SETTLEMENT OF SHALLOW FOUNDATIONS ON GRANULAR SOILS

Final Report

by

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EXECUTIVE SUMMARY

This report presents the results of a research project undertaken to provide a comprehensive state-of-the-art review of the procedures used by the geotechnical engineering profession to estimate the settlement of shallow foundations resting on granular soil deposits. A comprehensive review was made of the literature in order to summarize all of the existing design methods available and to assemble reported case histories involving documented settlement of shallow foundations on granular deposits. A Windows-based PC operated software package was developed which incorporates the majority of the common design methods and allows the operator to predict settlement of a proposed foundation using the available methods. The results of the work are presented in this report and accompanying Appendices that comprise the overall final report.

A stand alone Computer Program Users Appendix presents background of the programming language and a description of the software development. A users manual is included which provides step-by-step instructions on how to operate the software. The use of the software is illustrated in this Appendix by showing examples of calculated settlements for the FHWA footing load tests recently performed at Texas A&M University and other published cases. Additionally, in the final report a comparison is presented between the predicted settlement of a 3 m by 3 m footing using common SPT and CPT methods and actual settlement.

A standalone Case Histories Appendix contains a compendium of reported case histories involving settlement of shallow foundations resting on granular soil deposits. The compilation is limited to cases involving field large scale plate and footing tests and full size structures and is separated into several categories including tanks, mats and rafts, small footings (1m <B <3m) and large footings (3m< B <6m). Each case history includes a reference for the data, reported foundation geometry, reported soil conditions and properties, and reported load and settlement observations. The case histories serve to allow a check of full-scale performance on the accuracy of a given prediction method.

Recommendations are given at the end of this report for improvements in site characterization and improvements in settlement analyses. It is anticipated that the implementation of these recommendations will produce a better approach to the evaulation of settlements of shallow foundations on granular soils.

The results of the project indicate that a number of methods are outdates and should not be used in predicting settlements. Other, more modern methods, appear to give more reasonable approaches and results. The results presented in this report should enable design engineers to formulate exploration programs better suited for evaluating settlement and should allow more reliable settlement estimates to be made. This will allow more frequent use of cost-effective shallow foundation systems in lieu of more expensive deep foundation systems for transportation related structures such as highway bridges.

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1.0 INTRODUCTION

This report presents a state-of-the-practice review of the procedures available for estimating the settlement of shallow foundations on granular soil deposits. In transportation related construction, many situations occur in which shallow foundations may be used to support structural loads. A common example is the support of dry crossings located at bridges. The use of shallow footings in lieu of deep foundation systems provides a much more economical system, and can result in substantial cost savings for a project. Unfortunately, the uncertainty involved in the estimation of settlement of shallow footings on granular soil deposits, e.g., sand, sand and gravel, etc. presents a monumental task to the designer.

Abundant sand and gravel deposits are present throughout the Commonwealth of Massachusetts. This makes the use of shallow foundations an attractive alternative to deep foundations for the support of bridge piers and abutments. In most cases, the limit equilibrium or bearing capacity provided by these deposits is sufficient to provide support. However, settlement estimates for these structures made by using most traditional methods lead to predicted excessive settlements, which are not considered tolerable for most situations.

Unlike most other soil deposits, granular soil materials do not allow undisturbed sampling without great difficulty and expense to provide reliable specimens for laboratory testing to characterize the nature of the material or the stress-strain properties for use in settlement calculations. This means that the evaluation of the deformation characteristics of granular soils will usually be evaluated by field tests employing in situ techniques.

The subject of calculating settlement of footings on granular soils has received considerable attention in the past forty years. It is one of the most favorite subjects in geotechnical engineering which periodically becomes popular to study. Unfortunately, even though there have been a number of reviews of the subject in recent years, none of these reviews have singularly provided a thorough and comprehensive study of the subject. In addition, none of the previous work has attempted to provide a unifying concept to the problem.

This report is divided into a number of sections presenting an updated state-of-the-practice summary of the problem, a review of the variables affecting the deformation behavior of granular soils and a detailed description of the various techniques which have been proposed in the past to calculating settlements. In addition, a new unifying concept is presented which allows a first approximation to be made based on the coupling concept of relating the load-settlement behavior with the limit equilibrium condition of the foundation. The information presented in this report is accompanied by two additional volumes which present summaries of individual case histories of footing load tests and background of a Windows based personal computer software program which was developed as a part of this project.

2.0 BACKGROUND - SETTLEMENT OF SHALLOW FOOTINGS ON GRANULAR SOILS

Calculations of foundation settlements are a basic and fundamental component of foundation engineering and is a common procedure performed by practicing geotechnical engineers. The deformation behavior of shallow foundations deriving their support from primarily granular particulate soil deposits such as sands and gravels largely controls the final design of structures resting on these materials. This is largely due to the fact that the limit equilibrium behavior, i.e., the bearing capacity, of shallow foundations resting on granular deposits is typically of such a large magnitude, that the allowable settlement criteria established by the engineer will control the overall design.

In transportation related construction, one of the most common uses of shallow foundations is in the support of bridge structures, especially in dry crossing situations, where highway overpasses are needed for crossing over other highways, railroads or other structures. Provided that settlements can be accurately estimated, a shallow foundation provides a more economical foundation then either driven or drilled deep foundations.

Bozozuk (1978) presented the results of a performance survey of existing bridges in the U.S. and Canada to determine the movement that could be tolerated by a structure. Based on the results of the performance of about 120 abutments and piers on spread footings, the vertical movements ranged from approximately 0 to 1000 mm, while the horizontal movements ranged from 0 to 150 mm. A performance rating was established, as shown in Figure 2.1, which suggests that horizontal movements affected the structures more than did vertical movements. Additionally, as can be seen in Figure 2.1, the maximum tolerable or acceptable vertical movement of either a pier or abutment was suggested as 50 mm (2 in.). Moulton (1986) surveyed a large number of existing highway bridges in the U.S. and showed that generally, most bridges can tolerate more than 25 mm of settlement, that bridges founded on spread footings do not settle more than bridges on piles and that damage to bridges cannot be attributed to spread footing foundations more than to pile foundations. The tolerable movement of bridges and the use of shallow foundations has also been discussed by Wahls (1983) and Yokel (1990).

A number of studies have been published in the past 40 years comparing the results of calculated settlements with observed settlement of shallow foundations on granular soils. Some of these studies have been related to proposing a new settlement prediction method, while others have attempted to provide a comparison among various methods to evaluated whether or not any one particular method appears to provide superior accuracy over another. Most notably, review papers which summarize settlement observations or provide comparisons between predicted and observed settlements have been presented by Alpan (1964); Schmertmann (1970); Jorden (1977); Arnold (1980); Burland and Burbidge (1985); Jeyapalan and Boehm (1986); Maail (1987); and Berardi and Lancellotta (1991).

The reliability of settlement estimates for shallow foundations on granular soils has also received considerable attention and has been discussed by Schultze and Sievering (1977); Tan and Duncan (1991); Nova and Montrasio (1991a, 1991b); Cherubini and Greco (1991); and Berardi and Lancellotta (1994).

Investigations of the settlement behavior of bridge abutments and piers resting on granular soils, which is the primary focus of this project, have also been performed by a number of researchers. Table 2.1 summarizes previous reported studies involving settlement of bridges, piers and abutments on granular soils.

Most of the available methods for predicting the settlement of shallow foundations on granular soils rely on the results of in situ tests. The results of the tests are either used: (1) to estimate an elastic modulus of the soil which is used in an elastic analysis; (2) directly to estimate settlement using an empirical correlation; or (3) to estimate some other soil property, such as relative density, and then an estimate of settlement is made.

A review of previous comparisons made between predicted and observed settlement of shallow footings in sands or sands and gravels reveals that no single method works better than any other method in all cases. Some methods appear to work better than others and it appears that more recent methods are promising. This may be in part related to the fact that our understanding of soil behavior has increased but may also be the result of careful consideration of all of the factors that may influence performance of an individual foundation.

The majority of available methods for estimating settlement assume a linear response between load and deformation of granular material (i.e. a constant modulus). Additionally, there has been little effort to relate the settlement or relative displacement to the <u>level</u> of a load; i.e. - relative to an ultimate or failure load. This appears to be the primary deficiency in existing methods.

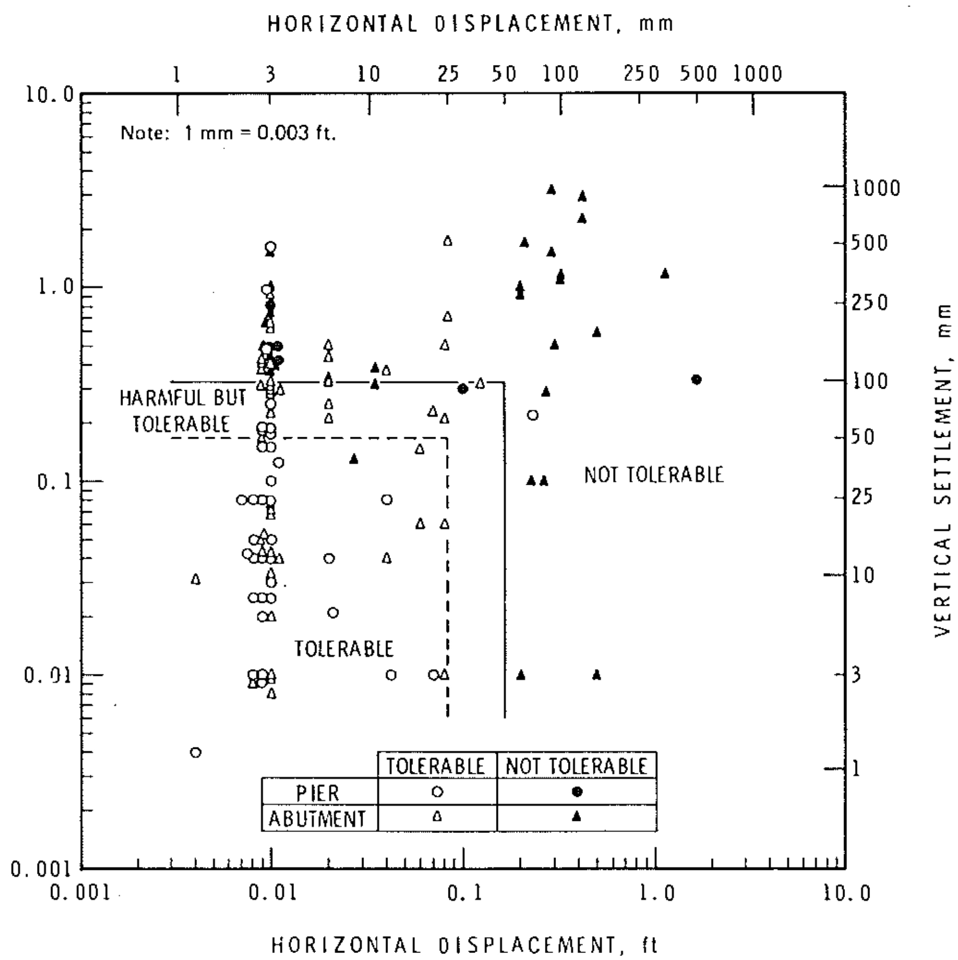


Figure 2.1 Engineering Performance of Bridge Abutments and Piers on Spread Footings (from Bozozuk 1978).

Table 2.1 Previous Investigations of Settlement of Bridge Foundations on Granular Soil.

Bridge	Reference
Afsnee Drongen St. Deny's - Western Beernem (Highway No. 70) Aalter Gentbrugge Beernem (Wellingstreet) Loppem	DeBeer (1948)
No. 13 St. Deny's - Westerm No. 14 St. Deny's - Westerm No. 18 Merelbeke No. 19 Merelbeke No. 20 Melle No. 22 Gontrode No. 24 Westerm No. 25 Oordegem No. 39 Loppem	Marivoet (1953)
No. 49 Marie - Aalter No. 29 Loppem No. 73 Drongen No. 75 Drogen No. 43 Beernem No. 14 St. Deny's - Westerm	DeBeer and Martens (1957)
No. 23 No. 24	Wennerstrand (1979)
Kolari Alvsbyn	Bergdahl & Ottosson (1982)
Burlington Cheshire Providence Colliersville Uxbridge Chester Manchester (Conrail) Manchester (Tolland) Manchester (Rt. 84)	Gifford et al. (1987)
Ottawa	Felio and Bauer (1994)

3.0 DESIGN APPROACHES FOR SETTLEMENT ESTIMATES

3.1 Introduction

As indicated in Section 2.0, there are essentially two separate design approaches that can be taken when using the results of in situ tests in geotechnical design: (1) Indirect Design and (2) Direct Design. In general, most engineers are currently using an indirect approach to apply the results of in situ tests to specific problems, although there are a number of cases in which direct design may be useful and appropriate.

3.2 Indirect Design

The indirect design approach relies on the interpretation of the results obtained from in situ tests to determine conventional design parameters of soils and the subsequent application of these parameters in a more-or-less traditional design methodology. An example of indirect design would be to use the results of a field vane test to estimate the undrained shear strength of a clay (S_u) which would then be used in the well known bearing capacity equation to predict the undrained end-bearing capacity of a driven pile (e.g., $Q_{end} = 9S_u$). This approach is one which most engineers would be more comfortable with since it uses design procedures that they have traditionally used and with which they are most familiar. A drawback to this design approach is that a transformation must be made between the measurement made in the test and the specific soil property needed for the design. In the example of the field vane test given above, one actually measures the torque in the test and uses it to obtain the undrained shear strength of the soil by making a series of assumptions relative to the behavior of the soil, drainage conditions, failure surface, shear stress distribution, strain rate, etc; all of which can influence the resulting estimate of undrained strength. On top of this, in the case of the field vane test, experience has shown that often times the results of the test do not always accurately predict performance and in many cases an "adjustment" factor is needed to match test results and field performance. For example, the vane strength correction factor introduced by Bjerrum (1972) for the application of field vane results to embankment design is widely used by practicing engineers, even though they may not be fully aware of the rationale behind it.

Engineers must be critically aware of how such transformations from field test measurements to soil properties are made, on what basis they were developed, what limitations may be imposed or implied, and they should scrutinize these procedures to determine if they are appropriate for a given design situation. Most transformations are based on some theoretical foundation, such as interpretation of the Cone Penetration Test (CPT) and piezocone test (CPTU) results using deep penetration theory, or the interpretation of the pressure meter test (PMT) using cavity expansion theory, however, even these theories have a number of implicit assumptions regarding generalized soil behavior, which are not applicable to all soils.

3.3 Direct Design

In contrast to the indirect design approach, a direct design approach gives the engineer the opportunity to pass directly from the measurement made during the in situ test to the performance of a foundation without the need to evaluate intermediate soil parameters. An example of this

approach would be to use the pressure/expansion curve of prebored pressuremeter to predict the lateral load/deflection characteristics of a drilled shaft. The test procedure closely approximates the construction and load/deformation sequence of the full-scale foundation element by requiring a predrilled hole and subsequently applying load in the lateral direction. This means that the in situ test essentially acts as a prototype of the full-scale member. There are obvious limitations with this approach, since most in situ tests do not model typical geotechnical problems and therefore do not actually serve as prototypes. Other obvious examples of direct design would be the application of CPT tip and sleeve resistance in the design of driven piles.

A direct design approach eliminates most of the assumptions involved in the indirect approach since the results of the test are being used directly in design; i.e. there is no intermediate transformation to a specific soil property. Additionally, the use of traditional algorithms to evaluate performance is eliminated and the performance is directly related to the test results. A drawback of this approach is that an appropriate model is needed to allow input of the field test results into the design model. Unfortunately, only a few models are available.

In the following sections of this report, methods for predicting the settlement of shallow foundations resting on granular soils using both direct and indirect design approaches are presented. By far, the most common methods use the indirect design approach. In most of the methods presented, the evaluation of a specific soil property from the results of an in situ test for use in estimating settlement is based on empirical observations. Users of a specific method should be extremely careful in their approach since the basis of the correlation is in most cases unclear.

4.0 ELASTIC SOLUTIONS FOR ESTIMATING SETTLEMENT

A number of solutions have been suggested for calculating shallow foundation settlements based on elastic methods. This section of the report discusses the generalized elastic approach based on the theory of elasticity and modifications thereof which have been described in the literature. In subsequent sections of this report, other settlement prediction methods are presented which are similar to an elastic approach but obtain soil modulus values from specific in situ tests, as recommended by individual authors. Therefore, the reader will note some obvious and unavoidable overlap in the discussion.

4.1 Generalized Elastic Solution

The general expression for the elastic deformation of a uniformly loaded plate resting on the surface of a uniform, homogeneous, isotropic, semi-infinite elastic half-space can be obtained from the solution presented by Boussinesq or a general theory of elasticity text and has the form:

$$s = [(qB/E)]\ I \qquad [4.1]$$

where:

s = deformation
q = applied foundation stress
B = foundation width
E = Young's modulus
I = influence factor

The influence factor I is included to account for the shape of the foundation and the thickness of the compressible zone. Values of I which are often used with Equation 4.1 were first presented by Steinbrenner (1934) and are reproduced by Terzaghi (1943), Lambe and Whitman (1969), and Bowles (1988). The full form of the equations for the influence factor, I, includes ratios of foundation length/foundation width (L/B), ratios of depth of elastic layer/foundation width (H/B) and Poisson's ratio, μ. The influence factor can be stated as (Taylor and Matyas 1983):

$$I = (1 + \mu)\ [(1 - \mu)\alpha_o - \alpha_1] \qquad [4.2]$$

The factors α_o and α_1 are shown in Figure 4.1, and may be used to calculate values of I for any value of Poisson's ratio between 0 and 0.5 as shown in Figure 4.2.

Giroud (1972) presented values of I for different values of L/B and μ which were based on an exact solution given by Burmister (1956). Taylor and Matyas (1983) made a comparison between the Steinbrenner and Giroud influence factors and found good agreement for all values of H/B $\geq$ 2 and acceptable agreement for all values of H/B > 0.5 for values of Poisson's ratio between 0 and 0.5 as shown in Figure 4.3.

Bowles (1982, 1988) gives the formulation for the Steinbrenner influence factor as:

$$I = F_1 + [(1-2\mu)/(1-\mu)]F_2 \qquad [4.3]$$

Values of F_1 and F_2 for different ratios of H/B and L/B are tabulated by Bowles (1988) and are given in Figure 4.4.

Janbu et al. (1956) presented a chart for the influence factor for depth of the layer referred to as μ_1 as shown in Figure 4.5, which is taken from Christian and Carrier (1978). Christian and Carrier (1978) noted that while there was some uncertainty, the curves presented by Janbu et al. (1956) and shown in Figure 4.5 were apparently obtained from the Steinbrenner approximate method with settlements averaged over a rectangular area. The values of μ_1 from Figure 4.5, are essentially the same as from the Steinbrenner equation for H/B > 5. For H/B less than 5, the calculated values of μ_1 are about 75% of those in Figure 4.5. This suggests that a factor $(1-\mu^2)$, which for $\mu = 0.5$ would be 0.75, was left out of the calculation of μ_1 shown in Figure 4.5. Therefore, Christian and Carrier (1978) suggested that for H/B less than about 5, the values of μ_1 shown in Figure 4.5 should be corrected by a factor of $(1 -\mu^2)$. An improved chart for μ_1 for Possion's ratio = 0.5 was presented by Christian and Carrier (1978) by incorporating Girouds (1972) results for effect of depth and is shown in Figure 4.6.

Since the influence factor I or μ_1 is intended for use with foundation loads applied at the surface of a layer, an additional correction factor is often applied for the effects of embedment or location of a footing beneath the surface. Fox (1948) presented a method to account for the effect of foundation embedment by computing the ratio between the average settlement of a vertically loaded area located at some depth within a semi-infinite elastic half space and the average settlement of the same loaded area located on the surface of the same half space. Therefore, the parameter is the average ratio of average settlements of flexible areas. Unfortunately, Fox (1948) only presented a chart for the parameter for a Poisson's ratio of 0.5 which is shown in Figure 4.7.

Christian and Carrier (1978) compared the chart presented by Janbu et al. (1956) shown in Figure 4.8, with the chart of Fox (1948) and found that they were essentially the same. Values of the Fox correction factor have been tabulated for different values of Poisson's ratio by Bowles (1988) as a function of L/B and D/B. It should be noted, however, that the depth of the soil layer beneath the base of the foundation is not included in the formulation of Fox (1948) and therefore the correction factor values actually only apply to a foundation embedded in an elastic half space. There is no known solution for influence or embedment factors for an embedded foundation resting on a soil of finite thickness and underlain by a rigid base. Bowles (1982, 1988) presents a chart of Fox embedment correction factors for different values of Poisson's ratio as shown in Figure 4.9.

Christian and Carrier (1978) noted that Burland (1970) had proposed revised values of the Fox correction factor to account for the fact that the Fox factor tended to "overstate the case" and

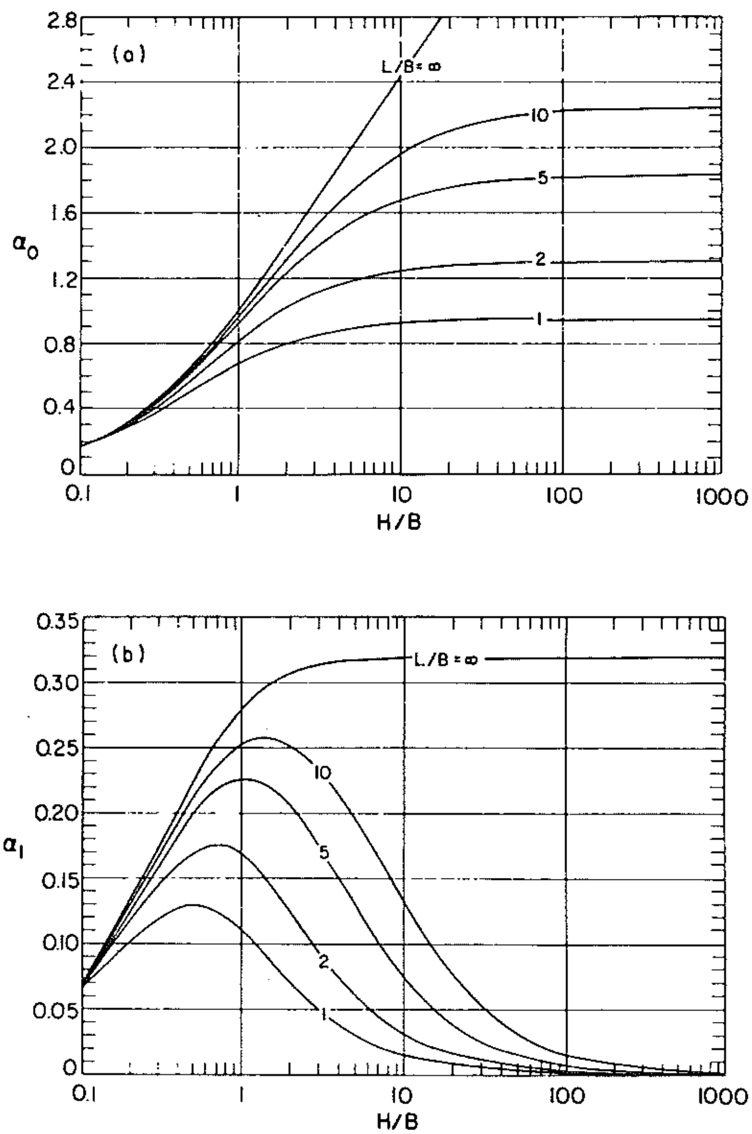


Figure 4.1 Factors a_o (a) and a_1 (b) for Determining the Steinbrenner Influence Factor I (from Taylor and Matyas 1983).

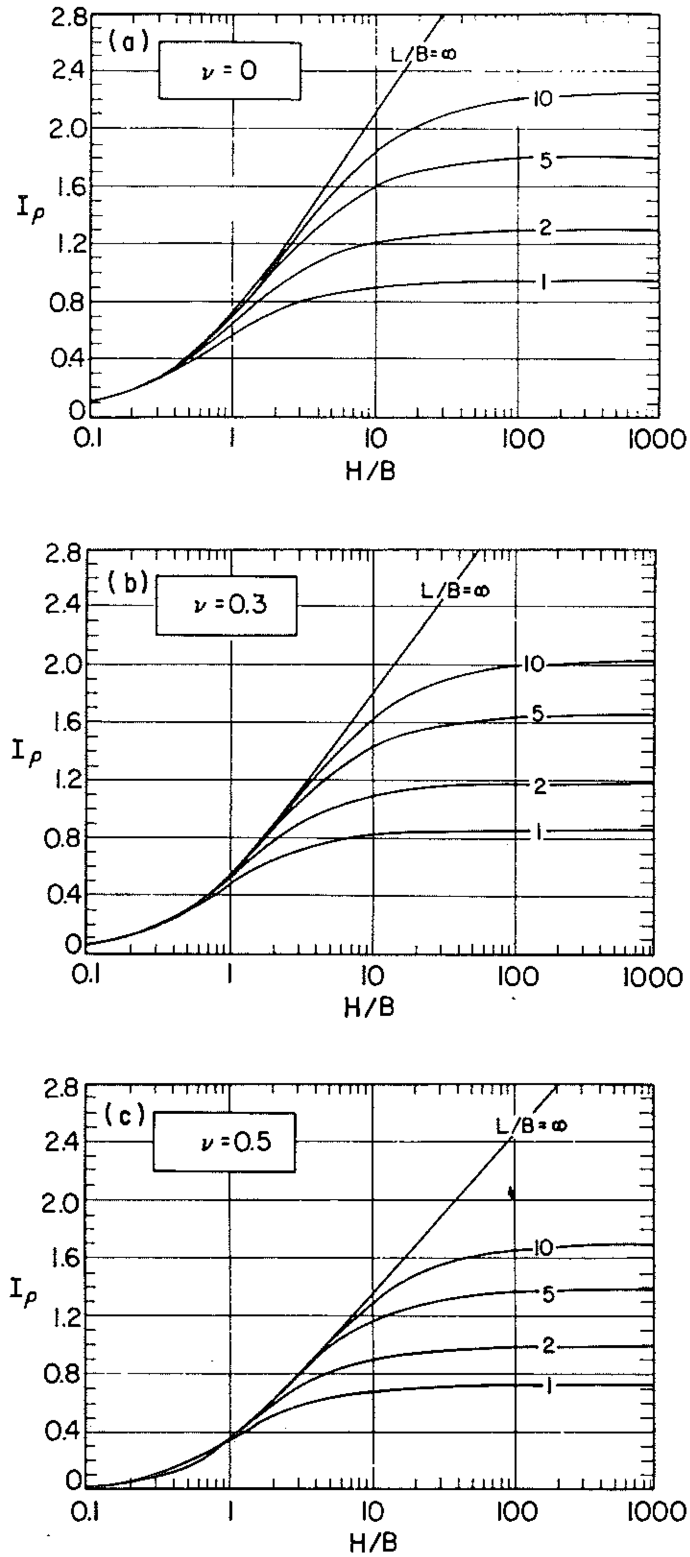


Figure 4.2 Steinbrenner Influence Factor, I, for Various Values of Poisson's Ratio (from Taylor and Matyas 1983).

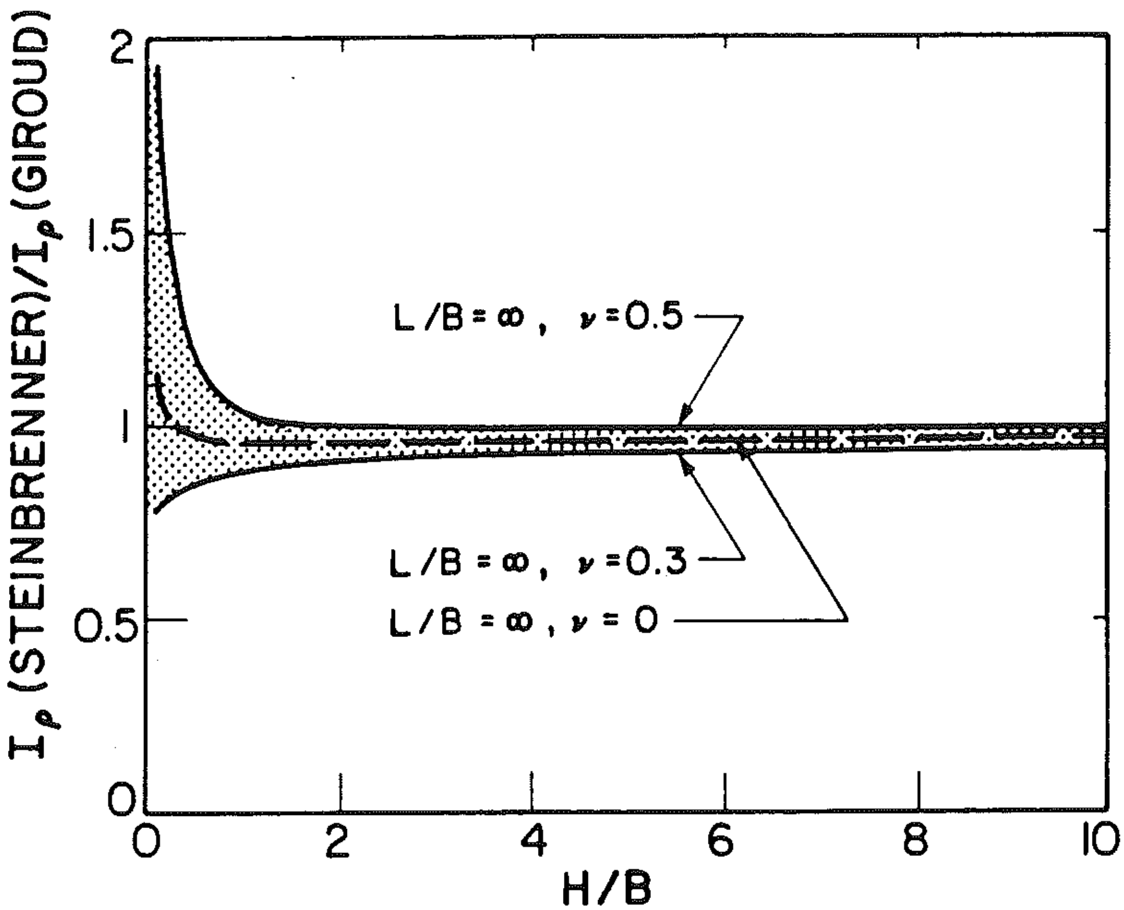


Figure 4.3 Comparison of Steinbrenner and Giroud Influence Factors (from Taylor and Matyas 1983).

H/B (1)	L/B 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

Figure 4.4 Values of F_1 and F_2 for Calculating Steinbrenner Influence Factors (from Bowles 1987).

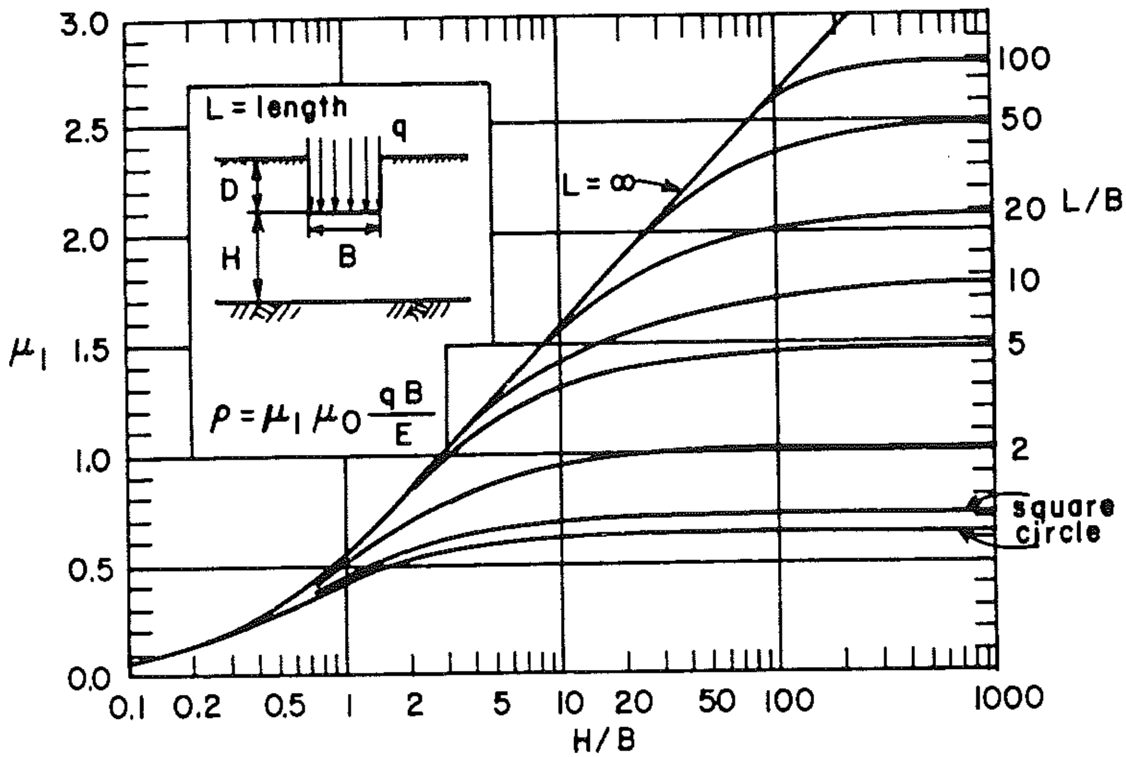


Figure 4.5 Janbu et al. (1956) Chart for Influence Factor (after Christian and Carrier 1978).

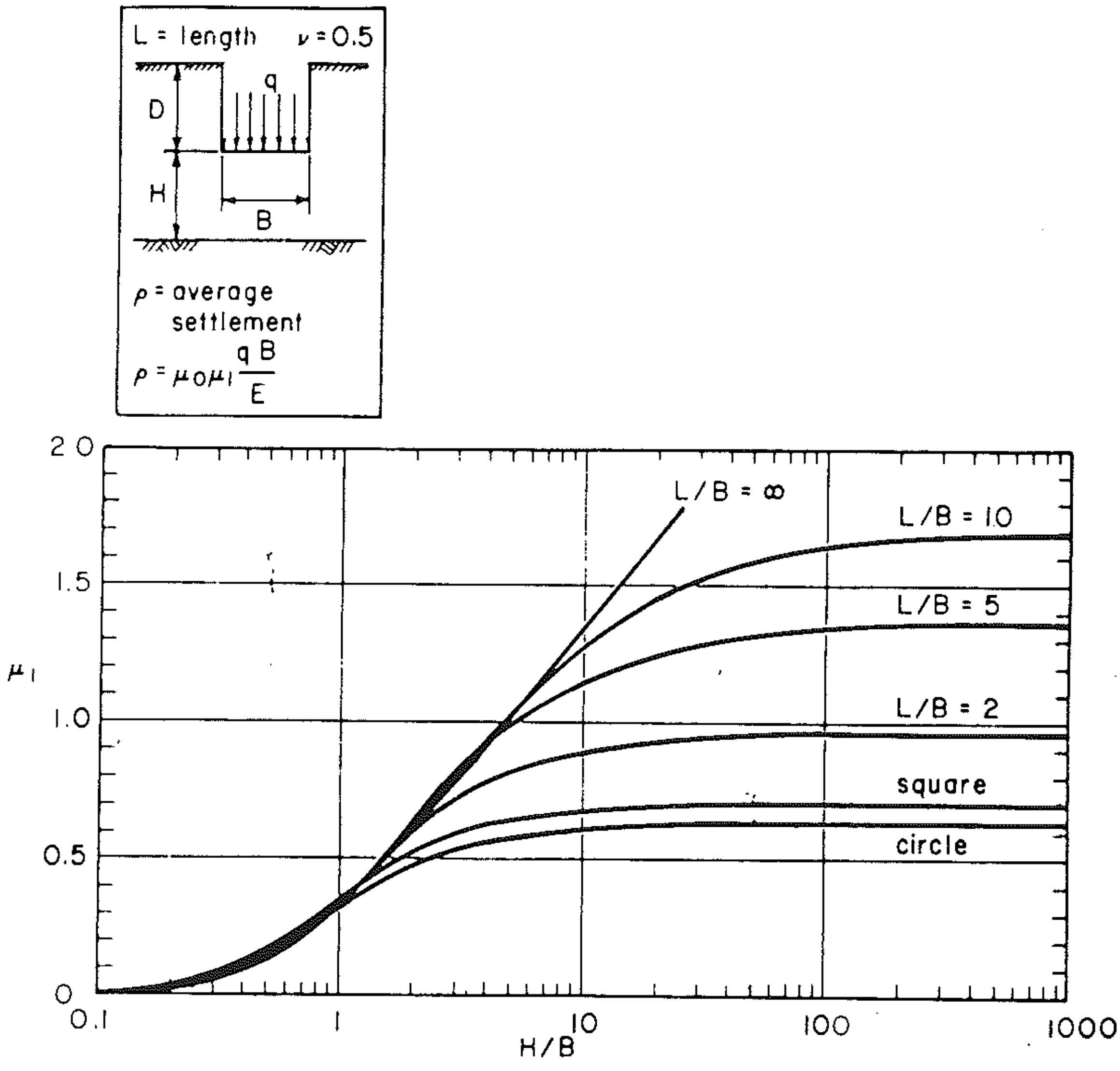


Figure 4.6 Improved Influence Factor Chart Proposed by Christian and Carrier (1978).

thus underestimate settlement. A comparison between the Fox (1948) and Burland (1970) charts for a circular area is shown in Figure 4.10. Christian and Carrier (1978) suggested that Burland's (1970) embedment correction factors be used with Giroud's (1972) depth factors as shown in Figure 4.11 but also noted that "ignoring embedment all together is nearly as good a procedure and may be the best approach when other effects .... are to be considered". Christian and Carrier (1989) restated their position in a discussion to Bowles (1987).

Other factors have been suggested to account for embedment of the foundation, e.g., Yamaguchi (1984), however, the factors presented by Fox, Janbu or Burland appear to be more often used in practice.

4.2 Tschebotarioff (1953, 1971)

Tschebotarioff (1953, 1971) suggested a simplified method of settlement analysis useful for footings resting on sands and other cohesionless soils. The method, as applied to square footings, assumes that the surface load is carried within the soil mass by a truncated pyramid of soil. The surface settlement is equal to the compression of the entire pyramid of height H. The total compression is the sum of the compressive strains of all of the successive horizontal layers dH of the pyramid. Each of the successive layers occupies a horizontal area $A = (b + 2H \tan\alpha)^2$, where α is defined as shown in Figure 4.12. For an assumed value of $\alpha = 30^\circ$, the settlement is given as:

$$s = (0.867\, qbC_s)/E \qquad [4.4]$$

where:

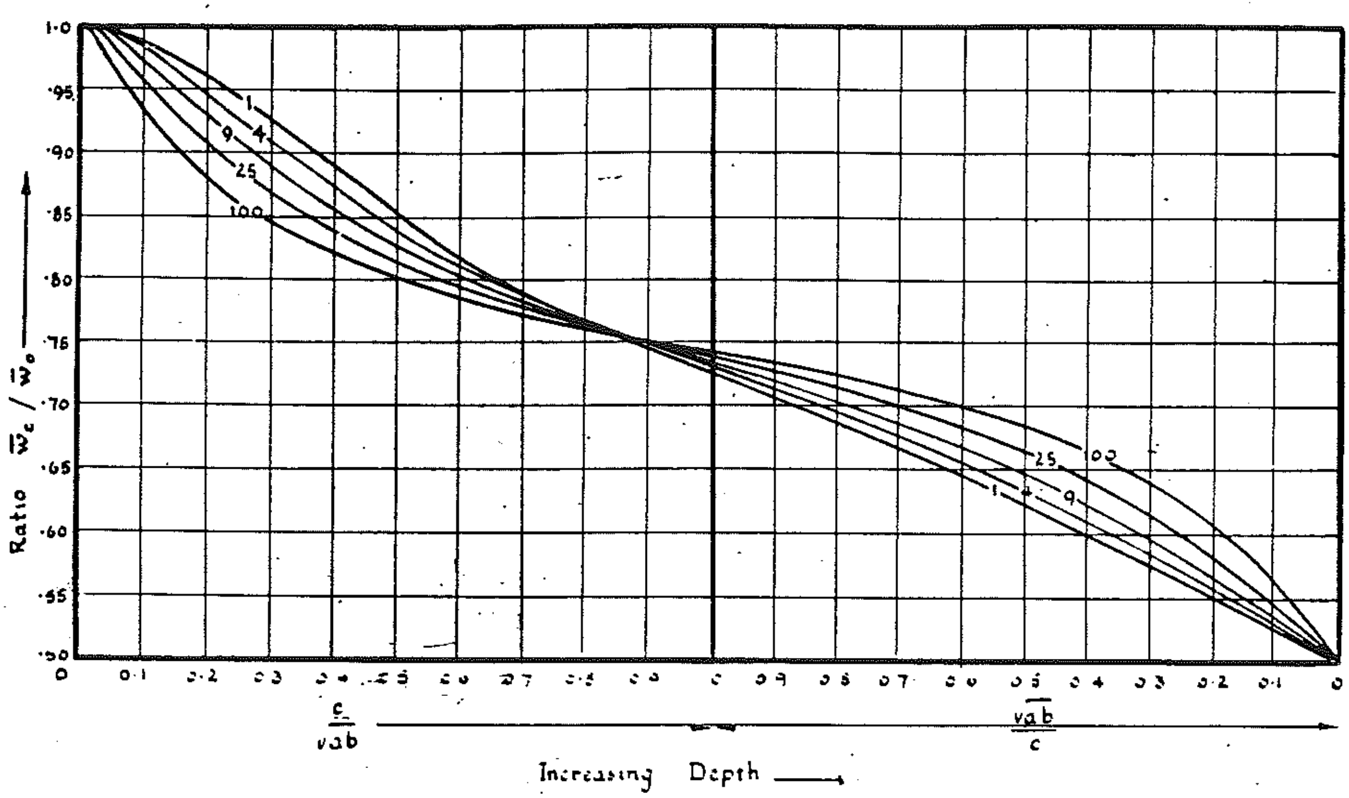
q = applied footing stress
b = footing width
C_s = layer thickness correction factor
E = Young's Modulus

The correction factor C_s is to account for values of H less than infinity. Values of C_s for various values of H/b are shown in Figure 4.13. For an infinity long strip footing of width b, the settlement may be obtained in a similar manner for $\alpha = 30^\circ$ as:

$$s = [(2.0qb)/ E] \log [1 + (1.154H)/b] \qquad [4.5]$$

4.3 Canadian Foundation Manual (1975, 1985, 1992)

The Canadian Foundation Manual (CFM) suggests that settlement estimates of footings may 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 to determine the



Ratio of Mean Settlements of Flexible Rectangular Footing a x b at Depth c and similar Footing at Surface
(Numbers on curves denote value of ratio $^a/_b$ which is constant along any one curve)

Figure 4.7 Fox (1948) Embedment Correction Factor.

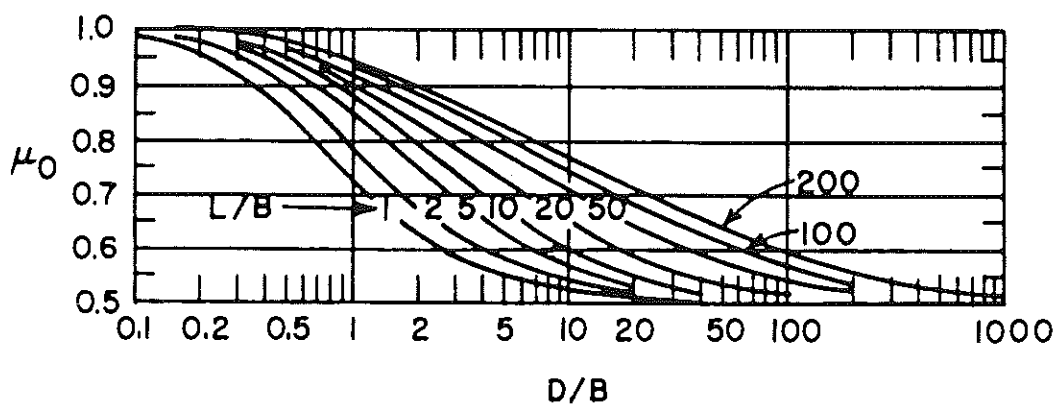


Figure 4.8 Embedment Correction Factor Chart Presented by Janbu et al. (1956; from Christian and Carrier 1978).

