands giving higher settlement ratios and dense sands giving lower settlement ratios. A comparison between the curves of Terzaghi and Peck and those presented by Bjerrum and Eggestad (1993) is shown in Figure 2.17. It has also been suggested by Meigh (1963) that the settlement ratio is dependent on the soil gradation with coarse, well graded soil having low settlement ratios and fine, uniformly graded soil having high settlement ratios.

Arnold (1980) suggested that the relationship presented by Terzaghi and Peck (1967) and given in Eq. 2.66 be modified as:

$$s_B / s_1 = (2B/(B^{\lambda}+1))^2 \qquad (2.67)$$

where:

$$\lambda = 0.788 + 0.002D_r \qquad (2.68)$$

where D_r = relative density

With this recommendation, the Terzaghi and Peck curve would correspond to a relative density of about 85% whereas the average Bjerrum & Eggestad curve would correspond to a relative density of about 35%.

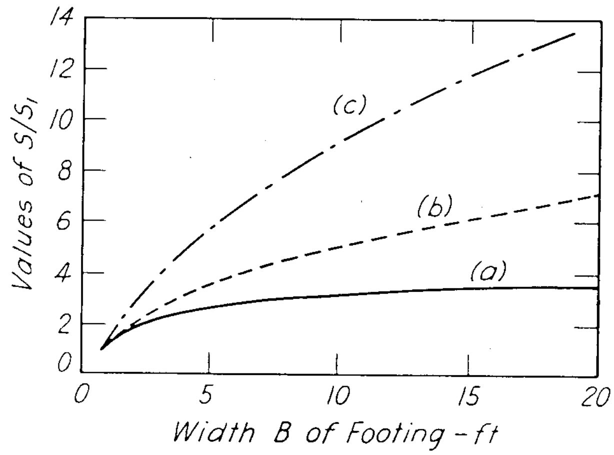
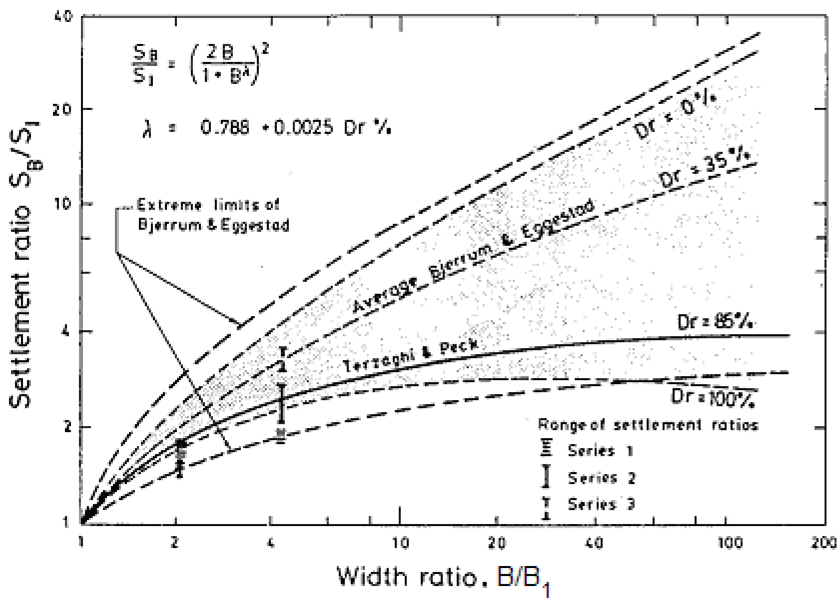


Figure 2.16 Settlement Ratio as a function of Footing Width, Terzaghi and Peck (1967)



(B_1 = 1ft, B is footing width)

Figure 2.17 Comparisons between Terzaghi and Peck and Bejerrum and Eggestad Curves, Lutenegger (1995)

2.5.2 Carrier and Christian (1973)

Carrier and Christian (1973) used the finite element method to solve the settlement and stress induced by rigid circular plate resting on a non-homogeneous elastic half-space defined by a Young's modulus (E) which increases linearly with depth according to:

$$E = E_0 + Kz \qquad (2.69)$$

where E_0 = Young's modulus at surface (i.e. z=0)
K = rate of increase in E with depth

z = depth

These solutions were compared with solutions in which E is assumed to be constant with depth and equal to E_0 and in which E at the surface is equal to zero but increases linearly with depth.