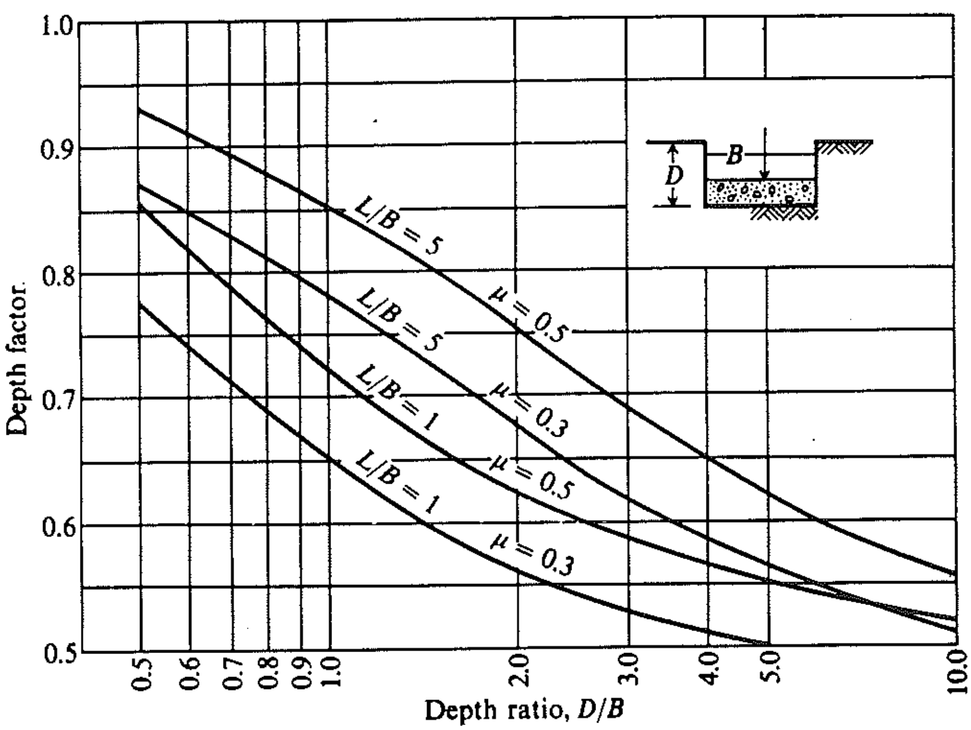


Figure 4.9 Fox Embedment Correction Factors (from Bowles 1982, 1988).

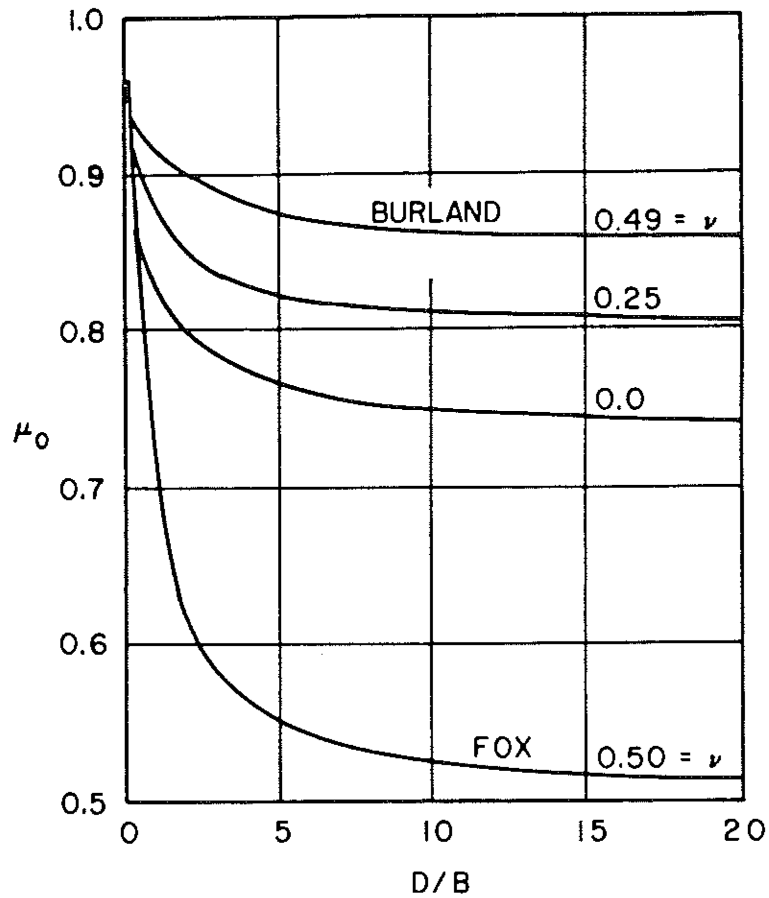


Figure 4.10 Comparison Between the Fox Embedment Correction Factor and the Factor Suggested by Burland (from Christian and Carrier 1978).

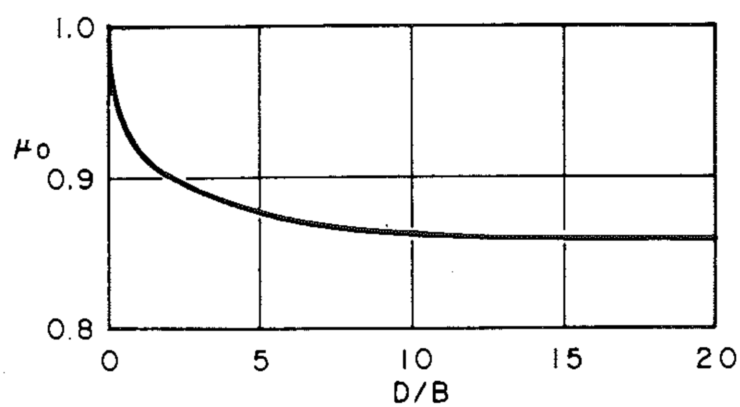


Figure 4.11 Embedment Correction Factor Recommended by Christian and Carrier (1978).

settlement of each layer. The layer strain, E_z, is determined according to:

$$E_z = q_z/E_s \quad [4.6]$$

where:

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

The total settlement is obtained from:

$$s = \Sigma\, E_z\, h_z \quad [4.7]$$

$$\text{or } s = \Sigma(q_z/E_s)h_z \quad [4.8]$$

where:

s = settlement
h_z = thickness of individual layers

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_o, the corresponding stress at a depth z is:

$$q_z = [q_o\, BL]/[(B+z)(L+z)] \quad [4.9]$$

For an infinitely long (strip) footing, Equation 4.9 becomes:

$$q_z = (q_oB)/(B + z) \quad [4.10]$$

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

$$s = (q_o\, B\, i_c)/E_s \quad [4.11]$$

where:

s = settlement
q_o = applied net footing stress
B = footing width
E_s = apparent modulus elasticity
i_c = influence factor

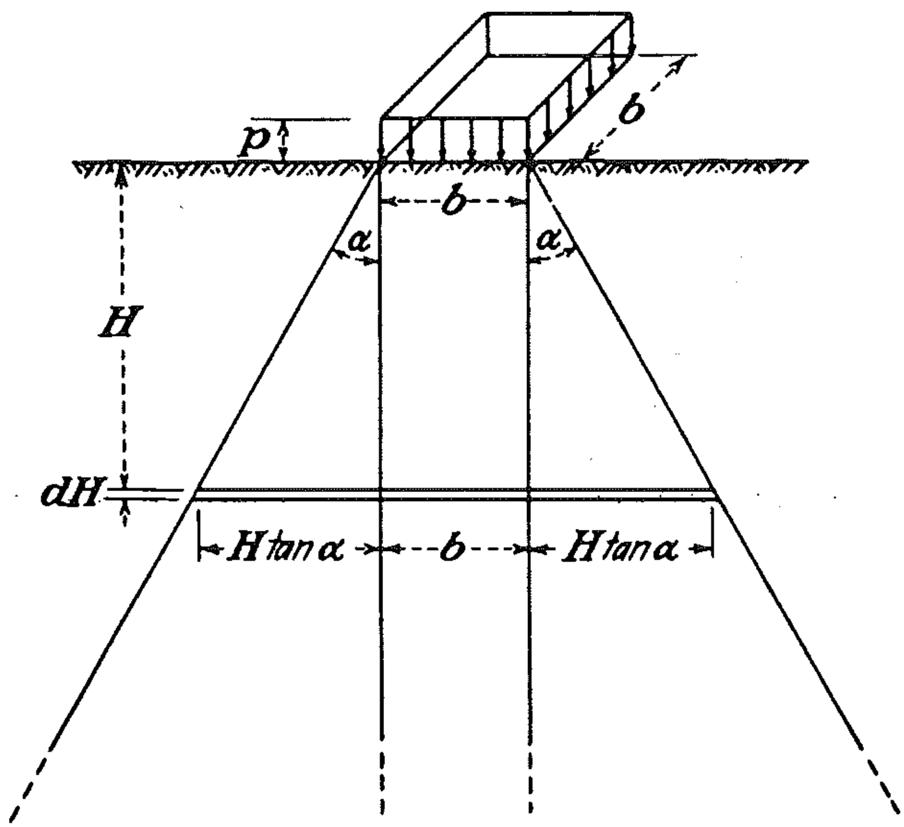


Figure 4.12 Compression of a Truncated Pyramid of Elastic Material
(after Tschebotarioff 1953, 1971).

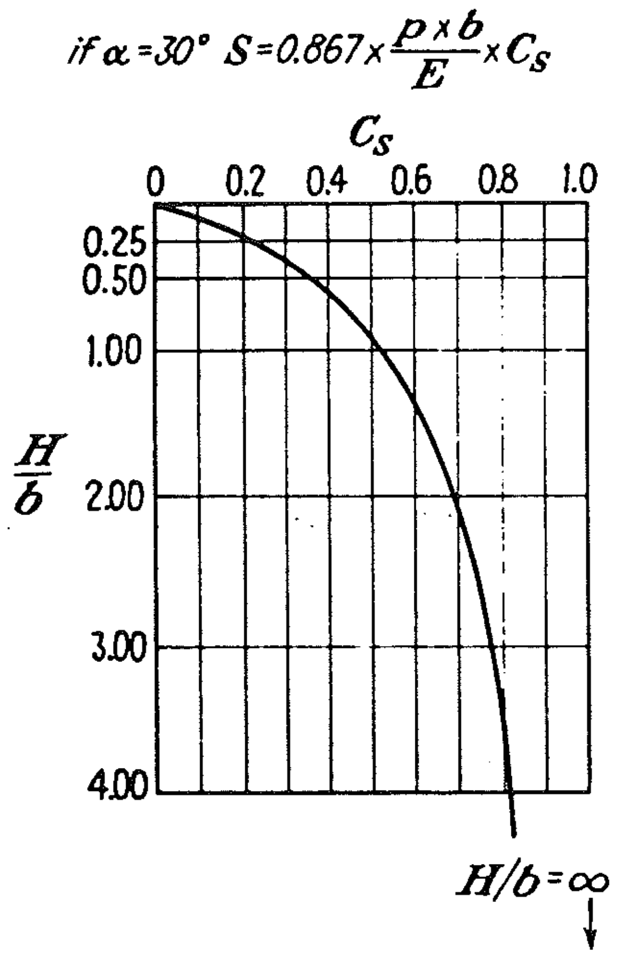


Figure 4.13 Layer Thickness Correction Factor, C_s (after Tschebotarioff 1953, 1971).

The influence factor, i_c, as presented in the CFM, is taken from Kany (1959) and is shown in Figure 4.14 for different values of z/B and L/B and therefore, like other influence factors, takes into account the layer thickness and foundation geometry.

4.4. Oweiss (1979)

A method known as the "Equivalent Linear Model" was presented by Oweiss (1979) which is essentially an elastic solution model in which the elastic deformation modulus of the soil is obtained from the standard penetration test (SPT) blowcounts. Settlements from individual soil layers beneath the foundation are calculated and the total settlement is obtained by summing all individual settlements. In this method, the settlement is calculated from the expression:

$$s = qB\sum_{i=1}^{n} (\psi_i/E_i) \qquad [4.12]$$

where:

s = settlement (ft.)
q = applied footing stress (ksf)
B = footing width (ft.)
I = individual layer
n = total number of layers
ψ_i = settlement factor of layer i
E_i = elastic modulus of layer i

Initially, the compressible zone beneath the foundation to a depth of at least D + 2B, where D = depth of the foundation, is divided into sublayers. The sublayers may be of any thickness and it may be convenient to define layer boundaries at obvious changes in soil properties, such as blowcount, grain-size, water table, etc. If there are no obvious distinct property variations, i.e., the compressible zone is more or less uniform, it is suggested that the zone between D and D + 2B be divided up into at least four or five sublayers to improve the accuracy of the settlement estimate.

For each soil layer, Oweiss (1979) suggested that the SPT blowcount should be corrected for overburden stress using the correction factor suggested by Peck and Bazaraa (1969) as:

$$N_c = 4N/(1+2\,p') \qquad \text{(for } p' \leq 1.5 \text{ ksf)} \qquad [4.13]$$
$$N_c = 4N/(3.25 + 0.5\,p') \qquad \text{(for } p' > 1.5 \text{ ksf)} \qquad [4.14]$$

where:

N_c = corrected blowcount

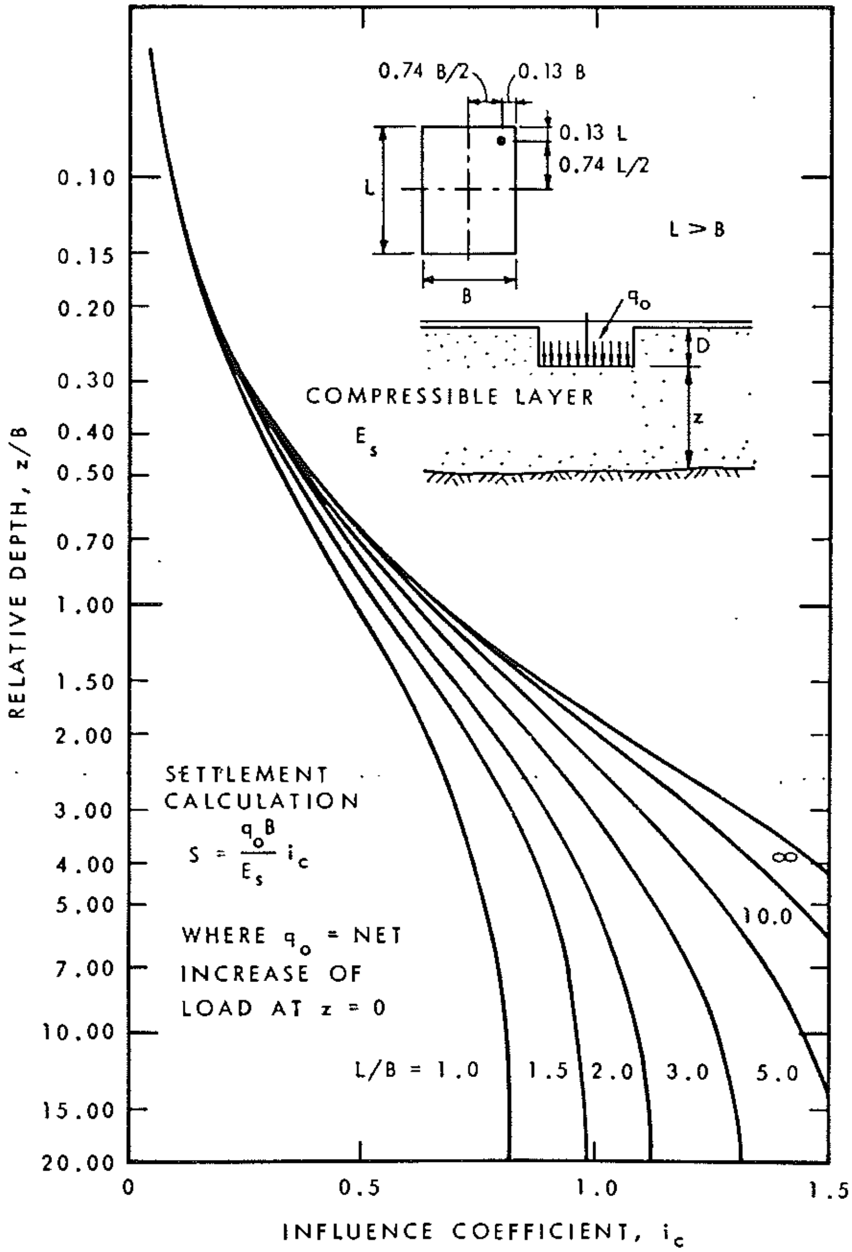


Figure 4.14 Chart for Influence Factor, i_c, after Kany (1959; from Canadian Foundation Manual, 1985).

N = field measured blowcount
p' = effective overburden stress at the location of the blowcount (in ksf)

The mean effective stress, σ'_{mo}, at the midpoint of each layer is calculated according to:

$$\sigma'_{mo} = [(1+2K_o)p']/3 \qquad [4.15]$$

where:

K_o = at rest coefficient of earth pressure
p' = effective overburden stress at the layer midpoint

Obviously, this calculation requires an estimate of both the unit weight of the soil and K_o, which may either be made based on other soil characteristics or may be estimated by other means.

The change in mean effective stress, $\Delta\sigma'_m$, at the midpoint of each layer, resulting from the applied footing stress, q, is given from:

$$\Delta\sigma'_m = \alpha q \qquad [4.16]$$

where:

α = an influence factor dependent on the depth and location of desired settlement estimate (i.e., edge or center for flexible footings). The value of α is obtained from Figure 4.15, after Oweiss (1979). For a rigid footing, the value of α may be estimated by interpolating midway between the edge and center curves shown in Figure 4.15.

The settlement factor, ψ_i, for each layer is calculated from:

$$\psi_i = (F_i) - (F_{i-1}) \qquad [4.17]$$

where:

F_i = factor at the bottom of each soil layer
F_{i-1} = factor at the top of each soil layer

The factors F_i and F_{i-1} are obtained from Figure 4.16. The depth to the bottom of each layer, Z_i and the top of each layer, Z_{i-1}, are used to evaluate values of F.

A strain parameter, λ_i, is calculated for each layer as:

$$\lambda_i = (\psi_i qB)/(zE_{max}) \qquad [4.18]$$

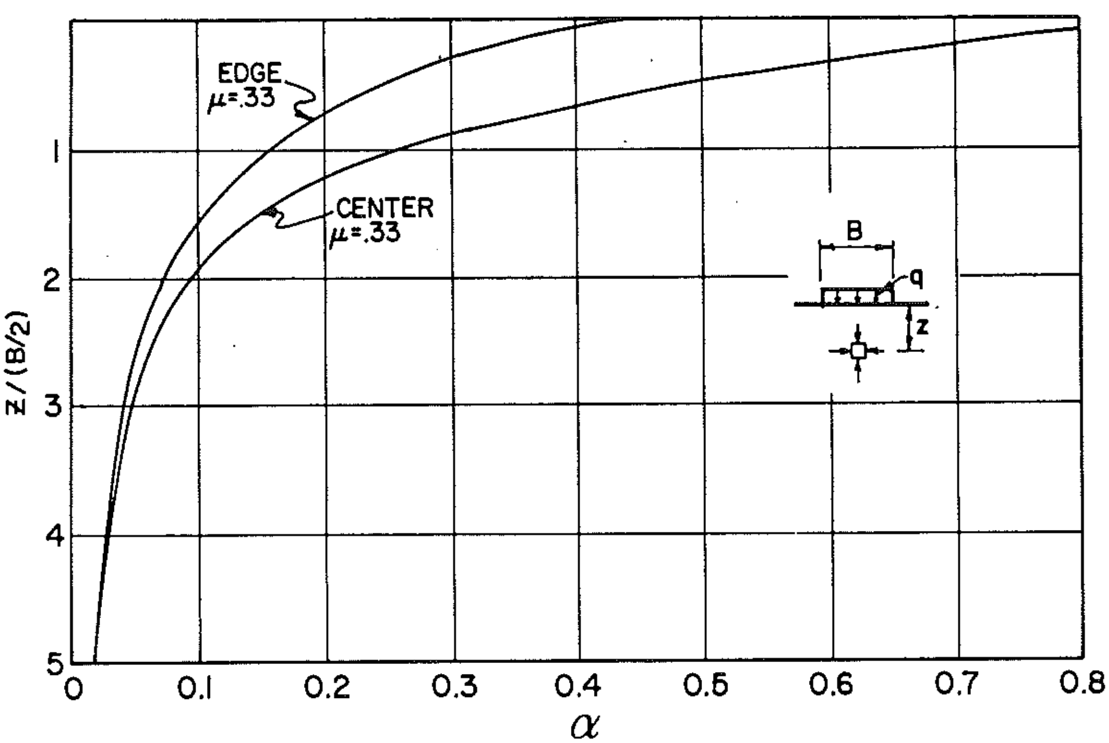


Figure 4.15 Oweiss (1979) Influence Factor, α.

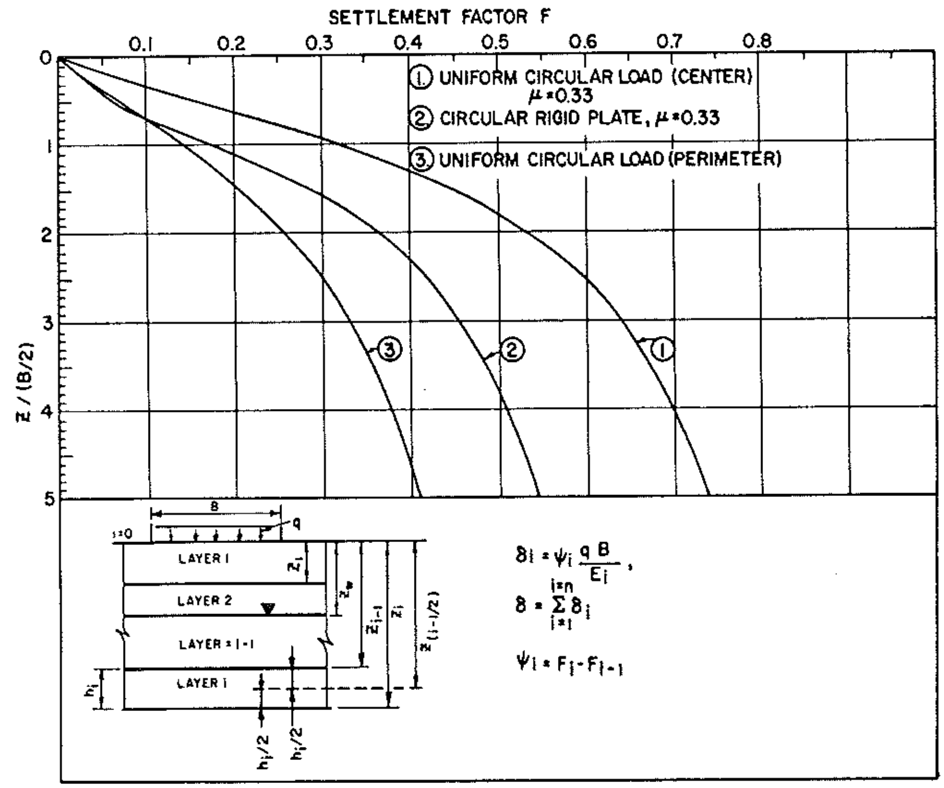


Figure 4.16 Oweiss (1979) Layer Factors.

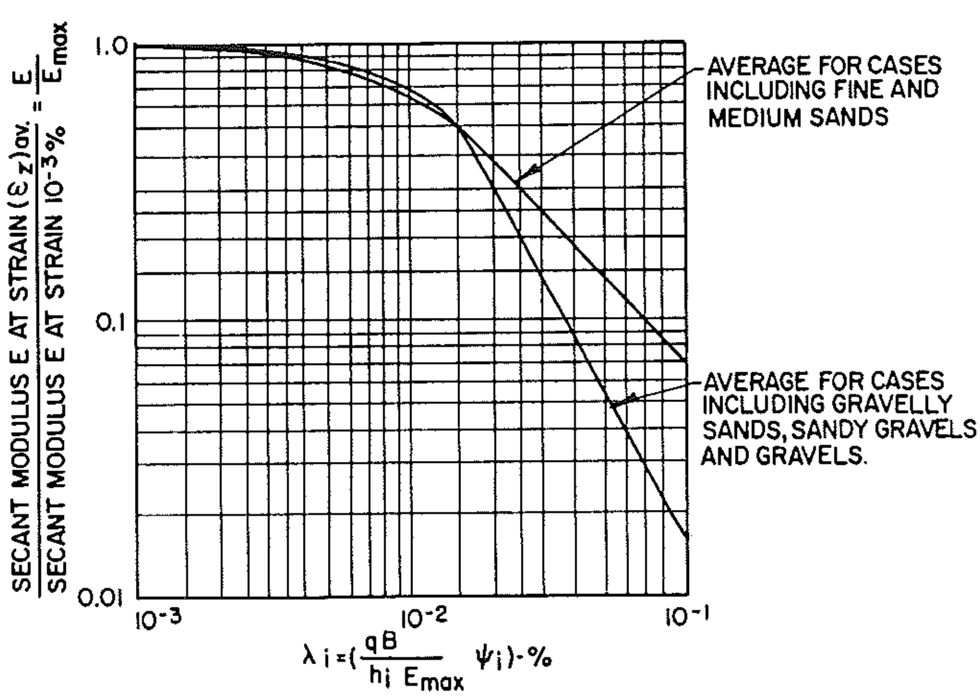


Figure 4.17 Oweiss (1979) Modulus Adjustment.

where:

z = layer thickness (in ft.)
E_{max} = maximum soil modulus (in ksf)

The value of E_{max}, the maximum soil elastic modulus which corresponds to a strain level of 0.001%, is obtained from:

$$E_{max} = K_{max} (\sigma'_{mo} + \Delta\sigma'_{m})^{0.5} \qquad [4.19]$$

where:

$$K_{max} = 17.2 (N_c)^{0.42} \qquad [4.20]$$

The strain parameter, λ_i, is then used to adjust the soil modulus, E_{max}, to give the "operational" soil modulus using the chart provided in Figure 4.17. The soil modulus for calculating the settlement of each layer is then determined from:

$$E_i = (E/E_{max})_i (E_{max})_i \qquad [4.21]$$

The settlement from each layer is calculated from:

$$s_i = (qB/E_i)\psi_i \qquad [4.22]$$

Total settlement of the compressible zone is then obtained from:

$$s = \sum_{i=1}^{n} s_i \qquad [4.23]$$

4.5 Das (1983)

The general elastic expression for settlement presented in Section 4.1 is for the settlement at the surface of a semi-infinite homogenous half-space. Das (1983) suggested a method to calculate the elastic settlement of a footing on a finite thickness compressible layer ($H < 10B$) by subtracting the settlement calculated for the same footing as if it were at a depth in the half space equal to the depth of the bottom of the compressible layer from the settlement calculated from the general elastic solution.

This method is performed as follows:

1 - Compute the settlement(s) of the footing on a semi-infinite half space using Equation 4.1.

2 - Compute the settlement of one corner of the footing at a depth equal to the bottom of the compressible layer (H) from:

$$s' = [(qB')(1-\mu^2)/2E]\, I_{sm} \qquad [4.24]$$

where:

$B' = B/2$
I_{sm} = modified Steinbrenner influence factor using H as the finite thickness

3 - Compute the settlement at the center of the flexible footing on a finite layer as:

$$s_f = s - 4(s') \qquad [4.25]$$

4.6 Bowles (1987)

Bowles (1987) presented a detailed reevaluation of the use of the general elastic solution for estimating settlement of footings on sand and suggested several practical considerations for modifying the method. The description of the method presented herein is taken from the step-by-step procedure given in Bowles (1988):

1 - Estimate the applied footing stress, q, as best as possible.

2 - For round footings, convert to an equivalent square.

3 - Determine the point where the settlement is to be computed (usually the center) and divide the base so the point is at the corner or common corner of contributing rectangles.

4 - Note that the thickness of the compressible zone contributing to settlement is not at $H/B \rightarrow \infty$, but is either:

a) z = 5B, or
b) z = depth to a "hard" layer if less than 5B.
A "hard" layer may be taken as that point where E_s in the hard layer is about $10E_s$ of the upper layer.

5 - Compute H/B' ratio. For H = z = 5B and for the center of the base, H/B' = 5B/0.5B = 10; for a corner H/B' = 5.

6 - Use the Steinbrenner equations along with the best estimate

of μ to calculate I. (The tables provided in Bowles (1988) which are also shown in Figure 4.4 may be used.)

7 - Estimate the Fox (1948) embedment correction factor using Figure 4.8.

8 - Obtain the weighted average E_s in the depth z = H. The weighted average can be calculated as:

$$E_{s(av)} = H_1E_{s1}+H_2E_{s2} + ...H_nE_{sn})/H \qquad [4.26]$$

The settlement calculation proceeds using Equation 4.1 and applying the Fox embedment correction factor as presented by Bowles (1982, 1988) shown in Figure 4.9.

Bowles (1987) suggested that based on Boussinesq stress distribution profiles and Schmertmann's (1970) strain profiles, for all practical purposes the soil mass below a depth of 4B to 5B has little influence on the settlement. The value of H = 5B was taken to be slightly conservative over using H = 4B and thus has a "substantial significance" on the Steinbrenner influence factor over taking H = ∞.

Additionally, it was reasoned that the soil modulus E_s generally increases with depth in homogenous sand deposits and therefore could be much larger at 5B than at the base of the foundation. It was suggested then that the average E_s over the depth H should be used and not the value in the zone of B to 2B beneath the foundation, as has been suggested by others. Reasonable values of E_s may be obtained from CPT or SPT results as indicated by Bowles from:

$$E_s = 2.5 \text{ to } 3\ q_c \quad \text{(in units of } q_c\text{)} \qquad [4.27]$$

$$E_s = 10\ (N + 15) \quad \text{(in ksf)} \qquad [4.28]$$

Bowles (1987) summarized a number of case histories from the literature and found good comparisons between observed and calculated settlements using this method. It was suggested that the reason that earlier estimates of settlement were poor were because E_s just below the base was used and that a semi-infinite half space was used which produced an error in the influence factors used.

4.7 Papadopoulos (1992)

Papadopoulos (1992) suggested a method of estimating the settlement of footings resting on granular soils of the elastic solution type as:

$$s = [(qB)/E_s]f \qquad [4.29]$$

where:

s = settlement
q = foundation stress
B = width of a rectangular foundation
E_s = constrained modulus of the soil for the appropriate stress range
f = a dimensionless factor which depends on soil stress history, geometry, loading and the relation between constrained modulus and effective stress.

According to Papadopoulos (1992) the settlement factor, f, is related to the stress history of the soil, the geometry of the foundation (depth and dimensions), the foundation loading, and the relation between the constrained modulus and the effective stress, σ', as shown in Figure 4.18. The influence of stress history and other factors, expressed in terms of the dimensionless factor, α, where α = the ratio of applied footing stress to the footing width times the effective soil unit weight, i.e.,

$$\alpha = q/(\gamma' B) \qquad [4.30]$$

is indicated in Figure 4.18.

The constrained modulus, E_s, is related to the effective stress for stresses $\sigma' \leq 600$ kPa by a linear expression:

$$E_s = E_{so} + \lambda\sigma' \qquad [4.31]$$

where:
E_{50} = constrained modulus for zero effective stress
λ = the rate of E_s increase with stress.

In practice, since it is difficult to evaluate λ from undisturbed samples, the alternative is to use an average E_s in settlement calculations evaluated from in situ tests and $\lambda = 0$. The following expressions for estimating soil modulus were suggested by Papadopoulos:

$$E_s = 2.5\, q_c \qquad \text{(for CPT results)} \qquad [4.32]$$
$$E_s = 7.5 + 0.8N \text{ (MPa)} \qquad \text{(for SPT results)} \qquad [4.33]$$

A comparison between the settlements predicted using this method and settlement observations using cases reported primarily by D'Appolonia et al. (1968), Schmertmann (1970), and Schultze and Sherif (1973) showed that in more than 90% of the cases the deviation of the estimated settlement from the measured settlement was ± 50%.

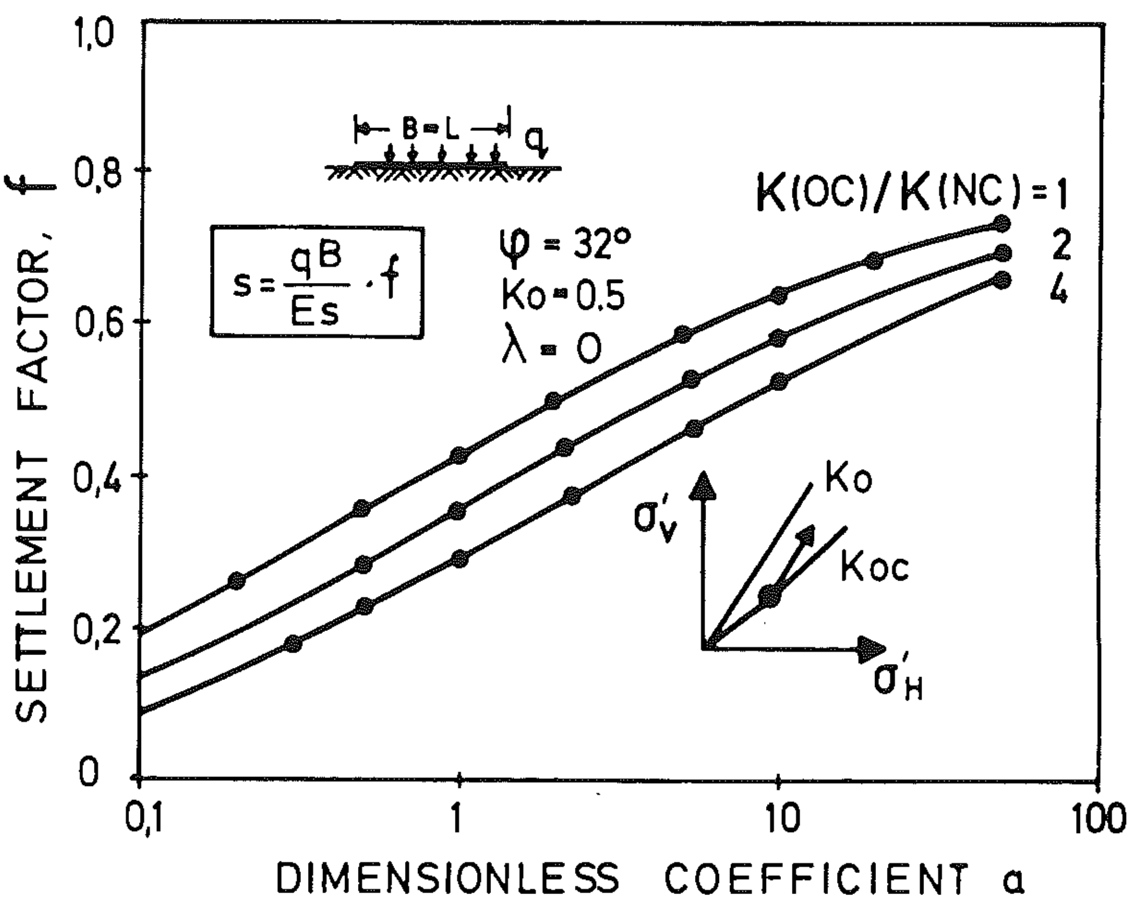


Figure 4.18 Papadopoulos (1992) Settlement Factor.

4.8 Wahls and Gupta (1994)

A method based principally on elastic stress-strain theory and designed for use with soil data from the SPT was recently presented by Wahls and Gupta (1994). Settlement is calculated from:

$$s = \sum_{i=1}^{n} (\Delta E_z \, \Delta Z)_i \qquad [4.34]$$

where:

s = settlement
ΔE_z = vertical strain in an element at depth Z
ΔZ = sublayer thickness

The compressible zone of soil is subdivided into a number of sub layers and the strain in each layer is calculated from:

$$\Delta E_z = (qI_s)/(E_{zo}(\sigma'_m)^{0.5}) \qquad [4.35]$$

where:

q = applied foundation stress
I_s = a strain influence factor
E_{zo} = modulus coefficient
σ'_m = mean stress

The use of five sublayers of equal thickness is recommended by Wahls and Gupta (1994) for this method. The maximum zone of influence was taken as 2B for $L/B \leq 3$ and 4B for $L/B > 3$. If the layer does not extend to the maximum depth of influence, the total thickness of the layer is used.

The strain influence factor, I_s, is a function of the applied stress, foundation geometry and Poisson's ratio. For Poisson's ratio equal to 1/3 (assumed by Wahls and Gupta (1994) as reasonable), the value of I_s is given as:

$$I_s = 4\,(I_z - I_m)/3 \qquad [4.36]$$

where:

I_z and I_m are stress influence factors given as:

$$I_z = \frac{2}{\pi}\left[\tan^{-1}\left(\frac{M}{2N(M^2+4N^2+1)^{0.5}}\right) + \frac{2MN}{(M^2+4N^2+1)^{0.5}}\left(\frac{1}{(M^2+4N^2)} + \frac{1}{(4N^2+1)}\right)\right] \qquad [4.37]$$

$$I_m = \frac{4}{3\pi}\left[\tan^{-1}\left(\frac{M}{2N(M^2+4N^2+1)^{0.5}}\right)\right] \qquad [4.38]$$

where:

$M = L/B$
$N = Z/B$
L = length of footing
Z = depth below footing

The value of the mean stress, σ'_m, is obtained from:

$$\sigma'_m = (\sigma'_m)_o + 0.5\Delta\sigma'_m = (\sigma'_m)_o + 0.5\, q\, (1 + \mu)\, I_m \qquad [4.39]$$

where:

$$(\sigma'_m)_o = (\sigma'_{vo})\,(1 + 2\, K_o)/3 \qquad [4.40]$$

The soil modulus, E_{zo} is obtained from:

$$E_{zo} = 43.8\,(1 + \mu)K_2(\sigma_{atm})^{0.5} \qquad [4.41]$$

where:

K_2 = coefficient that is a function of relative density and shear strain
$K_{2max} = K_2$ at shear strain = 0.0001%

$$= 0.6D_r + 16 \qquad [4.42]$$

where:
D_r = relative density (in %)

The relative density is obtained from the SPT corrected blowcount as:

$$D_r = [(N_c)/(A + B\sigma'_{vo})]^{0.5} \qquad [4.43]$$

Corrections to the SPT blowcount should include factors such as overburden stress, energy ratio, borehole diameter, and sampler geometry, however, Wahls and Gupta (1994) suggest that typically if the field blowcount is corrected for overburden, this will be used in Equation 4.43. The coefficients A and B were taken as 32 and 0.288, respectively. The value of K_2 is set equal to K_{2max} for initial loading. For any subsequent load increments, Figure 4.19 is used to reduce K_2 to account for the reaction of soil modulus with strain level.

Based on an initial comparison with case histories, it was suggested that a correction factor, C, should be applied to the settlement estimate using this method. Table 4.1 provides a summary of recommended correction factors.

Table 4.1 Correction Factors, C, for Wahls and Gupta (1994) Method

No. of Load Increments	$N \geq 15$	$N < 15$
1	4.5	11.25
10 or more	3.0	7.5

A comparison with observed settlements found that in more than 75% of 120 cases considered, predicted settlements were within 6 mm (1/4 in.) of observed settlements.

4.9 Estimating Soil Modulus from SPT and CPT

The settlement prediction methods presented in the previous sections require the input of the elastic modulus of the soil for evaluating settlement. Unfortunately the modulus of granular soils, like the modulus of all other soils is highly nonlinear. This has been recognized for a long time, but only recently been accounted for in a few of the design methods available. Since a general elastic approach may be useful in preliminary designs it is useful to review previous suggestions for estimating the elastic modulus of granular soils from in situ test results. It should be emphasized that some methods make explicit recommendations for estimating soil modulus from different tests. In order to appropriately use the method, those recommendations should be followed. The authors make no claims that any of the recommendations presented by individual investigators are superior or for that matter that methods using this approach are valid. The intent here is to only provide a brief summary of previously suggested correlations between in situ tests and reported soil modulus. Correlations presented in this section are for the SPT and CPT tests only. For example, Bowles (1988) has suggested a number of correlations between the results of both the SPT and CPT and soil modulus as shown in Table 4.2.

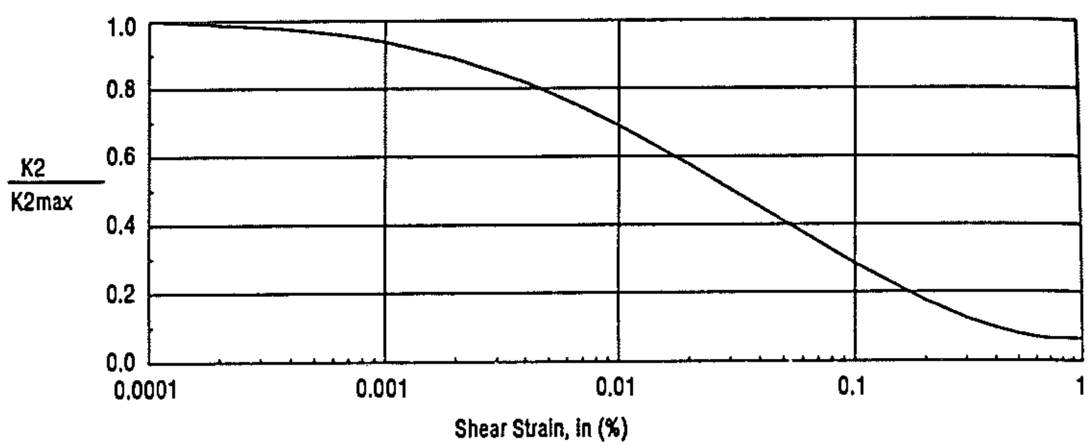


Figure 4.19 Wahls and Gupta (1994) Modulus Reduction.

4.9.1 Standard Penetration Test

Numerous suggestions have been made to use the SPT for estimating the elastic modulus of granular soils (e.g., Schultze and Menzenbach 1961; Schultze and Melzer 1965; etc). Most of these correlations have the form of:

$$E = a\,(N + b) \qquad [4.44]$$

where:

E = soil modulus
N = SPT blowcounts
a and b = constants (empirical factors)

Alternatively, other forms have been used. In addition to the correlations presented in Tables 4.2 and 4.3, a number of other suggestions have been made. These are summarized in Table 4.4.

Other attempts have been made to correlate the results of the SPT to the constrained modulus of the soil (M) as a function of overburden stress (e.g., Schultze and Melzer 1965). D'Appolonia et al. (1970) suggested correlations between M and SPT blowcount N recognizing the influence of stress history. These correlations are presented in the next section of this report and are subsequently shown in Figure 5.7.

Since the constrained modulus, M, is related to the elastic Young's modulus, E, as:

$$M = [\,E\,(1-\mu)\,] \,/\, [\,(1+\mu)\,(1-2\mu)] \qquad [4.45]$$

an estimate of Poisson's ratio is required to estimate E from M. For most granular soils in drained loading conditions, the constrained modulus probably varies in the range of 1.2E to 1.5E.

Unfortunately, the realization must be made that there is considerable scatter in suggested correlations between E or M and SPT blowcount N. This should in fact be not altogether unexpected since there is a considerable scatter in SPT results, even at a single site, because of large variations in test procedures that may occur. Additionally, since the source of correlations between modulus and N is highly variable and includes laboratory tests on reconstituted samples, results of field plate tests and results of settlement observations from full scale structures, the correlations will have implicit variability just because of differences in assumptions made. Additionally, since the modulus is strain level dependent, the correlations include comparisons at a range of strain levels.

4.9.2 Cone Penetration Test

The modulus of soils has also been correlated to the results of tip resistance measurements (q_c) obtained from the static CPT test. Most early correlations between q_c and E were of the general form:

$$E = \alpha q_c \qquad [4.46]$$

where:

α = a constant (empirical factor)

In addition to the correlations summarized by Bowles (1988) and presented in Table 4.2, Mitchell and Gardner (1975) had previously compiled a large number of reported correlations. These are summarized in Table 4.5. As with correlations presented between soil modulus and SPT results, the expressions indicated in Table 4.5 similarly show a very wide scatter. Additional suggestions are given in Table 4.6 for more recent work.

Since the performance of the CPT involves considerably less variation than the SPT and is prone to less errors in execution, it is suspected that the primary source of scatter indicated in Tables 4.5 and 4.6, and for that matter in Tables 4.3 and 4.4 is the soil itself and not the test method. Variations in soil mineral composition, initial void ratio, grain-size distribution, stress history, etc., as well as differences in initial effective stress level (octahedral) and change in stress during loading result in differences in the "operational" or "apparent" modulus of elasticity producing deformation. These factors, combined with the stress and strain level dependency of a "local" soil modulus for a given application all affect the reported correlations between the so-called soil modulus and in situ test results.

In recent years, the use of large calibration chamber tests on reconstituted samples of sands have helped to elucidate certain key variables that can influence correlations between modulus and CPT results. For both normally consolidated and overconsolidated sands, the ratio of constrained modulus, M, to CPT tip resistance, q_c, decreases with increasing relative density, all other factors being equal. Kulhawy and Mayne (1990) have summarized a number of available chamber test results which are shown in Figure 4.20.

Jamiolkowski et al. (1988) have also noted that for a given sand the ratio M/q_c and E/q_c is clearly related to stress history and current stress level for sands at different relative density. These results are presented in Figures 4.21 and 4.22, respectively.

Because of the wide range in correlation constants that may exist between the results of in situ penetration tests and a singular value of soil modulus it is doubtful that any method which relies on these techniques for the accuracy of settlement estimates will be of much value, other than that created by local correlations developed from full scale field observations of performance. However,

there are several techniques that have recently been suggested that account for nonlinearity in modulus and show distinctly strong correlations between observed "operational" modulus and the results of SPT or CPT tests. These methods are discussed further in subsequent sections of this report.

Table 4.2 Estimates of Soil Modulus from SPT and CPT (after Bowles 1988).

Soil	SPT	CPT
Sand (normally consolidated)	$E_s = 500(N + 15)$ $E_s = (15000 \text{ to } 22000) \ln N$ $E_s^{\S} = (3500 \text{ to } 50000) \log N$	$E_s = 2 \text{ to } 4q_c$ $E_s^{\dagger} = (1 + D_r^2)q_c$
Sand (saturated)	$E_s = 250(N + 15)$	
Sand (overconsolidated)	$E_s^{\ddagger} = 18000 + 750N$ $E_{s(OCR)} = E_{s(nc)}(OCR)^{1/2}$	$E_s = 6 \text{ to } 30q_c$
Gravelly sand and gravel	$E_s = 1200(N + 6)$ $E_s = 600(N + 6) \quad N \leq 15$ $E_s = 600(N + 6) + 2000 \quad N > 15$	
Clayey sand	$E_s = 320(N + 15)$	$E_s = 3 \text{ to } 6q_c$
Silty sand	$E_s = 300(N + 6)$	$E_s = 1 \text{ to } 2q_c$
Soft clay	-------	$E_s = 3 \text{ to } 8q_c$

Notes:
E_s in kPa for SPT and units of q_c for CPT; divide kPa by 50 to obtain ksf.
N values should be estimated as N_{55} and not N_{70}
† Vesic' (1970)
‡ Author's equation from plot of D'Appolonia et al. (1970).
§ USSR (and may not be standard blowcount N).

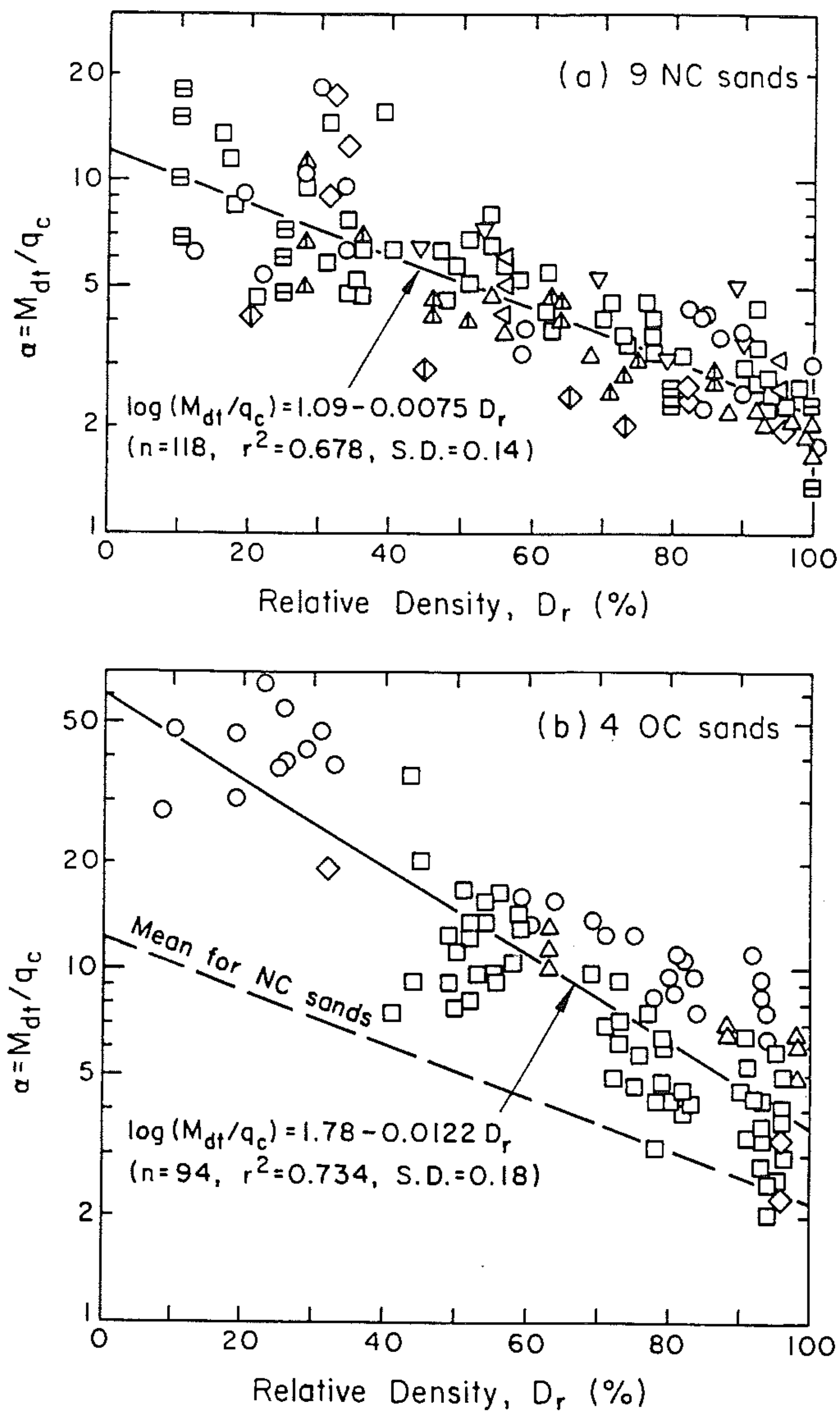


Figure 4.20 Variation in M/q_c with Relative Density (from Kulhawy & Mayne 1990).

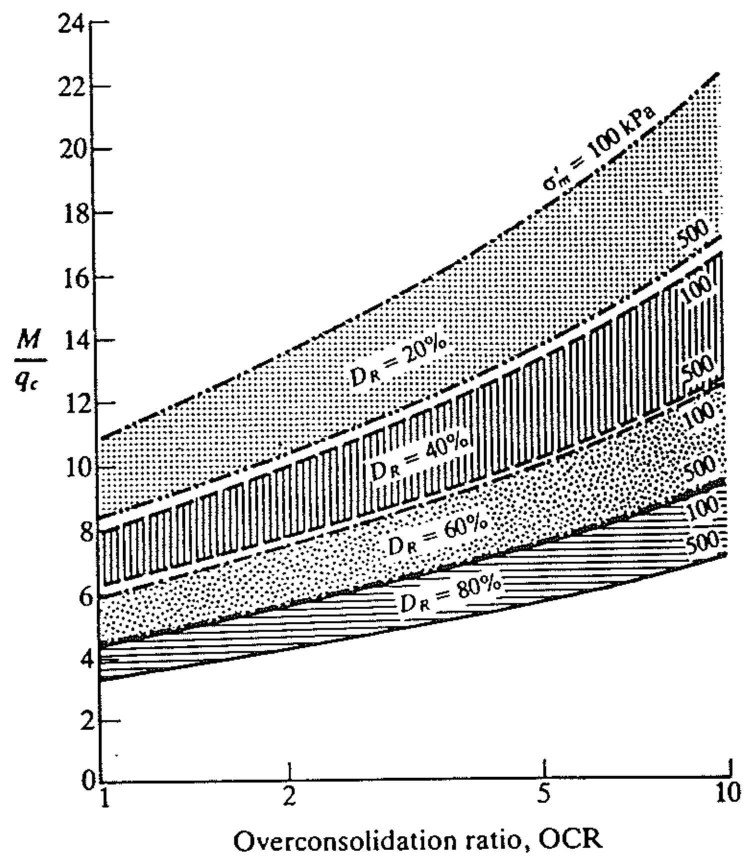


Figure 4.21 M vs. q_c for Ticino Sand (from Jamiolkowski et al. 1988).

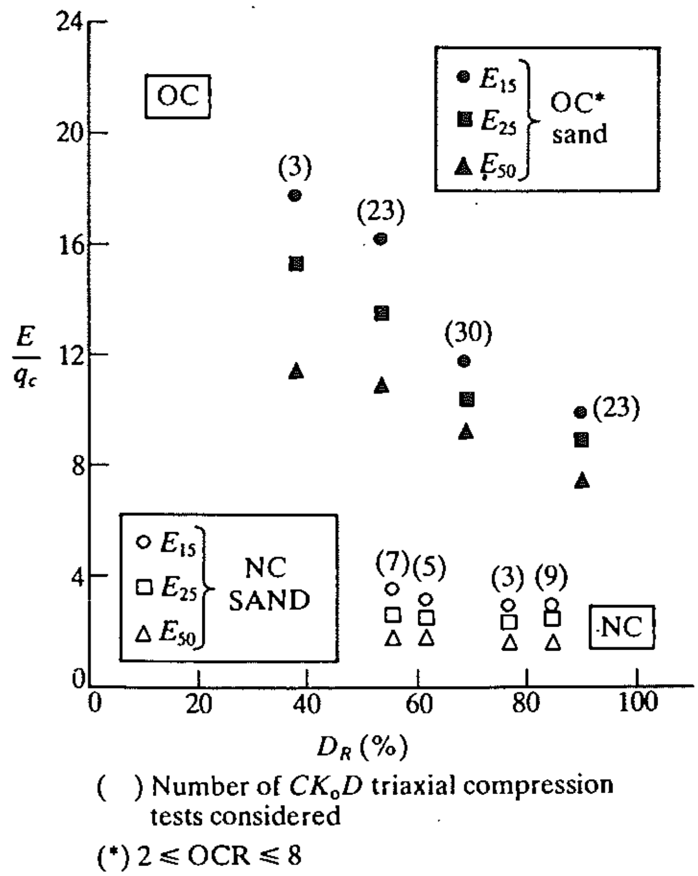


Figure 4.22 E vs. q_c for Ticino Sand (from Jamiolkowski et al. 1988).

Table 4.3 Soil Modulus from Standard Penetration Resistance (modified from Mitchell and Gardner 1975)

Reference	Relationship	Soil Types	Basis	Remarks
Schultze and Melzer (1965)	$E_s = Na_{0.522}$ kg/cm^2	Dry sand	Penetration tests in field and in test shaft. Compressibility based on e, $e_{max.}$ and $e_{min.}$ (Schulzte and Moussa, 1961)	Correlation Coefficient = 0.730 for 77 tests
Webb (1969)	$E_s = 5(N+15)$ tons/ft^2 $E_s = 10/3(N+5)$ tons/ft^2	Sand Clayey Sand	Screw Plate Tests	Below water table
Farrent (1963)	$E_s=7.5(1-m^2)N$ tons/ft^2 m=Poisson's ratio	Sand	Terzaghi and Peck loading settlement curves	
Begemann (1974)	$E_s= 40 + C(N-6)$ for N>15 kg/cm^2 $E_s= C(N+6)$ for N<15 kg/cm^2 C= 3(silt with sand) to 12(gravel with sand)	Silt with sand to gravel with sand		Used in Greece
Trofimenkov (1974)	$E_s= (350$ to $500)$ logN (kg/cm^2)	Sand		USSR practice

Notes:
N is penetration resistance in blows per 30 cm. (blows/ft.)
E_s = soil modulus
e = void ratio

Table 4.4 Other Expressions for Soil Modulus from SPT

Equation	Soil Type	Reference
$E_s^{4/3} = (44N)$ (tsf)	Sand	Chaplin (1963)
$E_s = 48 + 4N$ (tsf)	Sand	Webb (1969)
$E_s = 7(N)^{0.5}$ (MPa)	Sand	Denver (1982)
$E_s = 3.5N$ to $40N$ (MPa)	Sand	Clayton et al. (1980)
$E_s = 7.5 + 0.8N$ (MPa)	Sand	Papadopoulos (1982)

Table 4.5 Soil Modulus from Cone Penetration Test (modified from Mitchell and Gardner 1975)

Reference	Relationship	Soil Types	Remarks
Buisman (1940)	$E_s = 1.5q_c$	Sands	Over predicts settlements by a factor of about two
Trofimenkov (1964)	$E_s = 2.5\ q_c$ $E_s = 100 + 5\ q_c$	Sand	Lower limit Average
De Beer (1967)	$E_s = 1.5\ q_c$	Sand	Overpredicts settlements by a factor of two
Schultze and Melzer (1965)	$E_s = (1/m_v)v\sigma^{0.522}$ where $v = 301.1 \log q_c - 382.3p_o + 60.3 \pm 50.3$	Dry Sand	Based of field and lab penetration tests compressibility based on e, e_{max} and e_{min} Correlation coefficient = 0.778 for 90 tests valid for p_o = 0 to 0.8 kg/cm^2
Bachelier and Parez (1965)	$E_s = \alpha q_c$		
	$\alpha = 0.8\text{-}0.9$	Sand	
	$\alpha = 1.3\text{-}1.9$	Silty sand	
	$\alpha = 3.8\text{-}5.7$	Clayey-sand	
	$\alpha = 7.7$	Soft clay	
De Beer (1967)	$A = C(A_{oed}/C_{oed})$	Over-consolidated sand	C from field tests A_{oed} and C_{oed} from lab oedometer tests $C_{oed} = 2.3(1+e)/C_C$ $A_{oed} = 2.3(1+e)/C_S$
Thomas (1968)	$E_s = \alpha q_c$ $\alpha = 3 - 12$	3 sands	Based on penetration and compression tests in large chambers Lower values of a at higher values of q_c; attributed to grain crushing

Table 4.5 Cont'd

Reference	Relationship	Soil Types	Remarks
Webb (1969)	$E_s = 2.5(q_c+30)$, tsf	Sand below water table	Based on screw plate tests Correlated well with settlement of oil tanks
	$E_s = 1.67(q_o+15)$, tsf	Clayey sand below water table	
Meigh and Corbett (1969)	$E_s = 1/m_v = \alpha q_o$	Soft silty clay	
Vesic (1970)	$E_s = 2(1+D_R^2)q_c$ D_R = relative density	Sand	Based on pile load tests and assumptions concerning state of stress
Schmertmann (1970)	$E_s = 2q_c$	Sand	Based on screw plate tests $\Delta\sigma = 2$ tsf

Table 4.5 Cont'd

Reference	Relationship	Soil Types	Remarks
Gielly et al. (1969) Sanglerat et al. (1972)	$E_s = \alpha q_c$		Based on 600 comparisons between field penetration and lab oedometer tests
	$q_c < 7$ bars $3 < \alpha < 8$ $7 < q_c < 20$ bars $2 < \alpha < 5$ $q_c < 7$ bars $1 < \alpha < 2.5$	Clays of low plasticity (CL)	
	$q_c > 20$ bars $3 < \alpha < 6$ $q_c < 20$ bars $1 < \alpha < 3$	Silts of low plasticity (ML)	
	$q_c < 20$ bars $2 < \alpha < 6$	Highly plastic silts and Clays (MH, CH)	
	$q_c < 12$ bars $2 < \alpha < 8$	Organic silts (OL)	
	$q_c < 7$ bars: $50 < w < 100$ $1.5 < \alpha < 4$ $100 < w < 200$ $1 < \alpha < 1.5$ $w > 200$ $0.4 < \alpha < 1$	Peat and organic clay (Pt, OH)	
	$20 < q_c < 30$ bars $2 < \alpha < 4$ $q_c > 30$ bars $1.5 < \alpha < 3$	Gravel	
	$q_c < 50$ bars $\alpha = 2$ $q_c > 100$ bars $\alpha = 1.5$ $q_c > 12$ bars $w < 30\%$ $C_c < 0.2$ $q_c < 12$ bars $w < 25\%$ $C_c < 0.2$ $25 < w < 40\%$ $0.2 < C_c < 0.3$ $40 < w < 100\%$ $0.3 < C_c < 0.7$ $q_c < 7$ bars $100 < w < 130\%$ $0.7 < C_c < 1$ $w < 130$ $C_c > 1$	Sand	

Table 4.5 Cont'd

Reference	Relationship	Soil Types	Remarks
Bogdanovic' (1973)	$E_s = \alpha q_o$		Based on analysis of silo settlements over a period of 10 years
	$q_o > 40$ kg/cm^2 $\alpha = 1.5$	Sands, sandy gravels	
	$20 < q_o < 40$ $\alpha = 1.5 - 1.8$	Silty saturated sands	
	$10 < q_o < 20$ $\alpha = 1.8 - 2.5$ $5 < q_o < 10$ $\alpha = 2.5 - 3.0$	Clayey silts with silty sand and silty saturated sands with silt	
Schmertmann et al. (1978)	$E_s = 2.5\ q_o$	NC sands	L/B=1 to 2 axisymmetric
	$E_s = 3.5\ q_o$	NC sands	L/B = ≥ 10 plane strain
DeBeer (1974b)	$C > 3/2(q_c/\sigma_0)$	NC sands	Belgian practice
	$A > \varepsilon\ 3/2(q_c/\sigma_0)$	OC sands	$3 < \varepsilon < 10$, Belgian practice
	$E_s = 1.6\ q_c - 8$	Sand	Bulgarian practice
	$E_s = 1.5q_c$, $q_c > 30$ kg/cm^2 $E_s = 3q_c$, $q_c < 30$ kg/cm^2	Sand	Greek practice
	$E_s > 3/2\ q_c$ or $E_s = 2\ q_c$	Sand	Italian practice
	$E_s = 1.9\ q_c$ $E_s = 2.5(q_c + 3200)$, kPa $E_s = 1.67(q_c + 1600)$, kPa	Sand Fine to Medium sand Clayey sands, PI < 15%	South African practice
	$E_s = \alpha q_c$, $1.5 < \alpha < 2$	Sands	U.K. practice
Trofimenkov (1974)	$E_s = 3\ q_c$	Sands	U.S.S.R. practice
	$E_s = 7\ q_c$	Clays	
Alperstein and Leifer (1975)	$E_s = (11 - 22)q_c$	Overconsolidated sand	E_s determined by lab tests on reconstituted samples of sand
Dahlberg (1974)	$E_s = \alpha q_c$ $1 < \alpha < 4$	NC and OC sand	E_s back-calculated from screw plate settlement using Buisman-DeBeer and Schmertmann methods; α increases with increasing q_c

Table 4.6 Other Expressions for Soil Modulus from CPT

Expression	Soil	Reference
$E_s = 11q_c$	Sand	Lambrechts and Leonards (1978)
$E_s = 2.5q_c$	Sand	Roth et al. (1982)
$E_s = \alpha q_c$ $\alpha = 1.7$ to 4.4 average = 2.5	Med. sand	Das Neves (1982)
$E_s = 8(q_c)^{0.5}$, q_c in MPa	Sand	Denver (1982)
$E_s = 2.9q_c$	Sand	Garga and Quin (1974)

5.0 ESTIMATING SETTLEMENT FROM IN SITU TEST RESULTS

5.1 Introduction

In addition to the use of a simple general elastic solution and modifications to these methods for predicting settlements which was described in previous sections, there are a number of other methods which have been suggested for estimating settlement of shallow foundations on granular soils based principally on the results of in situ tests. As previously described, one of the most common ways that in situ tests can be used in this design problem is to estimate an elastic modulus, E, of the soil and then calculate settlement from a generalized elastic equation. This is a good illustration of the indirect design approach described in Section 3.0. Other, more direct design approaches have also been suggested, based largely on empirical observations.

In this section of the report, methods are described that have been suggested for predicting settlements using the results obtained from a variety of in situ field tests. The test methods included in this section are:

1 - Standard Penetration Test (SPT)
2 - Cone Penetration Test (CPT)
3 - Pressuremeter Test (PMT)
4 - Dilatometer Test (DMT)
5 - Plate Load Test (PLT)
6 - Drive Cone Test (DCT)

The design methods presented include a variety of approaches. Table 5.1 presents a summary of the various methods described in this report.

For each test method, the design methods are presented in chronological order so that the reader may develop an appreciation for progressive modifications made to different methods.

5.2 Standard Penetration Test

The use of the Standard Penetration Test to estimate the settlement of shallow foundations on granular soils is deeply seated within the practice of geotechnical engineering. The authors feel confident in suggesting that this test has been and remains to be the most often used tool to make such estimates. This is in part due to the fact that the test is widely available, easily understood, and low cost even though the test is subject to wide variations in procedures and results despite being standardized by ASTM. However the test remains the most commonly used in situ test in practice. A detailed description of the SPT equipment, procedures and problems associated with the test is presented in Appendix A. A recommended practice is also suggested as a part of Appendix A for MHD to follow.

Table 5.1 Methods to Evaluate Settlement of Granular Soils from In Situ Tests.

STANDARD PENETRATION TEST
Terzaghi & Peck (1948, 1967) Meyerhof (1956, 1965) Hough (1959, 1969) Teng (1962) Sutherland (1963) Alpan (1964) D'Appolonia et al. (1968) Bowles (1968) Peck & Bazaraa (1969) Webb (1969) D'Appolonia et al. (1970) Parry (1971) Schultze & Sherif (1973) Peck et al. (1974) Meyerhof (1974) Arnold (1980) NAVFAC (1982) Burland & Burbidge (1985) Stroud (1989) Berardi et al. (1991) Anagnostopoulous et al. (1991)
CONE PENETRATION
DeBeer & Martens (1957) Meyerhof (1956, 1965, 1974) DeBeer (1965) Thomas (1968) Schmertmann (1970) Berardi et al. (1991) Robertson (1991)
PRESSUREMETER TEST
Menard & Rousseau (1962) Martin (1977, 1987) Baguelin et al. (1978) Briaud (1992)

Table 5.1 Cont'd

PLATE LOAD TEST
Terzaghi & Peck (1948, 1967) Barata (1973) Carrier & Christian (1973) Parry (1978) Ghionna et al. (1991)
DILATOMETER TEST
Schmertmann (1986) Elastic Approach Leonards & Frost (1988)
DRIVE CONE TEST
Farrent (1963)

Because of the wide spread historical use of the SPT in site investigations, a large number of methods have been suggested during the past 45 years for using the results of the test to predict the settlement of shallow foundations on granular soils. Nearly 30 individual different methods or suggested modifications to methods have been presented in the literature. In this section, the following prediction methods are described:

1. Terzaghi and Peck (1948, 1967)
2. Meyerhof (1956, 1965)
3. Hough (1959, 1969)
4. Teng (1962)
5. Sutherland (1963)
6. Alpan (1964)
7. D'Appolonia et al. (1968)
8. Bowles (1968)
9. Peck and Bazaraa (1969)
10. Webb (1970)
11. D'Appolonia et al. (1970)
12. Parry (1971)
13. Schultze and Sherif (1973)
14. Peck et al. (1974)
15. Meyerhof (1974)
16. Arnold (1980)
17. Navfac DM7 (1982)
18. Burland and Burbidge (1985)

19. Stroud (1989)
20. Berardi et al. (1991)
21. Anagnostopoulos et al. (1991)

For each settlement method presented in this section, each of the terms of the equation is defined and the units of each term are presented. In most cases, every reasonable attempt had been made to retain the same form and terms used in the original equation presented by individual authors and to use original figures presented by each author.

Other uses of the SPT to predict settlement of shallow foundations on granular soils not presented in this report have been described, for example, by Sutherland (1974) and Louw (1977).

Most of the methods presented in this section are considered empirical. That is, they rely on either a direct correlation between the average N blowcount value (either corrected or uncorrected) and observed settlement, or the blowcount value is used to obtain an intermediate design parameter, the correlation of which is based on observations.

It would be nearly impossible to list all of the applications or case histories where the results of the SPT have been used to predict settlement. The writers believe that it is fair to rely on common knowledge to suggest that methods using the SPT are likely the most common techniques in use by practicing engineers for predicting settlement of footings on granular soils.

5.2.1 Terzaghi and Peck (1948, 1967)

The Terzaghi and Peck settlement method is based primarily on shallow foundation bearing capacity charts developed using the bearing capacity equations presented by Meyerhof (1956). The charts are used to obtain the allowable bearing capacity (although the F.S.used is not stated) for different footing widths and SPT blowcounts values with the maximum settlement not greater than 25 mm (1 in.) and a differential settlement not greater than 19 mm (3/4 in.) at a given allowable bearing capacity. According to Terzaghi and Peck, square and continuous footings of the same width show similar settlement behavior for the same soil and loading intensity. Settlement is given as:

$$s = (8q/N)(C_wC_d) \qquad \text{(for B} \leq \text{4ft.)} \qquad [5.1]$$
$$s = (12q/N)[B/(B+1)]^2C_wC_d \qquad \text{(for B} > \text{4 ft.)} \qquad [5.2]$$
$$s = (12q/N)C_wC_d \qquad \text{(for rafts)} \qquad [5.3]$$

These expressions can also be stated in a general form as:

$$s = (3q/N)(2B/B+1)^2\, C_wC_d \qquad [5.4]$$

where:

s = settlement (in inches)
q = net footing stress (in tsf)
N = uncorrected (field) blowcounts
B = footing width (in ft.)
C_w = water correction
$= 2 - (W/2B) \leq 2.0$ for surface footings [5.5]
$= 2 - 0.5\ (D/B) \leq 2.0$ for fully submerged, embedded footing; $W \leq D$ [5.6]
C_d = embedment correction
$= 1 - 0.25\ (D/B)$ [5.7]

where:

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

The uncorrected SPT blowcount data are used in calculating settlement, however, if the sand is dense, saturated and very fine or silty (e.g., abundant fines content), it was recommended that the blowcount should be corrected according to:

$$N_c = 15 + 0.5\ (N - 15) \text{ for } N > 15 \qquad [5.8]$$

The correction for water table applies to cases where ground water is at or above the base of the footing (complete submerged case). 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 correction is often omitted from the settlement estimates using this method since the method is generally considered to be overly conservative.

5.2.2 Meyerhof (1956, 1965)

Meyerhof (1956) suggested that the allowable bearing pressures for a footing on granular soils could be estimated based on the results of SPT blowcounts. 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 cause 25 mm (1 in.) of total settlement, Meyerhof (1956) proposed the following expression for dry and moist sands:

$$q_a = N/8 \qquad \text{(for } B \leq 4 \text{ ft.)} \qquad [5.9]$$
$$q_a = N(1 + 1/B)^2/12 \qquad \text{(for } B > 4 \text{ ft.)} \qquad [5.10]$$
$$q_a = N/10 \qquad \text{(approximately, for any B)} \qquad [5.11]$$

where:

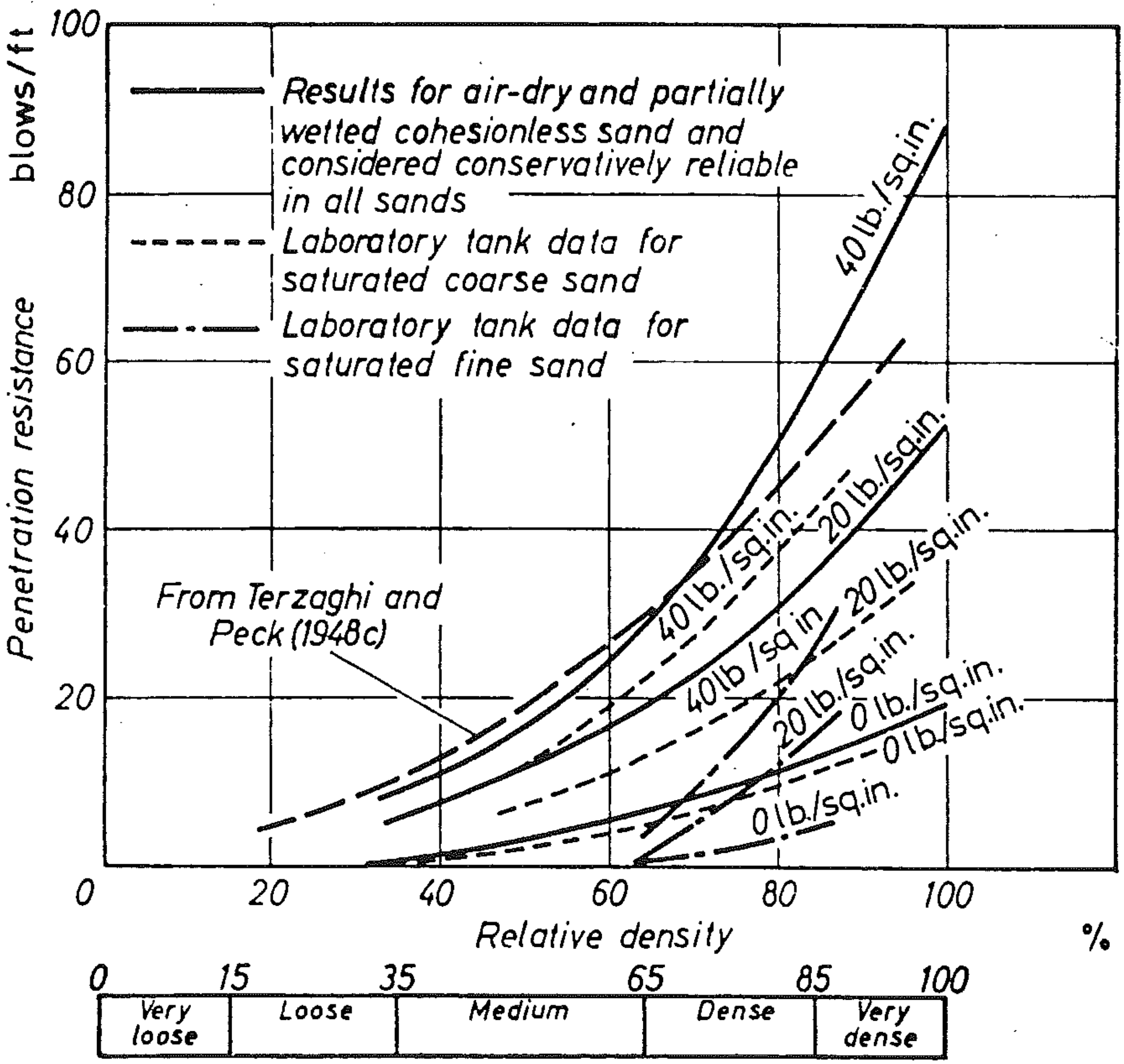


Figure 5.1 Gibbs and Holtz (1957) SPT Correction.

q_a = allowable bearing pressure (tsf)
N = uncorrected SPT blowcount
B = footing width (ft.)

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

$$N_c = 15 + 0.5\,(N - 15) \qquad [5.12]$$

which is the same as Equation 5.8.

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 Equations 5.9, 5.10 and 5.11 should be reduced to account for position of the water table as 1/2 for fully submerged.

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

$$s = 4q/N \quad \text{(for } B \le 4 \text{ ft.)} \qquad [5.13]$$
$$s = [6q/N][B/(B+1)]^2 \quad \text{(for } B > 4 \text{ ft.)} \qquad [5.14]$$
$$s = 6q/N \quad \text{(for rafts)} \qquad [5.15]$$

where:

s = settlement (in inches)
q = footing stress (in tsf)
N = uncorrected blowcounts
B = footing width (in ft.)
C_d = embedment correction factor

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

5.2.3 Hough (1959, 1969)

Hough (1959) presented an approach for calculating settlements of foundations on sands which is similar in form to calculating the one-dimensional consolidation settlement of structures on clay. The subsurface is divided into a number of appropriate layers, the change in vertical

the compression of the layer is calculated. The general expression for settlement is:

$$s = \sum_{o}^{z} (1/C)\, \Delta z \log [(\sigma'_{vo} + \Delta\sigma'_v)/\sigma'_{vo}] \qquad [5.16]$$

where:

s = settlement (in ft.)
C = bearing capacity index = $(1+e_o)/C_c$
Δz = thickness of the layer (in ft.)
σ'_{vo} = initial vertical effective stress at the mid-height of the layer
$\Delta\sigma'_v$ = change in vertical effective stress at the mid-height of the layer
z = thickness of the compressible zone

The change in vertical effective stress resulting from the foundation loading is obtained from elastic theory simple stress distribution charts such as Bousinesq. The thickness over which compression takes place is assumed to be equal to the depth where significant stress increases, i.e., $\Delta\sigma'_v/q$ is equal to 10%. Alternatively, it was suggested that one may use an approximate stress distribution method to obtain $\Delta\sigma'_v$ as:

$$\Delta\sigma'_v = p/(B+h)^2 \qquad \text{(for square footing)} \qquad [5.17]$$
$$\Delta\sigma'_v = p/[(B+h)(L+h)] \qquad \text{(for rectangular footing)} \qquad [5.18]$$

where:

p = applied footing load
L = length of footing
h = depth

No mention was made by Hough (1959) to apply any correction factors to the SPT blowcounts and therefore the bearing capacity index, C, was originally presented by Hough (1959) as a function of the uncorrected field SPT blowcounts as shown in Figure 5.2. Even in his later textbook on soil mechanics, Hough (1969) makes no mention of correcting SPT blowcounts but presented a new chart showing the relationship between SPT blowcounts and bearing capacity index. The new chart is shown in Figure 5.3 for various soil types.

There is a significant difference in the Charts presented in Figures 5.2 and 5.3, therefore, it is important that users of this method identify which chart is being used. For example, for a N = 20 in a "well graded silty sand and gravel", Figure 5.2 gives a bearing capacity index of approximately 78 while Figure 5.3 gives a value of about 50 which is a difference of almost 50%.

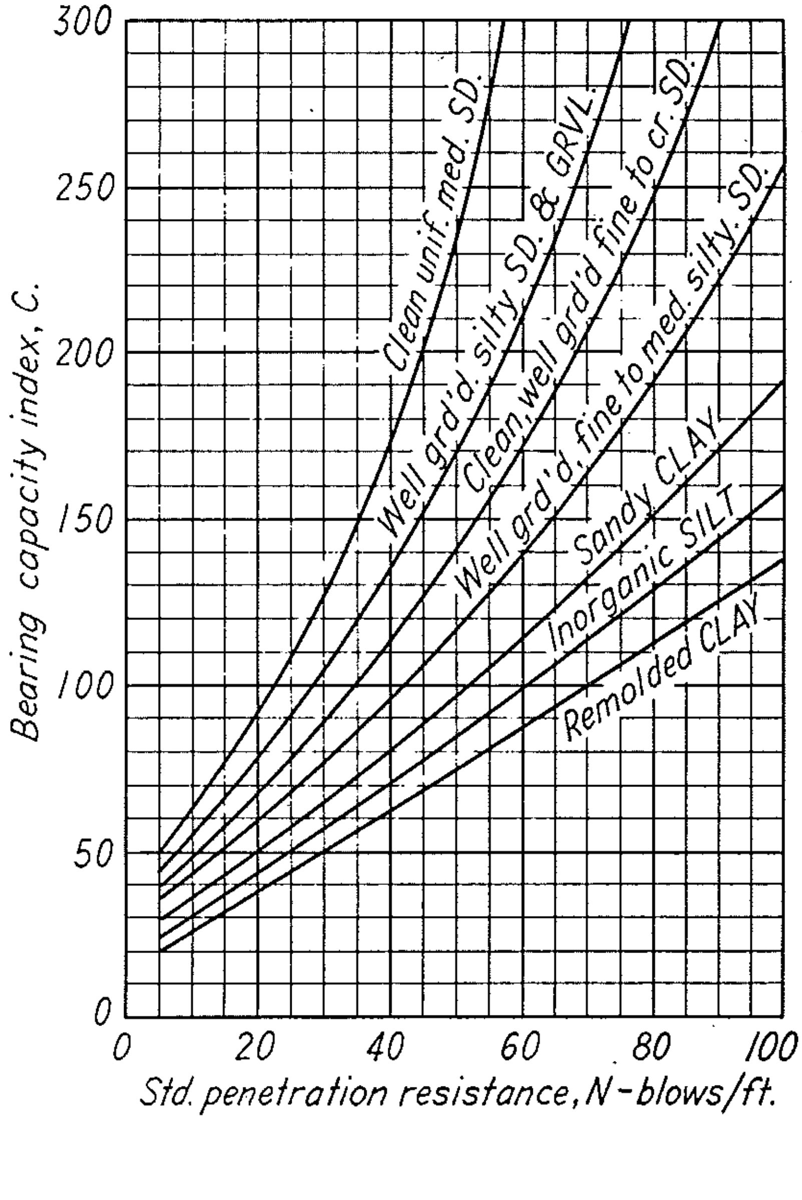


Figure 5.2 Hough (1959) Bearing Capacity Index.

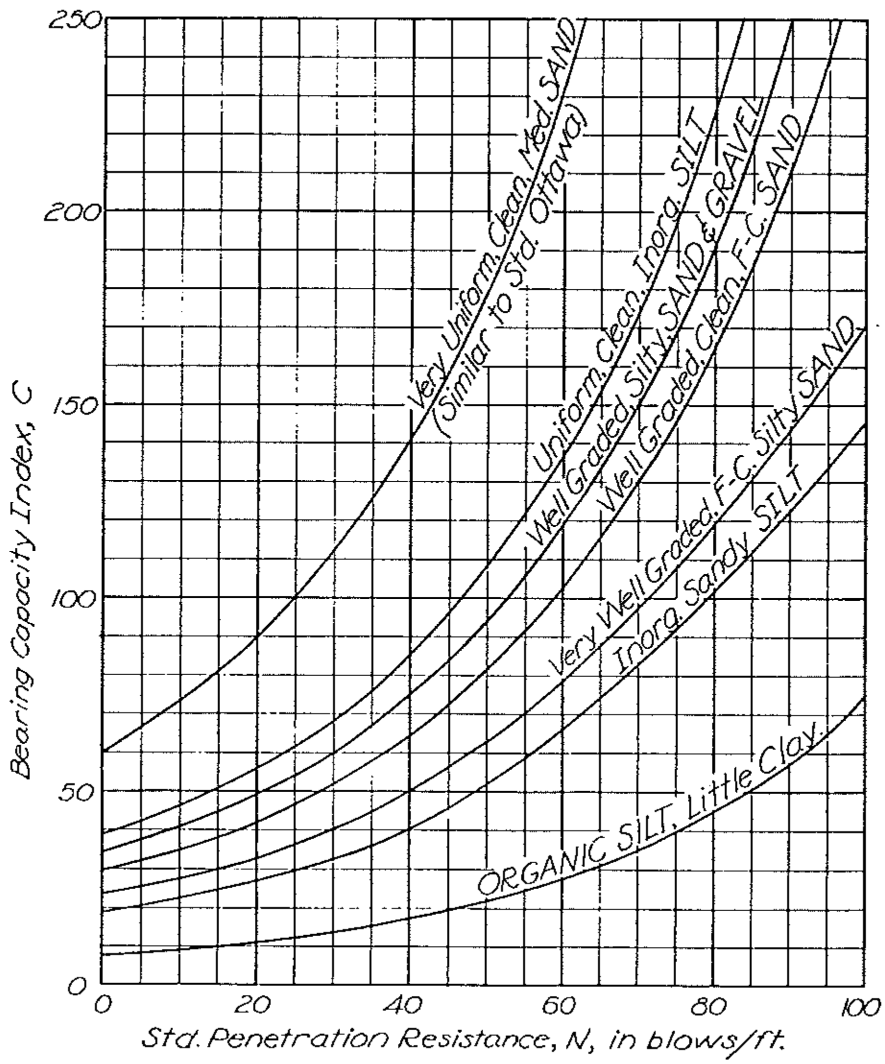


Figure 5.3 Hough (1969) Bearing Capacity Index.

5.2.4 Teng (1962)

An interpretation of the Terzaghi and Peck (1948) bearing capacity chart was presented by Teng (1962) for estimating settlement of shallow foundations on sand. Teng (1962) put the Terzaghi and Peck (1948) chart in equation form. The settlement is determined from:

$$s = [q/(720(N_c - 3))]\,[2B/(B+1)]^2\,[1/(C_w)(C_d)] \qquad [5.19]$$

where:

s = settlement (in inches)
q = net footing stress (in psf)
N_c = corrected blowcount
B = footing width (in ft.)
C_w = water table correction
$= 0.5 + 0.5\,(W/D)/(B) \geq 0.5$ for water at and below footing base [5.20]
C_d = embedment correction
$= 1 + (D/B) \leq 2.0$ [5.21]

The blowcount correction is similar to the Gibbs and Holtz (1957) correction shown in Figure 5.1 and the corrected blowcount for use in Equation 5.19 is obtained as:

$$N_c = N\,[50/(p' + 10)] \qquad [5.22]$$

where:

p' = effective overburden stress at the middepth where the blowcount is taken (about $D + (B/2)$, in psi ($\leq$ 40 psi)

5.2.5 Sutherland (1963)

Sutherland (1963) suggested a slight modification of the Terzaghi and Peck (1948) settlement method to incorporate the Gibbs and Holtz (1957) correction to SPT blowcounts for effective overburden stress. The blowcount vs. relative density curve suggested by Terzaghi and Peck was superimposed onto the chart proposed by Gibbs and Holtz as shown in Figure 5.4. In order to obtain the "equivalent" or corrected blowcount, N_E, the field or uncorrected blowcount, N, is used to estimate relative density by locating the intersection between N and the appropriate effective overburden stress curve. A perpendicular line is then projected upward to intersect the Terzaghi and Peck curve. The corrected blowcount for use in settlement calculations is obtained by projecting horizontally. The settlement calculations are then made using the same procedure as presented by Terzaghi and Peck.

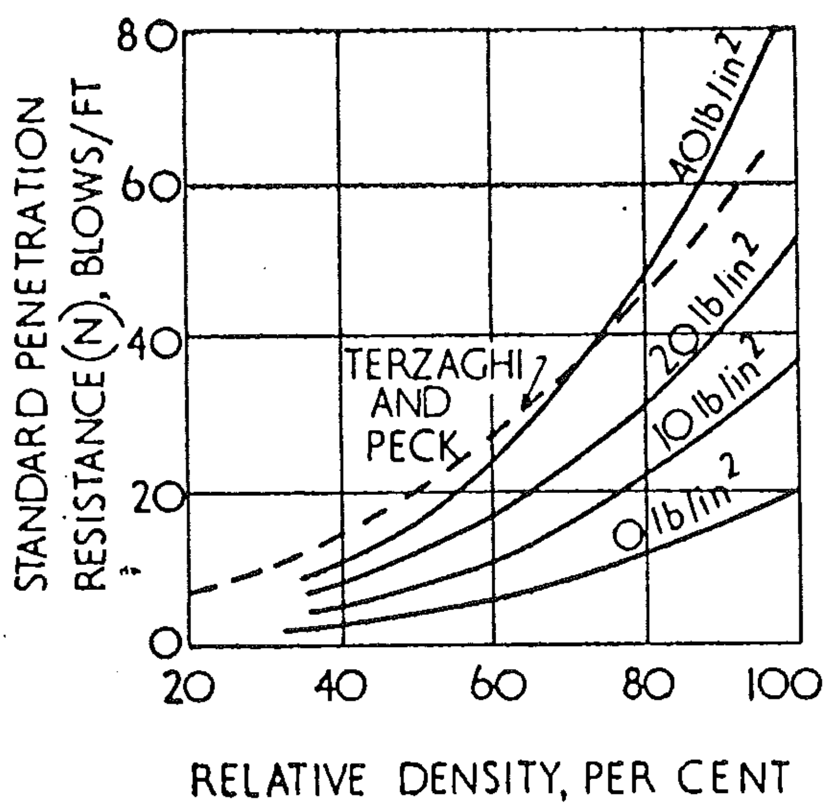


Figure 5.4 Sutherland (1963) Chart for Corrected Blowcount.

Sutherland (1963) compared the results of this procedure with a number of reported case histories in which the results of plate bearing tests were available and found that this procedure provided an improvement over the original Terzaghi and Peck procedure, however, it still tended to overestimate settlement since it appeared that the allowable bearing pressure was underestimated by almost 40%.