The results were presented by considering the elastic ratio as a function of a foundation width, similar to what had previously been presented by Terzaghi and Peck (1948, 1967) and Bjerrum and Eggestad (1963). Figure 2.18 presents solutions for various ratios of E_0 / K ranging from 0 to ∞. These results indicate that the settlement ratio of footings on a non-homogenous half-space increases linearly with the logarithm of the footing width. This observation means that if the result of plate load tests are available for a particular site (preferably for B=0.3m and at least one additional size between 0.6 to 1 m) an appropriate value of E_0 / K may be obtained and then the settlement of the production footings may be estimated based on extrapolation.

An alternative approach is by using the results of penetration tests, such as the CPT or SPT to evaluate the variation in the soil modulus with depth to obtain the value of K. The value of E_0 would then be obtained, as before, using a plate load test on a 0.3m (1 ft) wide plate.

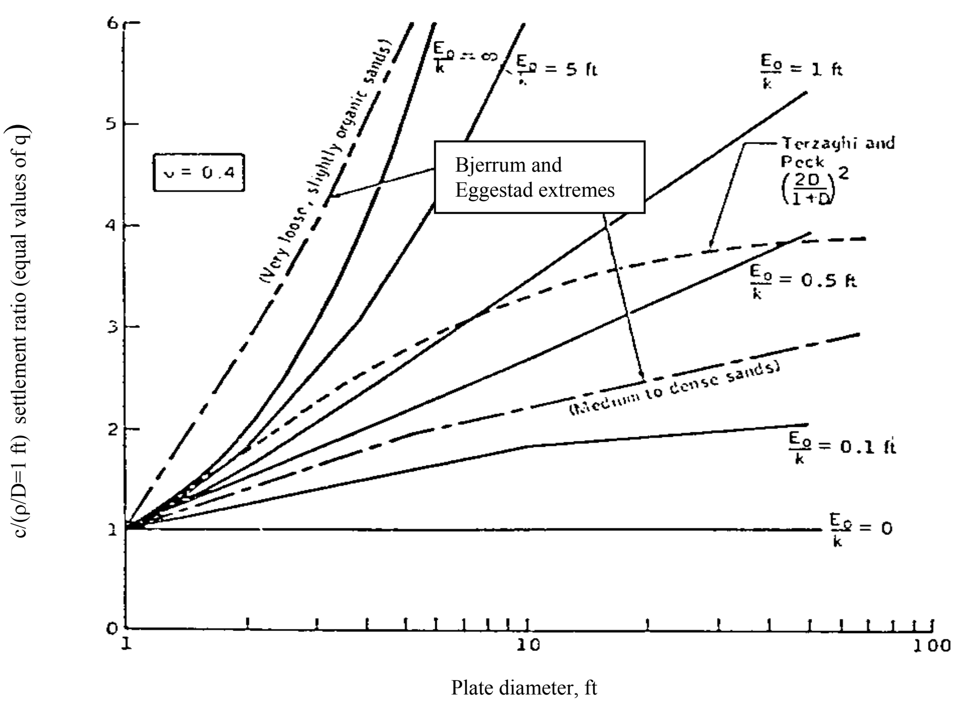


Figure 2.18 Settlement Ratio Curves Presented by Carrier and Christian (1973)

2.5.3 Das (2004)

In his text book, Das gives the following reduction to estimate the ultimate and allowable bearing capacities of footings using prototype test footing on the soil.

For tests in clay:

$$q_{u(F)} = q_{u(P)} \qquad (2.70)$$

where $q_{u(F)}$ = ultimate bearing capacity of the proposed foundation

$q_{u(P)}$ = ultimate bearing capacity of the test plan

For tests in sandy soil:

$$q_{u(F)} = q_{u(P)} \frac{B_F}{B_P} \tag{2.71}$$

where B_F = width of the foundation

B_P = width of the test plate

The allowable bearing capacity of a foundation, based on settlement considerations and for a given intensity of load, q_0, is

$$S_F = S_P \frac{B_F}{B_P} \text{ (for clay soil)} \tag{2.72}$$

and $$S_F = S_P \left(\frac{2B_F}{B_F + B_P}\right)^2 \text{ (for sandy soil)} \tag{2.73}$$

2.5.4 Alpan (1964)

An additional settlement method based primarily on the Terzaghi and Peck (1948) approach was presented by Alpan (1964). This method indirectly uses a corrected blow count to evaluate a modulus of subgrade reaction from a plate loading test.

The method assumes that the settlement response of a shallow footing resting on sands will be linear in the range of allowable bearing pressures (i.e., $q_{ult}/2.5$) and is given as:

$$s = s_0[2B/(B+1)]^2 mC_w \tag{2.74}$$

where s = settlement (in inches)

s_0 = settlement of a 1 ft^2 plate (in inches)

B = footing width (in ft.)

m = shape correction factor (Table 2.7)

C_w = water table correction factor

$= 0.5(D/B) \le 2.0$ for water located immediately below the footing