5.2.6 Alpan (1964)

Another settlement method based primarily on the Terzaghi and Peck (1948) method was presented by Alpan (1964). This method indirectly uses a corrected blowcount 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:

$$s = s_o\,[2B/(B+1)]^2\,mC_w \qquad [5.23]$$

where:

s = settlement (in inches)
s_o = settlement of a 1 ft^2 plate (in inches)
B = footing width (in ft.)
m = shape correction factor
C_w = water table correction factor
= 2.0 = 0.5 (D/B) ≤ 2.0 for water located immediately below the footing [5.24]

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

$$s_o = \alpha q B \qquad [5.25]$$

where:

q = footing stress (in tsf)
α = a constant (dependent upon the corrected blowcount N_c)

The blowcount value at the foundation level is first used to estimate the relative density of the sand, D_r, using the suggestions of Gibbs and Holtz (1957) which was put into a more convenient form by Coffman (1960) as shown in Figure 5.5. The correction factor, α, is shown in graphical form in Figure 5.6. Note that two charts are suggested by Alpan (1964); one for corrected blowcount values between 5 and 50; and another for corrected blowcount values between 25 and 80.

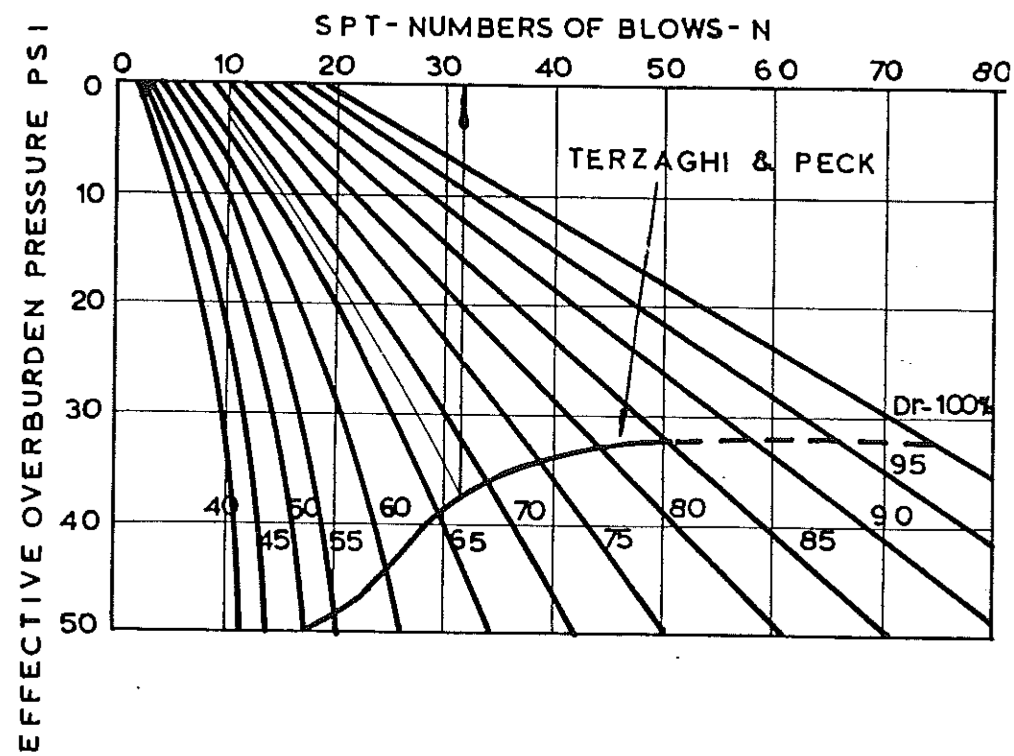


Figure 5.5 Coffman (1960) Interpretation of Gibbs and Holtz SPT Correction.

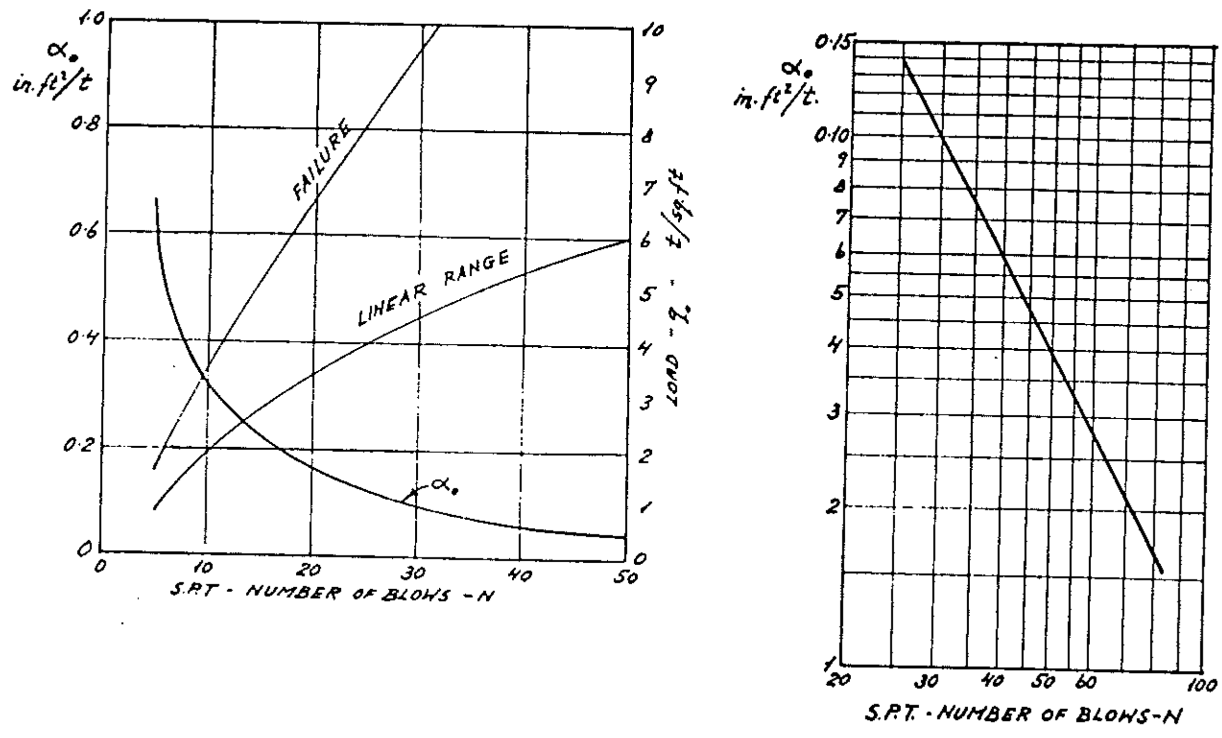


Figure 5.6 Alpan (1964) Correction Factors.

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

For very fine sand or silty sand, since the blowcount value may be too high, leading to an overestimate of relative density and thus a underestimation of the settlement, Alpan suggested using the correction presented by Terzaghi and Peck for N values greater that 15 as:

$$N_c = 15 + 0.5\,(N - 15) \qquad [5.8]$$

An additional shape correction was suggested by Alpan to account for foundation geometry. Shape correction factors, m, are presented in Table 5.2.

Table 5.2 Shape Correction Factors For Alpan (1964) Method

	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

5.2.7 D'Appolonia et al. (1968)

In an extensive study of the settlement performance of a large number of footings on sand, D'Appolonia et al. (1968) used a modification of the Terzaghi and Peck (1948, 1967) and Meyerhof (1956, 1965) methods to predict settlement. Based on their observations, they suggested that settlement should be estimated as:

$$s = [16q/(3N_c)]\, C_d \qquad \text{(for } B \leq 4 \text{ ft.)} \qquad [5.26]$$
$$s = (8q/N_c)\, [B/(B+1)]^2\, C_d \qquad \text{(for } B > 4 \text{ ft.)} \qquad [5.27]$$
$$s = (8q/N_c)\, C_d \qquad \text{(for rafts)} \qquad [5.28]$$

where:

s = settlement (in inches)
q = footing stress (in tsf)
B = footing width (in ft.)
N_c = corrected blowcounts
C_d = embeddment correction
= 1 - 0.25 (D/B) [5.29]

The depth or embedment correction factor is the same as the proposed by Meyerhof (1956). The blowcounts are to be corrected by the method described by Gibbs and Holtz (1957). Additionally, it will be noted that there is no water table correction used with this method.

The comparisons for footing settlement presented by D'Appolonia et al. (1968) were for vibratory compacted dune sand and would likely be considered overconsolidated. This should be kept in mind when using this procedure.

5.2.8 Bowles (1968)

Bowles (1968) and in later editions of his textbook (Bowles 1977, 1982, 1988) suggested that the allowable bearing pressures for 25mm (1 in.) of settlement presented by both Terzaghi and Peck and Meyerhof were too conservative and recommended an increase in the allowable pressure of approximately 50%. Assuming that the resulting settlement is linearly proportional to the allowable pressure, the estimated settlement for any pressure may be obtained by a simple ratio of the applied pressure to the allowable pressure. The charts presented by Bowles (1968) are presented and discussed in more detail in Chapter 6.0.

5.2.9 Peck and Bazaraa (1969)

A method similar to the Terzaghi and Peck (1948, 1967) method was presented by Peck and Bazaraa (1969) in a discussion to the paper published by D'Appolonia et al. (1968). It appears that the method is derived largely from work presented in Bazaraa's PhD Dissertation (1967). The method suggests a new blowcount correction to account for overburden stress and increases the allowable bearing capacity obtained by Terzaghi and Peck by 50%. The latter modification was incorporated to provide less conservative estimates. In addition, water table effects and an embedment correction were suggested. Settlement is calculated from the expression:

$$s = [(16q)/3N_c]\,[C_dC_w] \qquad \text{(for } B \leq 4 \text{ ft.)} \qquad [5.30]$$
$$s = [(8q)/N_c]\,[B/(B+1)]^2\,[C_dC_w] \qquad \text{(for } B > 4 \text{ ft.)} \qquad [5.31]$$
$$s = [8q/N_c]\,[C_dC_w] \qquad \text{(for rafts)} \qquad [5.32]$$

where:

s = settlement (in inches)
q = footing stress (in tsf)
N_c = corrected blowcount
B = footing width (in ft.)
C_d = embedment correction
$= 1.0 - 0.4\,[(\gamma D/q)]^{0.5}$ [5.33]

where:

γ = soil total unit weight

C_w = water table correction
$= \sigma'_{v\ dry}/\sigma'_{v\ wet}$ [5.34]
where σ'_v is computed at D + B/2

The water table correction factor C_w is the ratio of the effective overburden stress at D + B/2 of dry soil to effective overburden stress at D + B/2 at the location of the water table. If the water table is located below the depth D + B/2, then $C_w = 1.0$.

The corrected blowcount value is obtained from:

$N_c = (4N)/(1 + 2p')$ for $p' \leq 1.5$ ksf [5.35]
$N_c = (4N)/(3.25 + 0.5p')$ for $p' > 1.5$ ksf [5.36]

where:

p' = effective overburden stress (in ksf) at a depth of approximately D + B/2

5.2.10 Webb (1969)

Apparently, Webb (1969) was one of the first authors to suggest that a layered approach be used to estimate the total settlement from a footing resting on sand. Settlement is calculated from the expression:

$$s = \sum_{i=1}^{n} (\sigma_{zi}/E)\Delta z_i \quad [5.37]$$

where:

s = settlement (in ft.)
σ_{zi} = vertical stress in soil layer i produced by foundation stress q (in psf)
Δz_i = thickness of layer i (in ft.)
E = soil elastic modulus (in psf)

This method implies that the maximum strains occur immediately beneath the base of the foundation where the vertical stresses are maximum values. This is contrary to results from tests on small plates (e.g. Bjerrum and Eggestad (1963); Morgan and Gerrard (1971); and Schmertmann et al. (1978)) as well as elastic theory which indicate maximum strains occurring at depths of between 0.5 B and 0.75 B below the foundation base. The value of σ_{zi} is obtained from simple Bousinesq elastic theory.

The soil elastic modulus for use in Equation 5.37 is obtained directly from the uncorrected

SPT results as:

$E = 5(N + 15)$	for submerged fine to medium sands	[5.38]
$E = 3.33\ (N + 5)$	for clayey sands	[5.39]
$E = 4(N + 12)$	for average profiles	[5.40]

These correlations were developed on empirical observations between SPT results and field plate loading tests.

5.2.11 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 soil compressibility, M, which is obtained from SPT blowcounts. The settlement is calculated from the general elastic solution equation, as discussed previously in Section 4 as:

$$s = (qBI)/M \qquad [5.41]$$

where:

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

The influence factor I in Equation 5.41 is really the product of two factors, $(U_o)(U_I)$, which account for the geometry and depth of the footing and the depth to an incompressible layer. The factors U_o and U_I were developed by Janbu et al. (1956) and were previously presented in Section 4.

The blowcount 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 factors are applied. The soil modulus of compressibility is obtained from the SPT blowcount as:

$M = 196 + 7.9\ (N)$ (in tsf)	for NC Sand	[5.42]
$M = 416 + 10.9\ (N)$ (in tsf)	for OC Sand	[5.43]

Figure 5.7 presents the original figure of D'Appolonia et al. (1970).

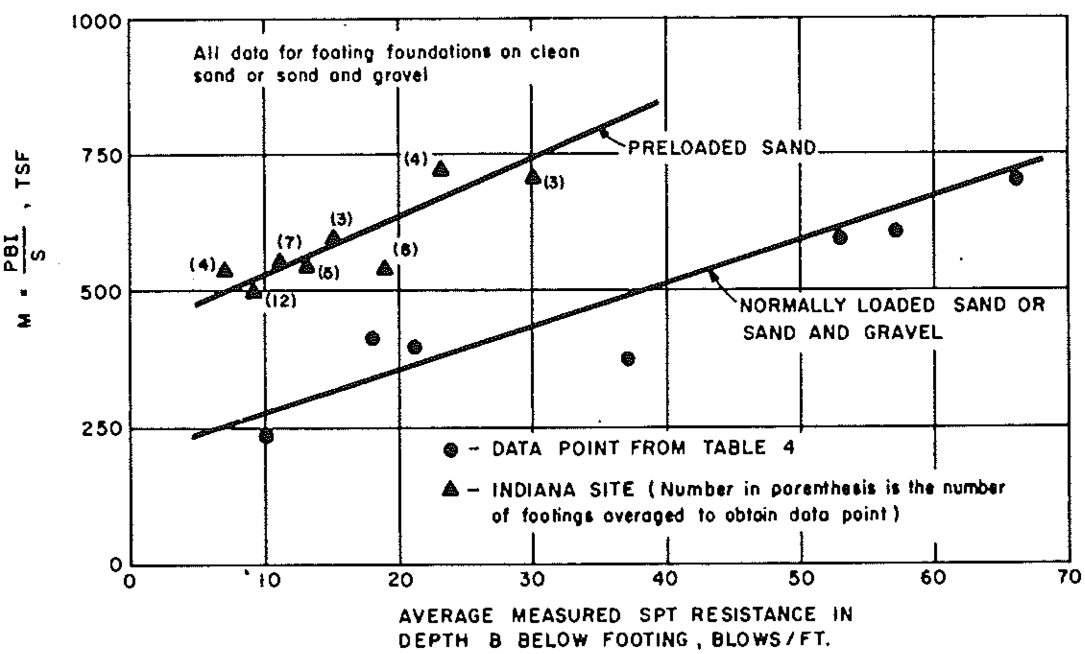


Figure 5.7 D'Appolonia et al. (1970) Correlation Between Modulus of Compressibility and SPT Blowcounts.

5.2.12 Parry (1971)

A method was proposed by Parry (1971) which is similar in form to the method of D'Appolonia et al. (1970) and not unlike the general elastic methods described in Section 4. Settlement is calculated from:

$$s = \alpha[qB/N_m]\,[C_d C_w C_t] \qquad [5.44]$$

where:

s = settlement (in inches)
α = a constant = 0.25
q = footing stress (in tsf)
B = footing width (in ft.)
N_m = weighted average of uncorrected N values between D and D = 2B
C_d = embedment correction factor
$= [1.3\,(0.75 + D/B)]/[1 + 0.25\,(D/B)]$ [5.45]
C_w = water table correction factor
$= 1 + (D_w)/(D + 3B/4)$ for $O<D_w<D$ [5.46]
$= 1 + [D_w(2B + D-D_w)]/[2B(D+0.75B)]$ for $O< (D_w - D) < 2B$ [5.47]

C_t = thickness of compressible sand stratum correction factor as shown in Figure 5.8.

This method assumes that in a uniform soil half of the settlement occurs within a depth of 3B/4 below the foundation level and the remaining half within a depth between 3B/4 and 2B below the foundation.

The value of α in Equation 5.44 was obtained by Parry (1971) by comparing predicted settlements with a number of plate bearing tests and several published field records of settlements from small and large footings, tanks, and rafts. With the exception of one reported case history of a chimney settlement, the range in ratio of calculated to measured settlement was from 0.8 to 2.6 averaging 1.2. In twenty of twenty-four cases the range was 0.8 to 1.5, averaging 1.1.

The value of the "representative" blowcount, N_m, for use in Eq. 5.44 was recommended as follows:

1- Where N varies consistently N_m should be taken as the "observed" value at a depth of 3B/4 below the foundation.

2 - Where N values do not vary consistently use the following procedure to obtain N_m:
a- take the average value of N between the foundation level and a depth of 3B/4 and multiply by 3, giving $3N_1$,

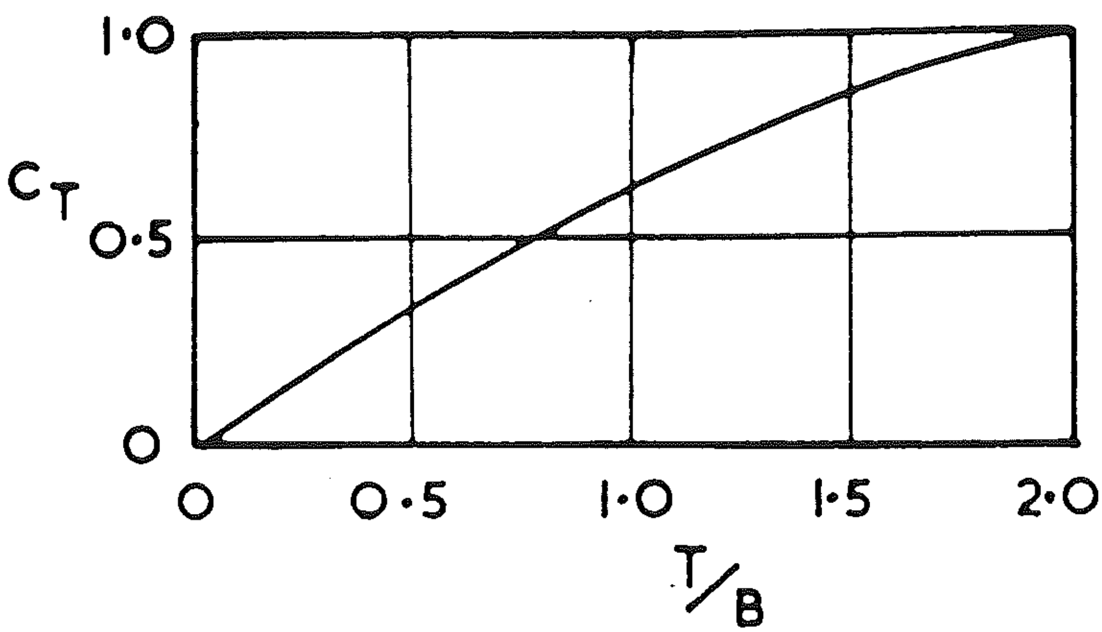


Figure 5.8 Parry (1971) Correction Factor for Layer Thickness.

b- take the average value of N between 3B/4 and 1.5B
and multiply by 2, giving $2N_2$,

c- take the average value of N between 1.5B and 2B giving N_3.

d- Add $3N_1 + 2N_2 + N_3$ and divide by 6 to get N_m.

5.2.13 Schultze and Sherif (1973)

Based on the results of a study of the observed settlements at 48 sites, Schultze and Sherif (1973) developed an empirical method to estimate the settlement of shallow foundations on sand using results of the SPT. The settlement expression is given as:

$$s = (QF_c)/(N^{0.87}C_d) \tag{5.48}$$

where:

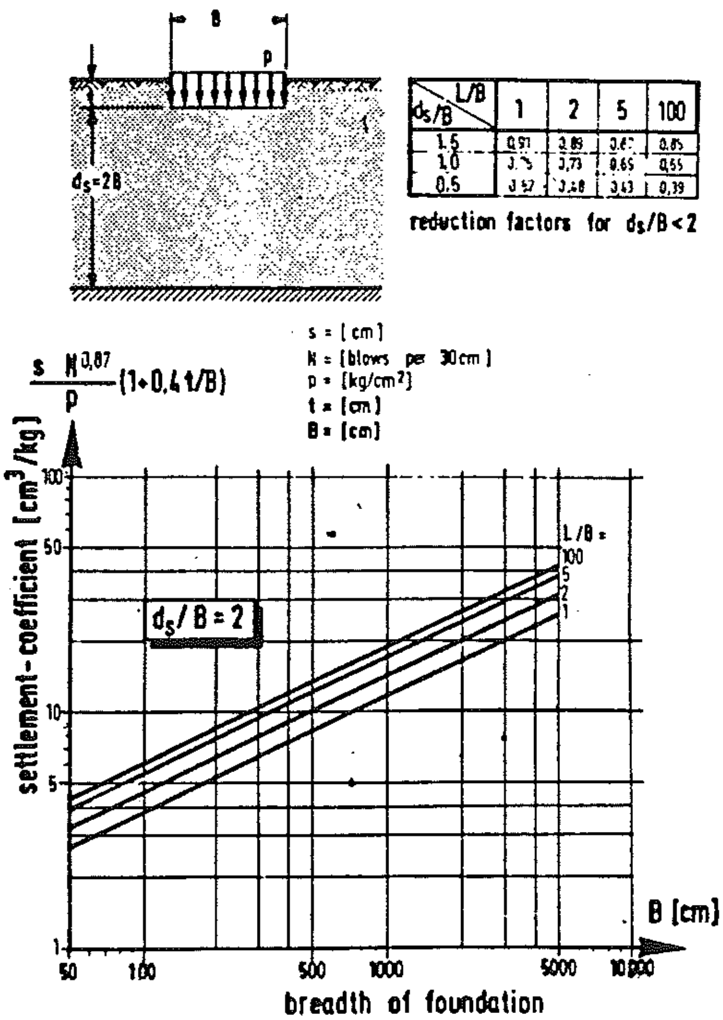
s = settlement (in cm)
Q = gross footing stress (surcharge not subtracted) (in kg/cm^2)
F_c = influence factor based on footing shape (in cm^3/kg)
N = uncorrected blowcount
C_d = embedment correction

$$= 1 + 0.4\,(D/B) \leq 1.4 \tag{5.49}$$

The exponent of 0.87 on the N value in Equation 5.48 is based on a statistical evaluation of their results. The influence factor, F_c, is a function of footing geometry as well as the depth below the footing, D_s, to an incompressible layer. Values of F_c for D_s/B less than 2.0 are given in Table 5.3. For $D_s/B > 2.0$ the chart shown in Figure 5.9 is recommended. The authors reported that the accuracy of this method in predicting settlements is generally about ± 40%.

Table 5.3 Influence Factors for Schultze and Sherif Method

D_s/B	L/ B			
	1	2	5	100
1.5	0.91	0.89	0.87	0.85
1.0	0.85	0.73	0.69	0.55
0.5	0.52	0.48	0.43	0.39



d_s/B \ L/B	1	2	5	100
1.5	0.91	0.89	[illegible]	0.85
1.0	[illegible]	0.73	0.65	0.55
0.5	0.57	0.48	0.43	0.39

reduction factors for $d_s/B<2$

Figure 5.9 Schultze and Sherif (1973) Influence Factor Chart.

5.2.14 Peck et al. (1974)

In their foundation engineering textbook, Peck, Hanson and Thornburn (1974) suggested a modification to the Terzaghi and Peck (1948, 1967) method by primarily adding a blowcount correction factor for overburden stress and a correction for groundwater located near the base of the footing. The expressions for settlement are:

$s = (q)/(0.11\ N_c\ C_w)$ (for medium sized footings (B> 2 ft.)) [5.50]
$s = (q)/(0.22\ N_c\ C_w)$ (for rafts) [5.51]

where:

s = settlement (in inches)
q = footing stress (in tsf)
N_c = corrected blowcount = NC_n
C_w = water table correction
$= 0.5 + 0.5\ (W/(D+B))$ [5.52]

The overburden correction to field blowcounts can be obtained from the expression:

$C_n = 0.77 \log (20/p')$ [5.53]

where:

p' = effective overburden stress (in tsf) for the measured blowcount at $D + B/2 \geq 0.25$ tsf

For effective overburden stress less than 0.25 tsf (generally in the upper 1.5 m (5 ft) a value of C_n between 1.5 and 2.0 may be assumed.

5.2.15 Meyerhof (1974)

Meyerhof's most recent expressions for settlement are further modifications of the earlier equations given (Meyerhof 1956, 1965) which are generally considered to be conservative. In this case, the settlement is given as:

$s = [(q)\ (B)^{1/2}/(2N)]\ [C_d]$ [5.54]
$s = [(q)\ (B)^{1/2}/N]\ [C_d]$ (for very fine or silty submerged sand) [5.55]

where:

s = settlement (in inches)
q = footing stress (in tsf)
B = footing width (in inches)

N = uncorrected blowcount
C_d = embeddment correction
= 1 - 0.25 (D/B) [5.56]

5.2.16 Arnold (1980)

Arnold (1980) presented a method for estimating the settlement of shallow foundations on sands which is based on establishing the relative density of a sand deposit, using empirical stress-strain relationships for sands of various densities, predicting the strains below a footing and integrating to give to total settlement. The relative density is obtained using the results of the SPT. The settlement equation presented by Arnold (1980) may be stated as:

$$s = 43.06B \sum_{z=0}^{2B} \Delta z \, [\alpha \ln(1/(1-I \cdot q/Q))]/[1+(3.281B)^m]^2 \qquad [5.57]$$

where:

s = settlement (in mm)
B = footing width (in meters)
α = an exponent
I = vertical strain influence factor
q = applied footing stress (kN/m^2)
Q = hypothetical ultimate pressure (kN/m^2)
m = an exponent
Δz = individual layer thickness (in meters)

The influence factor I is related to the depth below the footing and is taken at the mid depth for each soil layer within the compressible zone, assumed to be equal to 2B, as:

$$I = 1-0.5\,(z/B) \qquad [5.58]$$

Values of Q, α, and m are related to the relative density of the sand, D_r, which is obtained from the field blowcounts as:

$$D_r = 25.6 + 20.37\,[(1.26(N-2.4))/(0.0203\gamma H + 1.36) -1]^{0.5} \qquad [5.59]$$

where:

D_r = relative density (in %)
N = field SPT blowcount at mid-depth of layer
γH = effective overburden stress where N is determined (in kN/m^2)

Then:

$Q = 19.63\ D_r - 263.3\ (kN/m^2)$ [5.60]
$\alpha = 0.032766 - 0.0002134\ D_r$ (meters) [5.61]
$m = 0.788 + 0.0025\ D_r$ [5.62]

In order to perform the calculations, the compressible zone is subdivided into individual soil layers, and the settlement for each layer is calculated. The total settlement is then obtained by summing the individual settlements from each layer.

The basic assumption of this method is that at any horizontal plane below the surface of the sand, the relationship between vertical strain and vertical stress (as related to the stress applied to the surface) will be the same as that immediately under a loaded footing at the surface, with appropriate account for overburden stress. Arnold (1980) provided a check of the proposed method with the load-settlement results of 94 published case histories and found that in 50% of the cases the ratio of estimated to observed settlement was between 0.67 and 1.5, while in 73% of the cases, the ratio was between 0.5 and 2.0.

5.2.17 Navfac DM7 (1982)

A method based on the vertical modulus of subgrade reaction is presented in the Navfac design manual DM7.2 (1982) which is loosely based on the Terzaghi and Peck (1948, 1967) method. The modulus of subgrade reaction may be obtain by SPT blowcounts using correlations presented by Terzaghi (1955) or by using expressions suggested by Bazaraa (1967). The expression for settlement is:

$s = [(Cq)/K_v]\ [B/(B+1)]^2\ C_w$ [5.63]

where:

s = settlement (in ft.)
q = footing stress (in tsf)
B = footing width (in ft.)
K_v = modulus of subgrade reaction (tsf)
C = coefficient based on footing width
= 4.0 + (20 - B)/10, for 20 ft. $\leq$ B $\leq$ 40 ft. [5.64]
= 4.0 for B < 20 ft. [5.65]
= 2.0 for B > 40 ft. [5.66]
C_w = water table correction factor
= 2.0 - [(W-D)/(1.5 B)] $\leq$ 2.0 for water to a depth of 1.5B below the foundation [5.67]

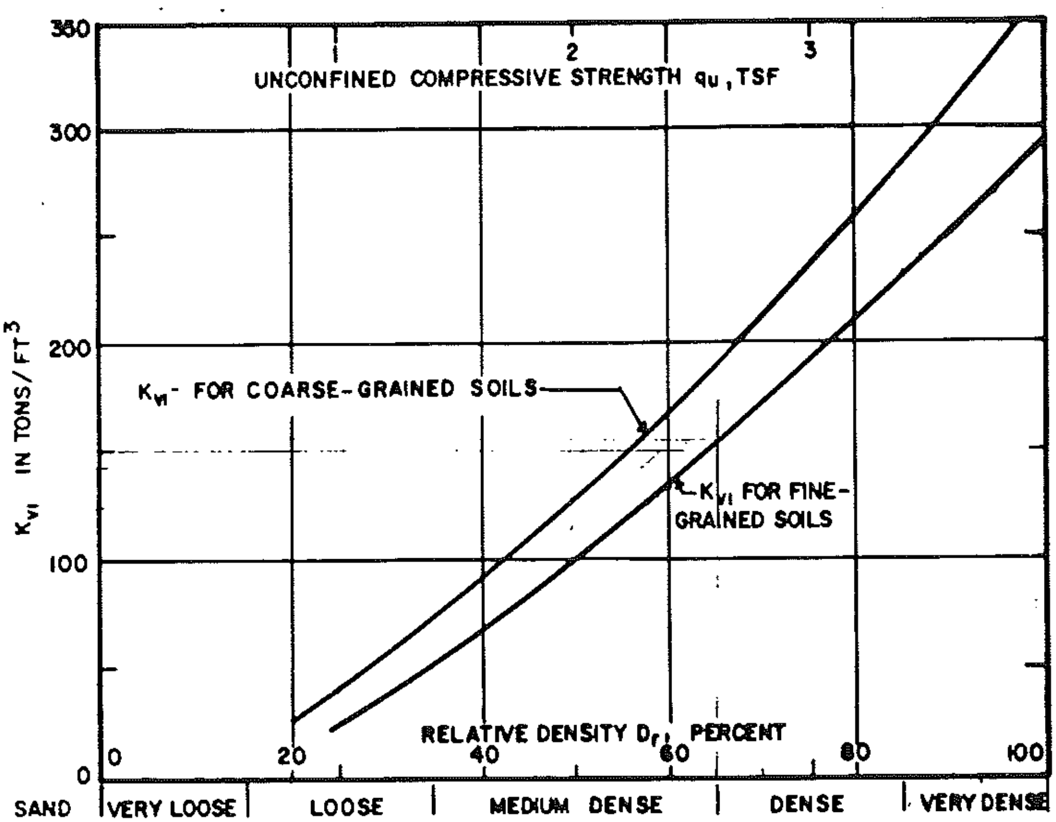


Figure 5.10 Navfac (1982) Correlation for Subgrade Reaction Modulus.

The correlation between K_v and relative density, D_r, presented by Navfac is given in Figure 5.10. To obtain an estimate of relative density from the SPT blowcount, the following equations, presented by Bazaraa (1967) are recommended:

$D_r = [N/20(1 + 2p')]^{1/2}$ for $p' \le 1.5$ ksf [5.68]
$D_r = [N/20(3.25 + 0.5\,p')]^{1/2}$ for p' 1.5 ksf [5.69]

where:

D_r = relative density (decimal)
N = uncorrected blowcount at a depth of approximately D + B/2
p' = effective overburden stress at a depth of D + B/2

Alternatively, Navfac DM7.1 presented a chart for estimating relative density from blowcounts, shown in Figure 5.11.

5.2.18 Burland and Burbidge (1985)

Burland and Burbidge (1985) presented another empirical method for using SPT results to estimate settlement of footings on granular soils. This method takes into account the load intensity, the shape of the footing and the depth of influence below the footing. It also considers whether the deposit is normally consolidated or overconsolidated and is based on a reevaluation of nearly 200 published case histories of observed settlements of various size footings.

Settlements are calculated from the following expressions:

$s = 0.14\, C_s C_I I_c\, (B/B_r)^{0.7}\, (q'/\sigma_r)\, B_r$ for NC soils [5.70]
$s = 0.047\, C_s C_I I_c\, (B/B_r)^{0.7}\, (q'/\sigma_r)\, B_r$ for OC soils and $q' \le \sigma'_c$ [5.71]
$s = 0.14\, C_s C_I I_c\, (B/B_r)^{0.7}\, ((q' - 0.67\,\sigma'_c)/(\sigma_r))$ for OC soils and $q' > \sigma'_c$ [5.72]

where:

s = settlement (in mm)
C_s = shape of factor
C_I = depth of influence correction factor
I_c = soil compressibility index
B = footing width (in meters)
B_r = reference width = 0.3 m
q' = net footing stress (in kPa)
σ_r = reference stress = 100 kPa
σ'_c = preconsolidation stress (in kPa)

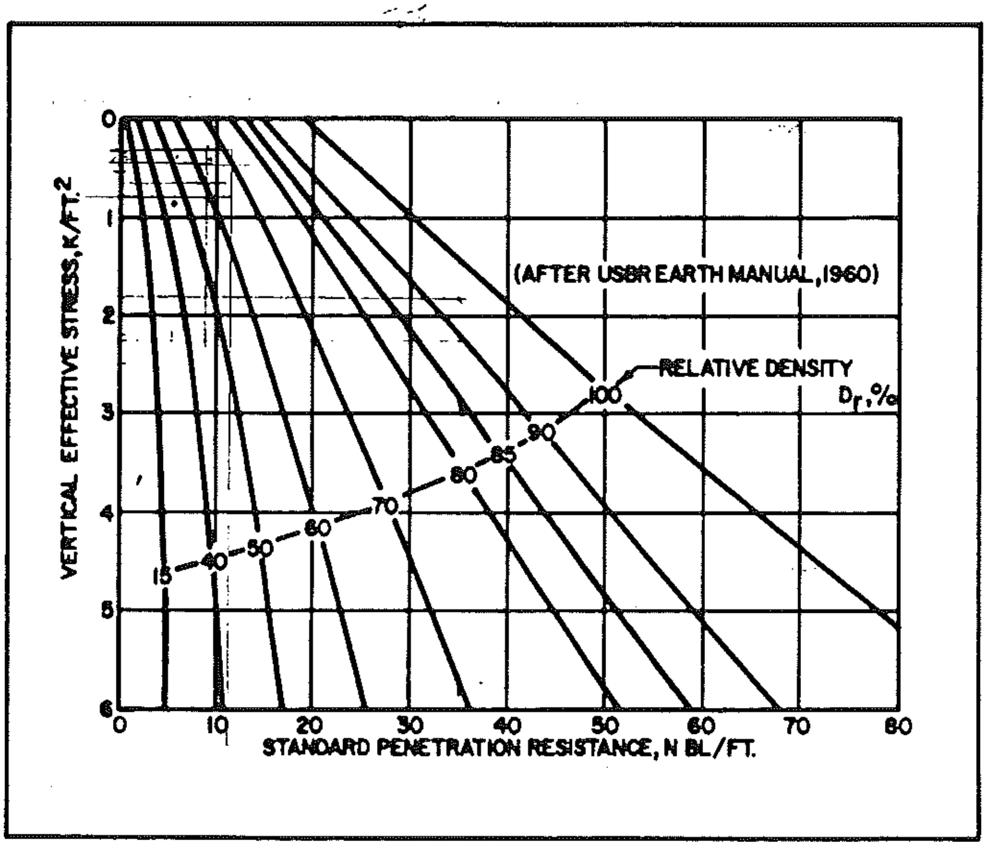


Figure 5.11 Navfac (1982) Correlation Between SPT Blowcount and Relative Density.

The shape factor is calculated as:

$$C_s = [(1.25\ L/B)/((L/B) + 0.25)]^2 \qquad [5.73]$$

where:

L = length of footing
B = footing width

Note that for a circular or square foundation the correction factor, C_s, is equal to 1.0 and for continuous or strip footings $C_s = 1.56$ as L/B tends to infinity.

The depth of influence factor, C_I, is obtained from:

$$C_I = (H/Z_I)\ (2\text{-}IH/Z_I) \leq 1.0 \qquad [5.74]$$

where:

H = depth from the bottom of the footing to the bottom of the compressible soil
Z_I = depth of influence below footing
$= 1.4\ (B/B_r)^{0.75}\ B_r$ [5.75]

If the SPT N_{60} values generally decrease with depth, use $Z_I = 2B$ or the depth to the bottom of the loose layer, whichever is less. The depth of influence correction factor, C_I, is only of concern when a loose soil is underlain by a much denser soil and the boundary between the two layers is within Z_I below the base of the footing.

The soil compressibility index, I_c, is calculated from the SPT blowcounts as:

$$I_c = 1.71/(\tilde{N}_{60})^{1.4} \quad \text{(for NC soils)} \qquad [5.76]$$
$$I_c = 0.57/\ (\tilde{N}_{60})^{1.4} \quad \text{(for OC soils)} \qquad [5.77]$$

where:

$\tilde{N}_{60}$ = average adjusted blowcounts

The blowcount values between the base of the footing and the depth of influence are used and should be corrected for energy only to give N_{60}. No overburden correction is applied. If the soil is a submerged dense very fine or silty sand with $N_{60} > 15$, N_{60} should be adjusted using the correction factor proposed by Terzaghi and Peck (1948). If the soil is gravelly sand or sandy gravel, Burland and Burbidge (1985) recommend multiplying N_{60} by an adjustment factor of 1.25.

5.2.19 Stroud (1989)

Stroud (1989) suggested an interesting approach that could be used to estimate settlement of shallow footings on sands and gravels based on the results of the SPT. Settlement of footings obviously involves estimating stiffness and like a number of other investigators, Stroud recognized that soil stiffness is strain dependent; i.e., the stiffness at small strains being greater that the stiffness at large strain. Following the work of Vesic (1973), Stroud noted that local shear failure of model footings of constant width essentially occurred at the same normalized settlement. Therefore as a first approximation, there appears to be a unique relationship between the degree of foundation loading, i.e., q/q_{ult} and the settlement for varying density. This suggested that q/q_{ult} is an indirect measurement of shear strain.

Since both soil modulus E' and SPT blowcount N vary with mean effective stress level in the ground the ratio E'/N_{60} and its variation with the degree of loading were investigated. Using the data from case histories of shallow foundation settlement compiled by Burland and Burbidge (1985), Stroud (1989) backcalculated the value of soil modulus E' from measured settlement. Figures 5.12 and 5.13 show the results obtained from a wide range of spread footings, raft foundations and large scale plate tests on normally consolidated and overconsolidated sands, respectively. To make use of Figures 5.12 and 5.13, it is necessary to follow the same methodology used by Stroud (1989).

In footing design, the value of q is known based on a trial footing size and anticipated loading and q_{ult} may be estimated using bearing capacity theory incorporating an allowance for local failure. The ultimate bearing capacity is obtained as follows:

1) The values of N_{60} of the material to a depth of about B below the foundation level is used to obtain ϕ' from Figure 5.14. For uniformly graded materials $\phi'_{cv} = 36^{o}$ were assumed.

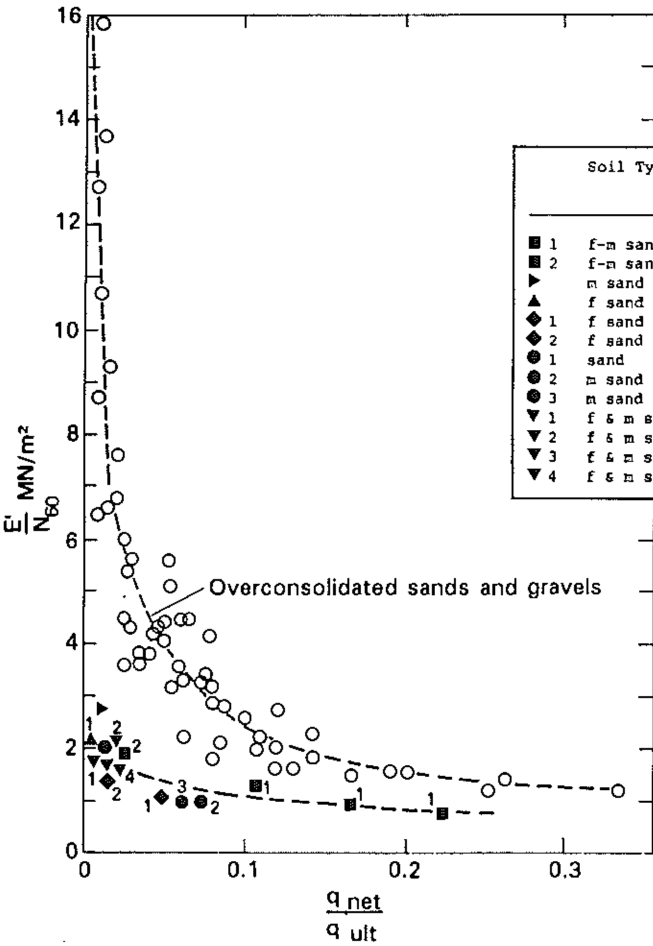
2) A distinction was made between normally consolidated and overconsolidated deposits. For overconsolidated deposits, an OCR = 3 was assumed.

3) Bearing capacity factors for use in calculating q_{ult} with allowance for local shear failure are obtained from Lambe and Whitman (1969) and Terzaghi (1943) as shown in Figure 5.15.

4) The standard Terzaghi bearing capacity equation is used to calculate q_{ult}.

5) q_{net} is taken as the gross effective bearing pressure less the previous existing overburden at the foundation level, i.e., γD_f.

Once the values of q_{net}, q_{ult} and N_{60} have been obtained the value of E' may be evaluated from Figures 5.12 and 5.13.



	Soil Type	N_{60}	B (m)	D (m)	t (m)	q_{net} (kN/m²)	Reference
■ 1	f-m sand	12	6	0	18	<180	Webb (1969)
■ 2	f-m sand	10	33	0	18	155	"
▶	m sand	8	11	3.5	13	76	Glick (1970)
▲	f sand	44	55	9	30	85	Dunn (1974)
◆ 1	f sand	14	26	2	26	220	Nonveiller (1963)
◆ 2	f sand	14	11	2	20	260	"
● 1	sand	28	23	0	7	180	Farrent (1963)
● 2	m sand	13	24	0	10	182	"
● 3	m sand	13	24	0	10	158	"
▼ 1	f & m sand	11	43	0	25	157	Davisson et al (1972)
▼ 2	f & m sand	11	34	0	25	148	"
▼ 3	f & m sand	11	38	0	25	228	"
▼ 4	f & m sand	11	27	0	25	146	"

Figure 5.12 Stroud (1989) Modulus for Normally Consolidated Sand.

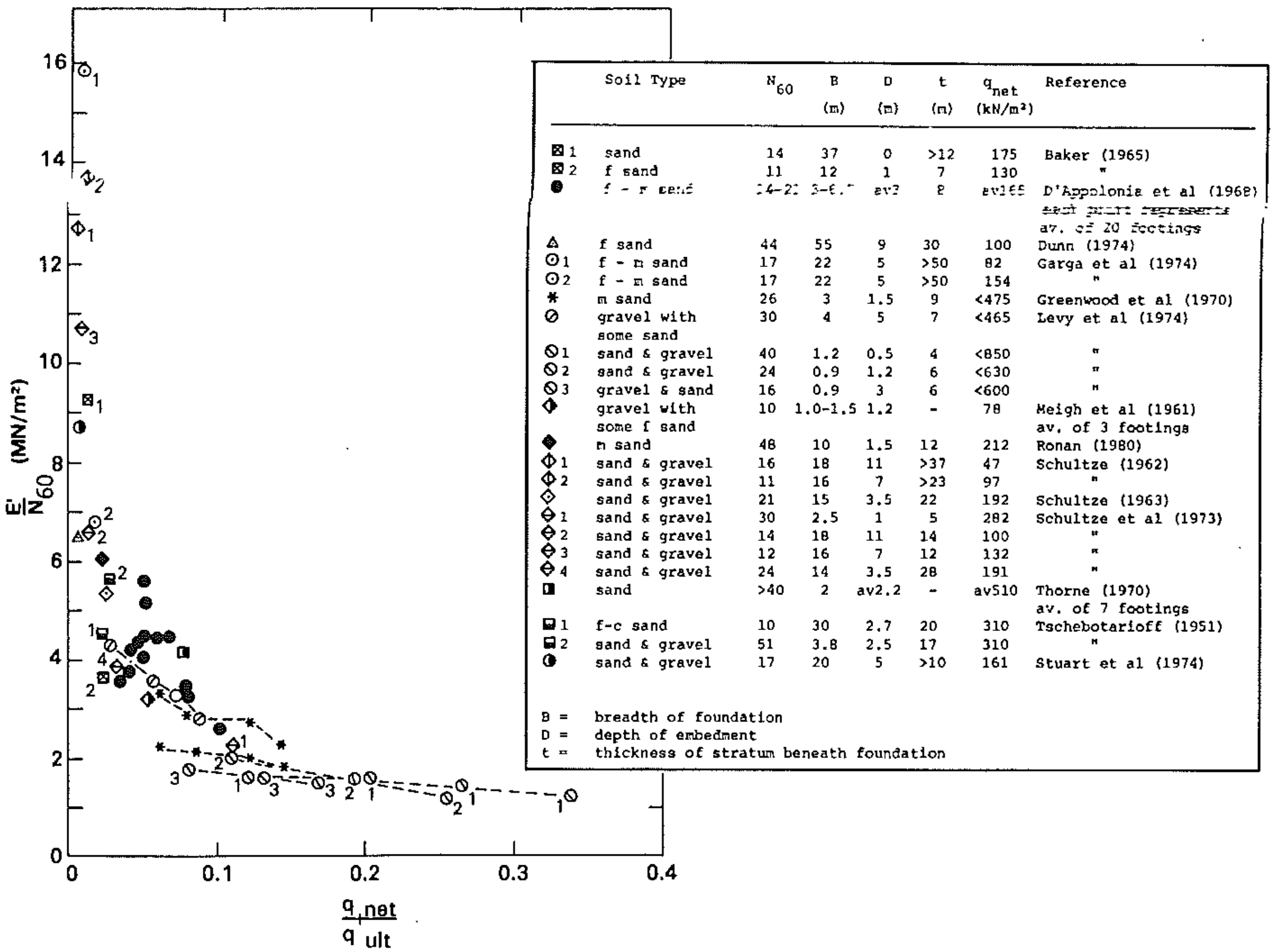


Figure 5.13 Stroud (1989) Modulus for Overconsolidated Sand.

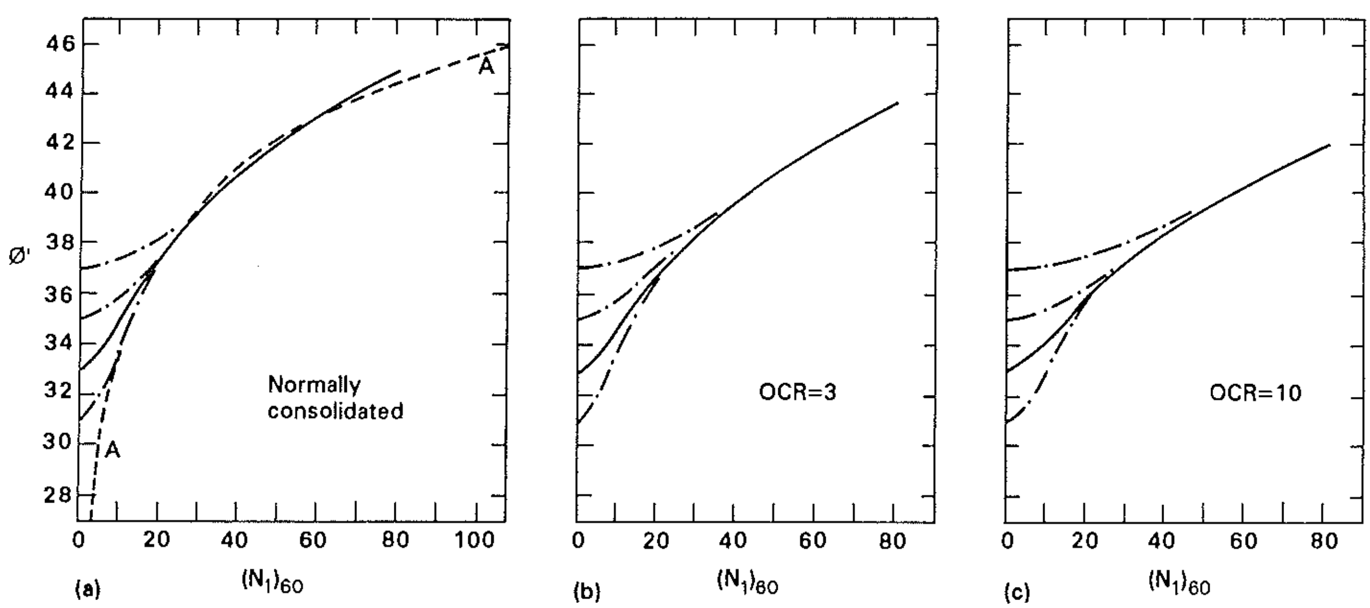


Figure 5.14 Soil Friction Angle from N_{60}.

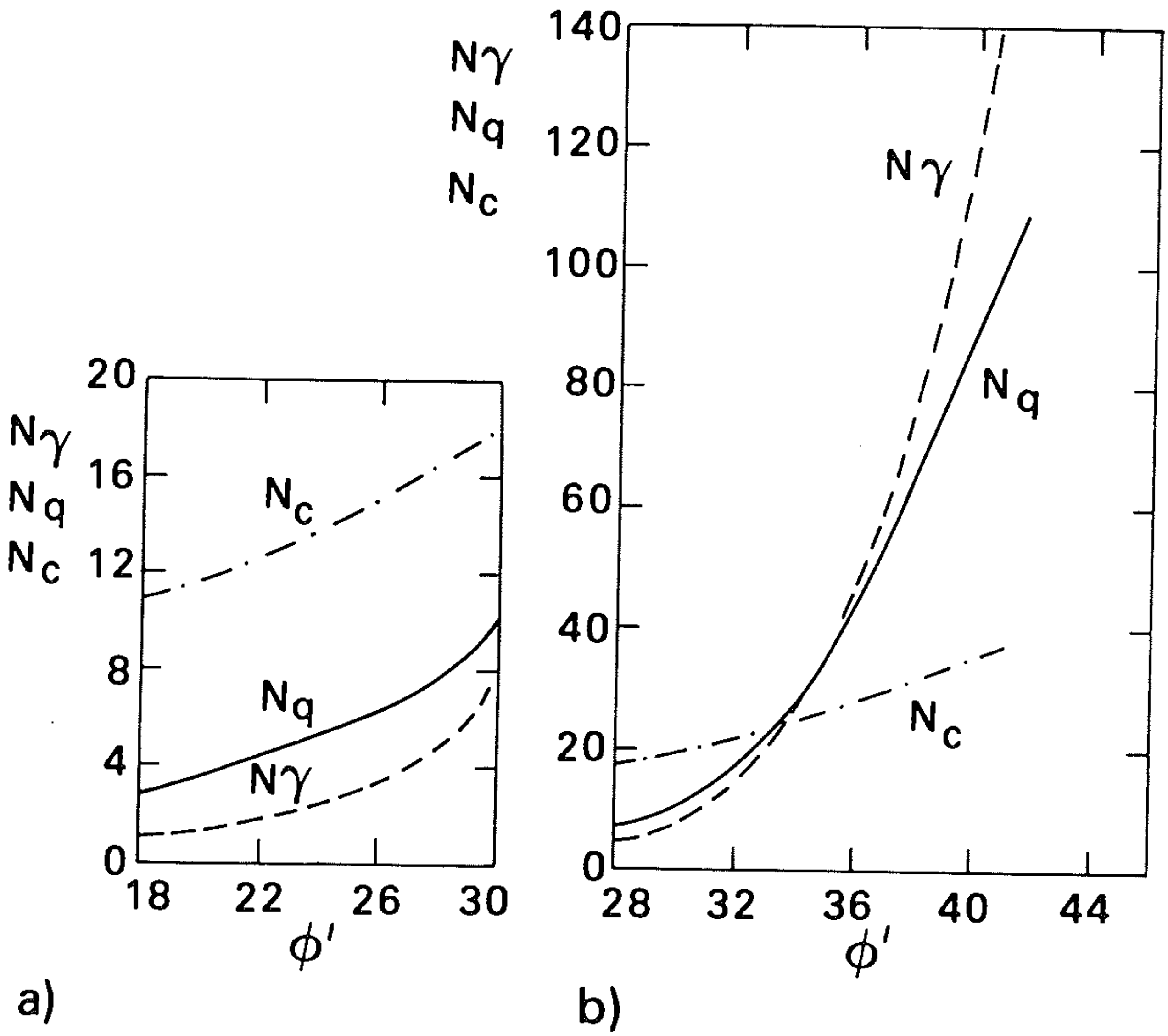


Figure 5.15 Bearing Capacity Factors from a) Lambe and Whitman and b) Terzaghi.

Settlement estimates are then made by using the linear elastic theory described in Section 4 and thus the stiffness is the average secant stiffness beneath the foundation under loading q_{net}. The settlement is calculated from:

$$\rho = (qBI\ \mu_1\ \mu_2)/E \qquad [5.78]$$

The influence coefficient I was estimated from the Steinbrenner charts presented in Lambe and Whitman (1969). Poisson's ratio was taken as 0.25. The foundation depth embedment factor μ_1 was based on Fox (1948; see Figure 4.7) and a factor $\mu_2 = 0.8$ was applied when the foundation is considered rigid.

Most reported case histories of settlement of shallow foundations do not provide a complete record of the load-settlement curve, but only give a single measured settlement observation at a single level of loading. Often, the loading is estimated and may not actually be known, particularly in the case of building foundations. Stroud (1989) found that the case histories presented by Burland and Burbidge (1985) represented working foundations generally having high factors of safety with q_{net}/q_{ult} less than about 0.1 which corresponds to a factor of safety of 10. Actual footing load tests and large plate tests with breadths of 1m to 3m were taken to higher degrees of loading, giving q_{net}/q_{ult} in the range of 0.1 to 0.4 corresponding to factors of safety of 10 to 2.5, respectively.

5.2.20 Berardi et al. (1991)

A review of the case histories and site information compiled by Burland and Burbidge (1985) was made by Berardi and Lancellotta (1991) who presented and alternative method for estimating settlement of footings on granular soils. Their approach was based on evaluating the operational soil stiffness for a more-or-less continuous mechanics approach. In this review, an elastic approach was taken, similar to that presented in Section 4 where settlement is calculated from:

$$s = [(q'_n)\ (B)/E']\ I_s \qquad [5.79]$$

where:

s = settlement (in meters)
q'_n = net footing stress (in MPa)
B = footing width (in meters)
E' = soil stiffness
I_s = factor which accounts for geometry, rigidity, Poisson's ratio, μ, and the depth of the compressible layer

While it has been demonstrated (Burland and Burbidge, 1985) that the depth of influence, H, is dependent on the variation of soil stiffness with depth and the foundation geometry, it was assumed that $H/B = 1$ for evaluating the factor I_s. Suggested values of I_s for this method are presented in Table 5.4 assuming $\mu = 0.15$.

The soil stiffness is evaluated using the corrected SPT blowcounts,

$$N_1 = C_n N_{av} \quad [5.80]$$

where the correction factor, C_n, is the effective overburden correction of Skempton (1986) as:

$$C_n = 2/(1 + \sigma'_{vo}) \quad [5.81]$$

where:

σ'_{vo} = effective vertical stress at a depth equal to B/2 (in kg/cm^2)

Table 5.4 Values of I_s for Berardi and Lancellota (1991) Method

H/B	L/B					
	1	**2**	**3**	**5**	**10**	**Circle**
1.0	0.56	0.65	0.67	0.68	0.71	0.52

The average blowcount value, N_{av}, is obtained over the depth of influence, again, assumed to be equal to the width of the foundation. The stiffness is evaluated from:

$$E' = K_E P_a [(\sigma'_{vo} + \Delta\sigma'_v/2)/P_a]^{0.5} \quad [5.82]$$

where:

K_E = modulus number
σ'_{vo} and $\Delta\sigma'_v$ = initial in situ vertical effective stress and change in vertical effective stress at a depth below the footing equal to B/2 (in kPa)
P_a = reference stress = 100 kPa

The modulus number, K_E, corresponds to a relative settlement (S/B) of 0.1% and is obtained from the corrected blowcount, N_1, from Figure 5.16. To account for the reduction of soil stiffness with strain leve, the modulus number K_E should be reduced according to Figure 5.17.

Berardi et al. (1991) illustrate an iterative procedure for calculating settlements using this procedure as shown by the chart presented in Figure 5.18. Essentially if the relative settlement calculated assuming S/B = 0.1% is within this value then the calculations are completed. If the relative settlement exceeds 0.1%, a new value of K_E is chosen which corresponds to the relative settlement and the calculations are repeated until there is reasonable convergence.

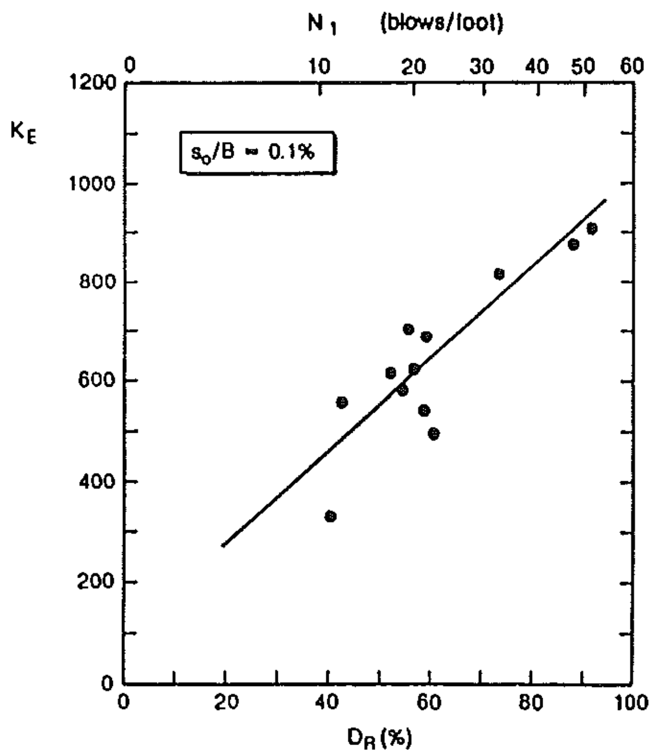


Figure 5.16 Berardi et al. (1991) Modulus Number.

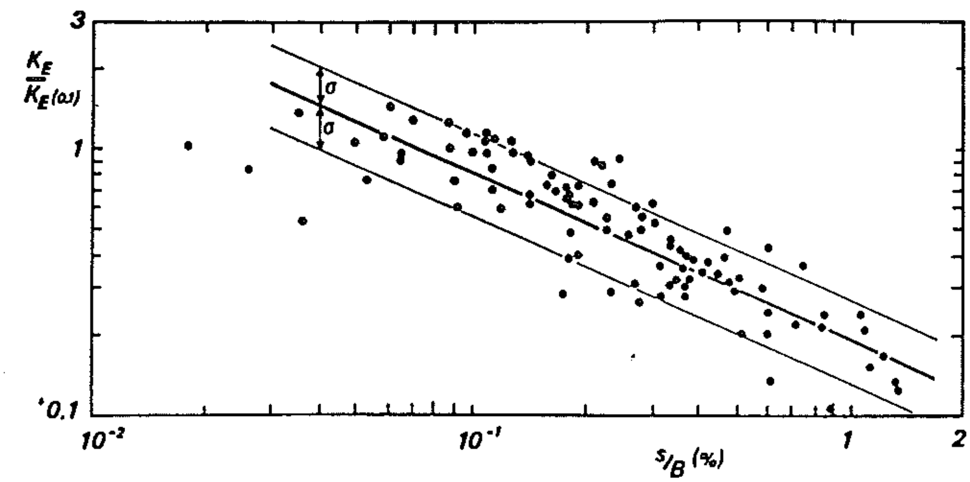


Figure 5.17 Berardi et al. (1991) Modulus Reduction.

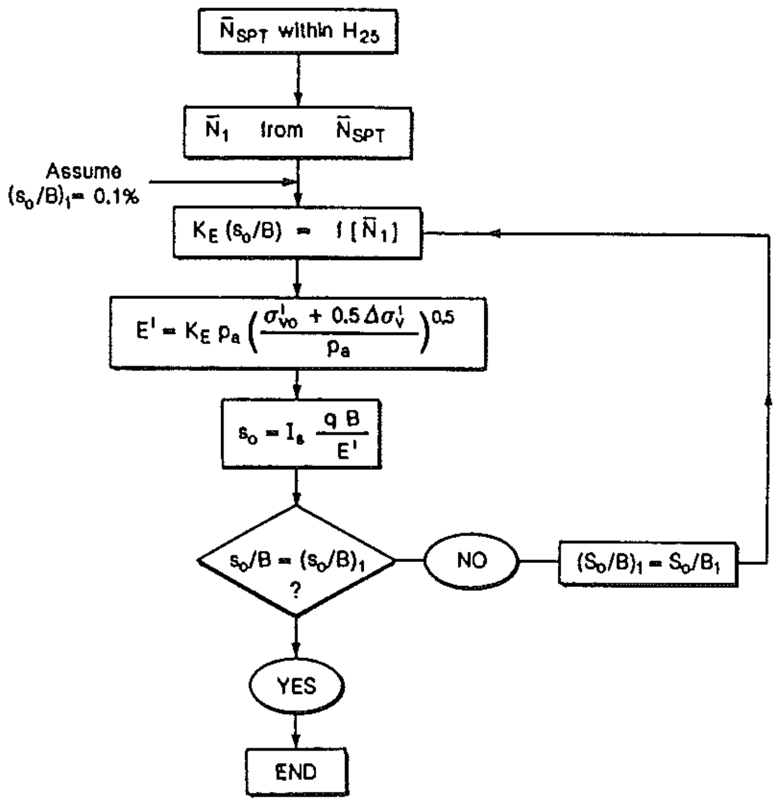


Figure 5.18 Berardi et al. (1991) Settlement Chart.

5.2.21 Anagnostopoulos et al. (1991)

Based on a statistical evaluation of measured settlements and multiple regression analyses, Anagnostopoulos et al. (1991) suggested grouping settlement estimates according to stiffness, e.g., loose, medium or dense sand as well as small vs large footings.

Settlements are calculated as:

$$s = [0.57\,(q)^{0.94}\,(B)^{0.90}]/N^{0.87} \quad \text{for zero} < N \leq 10 \qquad [5.83]$$
$$s = [0.35\,(q)^{1.01}\,(B)^{0.69}]/N^{0.94} \quad \text{for } 10 < N \leq 30 \qquad [5.84]$$
$$s = [604\,(q)^{0.90}\,(B)^{0.76}]/N^{2.82} \quad \text{for } N > 30 \qquad [5.85]$$
$$s = [1.90\,(q)^{0.77}\,(B)^{0.45}]/N^{1.08} \quad \text{for } B \leq 3 \text{ m} \qquad [5.86]$$
$$s = [1.64\,(q)^{1.02}\,(B)^{0.59}]/N^{1.37} \quad \text{for } B > 3 \text{ m} \qquad [5.87]$$

where:

s = settlement (in mm)
q = footing stress (in kPa)
B = footing width (in meters)
N = uncorrected blowcount

The SPT N value should be taken as the mean value in the zone of influence which is given as approximately equal to a zone of depth B below the foundation level, i.e., between D and D + B. No other correction factors are recommended.

The case histories used in the development of settlement expressions were obtained primarily from Schultze and Sherif (1973) and Burland and Burbidge (1985). Presumably one would calculate settlement by using the appropriate expression for both the SPT blowcount range and the appropriate footing width, and then average the two results to give a single settlement estimate.

5.3 Cone Penetration Test

The cone penetration test (CPT) has also been used extensively in the past to estimate settlement of shallow foundations on granular soils. Most of the approaches based on the results of the CPT rely on the tip resistance values obtained from the test. A description of the mechanics of the CPT is presented in Appendix B along with a brief discussion regarding the differences between mechanical and electrical cones.

In this section, several methods for estimating settlement using CPT results which have been presented in the literature are described. The methods include:

1. DeBeer and Martens (1957)

2. Meyerhof (1956,1965,1974)
3. DeBeer (1965)
4. Thomas (1968)
5. Schmertmann (1970)
6. Berardi et al. (1991)
7. Robertson (1991)

5.3.1 DeBeer and Martens (1957)

The method presented by DeBeer and Martens (1957) and also described by DeBeer (1965) is based on the semi-empirical Terzaghi-Buisman formula for calculating settlements:

$$s = (2.3/C) \log [(p'_o + \Delta p')/p'_o] H \qquad [5.88]$$

where:

s = settlement
C = constant of compressibility
p'_o = effective overburden stress
$\Delta p'$ = increment of stress at depth due to the footing stress
H = thickness of the layer

Settlement from individual soil layers can be calculated and then summarized to give the total settlement.

The constant of compressibility, C, is obtained from the CPT tip resistance, q_c, as:

$$C = 1.5\, q_c/p'_o \qquad [5.89]$$

The method was intended to provide a "safe upper limit of settlements" and compared with the settlements of several bridge abutments and piers generally gave estimates of about 2 times the observed settlement.

The value of $\Delta p'$ is obtained from Bousinesq stress distribution charts. The maximum depth of the zone of influence may be taken as the depth below the foundation to which the change in vertical effective stress equals 10% of the applied surface stress.

If the cone tip resistance, q_c, is constant with depth, it is suggested to use the Boussinesq equation to determine $\Delta p'$ as:

$$\Delta p' = (3q \cos^5\theta)/ 2\pi z^2 \qquad [5.90]$$

If q_c increases with depth, use the Buisman equation to determine $\Delta p'$ as:

$$\Delta p' = (2P \cos^6\theta)/\pi z^2 \qquad [5.91]$$

Additionally, it should be noted that this method is only applicable to normally consolidated sands.

5.3.2 Meyerhof (1956, 1965, 1974)

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

$$s = (q\,B)/(2q_c) \qquad [5.92]$$

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 of about 7.6 to 84 mm (0.3 to 3.3 in).

5.3.3 DeBeer (1965)

In 1965, DeBeer proposed a settlement analysis analogous to one dimensional settlement analysis for fine-grained soils not significantly different to the previous method presented by DeBeer and Martens (1957). For a soil deposit subdivided into N layers, the settlement may be calculated as:

$$s = \sum_{i=1}^{N} 1.535\,(\sigma'_{voi}/q_{ci})\,\log\left[(\sigma'_{voi} + \Delta\sigma'_v)/\sigma'_{voi}\right]\Delta h_i \qquad [5.93]$$

where:

s = settlement (in meters)
σ'_{voi} = initial effective stress in the ith layer (in kPa)
$\Delta\sigma'_v$ = increase in effective stress (in kPa)
q_{ci} = cone tip resistance (in kPa)
Δh_i = layer thickness (in meters)

5.3.4 Thomas (1968)

Thomas (1968) suggested a method based essentially on the calculation of consolidation settlements using an expression identical to that of DeBeer and Martens (1957). However, whereas the previous suggestion of DeBeer and Martens (1957) had been to define the constant of compressibility C as:

$$C = 1.5\ q_c/p'_o \qquad [5.94]$$

which can also be used to express the elastic modulus of the soil as:

$$E = 1.5\ q_c \qquad [5.95]$$

Thomas found that for a normally consolidated sand, the elastic modulus was related to cone tip resistance as $3q_c < E < 12q_c$, with the lower coefficient corresponding to high cone tip resistance and grain crushing.

The elastic expression of Section 4 was used to calculate settlements as:

$$s = IqB\ (1 - \mu^2)/E \qquad [5.96]$$

where:

s = settlement (in ft.)
I = an influence factor which depends on footing geometry (L/B)
q = applied footing stress (in tsf)
B = footing width (in ft.)
E = Elastic modulus (in tsf)
μ = Poisson's ratio

The influence factor, I, incorporating correction for embeddment was obtained using the expressions proposed by Fox (1948; see Figure 4.7). The elastic parameter $E/(1 - \mu^2)$ was obtained using the cone tip resistance values over a depth equal to 1.5 times the footing width and was taken as a function of q_c as shown in Figure 5.19.

Thomas (1968) found that the ratio of estimated to observed settlement, defined as R_s, was related to the level of loading, defined as a percentage of the ultimate bearing capacity as q/(B) (q_c) as shown in Figure 5.20. Schmertmann (1969) pointed out that this method tends to seriously underestimate settlement.

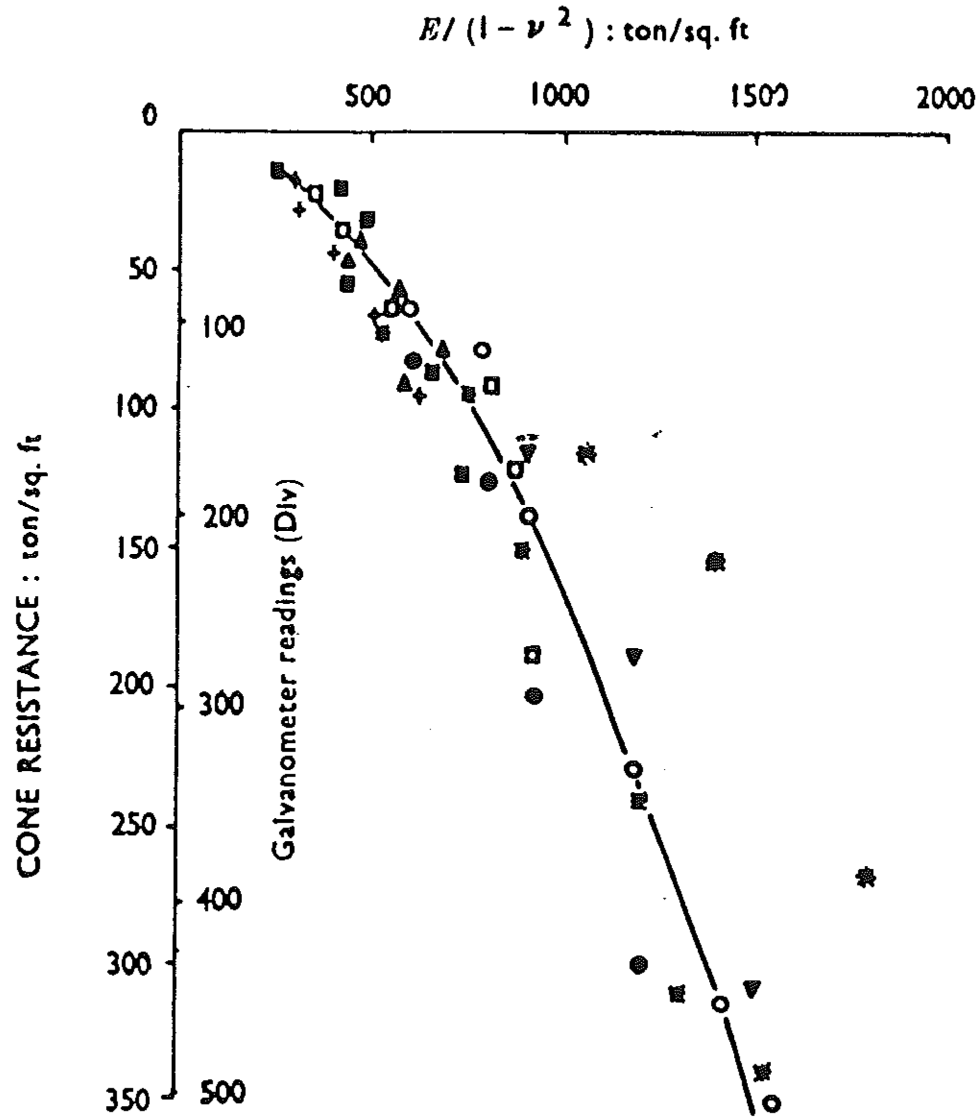


Figure 5.19 Thomas (1968) Elastic Modulus from CPT.

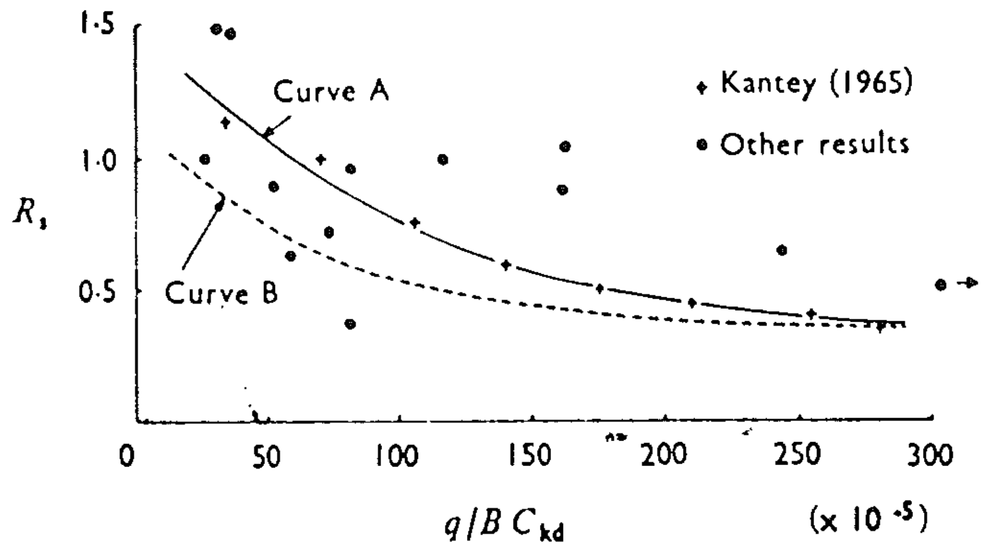


Figure 5.20 Settlement Ratio as a Function of Load Level (Thomas 1968).

5.3.5 Schmertmann (1970)

In what may now be considered as one of the most classical papers of applied soil mechanics, 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 referred to as the "2B - 0.6" method which described the approximate strain influence diagram proposed by Schmertmann to calculate settlements over a zone of influence equal to 2B below the footing.

As a result of later studies of the vertical strain distribution, Schmertmann et al. (1978) suggested subtle modifications to the strain influence diagram to account for differences in footing shape and load intensity. As presented herein, this method is referred to as the "Schmertmann (1970)" method but uses the improved strain influence diagrams presented by Schmertmann et al. (1978). Settlement is calculated from the expression:

$$s = C_1 C_2 q \sum_{i=1}^{n} (I_z/E_s) z_i \qquad [5.97]$$

where:

s = settlement (in ft.)
q = footing stress (tsf)
I_z = strain influence factor
E_s = soil modulus (tsf)
i = individual layer
n = total number of soil layers
z_i = thickness of individual layer (in ft.)
C_2 = depth correction factor

$$= 1.0 - 0.5\,(\gamma D/q) \geq 0.5 \qquad [5.98]$$

where:

γ = soil weight (in pcf)
D = depth of embedment
C_1 = creep correction factor

$$= 1.0 + 0.2 \log (t/0.1) \qquad [5.99]$$

where:

t = time in years

In order to obtain the strain influence factor, I_z, at the midpoint of each soil layer, it is necessary to construct the strain influence diagram. Harr (1966) has suggested other strain influence factors based on a probabalistic soil theory which are highly dependent on the in situ lateral stresses at rest and are therefore dependent upon K_o.

To construct the strain influence diagram for a particular case, the following approach is used:

<u>For axisymmetric footings (square and round)</u>

$I_z = 0.1$ at depth = 0
$I_z = 0$ at depth = 2B

Maximum I_z occurs at a depth of B/2 and has a value of:

$I_z = 0.5 + 0.1\ [\Delta q/\sigma'_{vp}]^{0.5}$

<u>For plane strain footings (L/B > 10)</u>

$I_z = 0.2$ at depth = 0
$I_z = 0$ at depth = 4B

Maximum I_z occurs at a depth of B and has a value of:

$I_z = 0.5 + 0.1\ [\Delta q/\sigma'_{vp}]^{0.5}$

where:

Δq = net applied footing stress
σ'_{vp} = initial vertical effective stress at maximum I_z for each loading case (i.e., B/2 for axisymmetric and B for plane strain.

Figure 5.21 presents a comparison of the original and modified strain influence diagrams.

The method is performed using the following simple steps:

1.- Divide the subsurface soil into sublayers (usually on the basis of differences).
2.- Obtain E_s in each sublayer.
3.- Obtain I_z at the midpoint of each sublayer.

4.- Use Equation 5.95 to calculate the total settlement.
5.- Apply correction factors C_1 and C_2 as appropriate.

The method of estimating settlement proposed by Schmertmann (1970) is primarily intended for use with cone penetration test data. The CPT has the obvious advantage of providing a near continuous record of penetration resistance, especially if an electric CPT is used and thus provides a larger data base for delineating individual important sublayers within the compressible zone as will as allowing statistical averaging of data within a layer.

Schmertmann (1970) suggested that based on screwplate tests, the soil modulus could be evaluated from:

$$E_s = 2q_c \qquad [5.100]$$

However, in their modification, Schmertmann et al. (1978) noted that $E_s = 2.5\ q_c$ would also have been a reasonable choice to evaluate the soil modulus and recommended:

$$E_s = 2.5\ q_c \text{ (for axisymmetric cases)} \qquad [5.101]$$
$$E_s = 3.5\ q_c \text{ (for plain strain cases)} \qquad [5.102]$$

Normally, the CPT is more efficient to conduct than the SPT, however, it suffers from the disadvantage of not providing a soil sample for visual classification. On most projects, an exploration program which combines the use of the SPT and the CPT would be desirable.

Schmertmann (1970) suggested that if only SPT results were available to the engineer, settlement predictions could still be made using his proposed method by converting SPT blowcount values to CPT cone tip resistance values using the q_c/N ratio. Schmertmann (1970) recommended that provisionally, the following q_c/N ratios could be used to convert N values to q_c:

Soil Type	q_c/N
silts, sandy silts, slightly cohesive silt-sand mixtures	2.0
clean, fine to medium sands and slightly silty sands	3.5
coarse sands & sand with little gravel	5
sandy gravels and gravel	8

Earlier, Sutherland (1963) had compiled a series of comparisons between q_c and N and found the following results:

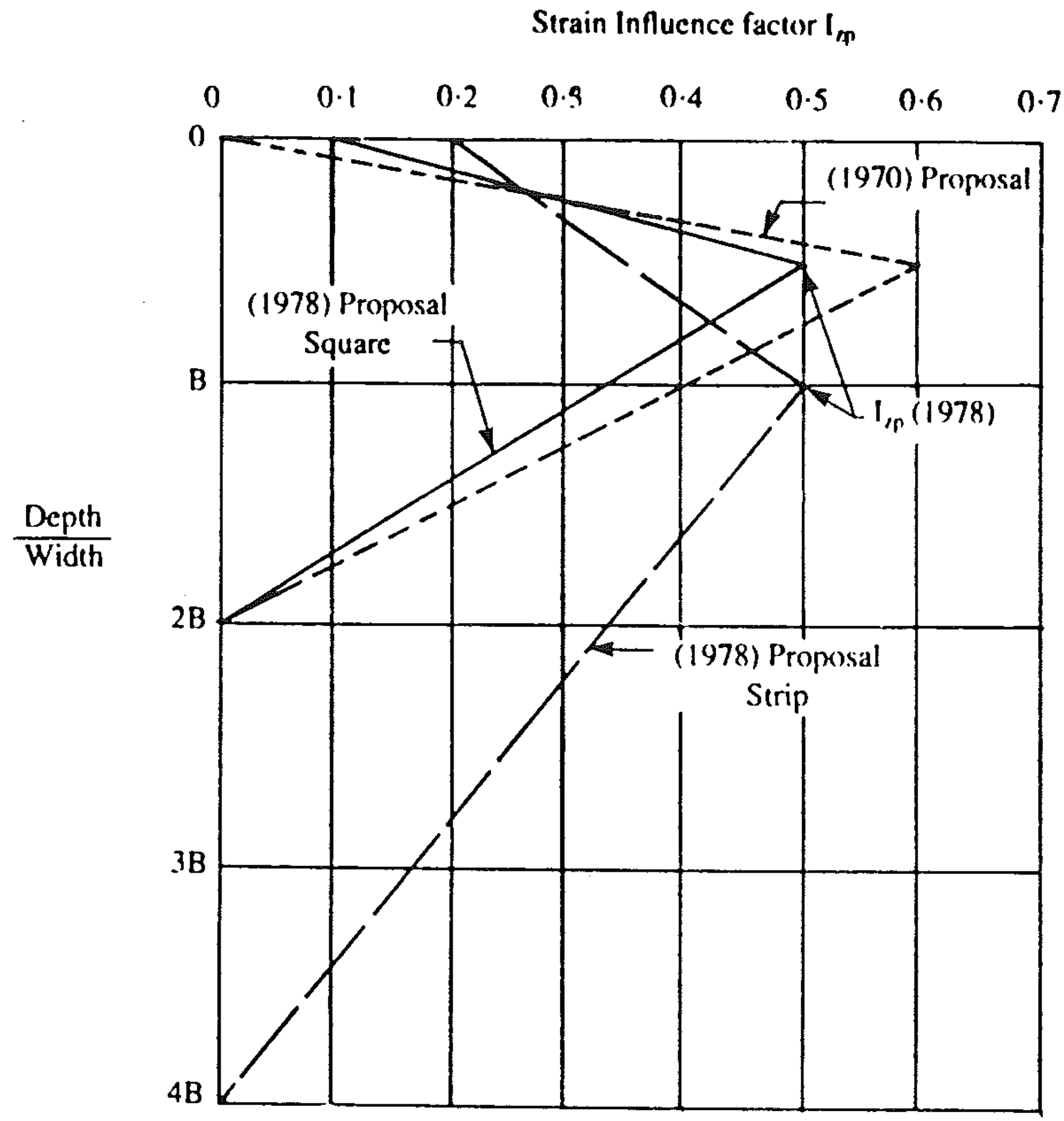


Figure 5.21 Schmertmann (1970) and Schmertmann et al. (1978) Strain Influence Factors.

Soil Type	q_c/N
sandy silt	2.5
sand & gravelly sand	3.6
fine sand and silty fine sand	4
fine to medium sand	4.8
sand with some gravel	8
medium and coarse sand	8
fine to medium sand	10
sand	10
gravelly sand	8-18
sandy gravel	12-16

In an attempt to quantify the q_c/N ratio in relation to the grain-size of the soil, Robertson et al. (1983) presented the chart shown in Figure 5.22 which correlates q_c/N values with mean grain-size, D_{50}, in mm, An alternative approach was presented by Muromachi and Kobayashi (1982) who showed that q_c/N could also be related to the amount of fines (% < #200 sieve) in predominantly granular soils as shown in Figure 5.23.

It is important to note that in order to make proper use of this settlement method with SPT blowcounts, a sufficiently large number of tests are needed within the zone of foundation influence. For this reason, it is suggested that drillers be instructed to perform continuous SPT's within a minimum depth of 2B below the proposed foundation level.

5.3.6 Berardi et al. (1991)

Berardi et al. (1991) suggested that an elastic approach could also be used along with the results of the CPT to estimate settlements using Equation 5.79. The values of the influence factor, I_s, is given in Table 5.5 and are based on a Poisson's ratio of 0.15. The operational soil stiffness, E'_s, for use in Equation 5.79 is obtained from results relating stiffness to CPT tip resistance, q_c, where the ratio of E'_s/q_c is a function of the normalized tip resistance and is different for different soil stress history. One such chart, as given by Berardi et al. (1991) was developed by Baldi et al. (1989) and is shown in Figure 5.24. In this figure, the modulus is defined at a vertical strain level of 0.1% which Berardi et al. (1991) suggest corresponds to the upper limit of the average strain of practical interest for foundations placed on normally consolidated sands and overconsolidated sands which may include structural fills compacted under well controlled conditions. For recent normally consolidated deposits, it may be desirable to reduce the stiffness to account for higher strain levels. In this case, a modulus reduction factor, varying from 0.5 to 0.75 may be appropriate depending on the anticipated strain level (up to about 0.5%).

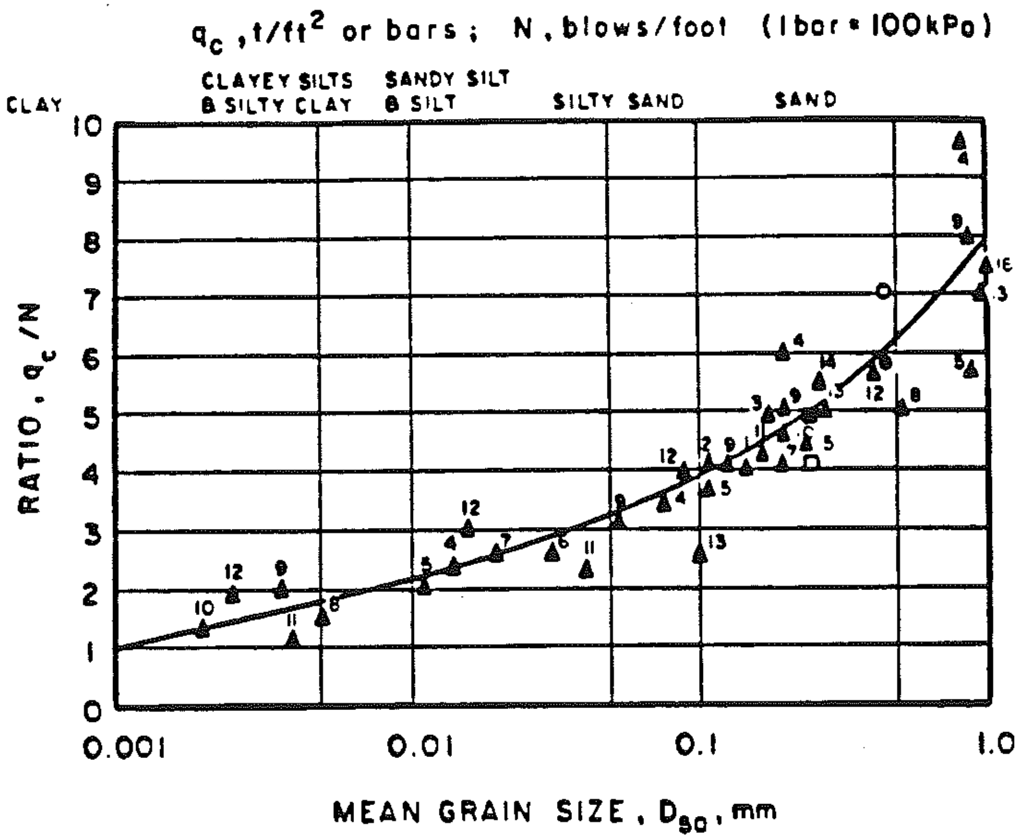


Figure 5.22 q_c/N vs. D_{50} (Roberston et al 1983).

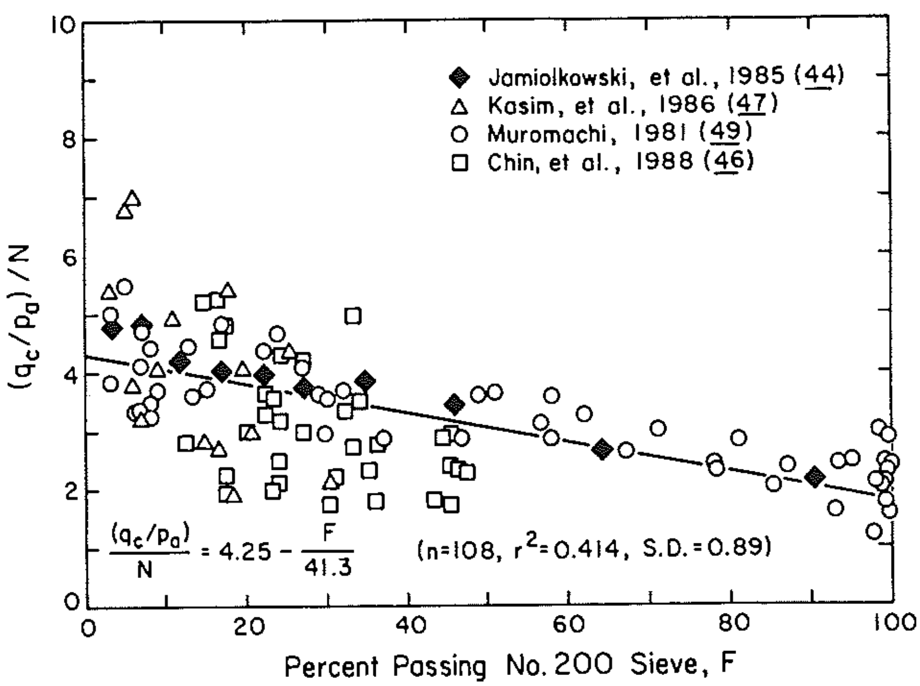


Figure 5.23 q_c/N vs % Fines (Muromachi and Kobayashi 1982).

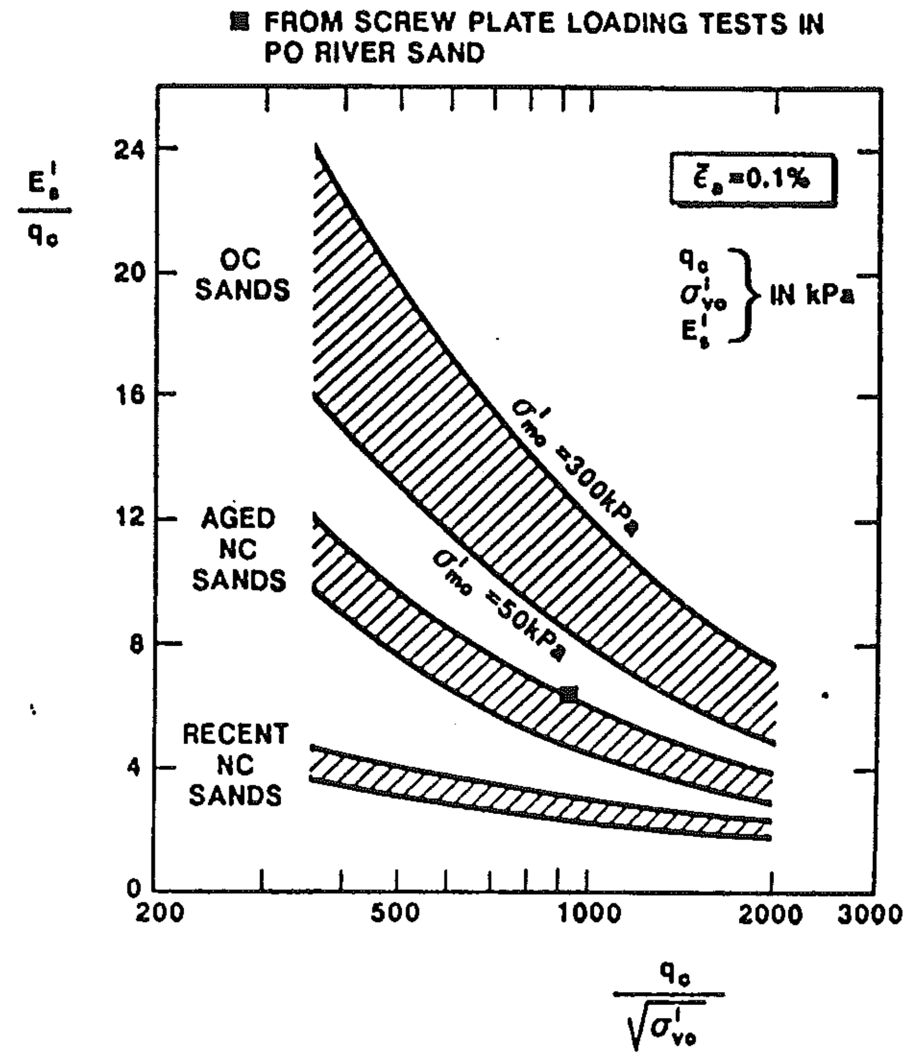


Figure 5.24 Evaluation of Drained Youngs Modulus from CPT.

Table 5.5 Influence Factors for Berardi et al. (1991)

H/B	L/B = 1	2	3	4	10	Circle
0.5	0.35	0.39	0.40	0.41	0.42	0.34
1.0	0.56	0.65	0.67	0.68	0.71	0.52
1.5	0.63	0.76	0.81	0.84	0.89	0.59
2.0	0.69	0.88	0.96	0.99	1.06	0.63

5.3.7 Robertson (1991)

Robertson (1991) suggested a method based on cone tip resistance to evaluate soil elastic modulus which is similar to the method presented by Stroud (1989) for SPT blowcounts. The operative soil modulus is related to the degree of loading of the foundation, relative to the ultimate calculated bearing capacity. Based on the theory of elasticity, settlement may be calculated as:

$$s = [I_s q_{net} B]\,[(1-\mu^2)/E'] \qquad [5.103]$$

where:

s = settlement
B = foundation width
q_{net} = net applied footing stress
μ = Poisson's ratio
I_s = influence factor which depends on footing geometry and rigidity and type of elasticitymodel
E' = drained Young's modulus

While Robertson (1991) does not give details for evaluating I_s or μ, it may be assumed that appropriate I_s values may be obtained from the charts presented by Christian and Carrier (1978) and that Poisson's ratio equals 0.25.

In order to obtain the drained Young's modulus, E', the normalized cone tip resistance, q_{ci} must be evaluated as:

$$q_{ci} = (q_c/P_a)(P_a/\sigma'_{vo})^{0.5} \qquad [5.104]$$

where:

P_a = atmospheric pressure
σ'_{vo} = vertical effective stress where q_c is measured

Since the correlation between q_c and E' is a function of the degree of loading it is necessary to

necessary to evaluate q_{ult}. Robertson (1991) suggested using bearing capacity factors for local shear presented by Lambe and Whitman (1969) and that ϕ be estimated using the correlations between q_{ci} and ϕ proposed by Robertson and Campanella (1983). Once the ratio q_{net}/q_{ult} is obtained, this value, along with the normalized cone resistance, q_{ci}, are used to estimate the E'/q_c ratio as shown in Figure 5.25 which is applicable for overconsolidated sand. Interpolation between the two curves shown is required. It may be necessary to reduce to E'/q_c ratio obtained by some factor when applied to normally consolidated sands. Robertson suggests:

1. reduce E' by a factor of 2 for aged NC silica sands
2. reduce E' by a factor of 3 for recent (<1,000 yrs) NC silica sands

5.4 Pressuremeter Test

Settlement predictions for shallow foundations made using the results of pressuremeter tests primarily rely on the evaluation of the pressuremeter modulus, E_m. A number of cases have been described in the literature on the successful use of the PMT for this purpose. A summary of reported cases for various soil condition is presented in Table 5.6. This section of the report describe a number of different methods which are currently available. The following methods are described:

1. Menard and Rousseau (1962)
2. Martin (1977, 1987)
3. Baguelin et al. (1978)
4. Briaud (1991)

5.4.1 Menard and Rousseau (1962)

Menard and Rousseau (1962) suggested that in a uniform homogeneous soil, a reliable estimate of settlement could be obtained from the PMT and would be composed of two components, one arising from the deviatoric strain tensor and one from the spherical strain tensor. They suggested the following semi-empirical expression:

$$s=\frac{2}{9E_m}q^*B_o\,[\lambda_d\frac{B}{B_o}]^{\alpha}+\frac{\alpha}{9E_m}q^*\lambda_c B \qquad [5.105]$$

which can be written as:

$$s=\frac{q^*}{9E_m}[2B_o(\lambda_d\frac{B}{B_o})^{\alpha} + \alpha\lambda_c B] \qquad [5.106]$$

where:

s = settlement

Table 5.6 Reported Use of Pressuremeter For Settlement Predictions of Shallow Foundations

Foundation	Soil Type	Reference
misc. cases	misc.	Calhoun (1969)
unknown	Till, sand & gravel, rock	Eisenstein & Morrison (1973)
unknown	gravel	Burgess & Eisenstein (1977)
misc. cases	misc.	Baguelin et al. (1978)
2.8m x 14.0m 3.3m x 14.5m	sand sand	Wennerstrand (1979)
400m x 70m	silty & clayey sand	Withiam & Christiano (1981)
9.5m x 10.1m 5m x 8.5m	sand & gravel sand	Bergdahl & Ottosson (1982)
unknown	residual	Barksdale et al. (1986)
4.3m x 4.3m 2.7m x 2.7m 4.0m x 4.0m	glacial till landfill, rubble fill sandy, silt	Lukas (1985)
3.5m x 6.9m	residual	Borden et al. (1988)
unknown	sand & gravel	Kummerle & Dumas (1988)
1.1m x 1.3m 2.3m x 2.5m 0.6m x 0.55m 1.6m x 2.0m unknown 20m x 50m	dense loose sand mixes dense sand, silt, sandy clay	Hansbo & Pramborg (1990)
1m x 1m 1.5m x 1.5m 2.5m x 2.5m 3m x 3m	sand	Briaud & Gibbens (1994)

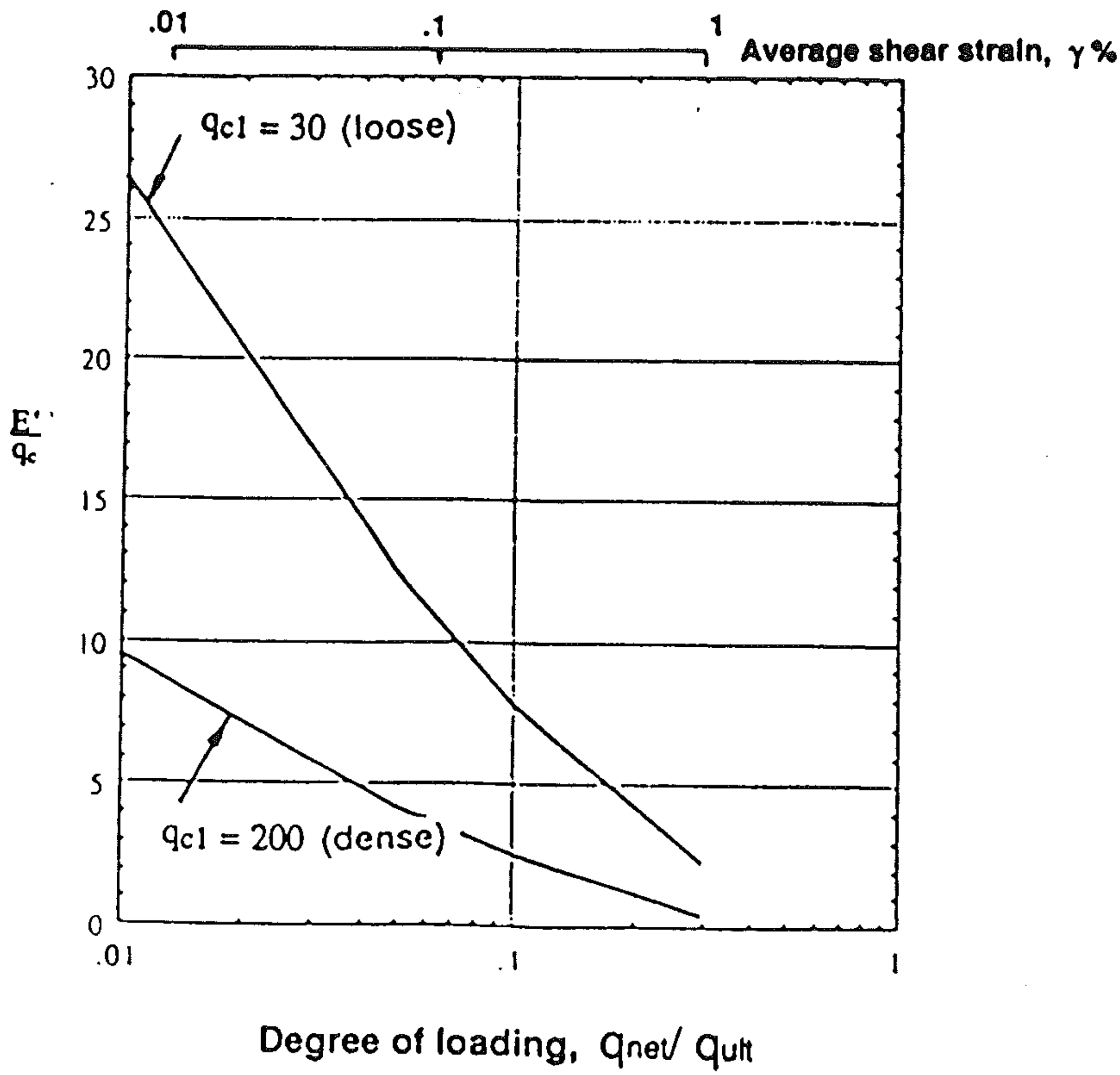


Figure 5.25 Estimate of E_s from q_c from Robertson (1991).

q^* = net footing pressure
B = footing width
B_o = reference width = 60 cm (2 ft)
E_m = PMT modulus (initial)
$\lambda_{c,d}$ = shape factors which are a function of foundation geometry
α = geological factor which is related to soil type and stress history

This method is also presented and discussed by Baguelin et al. (1978) and Briaud (1992). Values for $\lambda_{c,d}$ and α may be obtained from Figure 5.26 and Table 5.7 respectively. Note that values of α may also be chosen based on the ratio E_m/P^*_L, where P^*_L is equal to the net limit pressure obtained from the test; i.e., P_L-P_o.

Table 5.7 Rheological Factors for PMT

Soil Type	Peat		Clay		Silt		Sand		Sand & Gravel	
	E/p^*_L	α	E/p^*_L	α	E/p^*_L	α	E/p^*_L	α	E/p^*_L	α
Over-consolidated			>16	1	<14	2/3	>12	1/2	>10	1/3
Normally Consolidated	For all values	1	9-16	2/3	8-14	1/2	7-12	1/3	6-10	1/4
Weathered and/or remoulded			7-9	1/2		1/2		/13		1/4
Rock	Extremely Fractured $\alpha = 1/3$			Other $\alpha = 1/2$			Slightly fractured or extremely weathered $\alpha = 2/3$			

In Equation 5.105 the first term represents settlement resulting from the shear deformation arising from the deviatoric strain tensor, while the second term represents settlement resulting from the volumetric deformation arising from the spherical strain tensor. The deviatoric strain is significant down to a depth of at least 2B below the footing. The spherical strain decreases rapidly with increasing depth below the footing such that the volumetric settlement is only important at shallow depths, on the order of B/2 below the footing. For a wide foundation over a thin layer the volumetric settlement will dominate, whereas for footings on deep relatively uniform deposits, the shear settlement will dominate.

L/B	Circle	1 Square	2	3	5	20
λ_d	1	1.12	1.53	1.78	2.14	2.65
λ_c	1	1.10	1.20	1.30	1.40	1.50

Figure 5.26 Shape Factors for PMT.

In order to recognize the strain arising from these two components, it is recommended that Equation 5.105 be modified as:

$$s=\frac{2}{9E_d}q^{*}B_o\left[\lambda_d\frac{B}{B_o}\right]^{\alpha}+\frac{\alpha}{9E_c}q^{*}\lambda_c B \qquad [5.107]$$

where:

E_d = PMT modulus within the zone of the deviatoric strain tensor
E_c = PMT modulus within the zone of the spherical tensor

In most soil deposits, the value of the PMT modulus may vary with depth in which case equivalent values of PMT modulus, i.e., E^*_d and E^*_c must be obtained. In order to do this the soil below the footing is divided into a series of individual layers each with a thickness equal to B/2 down to a depth of 8B.

The value of E_c is taken from the first layer as E_1. E_d is taken as the equivalent modulus within the 16 layers (each of thickness B/2) under the footing as:

$$\frac{1}{E_d}=\frac{1}{4}\left(\frac{1}{E_1}+\frac{1}{0.85E_2}+\frac{1}{E_{3/4/5}}+\frac{1}{2.5E_{6/7/8}}+\frac{1}{2.5E_{9/16}}\right) \qquad [5.108]$$

where:

$E_{p/q}$ = the harmonic mean of the moduli of layers p to q

This procedure is illustrated in Figure 5.27. In cases where the PMT data are not available in individual layers, which will often be the case, the value of E_i for any individual layer must be estimated based on other data or site information. If a rigid boundary is encountered within a depth of 2B below the footing, this technique may lead to substantial errors.

5.4.2 Martin (1977, 1987)

Martin (1977, 1987) suggested a method to predict foundation settlements in piedomont residual soils using the PMT modulus and strain influence factor method suggested by Schmertmann (1970) for sands. Menard (1965) developed a rheological factor, α, to E_s as:

$$E_s=\alpha E_m \qquad [5.109]$$

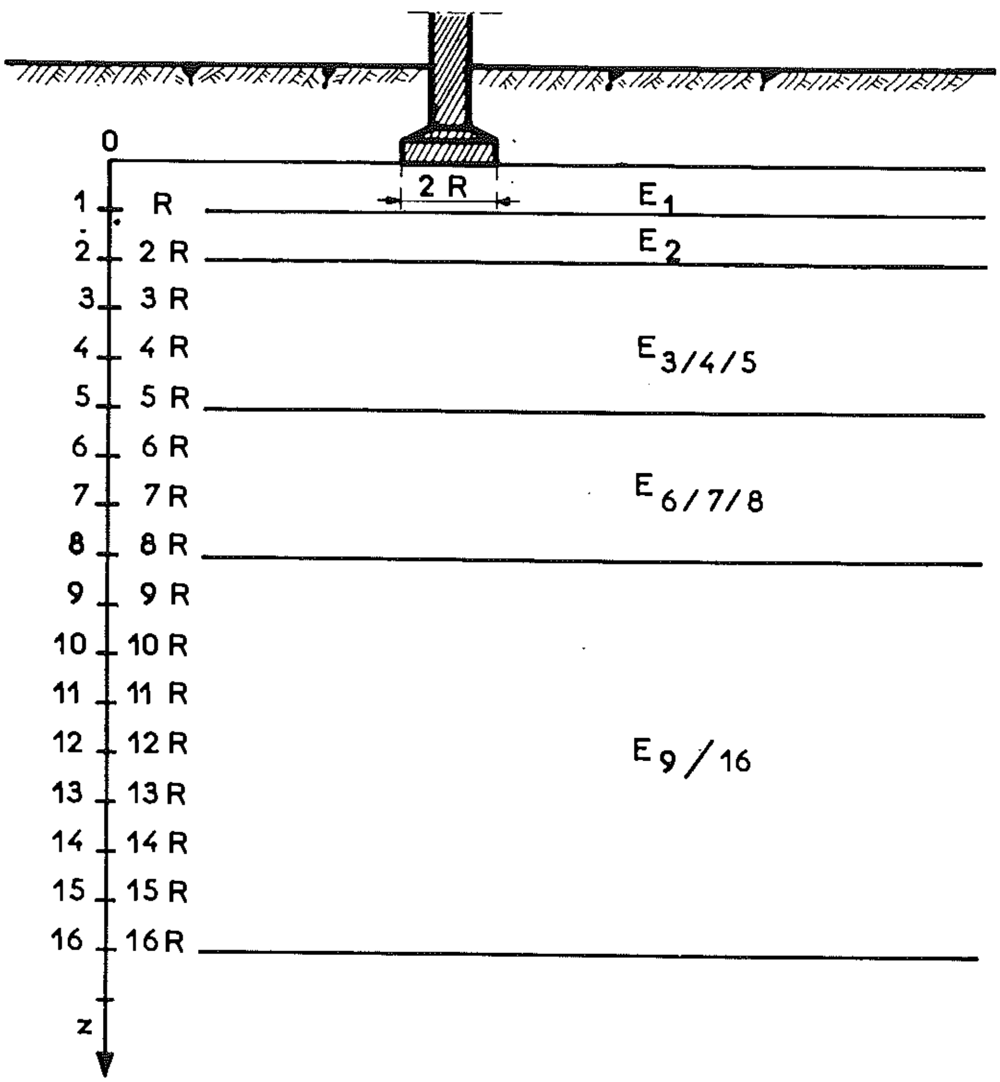


Figure 5.27 Evaluation of Harmonic Mean Modulus.

For different soils Menard suggested different values of α as: (1) sand & gravel, 1/3; (2) silt, 1/2; (3) clay. 2/3; (4) peat, 1. Based on the correlation exhibited between measured and predicted settlements, Martin (1977) found that the PMT modulus, E_m, closely approximates the soil modulus, E_s, i.e., $\alpha = 1$, for the soils investigated when using Schmertmann's 2B-0.6 strain influence factors.

5.4.3 Baguelin et al. (1978)

Baguelin et al. (1978) present a simplified approach to estimating foundation settlements for preliminary design as:

$$s = [q_{net}/E_m]f \qquad [5.110]$$

where:

s = settlement
q_{net} = net footing pressure = $q - \gamma D = q^*$
γ = soil total unit weight
D = foundation depth
f = an empirical coefficient which is a function of soil type and footing geometry
E_m = PMT modulus within a depth of 2B below the footing

The value of E_m should be adjusted to give the weighted average of values obtained in the zone 2B below the footing if the soil is not uniform. Values of f may be obtained from Figure 5.28 and will have units of length as shown. Both E_m and q_{net} must have the same units.

5.4.4 Briaud (1992)

An alternative approach which is presented by Baguelin et al. (1978) and Briaud (1992) is to use a simple elastic solution to calculate settlement as:

$$s = I_o I_1 (1 - \upsilon^2)\, q(B/E) \qquad [5.111]$$

where:

s = settlement
I_o, I_1 = influence factors
υ = Poisson's ratio
q = bearing pressure
B = footing width
E = PMT modulus within the zone of influence

This approach is essentially the same as other elastic methods presented, with the exception that the soil modulus is obtained directly from the PMT.

Soil Type	Peat		Clay		Silt		Sand		Sand and gravel	
	E_M/p_l^*	α	E_M/p_l^*	α	E_M/p_l^*	α	E_M/p_l^*	α	E_M/p_l^*	α
Over-consolidated			>16	1	>14	2/3	>12	1/2	>10	1/3
Normally consolidated		1	9–16	2/3	8–14	1/2	7–12	1/3	6–10	1/4
Weathered and/or remoulded			7–9	1/2		1/2		1/3		1/4

Rock	Extremely fractured	Other	Slightly fractured or extremely weathered
	$\alpha = 1/3$	$\alpha = 1/2$	$\alpha = 2/3$

Figure 5.28 Values of factor f for Baguelin et al. (1978).

5.5 Dilatometer Test

Marchetti (1980) suggested that the Dilatometer Modulus, E_D, could be used to estimate the one-dimensional constrained modulus, M_2 and provided an empirical approach to estimating settlement. This was also shown by Schmertmann (1981) to be a reasonable approach as evaluating soil stiffness.

The DMT has proven especially useful for rapid and economical preliminary estimates of settlements of shallow foundations and has been used in a number of different soils as summarized in Table 5.8. Each DMT produces a predicted modulus value at a particular point in the foundation soil, and at the particular effective stress condition existing at that point at that time. The engineer can calculate this condition by computing total vertical overburden pressure and the equilibrium water pressure at each point. All soil modulus values are effective stress dependent to varying degrees on vertical and horizontal pre-stress magnitudes, cementation, etc. In some problems it may be advisable to adjust the DMT-determined modulus values to better match the vertical stress changes imposed by the structure involved during its construction and service life as outlined by Schmertmann (1986).

Table 5.8 Reported Use of Dilatometer For Settlement Predictions of Shallow Foundations

Foundation	Soil Type	Reference
misc.	misc.	Hayes (1986)
misc.	misc.	Schmertmann (1986)
misc.	misc.	Saye & Lutenegger (1988)
misc.	residual	Mayne & Frost (1991)
0.9m x 0.9m	Sand	Skiles & Townsend (1994)

5.5.1 Schmertmann (1986)

Schmertmann (1986) recommended a procedure to estimate footing settlements based on the work by Janbu (1963, 1985) in which it was pointed out that among other things, using a simple modulus concept could significantly simplify the understanding and calculation of consolidation settlements. Janbu suggested using the following equation of calculate consolidation settlement:

$$s = \Sigma(\Delta\sigma_v'/M_i h_i) \qquad [5.112]$$

where:

s = settlement
h_i = the thickness of each "i" soil layer or sublayer being considered, $\Delta h_i = H$
i = the ith layer in a total of n layers (total thickness of H)
M_i = the applicable vertical 1-dimension modulus of compressibility in the "i" the soil layer
(= vertical stress increase/vertical strain)
(= vertical stress increase/volume strain)
$\Delta\sigma_v'$ = effective stress increase at the mid-height of each "i" layer that produces the settlement

The first term in brackets in Equation 5.112 computes the vertical strain, which is then multipled by the layer thickness to obtain the consolidation settlement for that layer, and all such layers are then summarized to give the total settlement.

The step-by-step procedure presented by Schmertmann is presented below.

Method A - Ordinary Method

1. Perform a DMT sounding at each settlement analysis location and determine profiles of M through the soil layers of interest.

2. Divide the compressible soils into layers and/or sublayers of similar soil type and stiffness.

3. Determine the average M value from the DMT results for each layer and sublayer in 2.

4. Calculate the vertical stress increase $\Delta\sigma_v$ at the mid-height of each layer and sublayer in 2 using any suitable method to calculate the vertical stress increase.

5. Calculate the 1-D settlement of each later or sublayer using the following equation:

   Settlement = (stress increase x thickness)/(modulus)
   = $\Delta\sigma_v H/M$

6. Obtain the total 1-D settlement by adding all the contributions from the layers and sublayers in step 5.

7. Make corrections to the settlement calculated in step 6, as appropriate and using DMT experience with similar soils and loadings.

Method B - Special Method

This method includes the extra steps 4.1 to 4.5, the sole purpose of which is to adjust M to the average vertical effective stress during the loading that produces the settlement of interest.

SPECIAL METHOD ADDITIONS TO STEP 4

4.1. Calculate the initial effective overburden stress σ_o at the mid-height of each layer and sublayer in step 1.

4.2 Determine the average p_c' and σ_d' value from the DMT results for each layer and sublayer in step 2.

4.3 Compare σ_D' vs. σ_o' (the effective overburden pressure at the time of the structure loading may not be the same as at the time of the DMT because of excavation, surcharge, dewatering, etc.)

4.4 Compare p_c' and (σ_o' + $\Delta\sigma_v$) and decide on which of the following cases applies to each layer or sublayer.

a. All virgin compression: use M for the normally consolidated (NC) case.

b. All recompression: use M for the OC case.

c. The stress increase spans part recompression and part virgin compression: use M from step 4.5 below.

4.5 Make adjustments to the average M values in step 3, as needed.

Schmertmann reported the results of 16 comparisons using this technique and field observations. The range in the ratio of DMT predicted/measured was 2.2 to 0.7 with an overall mean of about 1.1.

Corrections to the settlement computation to account for three-dimensional effects, secondary compression, aging, structural rigidity, etc. still should be applied as the engineer feels necessary.

5.5.2 Elastic Approach

Engineers may also wish to make a separate estimate of the "immediate" or "elastic"settlement, particularly in the case of cohesionless soils. Commonly used elastic settlementcomputation methods are all based on the theory of elasticity, and are given as:

$$s = CqB/E^* \qquad [5.113]$$

where:

s = settlement
q = net unit load increase over foundation area
B = least width of foundation loaded area (short side of rectangle of length L, diameter of circle)
C = "shape factor" accounting for shape of loaded area, depth of embedment, depth to rigid boundary below loaded area, point considered on loaded area (center, corner, edge), and rigidity of loaded area (usually perfectly flexible or rigid)
$E^* = E/(\mu^2)$; where E = Young's modulus and μ = undrained Poisson's ratio

While the DMT provides a "modulus" value (E_D) it is, however, obtained following disturbance from penetration. Therefore, adjustments to determine a more appropriate modulus maybe necessary. For example, Robertson et al. (1989) proposed multipication factors to obtain the initial tangent modulus, E_i from E_D and recommend using 2 for sands.

5.5.3 Leonards and Frost (1988)

A modification to the Schmertmann (1970) method for predicting settlement of shallow foundations on granular soils using the results of the CPT was presented by Leonards and Frost (1988) for use with the DMT. This method acknowledges the effects of overconsolidation on reducing the compressibility of soil and suggests evaluating the preconsolidation stress so that settlements in both the reload and virgin loading range may be evaluated. The step-by-step procedure as presented by Leonards and Frost (1988) is as follows:

1. Perform DMT and CPT soundings at appropriate locations through soil layers of interest.

2. Divide the soil profile into layers with similar characteristics for settlement calculations.

3. Determine the average q_c/σ'_{vo} ratio and K_D value for each layer.

4. Determine K(OC) according to:

   $$K(OC) = 0.376 + 0.095\ K_D - 0.0017(q_c/\sigma'_{vo})$$

5. Using the chart prepared by Marchetti (1985) from the Durgunoglu and Mitchell (1975) equations, determine ϕ_{ps}.

6. Calculate the value of ϕ_{ax} from:

   $$\phi_{ax} = \phi_{ps} - [(\phi_{ps} - 32°)/3]$$

7. Determine OCR from:

$OCR = [K(OC)/(1-\sin\phi)]^{(1/0.8\sin\phi)}$

8. Calculate the initial vertical effective stress at the center of the layer.

9. Determine the preconsolidation pressure at the test elevation as:

$p'_c = \sigma'_{vo}OCR$

10. Determine the stress increment at the center of the layer due to the applied load (assume 2:1)

11. Determine the final stress at the center of the layer as:

$\sigma'_f = \sigma'_{vo} + \Delta\sigma'$

12. Determine that portion of the load increment that will be in the OC range $R_z(OC)$ and in the NC range $R_z(NC)$ as:

$R_z(OC) = (P'_c - \sigma'_{vo})/(\sigma'_f - \sigma'_{vo})$
$R_z(NC) = (\sigma'_f - P'_c)/(\sigma'_f - \sigma'_{vo})$

13. Determine the average E_D value for the layer.

14. Determine the strain influence factor, I_z, for the layer from Schmertmann's B(2:2B approximation).

15. Calculate the settlement for the layer from:

$$s = c_1 q_{net} \sum_0^D I_z \Delta_z \left[\frac{R_z(OC)}{E_z(OC)} + \frac{R_z(NC)}{E_z(NC)} \right] \qquad [5.114]$$

5.6 Plate Load Test

The use of the plate load test has in the past been an attractive approach to predicting the settlement of shallow footings on granular soils, largely because the plate acts as a prototype foundation and load is applied in the same direction as anticipated by the foundation. In order for the results of a plate load test to be useful in predicting settlements the test must be performed on soil which is representative of that to be stressed by the foundation, which means that the surface where the plate test is to be performed must be undisturbed and that the soil throughout the zone of influence of the plate and the foundation is the same. However, since the stiffness of granular soils is related to the stress level in the ground (i.e. modulus is stress dependent) errors maybe associated with evaluating stiffness from small plate tests (typically 0.3m) and then extrapolating the results to larger footings.

5.6.1 Terzaghi and Peck (1948, 1967)

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

$$(\delta_B)/(\delta_1) = (2B/(B + 1))^2 \qquad [5.115]$$

where:

B = footing width (ft.)

For very large footings on the order of B > 8m the ratio tends to a maximum value of about 4 as shown in Figure 5.29.

Bjerrum and Eggestad (1963) demonstrated that there could be considerable scatter in the settlement ratio observed from different cases and that settlement ratios much larger than 4 could occur. They suggested the settlement ratio was dependent on density with loose sands giving higher settlement ratios and dense sands giving lower settlement ratios. A comparison between the Terzaghi and Peck curve and curves presented by Bjerrum and Eggestad (1963) is shown in Figure 5.30. It has also been suggested (Meigh 1963) that the settlement ratio may also be dependent on gradation of the soil with coarse, well graded soils having low settlement ratios and fine, uniformly graded soils having high settlement ratios.

Arnold (1980) suggested that the relationship presented by Terzaghi and Peck (1967) and given in Equation 5.115 be modified as:

$$s_B/s_1 = (2B/(B^{\lambda} + 1))^2 \qquad [5.115a]$$

where:

$$\lambda = 0.788 + 0.002\ D_r \qquad [5.115b]$$

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%.

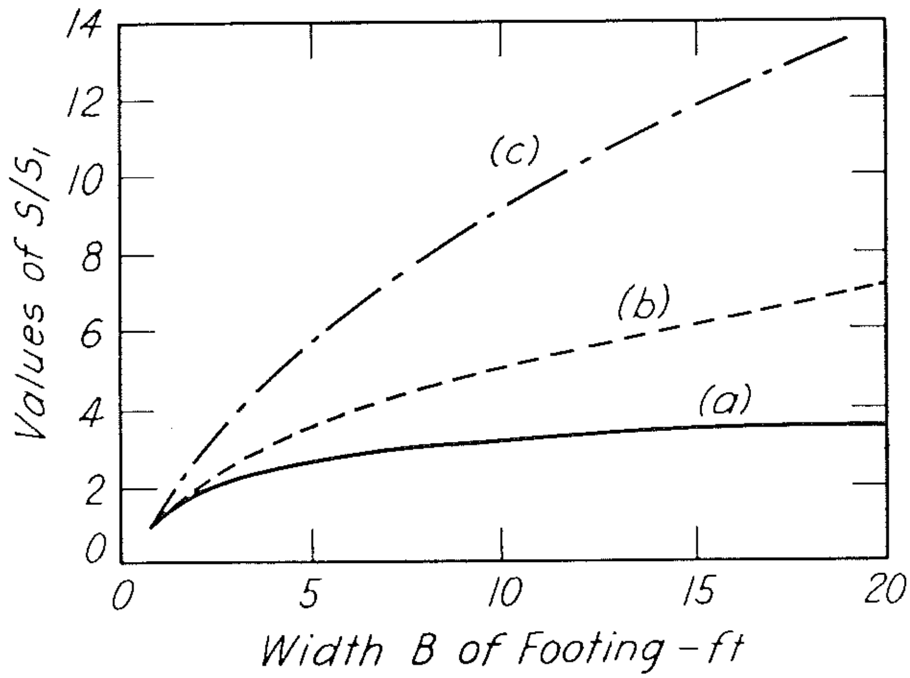


Figure 5.29 Settlement Ratio as a function of Footing Width.

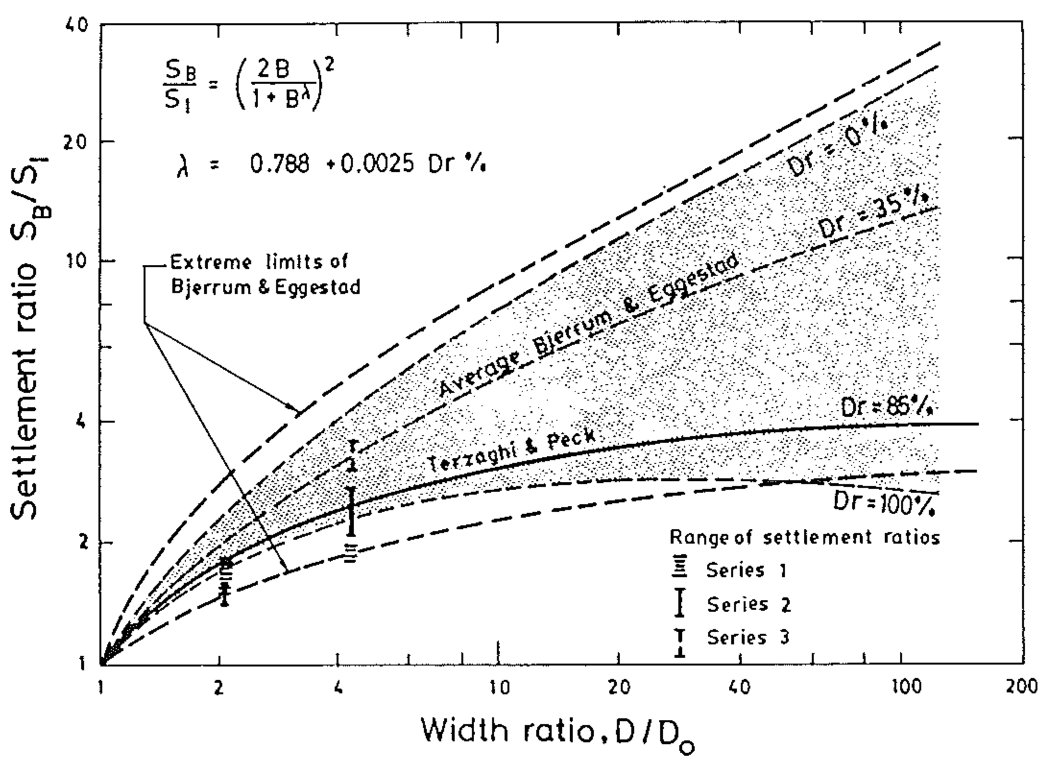


Figure 5.30 Comparison Between Terzaghi and Peck and Bjerrum and Eggestad Curves.

5.6.2 Barata (1973)

Barata also pointed out that the expression for settlement of plates on granular soils was severely limited in application and that in many cases the results obtained from this extrapolation would be in error. It was suggested that an expression based on the concepts suggested by Housel (1929) and Burmister (1947) would be of more general applicability.

According to Housel (1929) the settlement of a square plate with side B = 2b on the surface of the ground is given as:

$$s = n_o + m_o (P/A) \qquad [5.116]$$

where:

s = settlement
n_o & m_o = characteristic coefficients of the ground
P = perimeter of the plate
A = area of the plate

Burmister (1947) adopted the theory of elasticity to Equation 5.116 to obtain expressions for n_o and m_o as:

$$n_o = (C^s)/C_\delta(1-\mu^2) \qquad [5.117]$$
$$m_o = (E_o^s)/4C(1-\mu) \qquad [5.118]$$

where:

E_o = modulus of deformation at the ground surface
C = rate of modulus increase with depth (i.e. $E_z = E_o + C_z$)
μ = Poison's ratio
C_δ = coefficient dependent on the shape and rigidity of the plate

Barata (1973) presented a series of charts for different values of C and for different plate sizes, taking into account the initial modulus value, E_o , and assuming Poisson's ratio equals 0.3. The general relationship between the settlement of a plate with width equal to 0.3m (1ft) and any size plate was thus expressed as:

$$s/s_o = (B/B_o) ((E_o + CB_o)/(E_o + CB)) \qquad [5.119]$$

which may be restated as:

$$s/s_o = (B/B_o)(\theta) \qquad [5.120]$$

where:

$$\theta = [(E_o/C)+B_o] / [(E_o/C) + B] \quad [5.121]$$

for C=0 (E constant with depth) $\theta = 1$;
for $E_o = 0$ and $C>0$, (linearly increasing E with depth beginning at E = 0), $\theta = B_o/B$ and $S/S_o = 1$;
for $C<0$ (E decreases with depth, i.e. $E_z = E_o - C_z$), $\theta = B/B_o$.

5.6.3 Carrier and Christian (1973)

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

$$E = E_o + K \quad [5.122]$$

where:

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

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

The results were presented by considering the elastic settlement ratio as a function of foundation width, similar to what had previously been presented by Terzaghi and Peck (1948, 1967) and Bjerrum and Eggestad (1963). As shown in Figure 5.31, solutions were shown for various ratios of E_o/K ranging from 0 to ∞. These results show that the settlement ratio of footings on a non-homogenous half-space increases linearly with the logarithm of footing width.

This means that if the results of plate bearing tests are available for a particular site (preferably for B = 0.3m and 0.6 to 1m) an appropriate value of E_o/K may be obtained and then the settlement of production footings may be estimated based on extrapolation.

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

5.6.4 Parry (1978)

Parry (1978) recognized that the use of extrapolation formulas, such as that proposed by Terzaghi and Peck (1948, 1967) and others, which take no account of soil conditions may lead to errors in settlement estimates. Parry (1978) suggested that the results obtained from plate bearing tests could be used in conjunction with results of the SPT to extrapolate settlements as:

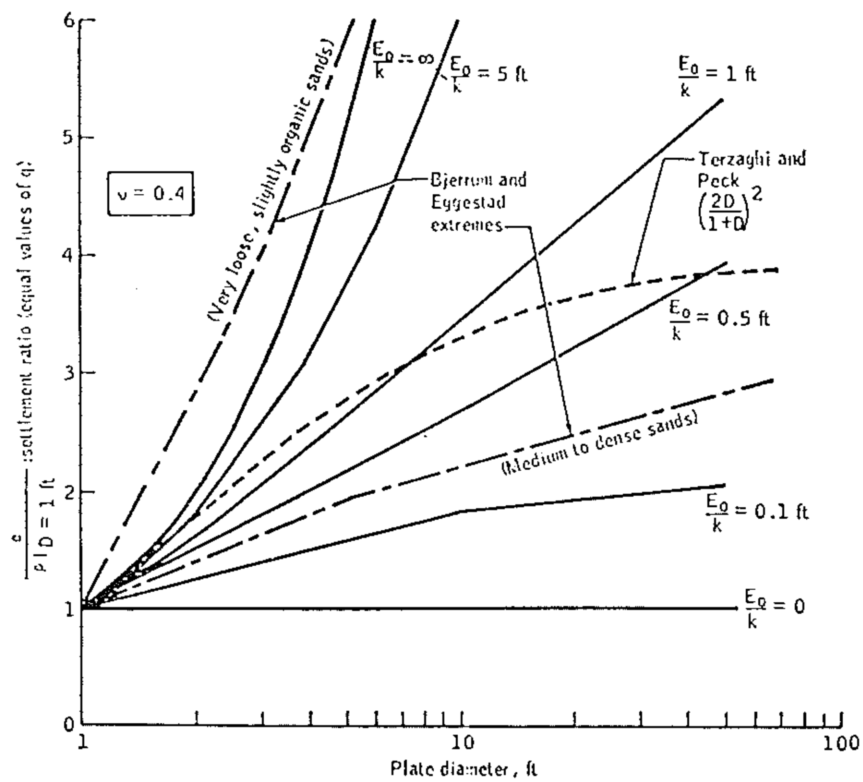


Figure 5.31 Settlement Ratio Curves Presented by Carrier and Christian (1973).

$$P_B = P_1 (B/B_1)((N_m)_1 /(N_m)_B) \quad [5.123]$$

where:

P_B = settlement of a foundation of width B
P_1 = settlement of plate of width B_1
$(N_m)_B = N_m$ for the foundation

Note that any size plate may be used in this technique.

The value of N_m is obtained as:

$$N_m = 1/6 (3N_1 + 2N_2 + N_3) \quad [5.124]$$

where:

N_1 = N from the base of the foundation to a depth of 2/3 B
N_2 = N from 2/3B to 4/3B
N_3 = N from 4/3B to 2B

Where N shows a consistent trend over the depth 2B below the foundation, N_m may be taken as the N value at a depth of 3/4B below the foundation.

5.6.5 Ghionna et al. (1991)

A technique which uses the results of plate load tests in conjunction with conventional linear elastic theory and accounting for the dependence of soil stiffness on stress and strain was presented by Ghionna et al. (1991). The application of elastic theory to plate tests allows for the evaluation of Young's modulus according to the expression suggested by Oweiss (1979) as:

$$E = 0.71q(s/B) \quad [5.125]$$

where:

q = average applied stress
s = settlement
B = footing width

The results of plate tests show that resulting values of Young's modulus, evaluated using Equation 5.125, are nonlinear and decrease with increasing values of q. Soil deformation parameters are obtained by assuming a hyperbolic stress-strain model for soils and assuming that settlements or soil strains occur over a zone below the foundation related to foundation geometry; i.e., 2B for square

and circular foundation; 4B for strip footings (L/B>10) and intermediate values for intermediate cases.

Settlement is determined from:

$$s = [1/K_i][(qBI(1-\mu_2))/(\sigma'_{oct})^n_{av} - (qBI(1-\mu^2))/(\sigma'_{oct})^{1-n}_{av}C_fH_i] \quad [5.126]$$

where:

q = applied footing stress
I = influence fator to account for footing rigidity
$(\sigma'_{oct})_{av} = [(1 + 2K_o)/3]\sigma'_{vo} + \alpha q$
K_o = at-rest coefficient of earth pressure
σ'_{vo} = effective vertical overburden stress
α = ratio between $\Delta\sigma'_{oct}$ induced by q at depth z; and q (determined from linear elastic theory)
K_i & C_f = constants determined from the plate test
H_i = zone of influence for the foundation

The constants K_i and C_f are determined from a plot of normalized load-settlement results from the plate load test as shown in Figure 5.32. These transformed hyperbolic data give a linear curve with slope of $1/C_f$ and intercept of $1/K_i$.

Numerous authors (e.g., Oweiss 1979; Ismael 1991; Ortigosa et al. 1989 Papadopoulos 1992) have compared the observed settlement of footing tests with the nondimensional width ratio vs. settlement ratio curves of Bjerrum and Eggestad (1963) and others, and have found wide variations in results.

5.6.6 Burland et al. (1977)

Another interesting method for comparing the results of plate load and footing load tests was presented by Burland et al. (1977) and is shown in Figure 5.33. As indicated, the settlement per unit applied pressure is seen to increase with footing size and is a function of the relative density. This figure may be used to make preliminary estimates of settlement.

5.7 Drive Cone Test

The use of an impact driven point in the form of a cone penetrometer has frequently been suggested as an expedient substitute for the Standard Penetration Test (e.g; Mohan et al., 1971). Farrent (1963) had suggested that the test could be used to predict the settlement of footings on granular soils. In this case, a cone tip of 50mm (2in.) diameter with an apex angle of 60° was used. Comparisons made at two sites indicated that the blowcount values obtained by the drive cone (over a distance of 0.3m (1ft.)) were equivalent to SPT blowcount values over the same zone. The obvious advantages of the drive cone are increased frequency of test data and expedience in deployment and execution of the test.

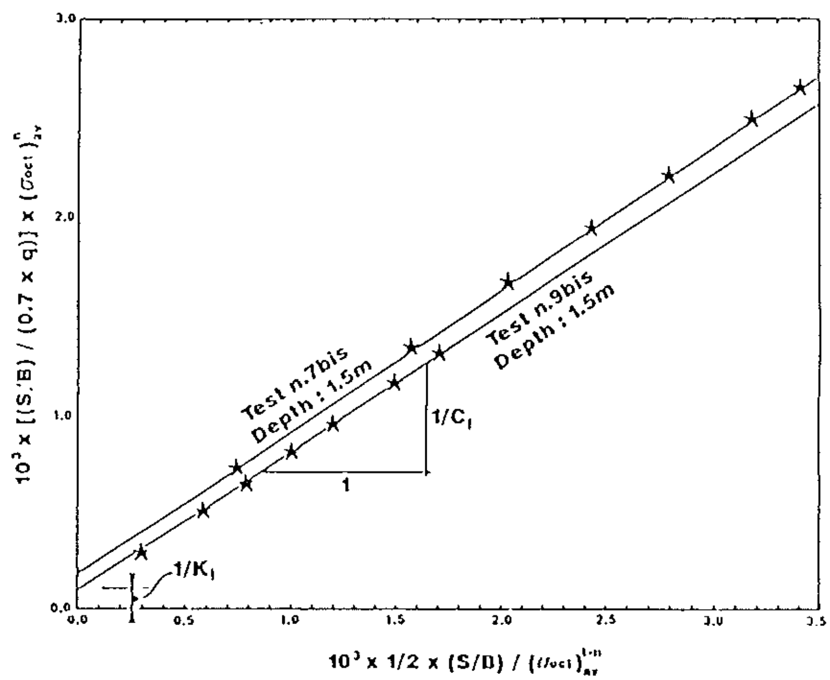


Figure 5.32 Evaluation of Constants K_i and C_f for Ghionna et al. (1991).

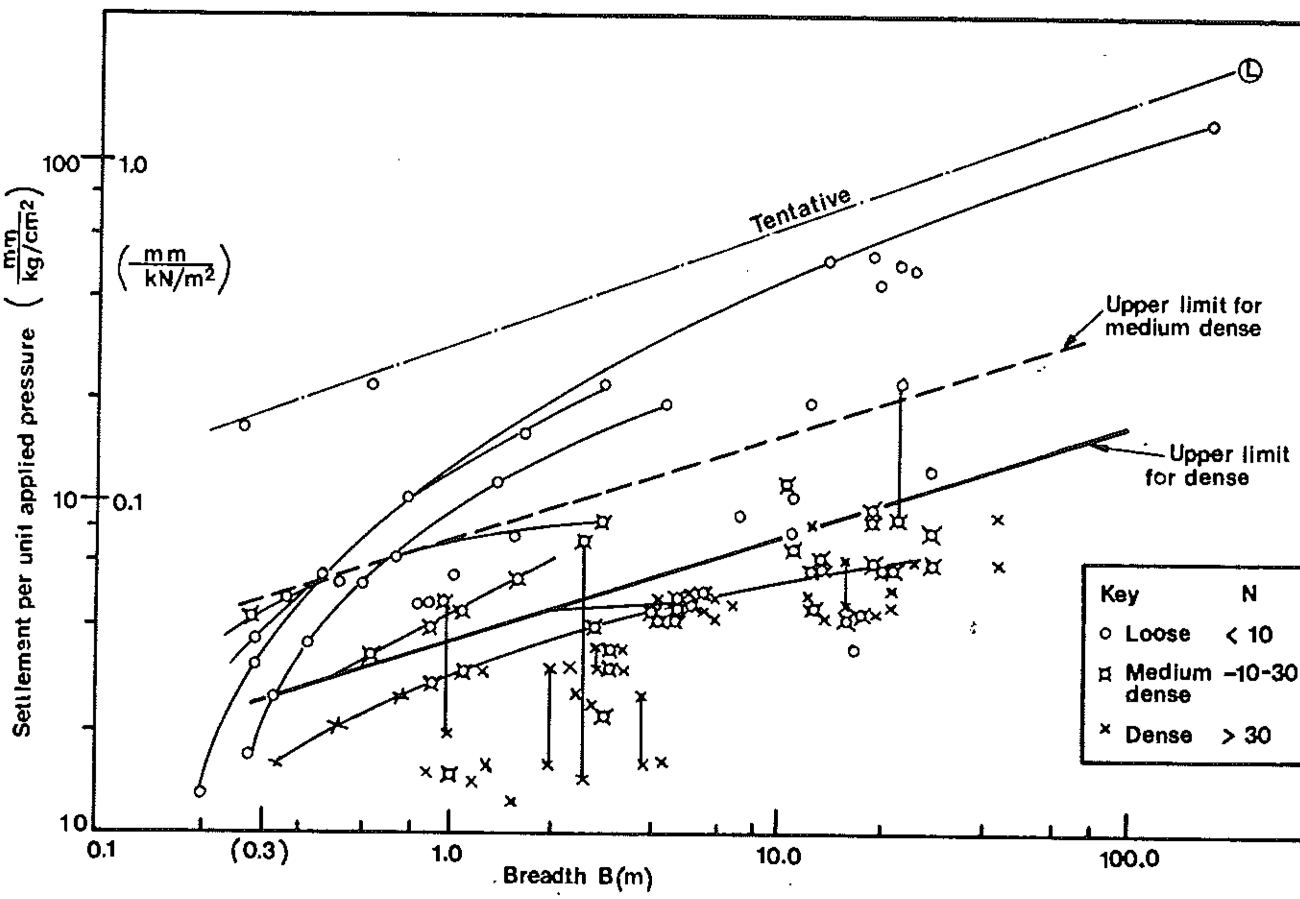


Figure 5.33 Settlement Ratio vs. Footing Width (from Burland et al. 1977).

The use of the drive cone test is well documented for sand deposits (e.g., Palmer and Stuart 1957, Mohan et al. 1970, and Muromachi and Kobayashi 1982) as well as in gravelly materials (e.g., Rao et al. 1982 and Hanna et al. 1986). Charts have been presented providing a comparison between SPT's and DCT's and in some cases allowable bearing capacity charts for shallow foundations on granular soils have been developed based on the results of DCT's (e.g., Mohan et al. 1971).

Farrent (1963) suggested the following approach could be used to calculate settlements based on the elastic approach presented by Terzaghi (1943):

$$s = KqB[(1 - \mu^2)/E] \qquad [5.127]$$

where:

s = settlement
K = a constant depending on the position where settlement is desired
q = applied footing stress
2B = width of the footing
μ = Poisson's ratio
E = Young's Modulus

It was further suggested that μ and E could be assumed constant provided that the applied stress did not exceed about 1/3 to 1/2 of the ultimate bearing capacity (with FS = 2 to 3). Based on back calculation of load-settlement curves presented by Terzaghi and Peck (1948) for different values of SPT penetration resistance, Farrent suggested:

$$E/(1-\mu^2) = 15{,}000N \text{ (psf)} \qquad [5.128]$$

where:

N = SPT or DCT blowcount.

5.8 Comparison of Methods

To illustrate results that can be obtained using various methods, settlement calculations were performed using the SPT and CPT results obtained at the site of the FHWA footing load tests at Texas A & M University and reported by Briaud and Gibbens (1994). Settlement estimates were made for a 3m x 3m footing (Test Footing #1) using the applied pressure corresponding to a settlement of 25mm (1in.) obtained from the actual load vs. settlement curve. The results of these estimates are presented in Tables 5.9 and 5.10. The resulls presented in both of these tables indicate a wide range of estimated settlement, from about 18 to 124mm, however, there are a number of methods that show very close agreement with the observed value.

Table 5.9 Comparison of Settlement Estimates (SPT Methods)

Method	Settlement (mm)
Terzaghi & Peck (1948, 1967)	69
Meyerhof (1956)	41
Meyerhof (1965)	37
Hough (1959)	66
Hough (1969)	107
Teng (1962)	22
Sutherland (1963)	28
Alpan (1964)	56
D'Appolonia et al. (1968)	19
Bowles (1968)	59
Peck & Bazaraa (1969)	29
Webb (1969)	124
D'Appolonia et al. (1970)	32 if assume NC
D'Appolonia et al. (1970)	18 if assume OC
Parry (1971)	34
Schultze & Sherif	25
Peck et al. (1974)	44
Meyerhof (1974)	38
Arnold (1980)	21
NAVFAC	37
Burland & Burbidge (1985)	32 if assume NC
Stroud (1989)	23
Berardi et al. (1991)	19
Anagnostopoulos et al. (1991)	23

Table 5.10 Comparison of Settlement Estimates (CPT Methods).

Method	Settlement (mm)
DeBeer & Martens (1957)	42
Meyerhof (1956,1965,1974)	125
DeBeer (1965)	42
Thomas (1968)	20
Schmertmann (1970)	90
Berardi et al. (1991)	20
Robertson (1991)	34

6.0 ALLOWABLE BEARING CAPACITY CHARTS

In the past 45 years, a number of suggestions have been made to directly relate the results obtained from in situ tests to the allowable bearing pressure of shallow foundations on granular soils. Most of these schemes have made use of the results of the Standard Penetration Test (SPT) and the Cone Penetration Test (CPT) and give the allowable bearing pressure in relation to the width of the footing. In general, the allowable pressure is intended to provide the footing pressure that will produce a footing settlement of 25 mm (1 in.) and also provide a sufficiently large factor of safety against a bearing capacity failure. These types of charts appear in a number of textbooks on soil mechanics and foundation engineering. In this section of the report, the historical development of allowable bearing capacity charts will be described and a comparison will be made of the various available charts. In some cases, these charts or the equations represent an alternative method for estimating settlements over the various methods presented in Section 5. In other cases, it will be noted that the method is essentially the same as previously given.

6.1 Terzaghi and Peck (1948, 1967)

Terzaghi and Peck (1948) were the first to suggest a direct relation between the SPT blowcounts and allowable soil pressure for footings on dry and moist sand. According to Terzaghi and Peck (1948) the chart, shown in Figure 6.1, "was prepared on the basis of present knowledge concerning the relation between the number of blows N per foot of penetration of the sampling spoon, the results of surface loading tests, and Equation 5.4.1.". In large part, the behavior of shallow footings was related to the relative density which was in turn related to the penetration resistance. The equation referred to was given by Terzaghi and Peck (1948) as:

$$S = S_1 \left[(2B)/(B+1)\right]^2 \qquad [6.1]$$

where:

S = settlement of a footing of width B (in inches)
S_1 = settlement of a footing 1 ft. wide subject to the same load per unit area (in inches)
B = footing width (in feet)

This expression was given as an approximation to a figure presented by Terzaghi and Peck (1948), shown in Figure 6.2, and credited by Terzaghi and Peck (1948) to "F. Kögler and others".

Note that the chart presented by Terzaghi and Peck (1948) only included three different curves for SPT blowcounts of 10, 30 and 50. No corrections were recommended for the blowcounts. It was recommended that "If N has a value other than those for which the curves are drawn, the allowable soil pressure is obtained by linear interpolation between curves". It was suggested that SPT's should be obtained between the base of the footings and a depth B every 0.8 m (2.5 ft.). Additionally, it was recommended that the lowest (average) value of N from any single test boring should be used for estimating allowable soil pressure.

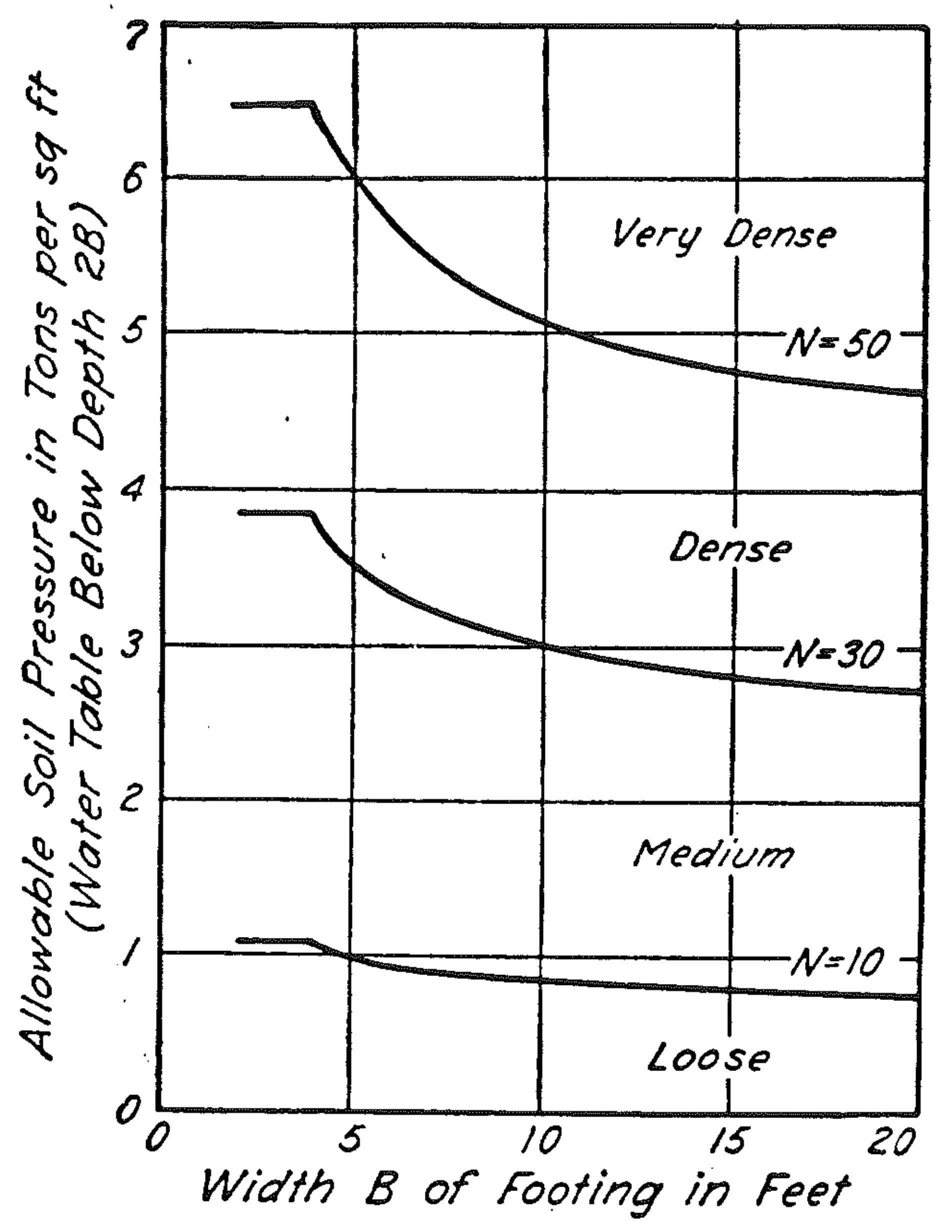


Figure 6.1 Terzaghi and Peck (1948) Allowable Soil Pressure Chart.

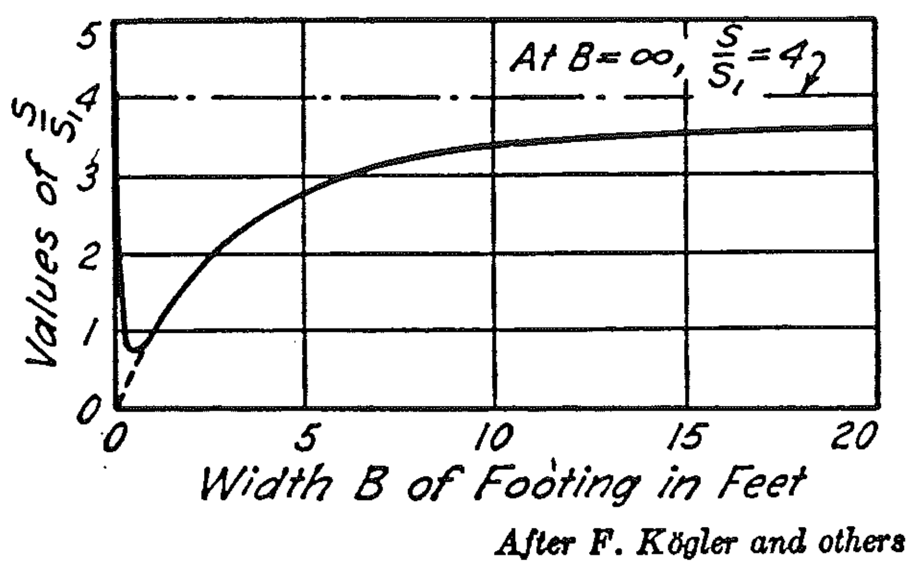


Figure 6.2 Relationship Between Settlement and Footing Size from Terzaghi and Peck (1948).

In saturated sand it was suggested that if the depth to width ratio, i.e., D_f/B, of the footing is small, the values obtained from Figure 6.1 should be reduced 50%. If D_f/B is close to 1, 66% of the values from Figure 6.1 can be tolerated; i.e., the values need only be reduced by 33%. The chart shown in Figure 6.1 is intended for conditions where the water table is located at a depth of at least 2B.

In the second edition of their book, Terzaghi and Peck (1967) presented essentially the same chart, which is shown in Figure 6.3. They did note however that the work of Bjerrum and Eggestad (1963) indicated that the ratio S/S_1 may increase more rapidly with increasing footing width if the sand is loose.

6.2 Taylor(1948)

In his textbook on soil mechanics, Taylor (1948) discussed the general considerations for determining allowable bearing capacity of footings on sands including size effects, density effects, and embedment. He suggested that the allowable soil pressure curve for a given soil would be composed of two parts as shown in Figure 6.4; and initial linear portion corresponding to the safe capacity (i.e., ultimate bearing capacity divided by a factor of safety, q_u/F); and a second linear portion corresponding to the maximum allowable settlement.

Taylor (1948) indicated that this second portion of the curve would show a decrease in allowable soil pressure with increasing width of the footing. This is because the coefficient of settlement, C_p, decreases slightly with increasing footing width. The allowable soil pressure based on allowable settlement is obtained from:

$$q_a = C_p \rho_a \qquad [6.2]$$

where:

q_a = allowable soil pressure (in tsf)
C_p = coefficient of settlement (in tsf/in.)
ρ_a = allowable settlement (in inches)

The coefficient of settlement is really then the slope of the pressure vs. settlement diagram (subgrade reaction modulus) obtained from plate or footing load tests. Taylor (1948) actually did not give any guidelines for determining allowable soil pressures from any type of penetration testing but only discussed allowable soil pressure in general terms.

6.3 Peck, Hanson and Thornburn (1953, 1974)

Peck et al. (1953) presented a modified chart for allowable bearing pressure "corresponding to 1 in. settlement of footings on sand" resembling the chart of Terzaghi and Peck (1948). As shown in Figure 6.5, the chart included more curves representing different SPT N values. Peck et al. (1953)

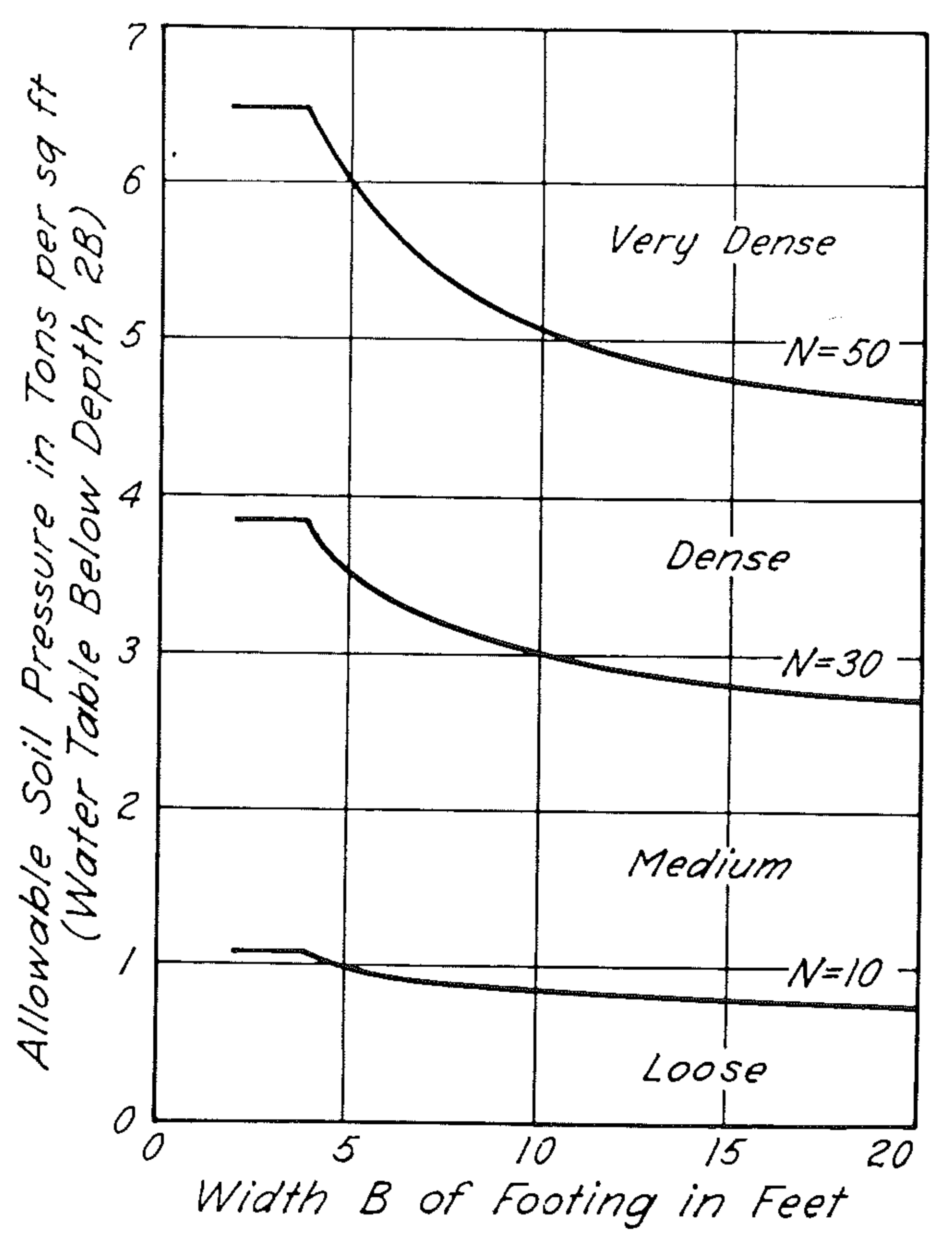


Figure 6.3 Terzaghi and Peck (1967) Allowable Soil Pressure Chart.

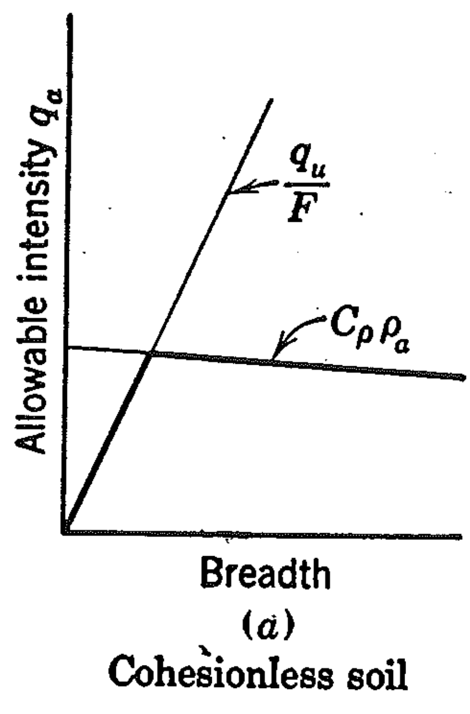


Figure 6.4 Taylor (1948) Allowable Soil Pressure Chart.

also noted that "If the pressure corresponding to some other amount of settlement is desired, it may be computed from the value given by the chart by assuming that the settlement varies directly as the soil pressure." This appears to be the first direct indication that there would be a linear relationship between settlement and footing pressure within the charts. Figure 6.5 was intended for situations where the water level was located at a level of at least a distance B below the base of the footing. They noted that "If the water level is near or above the base of the footing, the pressure corresponding to a 1-in. settlement should be taken as half the value given by the chart. For intermediate positions, the proper pressures may be obtained by interpolation."

In the updated version of their text, Peck et al. (1974), referring to the original chart of Terzaghi and Peck (1948), indicated that "The information then available was interpreted conservatively, so that in most instances that actual settlement of a footing proportioned on the basis of the relation would be less than 1 in. Experience has indicated that the relation was indeed conservative and sometimes excessively so; hence, various modifications have been suggested ...".

Accordingly, Peck et al. (1974) presented a set of curves for difference values of D_f/B corresponding to a settlement of 25 mm (1 in.) for the condition that the water table is at "great depth". The charts are presented in Figure 6.6. On the right hand of the charts, horizontal lines represent conditions where settlement controls, while on the left hand, diagonal lines indicated where bearing capacity controls.

6.4 Meyerhof (1956,1965)

Meyerhof (1956) suggested that the safe allowable bearing capacity of shallow foundations on dry or moist sands with respect to a bearing capacity failure could be estimated on the basis of penetration tests. Assuming a factor of safety of 3, it was recommended that safe bearing pressure could be obtained from:

$$q_s = NB(1+D/B)/30 \qquad \text{(in tsf)} \qquad [6.3]$$

and

$$q_s = q_c B(1+D/B)/120 \qquad \text{(in tsf)} \qquad [6.4]$$

where:

q_s = safe bearing pressure (in tsf)
N = average SPT blowcounts within depth B below base level of the footing
B = width of footing (in ft.)
D = depth of footing (in ft.)
q_c = average cone tip resistance within depth B below base level of the footing (in tsf)

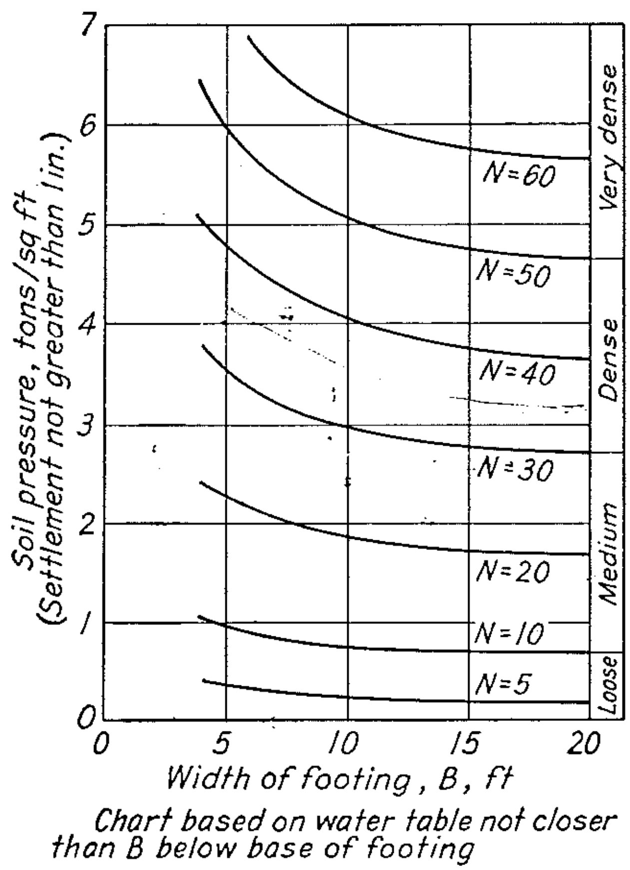


Figure 6.5 Peck et al. (1953) Allowable Soil Pressure Chart.

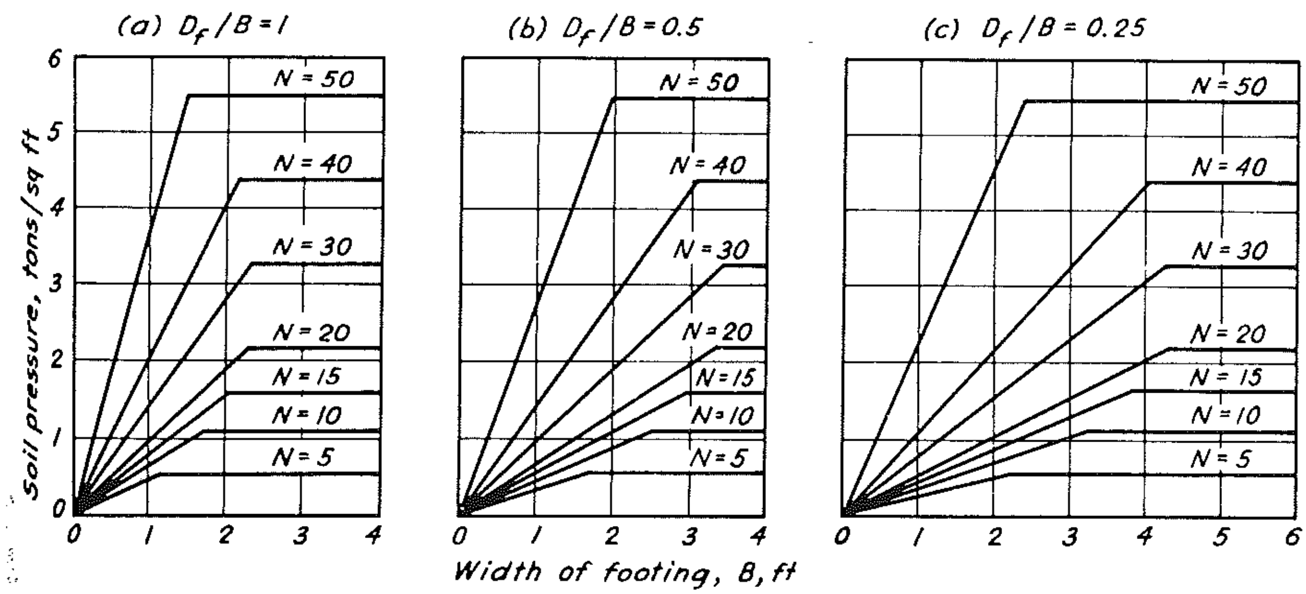


Figure 6.6 Peck et al. (1974) Allowable Soil Pressure Chart.

Meyerhof (1956) recommended that "For silty sands the bearing capacities...should be reduced by up to one-half, while for sand-gravel mixtures the values may be increased up to twice..." Additionally, he stated that "Full submergence of cohesionless soils reduces the effective unit weight and, thus, the bearing capacities... by about one-half. The bearing capacity is, however, not affected by a water table at a depth greater than about 1.5B below base level so that the bearing capacity for intermediate positions of the water table can be interpolated accordingly."

In a comparison with the results of plate loading tests, Meyerhof (1956) noted that "It is found that the observed ultimate bearing capacities are conservative and are about twice the estimated values in the case of 1 ft. wide footings. However, with increasing footing width the angle of internal friction at the ultimate bearing capacity decreases..., so that the proposed relationships are considered reasonable for the footing widths commonly used in practice."

Meyerhof (1956) recognized that the allowable bearing pressure may be less than the safe bearing pressure if settlement under the safe bearing pressure is excessive. Taking a total settlement of 25mm (1 in.) as producing a maximum tolerable differential settlement of 17.5mm (0.75 in.) he suggested the following expressions for the allowable soil pressure to produce a settlement of 25mm (1 in.):

$$q_a = N/8 \text{ (in tsf)} \qquad \text{(for } B \leq 4 \text{ ft.)} \qquad [6.5]$$
$$q_a = N(1+1/B)^2/12 \text{ (in tsf)} \qquad \text{(for } B > 4 \text{ ft.)} \qquad [6.6]$$

or

$$q_a = N/10, \text{ approximately, irrespective of } B \qquad [6.7]$$

Additionally, for CPT data:

$$q_a = q_c/30 \text{ (in tsf)} \qquad \text{(for } B \leq 4 \text{ ft.)} \qquad [6.8]$$
$$q_a = q_c\,(1+1B)^2/50 \text{ (in tsf)} \qquad \text{(for } B > 4 \text{ ft.)} \qquad [6.9]$$

or

$$q_a = q_c/40, \text{ approximately (irrespective of } B) \qquad [6.10]$$

Meyerhof (1956) noted that "irrespective of the relative density of the soil, the bearing pressure is governed by settlement considerations, i.e., the allowable bearing pressure is less that the safe bearing pressure, if the footing width exceeds 3 to 4 ft. depending on the footing depth. For raft and pier foundations it has been suggested that twice the allowable bearing pressure of footings can be used. For both types of foundations settlement governs the bearing pressure in all practical cases and the allowable pressure is then given by twice the values estimated by (Equations 6.5 to 6.10). Since submergence of cohesionless soils increases the settlement by about the same amounts as the bearing

capacity is reduced, the allowable bearing pressure should be reduced with position of the water table as indicated above for the ultimate bearing capacity."

Meyerhof (1965) gave essentially the same expressions for allowable bearing pressure as:

$p_a = Ns_a/8$ (for $B \leq 4$ ft.) [6.11]
$p_a = (Ns_a/12)((B+1)/B)^2$ (for $B > 4$ ft.) [6.12]

$p_a = Ns_a/12$ for rafts [6.13]

where:

p_a = allowable bearing pressure (in tsf)
N = SPT blowcounts (corrected for compact and dense, submerged silty sand)
B = foundation width (in ft.)
s_a = allowable settlement (in inches)

Meyerhof noted that these relationships are "sensibly independent of the shape of the foundation, and for a foundation depth approaching the width B, an increase of the allowable pressure by approximately one-third is sometimes made." It was also noted that "Even without making any allowance for the ground-water conditions, which are already reflected in the measured values of N,... the method still furnishes conservative bearing pressures."

6.5 Teng (1962)

The empirical allowable bearing pressure chart for a settlement of 25 mm (1 in.) of Terzaghi and Peck (1948) was put into equation form by Teng (1962) as:

$q_a = 720\ (N - 3)\ [(B+1)/(2B)]^2\ R'_w$ [6.14]

where:

q_a = net allowable bearing pressure (in psf)
B = footing width (in feet)
N = SPT blowcount
R'_w = reduction factor for water level

The water table adjustment factor R'_w is shown in Figure 6.7.

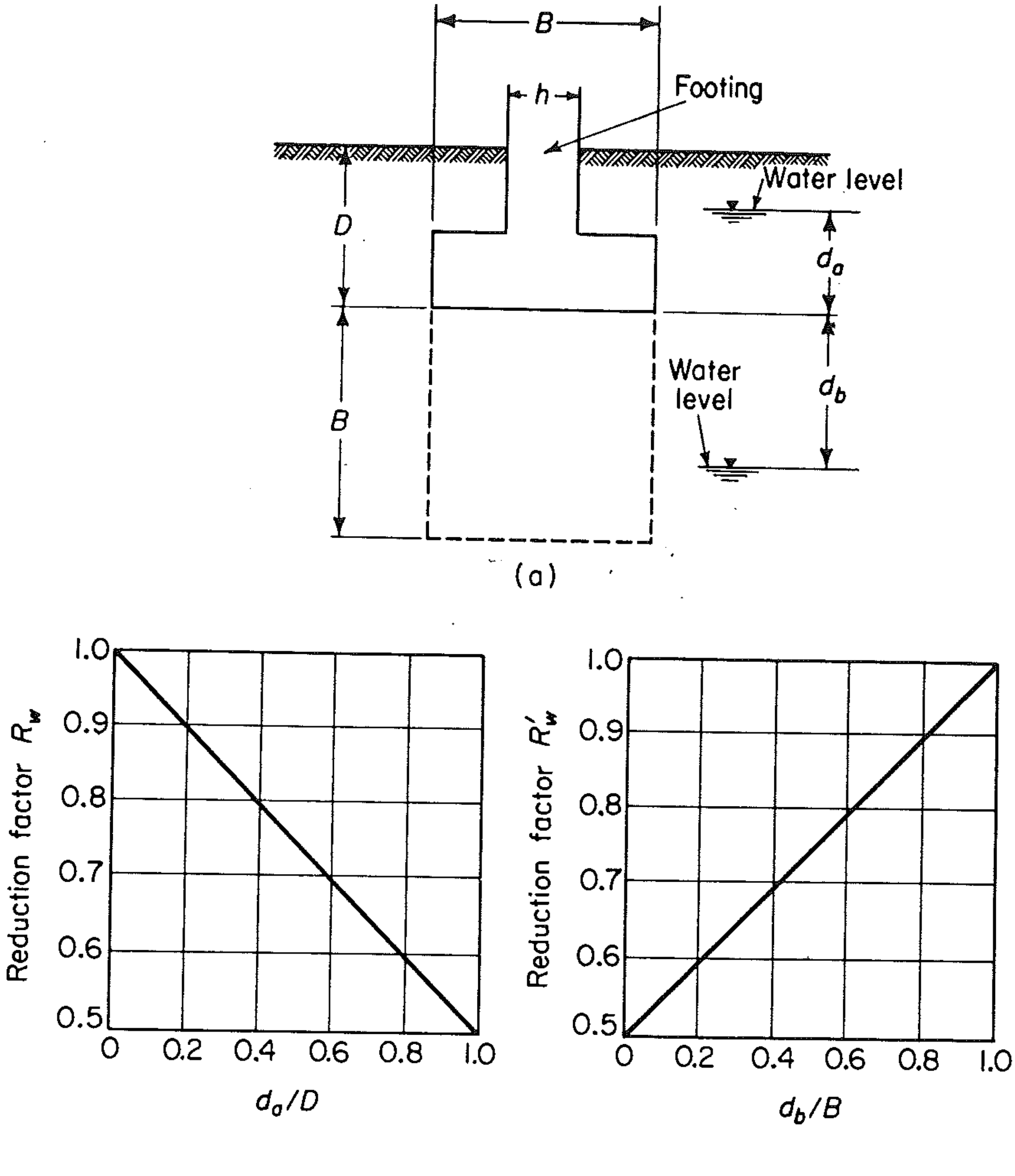


Figure 6.7 Teng (1962) Water Table Adjustment Factor.

6.6 Sowers (1962)

Sowers (1962) in Leonards (1962) presented a chart illustrating the relation between allowable foundation pressure and standard penetration resistance for small structures with column loads between 25,000 and 100,000 lb. The resulting chart for different soils is shown in Figure 6.8. Sowers (1962) recommended using the average penetration resistance for a depth of 1.5B below the footing.

6.7 Bowles (1968, 1977, 1982, 1988)

Bowles (1968) suggested that the chart of Terzaghi and Peck (1948) and the expressions presented by Meyerhof (1956, 1965) were too conservative, stating that "There seems, at present, to be considerable evidence... that both of these proposals ... are too conservative. These data indicate that the allowable soil pressure could be increased possibly as much as two or three times without excessive settlements occurring, and further, that the location of the groundwater beneath the footing is not as critical as presented...". Accordingly, Bowles (1968) recommended increasing the allowable soil pressures to produce 24 mm (1 in.) of settlement and presented charts for both the Terzaghi and Peck Method and Meyerhof Method. These charts are shown in Figure 6.9 and 6.10, respectively.

In the 1982 edition of his text, Bowles (1982) had dropped the modified Terzaghi and Peck (1948) chart and had only included a slightly different chart based on Meyerhof's Method than in previous editions, Figure 6.11. This chart also appears in Bowles (1988).

6.8 Mohan, Aggarwal and Tolia (1971)

A chart incorporating the corrected SPT values and the use of dynamic cone penetration tests (DCT) for determining allowable soil pressure was presented by Mohan et al. (1971) as shown in Figure 6.12. The allowable soil pressure is written approximately as:

$$q_a = (Ns_a)/(7P+5) \qquad \text{(for B< 1.5 m)} \qquad [6.15]$$
$$q_a = [(Ns_a)/(14P + 10] [(B+1)/B]^2 \qquad \text{(for B} \geq \text{1.5 m)} \qquad [6.16]$$
$$q_a = (Ns_a)/(11P+8) \qquad \text{(for rafts)} \qquad [6.17]$$

where:

q_a = allowable soil pressure (in kg/cm^2)
B = footing width (in meters)
N = SPT blowcount
P = overburden pressure (in kg/cm^2)
s_a = allowable settlement (in cm)

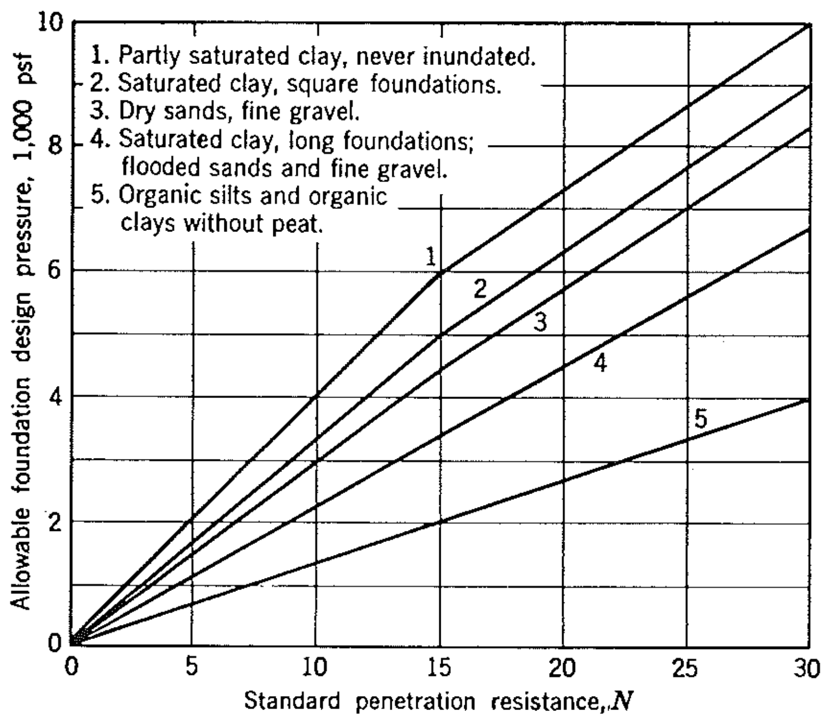


Figure 6.8 Sowers (1962) Allowable Soil Pressure Chart.

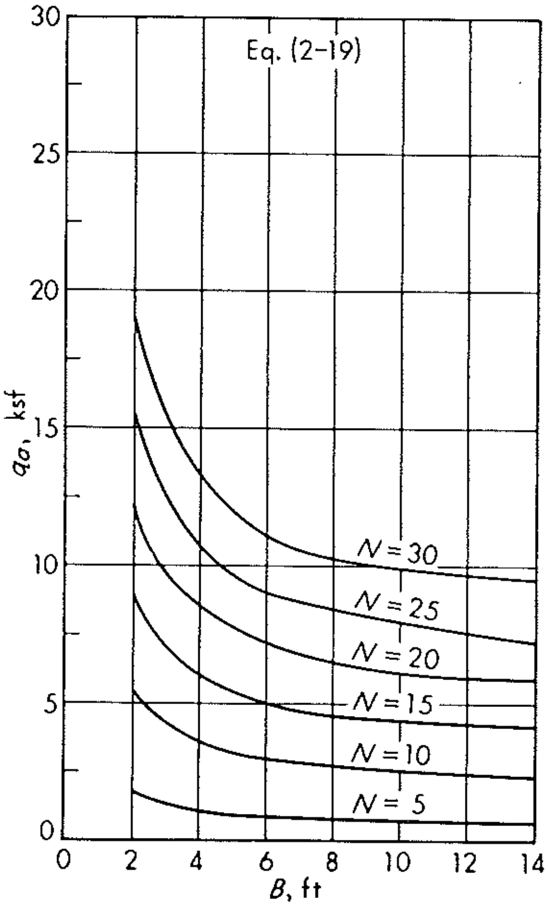


Figure 6.9 Bowles (1968) Modified Terzaghi and Peck Allowable Soil Pressure Chart.

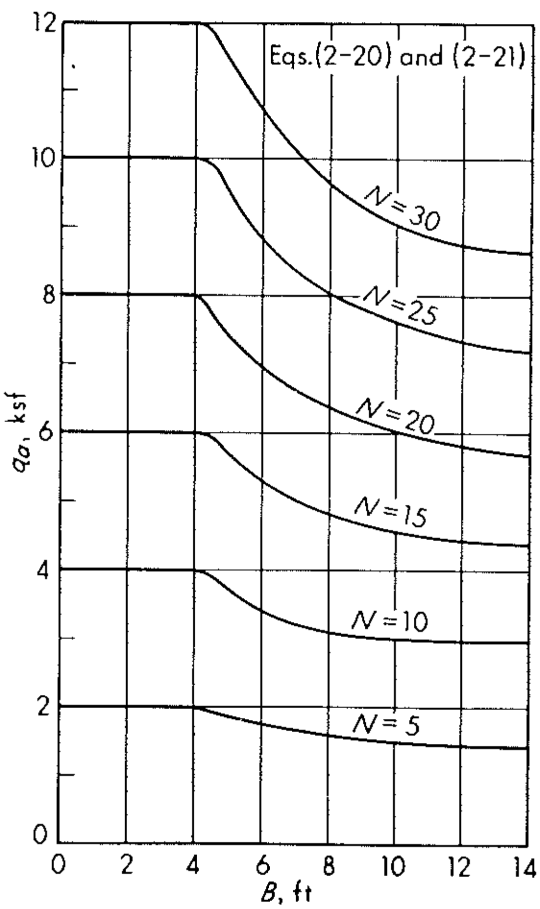


Figure 6.10 Bowles (1968) Allowable Soil Pressure Chart Based on Meyerhof.

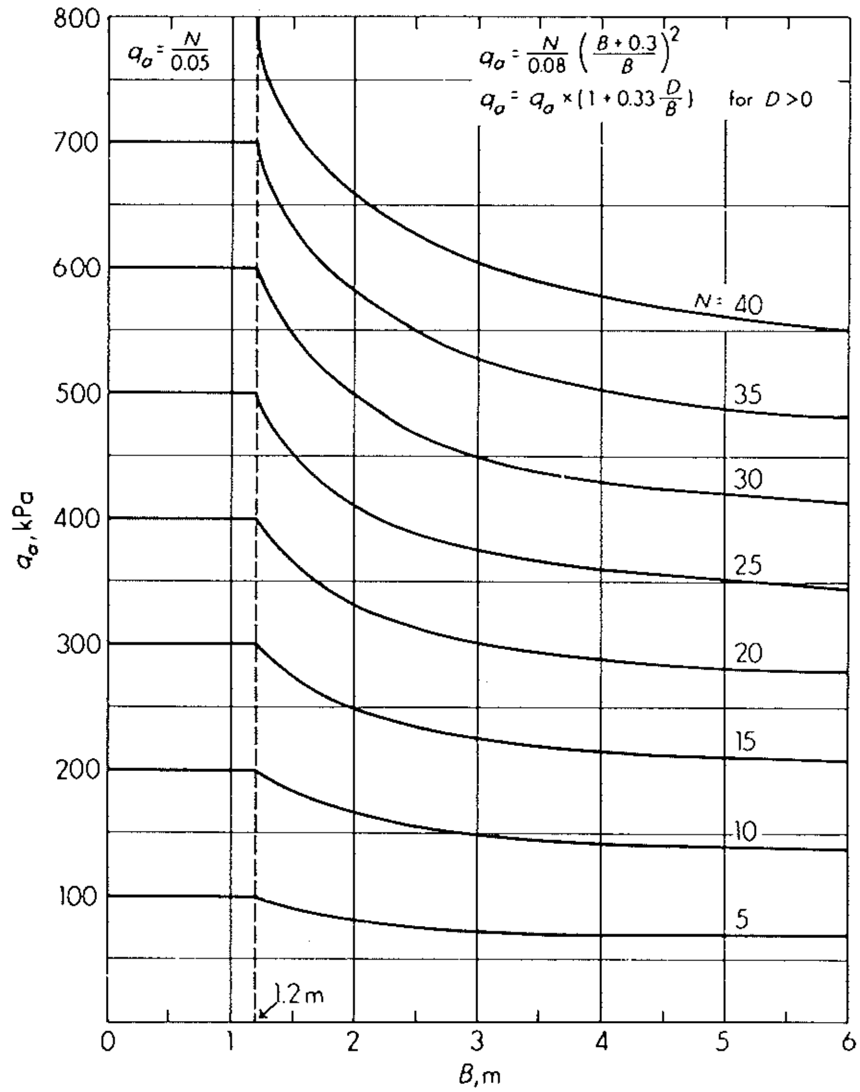


Figure 6.11 Bowles (1982) Meyerhof Allowable Soil Pressure Chart.

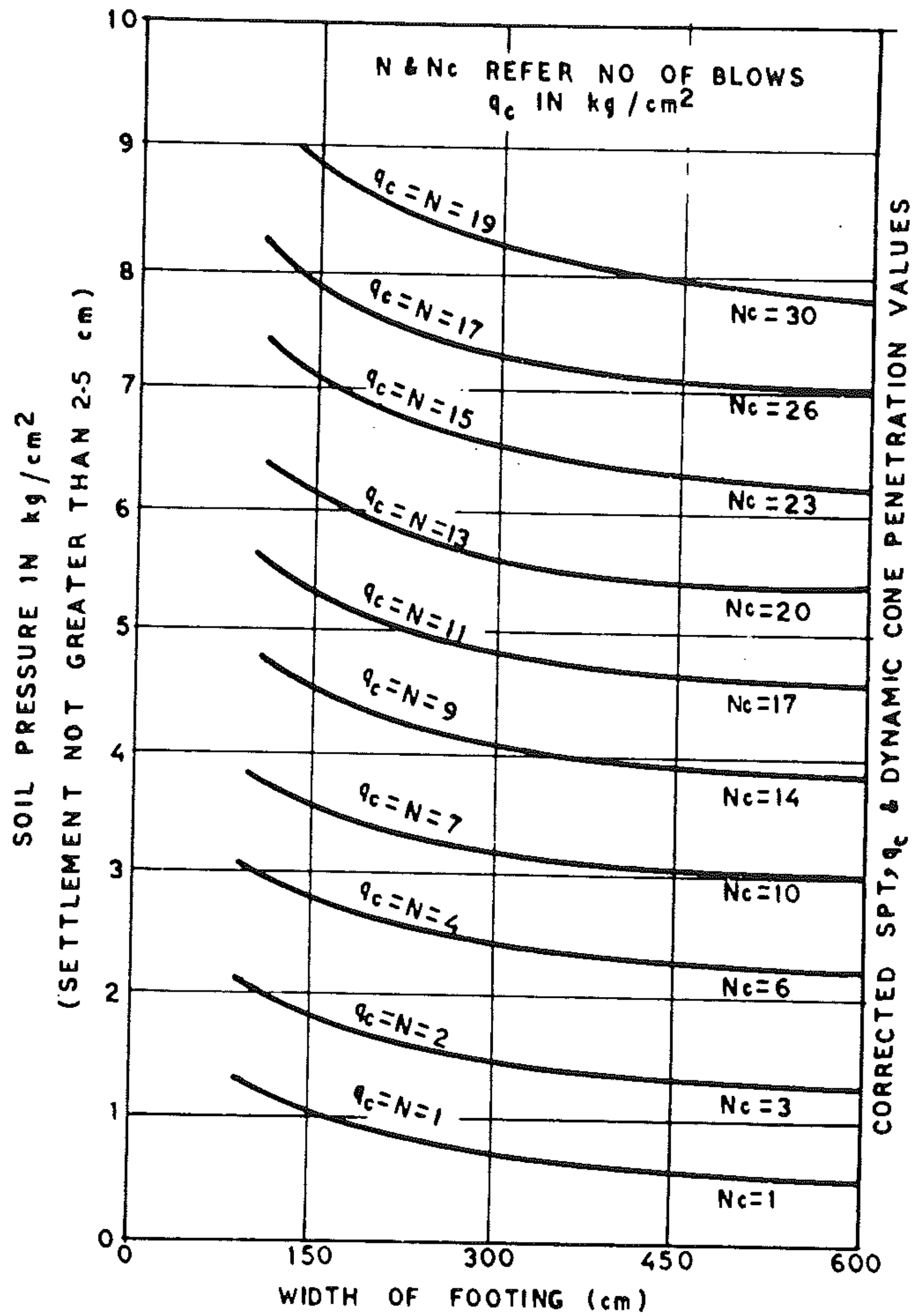


Figure 6.12 Mohan et al. (1971) Allowable Soil Pressure Chart.

It was stated by Mohan et al. (1971) that "These equations can be directly used for SPT value as measured without making any SPT corrections since these equations include a correction factor for the overburden pressure at any particular depth."

6.9 Canadian Foundation Engineering Manual (1975, 1985, 1992)

The first edition of the Canadian Foundation Engineering Manual (CFM, 1975) presented charts for allowable soil pressures beneath footings on non-cohesive soils as determined by bearing capacity taken from Peck et al. (1953) and shown in Figure 6.13. Apparently, these charts do not take into account settlement and only consider bearing capacity.

The allowable bearing pressure was also related to the results of dynamic cone penetration tests (CFM 1975) by correlating the DCT to the SPT by:

$$N_{cone} = 1.5N \qquad [6.18]$$

Equation 6.18 is intended for use with a 57.2 mm (2.25 in.) diameter 60° cone driven by a 140 lb hammer with a 30 in drop. N_{cone} is the number of blows per foot of penetration.

Alternatively, the allowable bearing pressure can be estimated as:

$$q_a = R_d/20 \qquad [6.19]$$

where:

$$R_d = (M^2H)/[Ae(M+P)] \qquad [6.20]$$

where:

R_d = unit resistance (lb/ft^2)
M = mass of hammer (lb)
H = height of fall of hammer (ft.)
e = penetration per blow (ft.)
P = mass of pipe (lb)
A = cross sectional area of cone (ft^2)

For shallow foundations, the results of the CPT may be used to estimate the allowable bearing pressure of shallow foundation as:

$$q_{allow} = 0.1\, q_c \qquad [6.21]$$

or using the relationship of Meyerhof (1956) which was put into graphical form and is shown in Figure 6.14.

Later editions of the Canadian Foundation Engineering Manual (1985, 1992) present SI versions of the design charts of Peck et al. (1974) shown in Figure 6.14 and the chart for correcting the SPT blowcounts suggested by Peck et al. (1974) as given in Figure 6.16. They also give the chart shown in Figure 6.14 but dropped the use of the dynamic cone test.

6.10 McCarthy (1977)

A chart of allowable bearing pressure related to footing width and corrected SPT blowcount values was presented by McCarthy (1977) and is shown in Figure 6.17. Table 6.1 presents the recommendations for a correction factor to be applied to field blowcounts taken from McCarthy (1977). It is recommended to use the average corrected N value in the zone between the bottom of the footing and a depth of about 1.5B.

Table 6.1 Correction Factor, N_{Design}/N_{field}, for SPT Values (from McCarthy, 1977)

Vertical Effective Pressure ksf	Approximate Depth for SPT Sample, (Feet)	Correction Factor
0.25	2	1.7
0.50	4-5	1.4
0.75	7	1.2
1.0	9-10	1.1
1.5	14-15	0.95
2.0	18	0.90
2.5	22-24	0.85
3.0	26-28	0.80
4.0	35-38	0.75

The chart shown in Figure 6.17 assumes the water table to be a depth greater than B below the base of the footing. McCarthy recommended that "For the condition where the water table is at the base of the foundation, a one-third reduction in the bearing pressure values should be applied. A linear interpolation can be assumed for water table depths intermediate between the foundation level and a distance B below it."

McCarthy (1977) also presents a chart for allowable bearing capacity of footings based on the CPT, shown in Figure 6.18, which appears to be taken from the suggestion of Meyerhof (1956, 1965) and is the same as presented by the Canadian Foundation Engineering Manual (1975, 1985, 1992). By assuming a q_c/N ratio of 4 for sand, McCarthy (1977) also presented a chart for 25 mm (1 in.) settlement based on CPT tip resistance, which is shown in Figure 6.19.

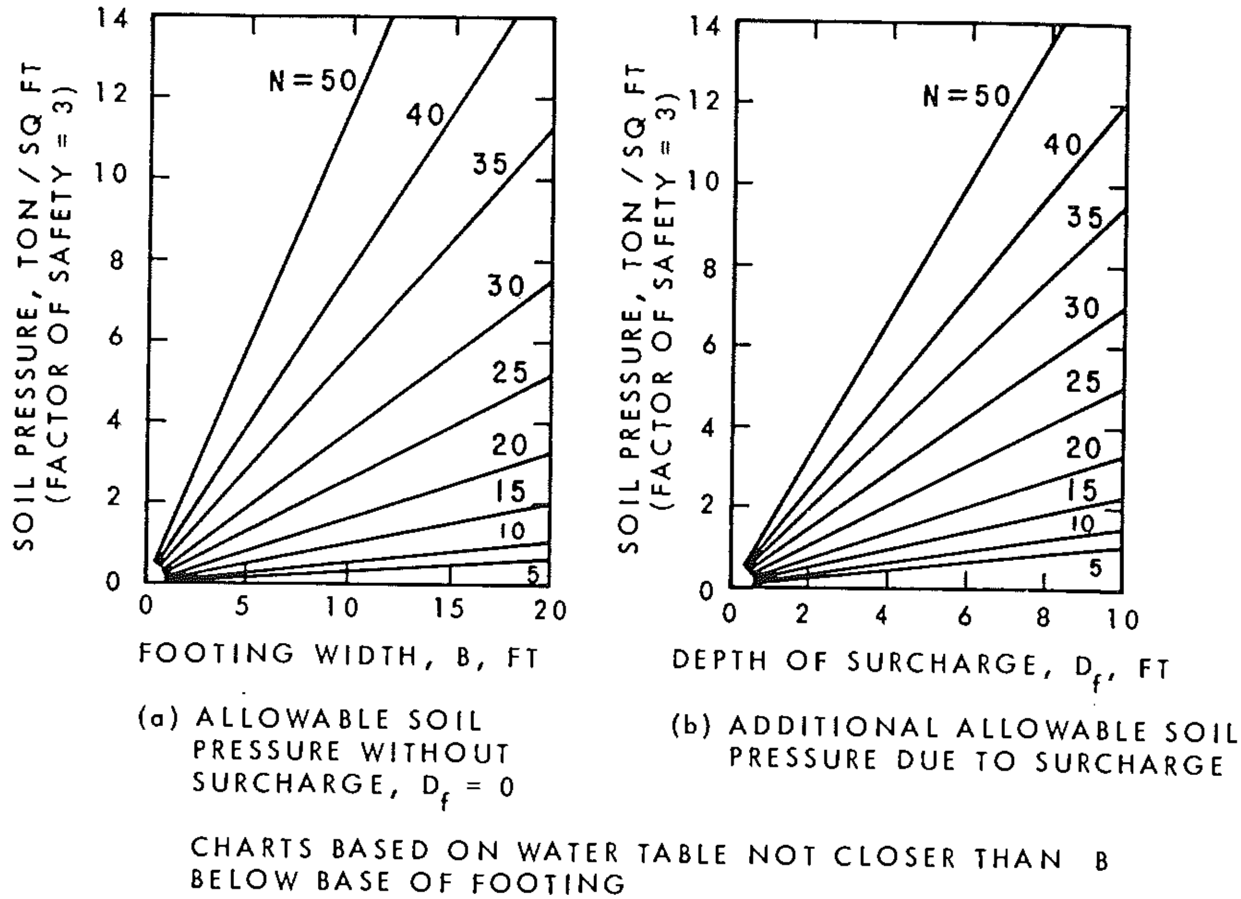


Figure 6.13 CFM (1975) Allowable Bearing Capacity Chart Based on SPT.

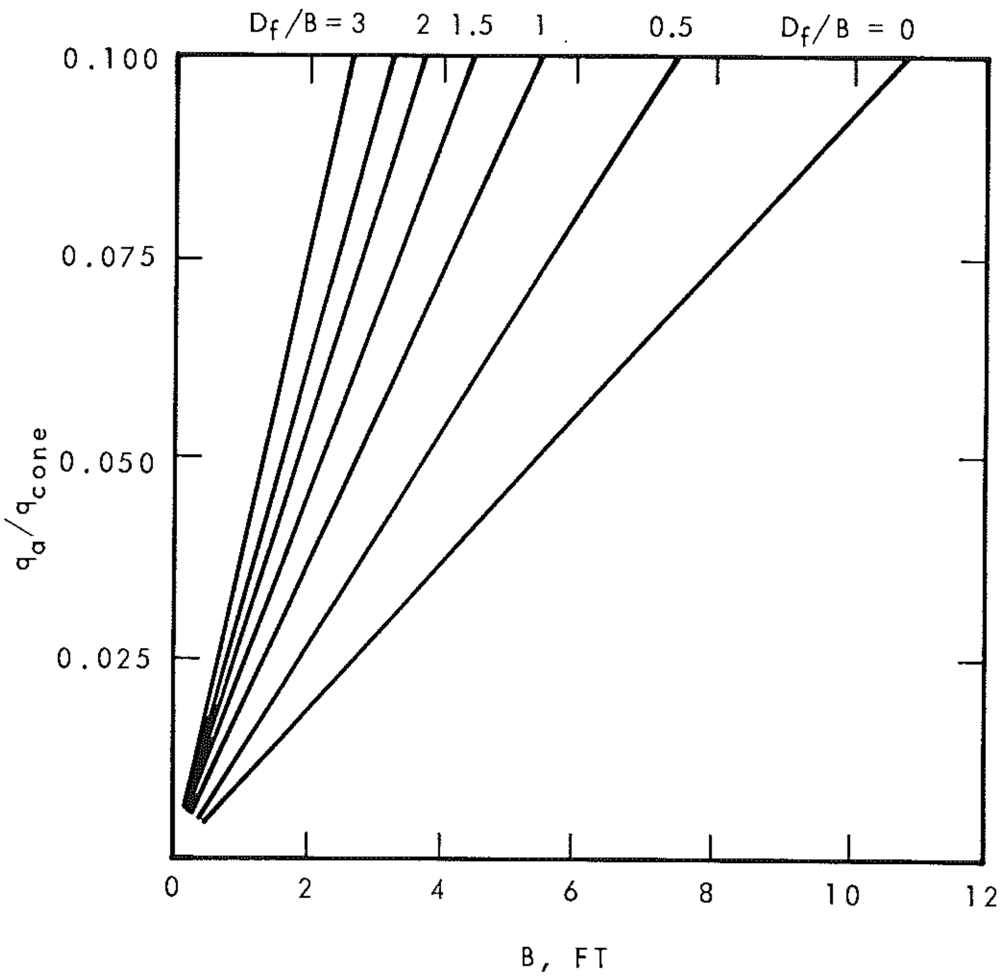


Figure 6.14 CFM (1975, 1985, 1992) Allowable Bearing Capacity Chart Based on CPT.

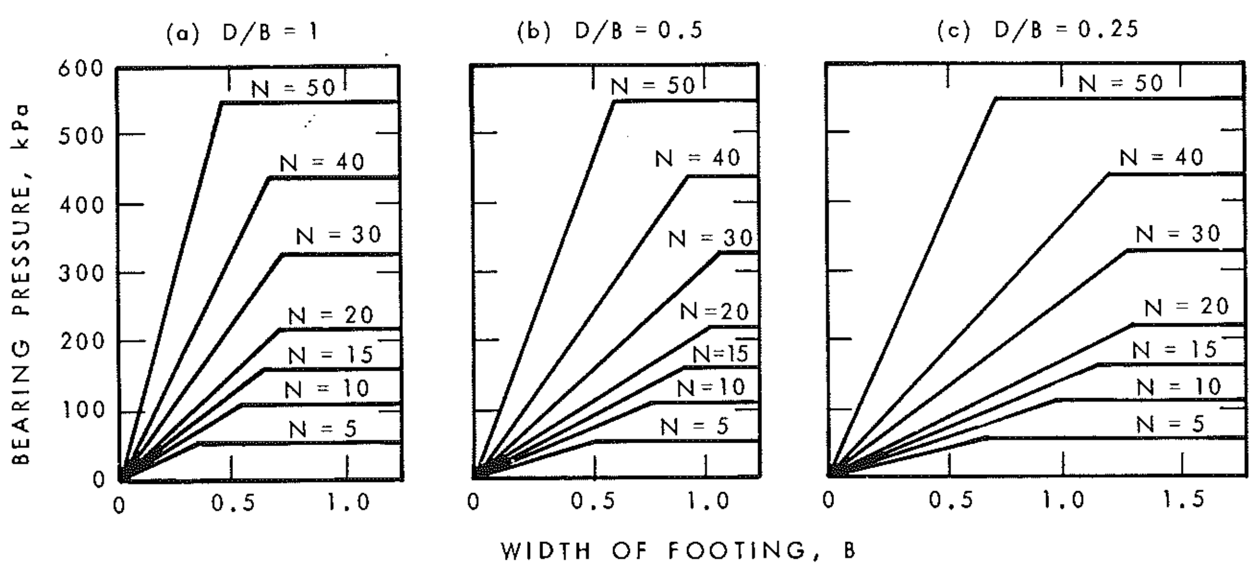


Figure 6.15 CFM (1985, 1992) Allowable Bearing Capacity Chart Based on SPT.

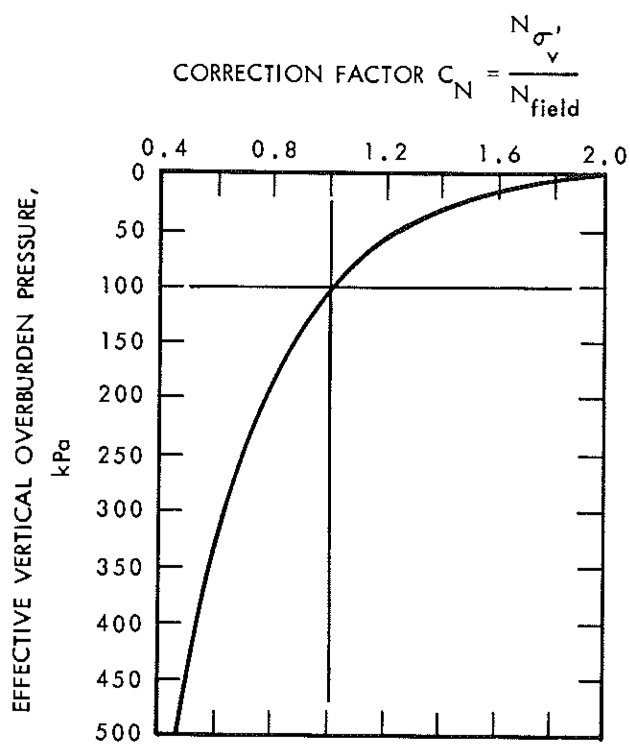


Figure 6.16 CFM (1985, 1992) SPT Correction.

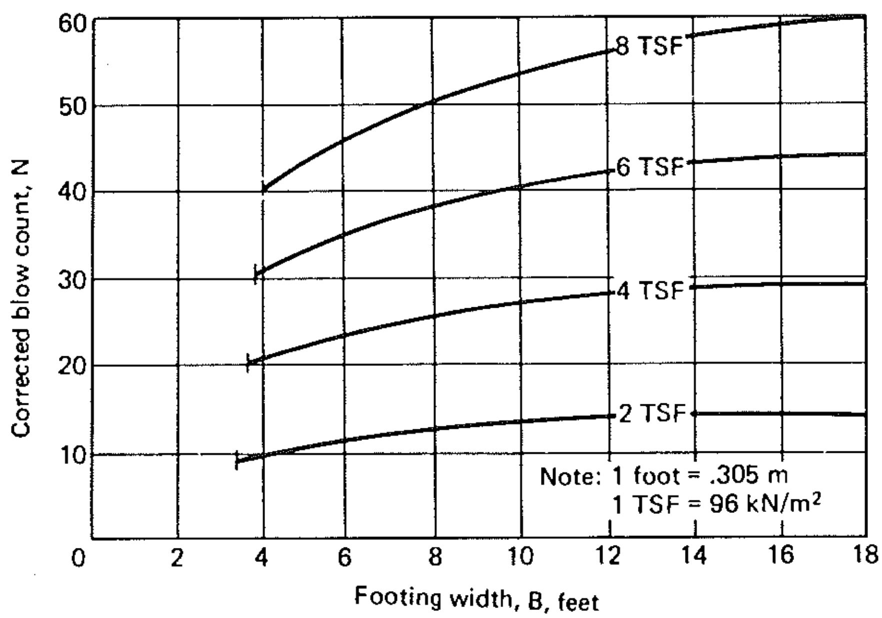


Figure 6.17 McCarthy (1977) Allowable Soil Pressure Chart Based on SPT.

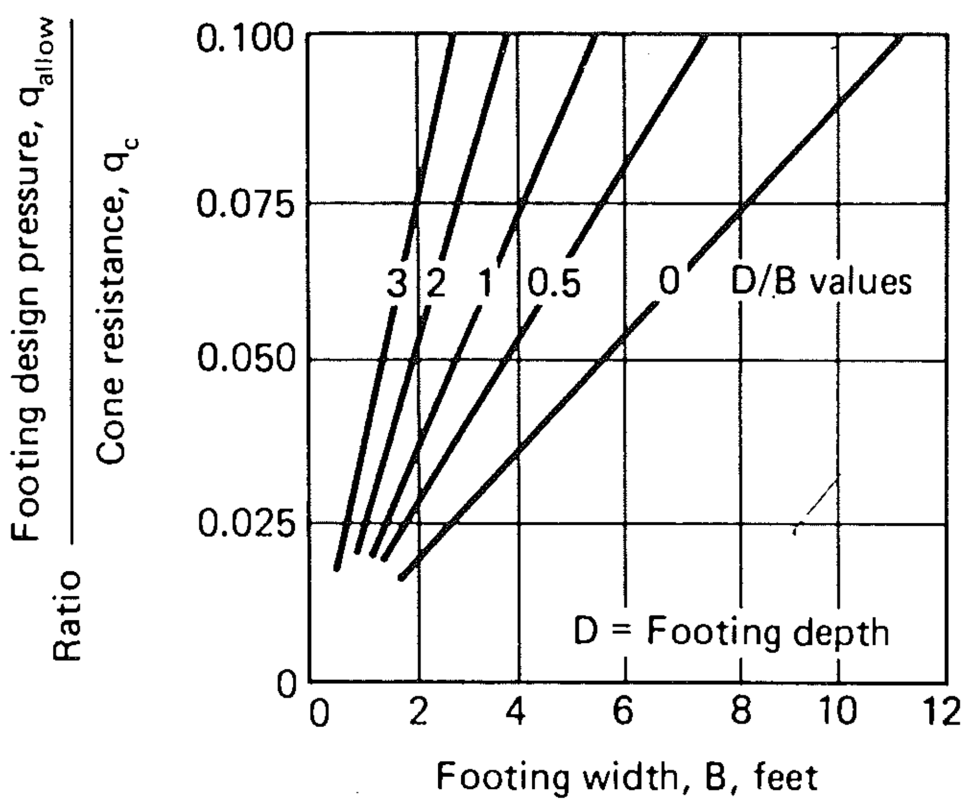


Figure 6.18 McCarthy (1977) Bearing Capacity Chart Based on CPT.

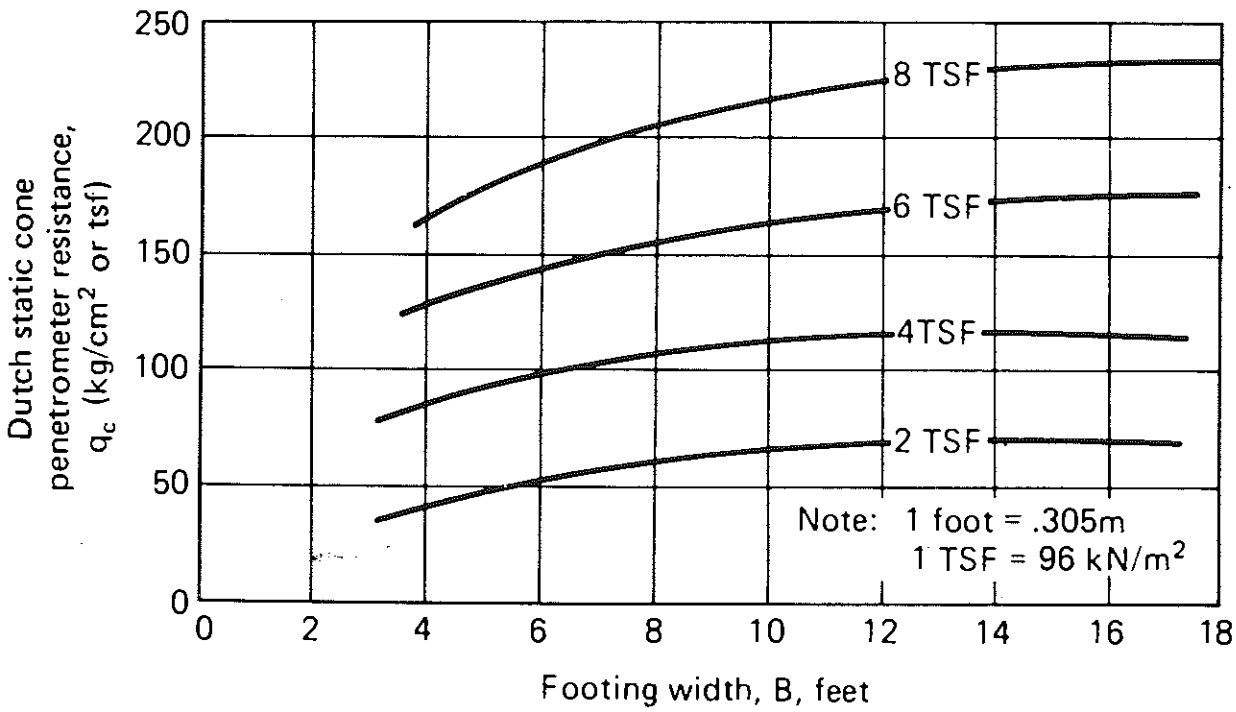


Figure 6.19 McCarthy (1977) Allowable Soil Pressure Chart Based on CPT.

6.11 Parry (1977)

A method for computing the ultimate bearing capacity of cohesionless soils for $D_f \leq B$ using the results of the SPT was proposed by Parry (1977), using an approximate relationship between N and ϕ as:

$$\phi = 25 + 28\,(N_m/\sigma'_{vo})^{0.5} \qquad [6.22]$$

where:

σ'_{vo} = effective vertical stress at the level where N_m is obtained.

The ultimate bearing capacity is given as:

$$q_{ult} = 30N_m \quad \text{(in kPa)} \qquad [6.23]$$

The blowcount value is taken as a "representative value of N", N_m, at a depth of 0.75 B below the foundation level. Parry (1971) had earlier suggested that settlements could be estimated from:

$$\rho = 200qB/N_m \qquad [6.24]$$

where:

ρ = settlement (in mm)
q = footing pressure (in MPa)
B = width (in meters)

Parry (1977) noted that "... for design purposes the calculated values should be increased by 50% ..." Therefore, for design, Parry (1977) recommended using:

$$\rho = 300qB/N_m \qquad [6.25]$$

Parry (1977) suggested that if the "allowable" settlement is taken as 20mm, Equation 6.25 could be rewritten as:

$$B = N_m/15q_a \qquad [6.26]$$

and noted that for this case (i.e., allowable settlement equal to 20mm) the design of footings less than 2.2 m wide will be governed by bearing capacity and the design of footings greater than 2.2 wide will be governed by settlement.

For comparison with the other methods presented in this section, we may set the allowable settlement equal to 25 mm and rearrange Equation 6.25 to give:

$$q_a = N_m/0.012B \ (kN/m^2) \tag{6.27}$$

Equation 6.27 may be programmed for various values of N_m and B to give a design chart. The resulting chart is shown in Figure 6.20.

6.12 Spangler and Handy (1982)

A chart relating SPT blowcounts, CPT tip resistance and footing width to net footing pressure was presented by Spangler and Handy (1982) based on the procedure for predicting settlements in sands presented by Schmertmann (1970) and Schmertmann et al. (1978). According to Spangler and Handy (1982) "The Terzaghi and Peck relationship, which was reproduced in previous editions of this text, has been found to be considerably in error by overestimating settlement of narrow footings and underestimating it for wide footings. In light of these more recent data, the use of the Terzaghi and Peck relationship and derivatives by Meyerhof (1965) and by Bowles (1977) is not recommended."

The resulting chart presented by Spangler and Handy (1982) is shown in Figure 6.21. Note that a q_c/N ratio of 4 was assumed so that this chart could also be used to estimate settlement using the results of the SPT.

6.13 Goel (1982)

Charts were presented by Goel (1982) giving the allowable footing pressure to produce 25 mm (1 in.) of settlement from shallow footings on sand, based on the CPT tip resistance. The charts shown in Figure 6.22 are for conditions of both dry and submerged state and for the range in footing size from 1.5m to 4.5 m. These charts were developed from footing load tests performed in a pit using three different sand compacted to relative densities ranging from 10 to 70%.

6.14 Navfac DM7 (1982)

The Navy Design Manual DM7 (1982) suggests determining allowable bearing pressure of footings on sand from CPT results using the method apparently given by Meyerhof (1956, 1965) and previously shown. The chart presented in DM7 appears exactly the same as that given in the Canadian Foundation Engineering Manual and is shown in Figure 6.23.

6.15 Hunt (1986)

Hunt (1986) presented the design charts of Peck et al. (1974), previously shown in Figure 6.6 and the chart based on the CPT, shown in Figure 6.24. He also presented a comparison of several of the methods, which is shown in Figure 6.25 and pointed out that "Based on investigations by Schmertmann (1970) and others, the Terzaghi and Peck curves [also Terzaghi and Peck (1967)] are considered conservative for footing widths B < 10 ft. (3 m) and unconservative for larger footing widths."

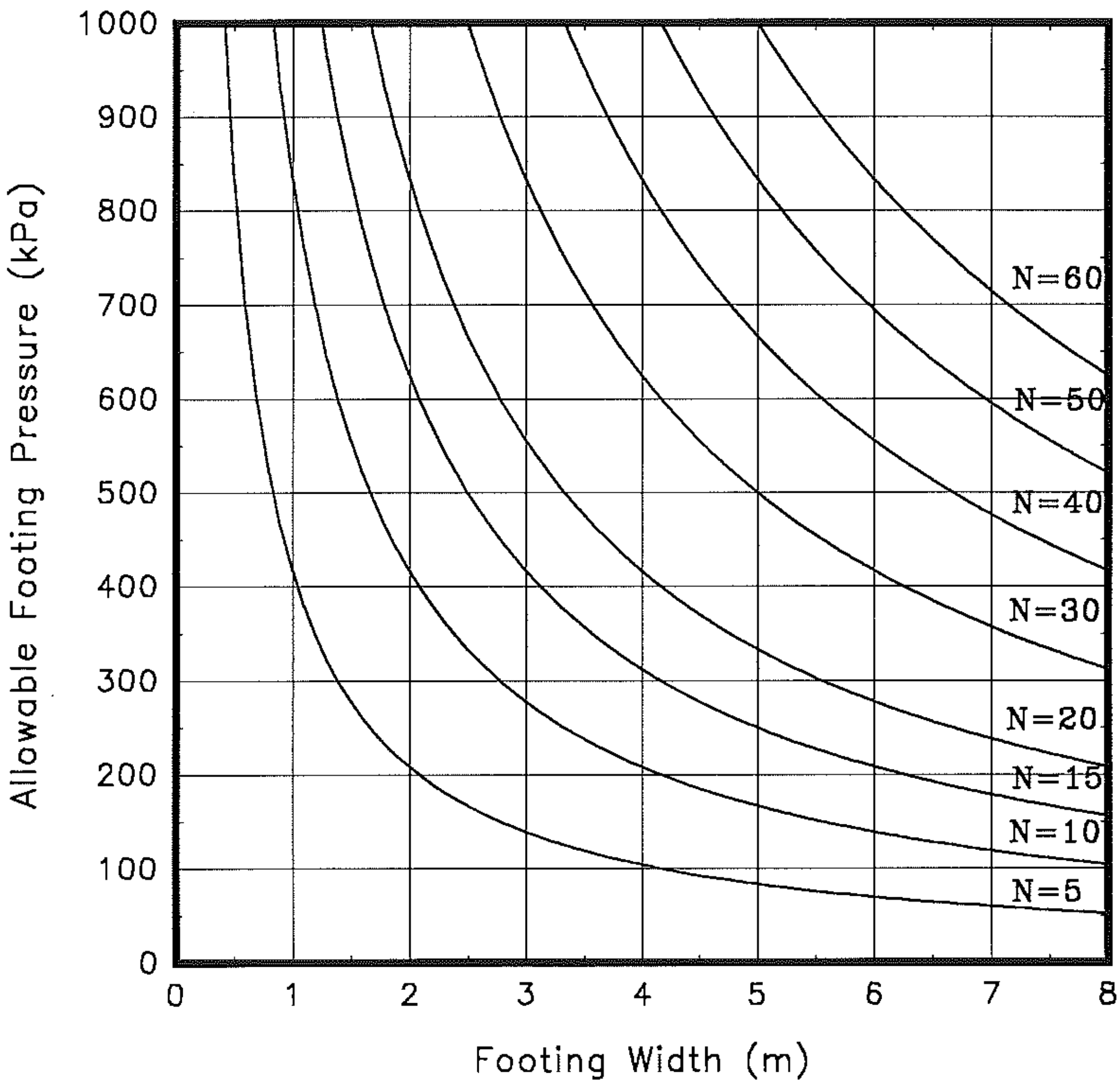


Figure 6.20 Design Chart Based on Parry (1977).

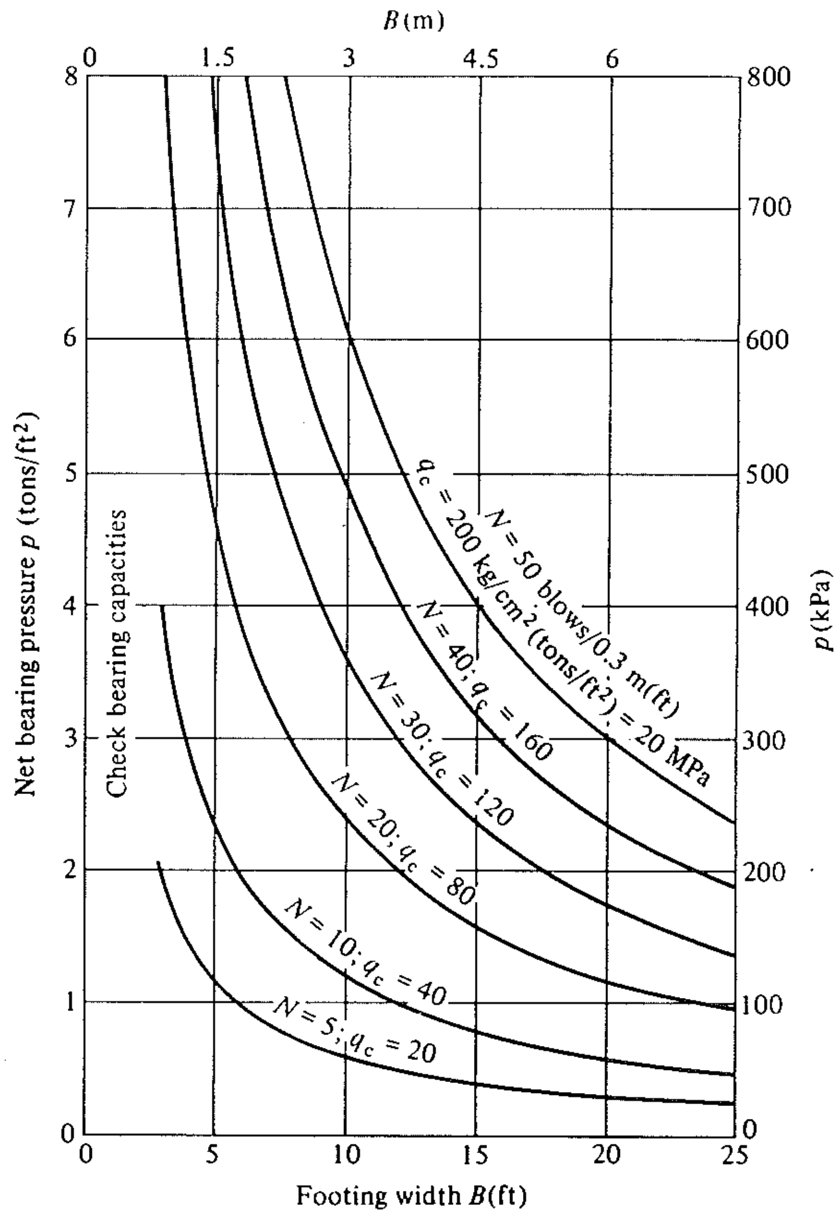


Figure 6.21 Spangler and Handy (1982) Allowable Soil Pressure Chart.

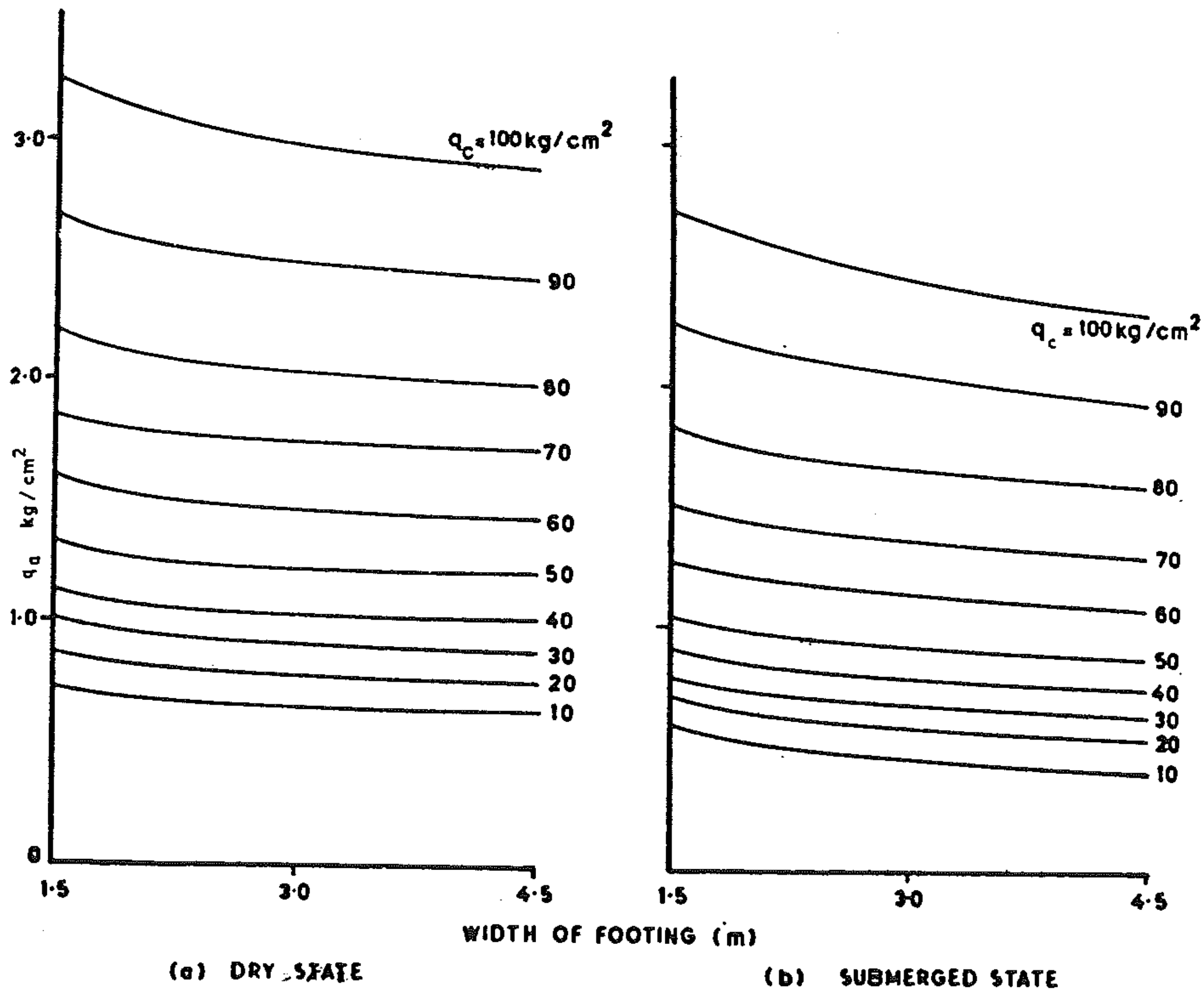


Figure 6.22 Goel (1982) Allowable Soil Pressure Charts Based on CPT.

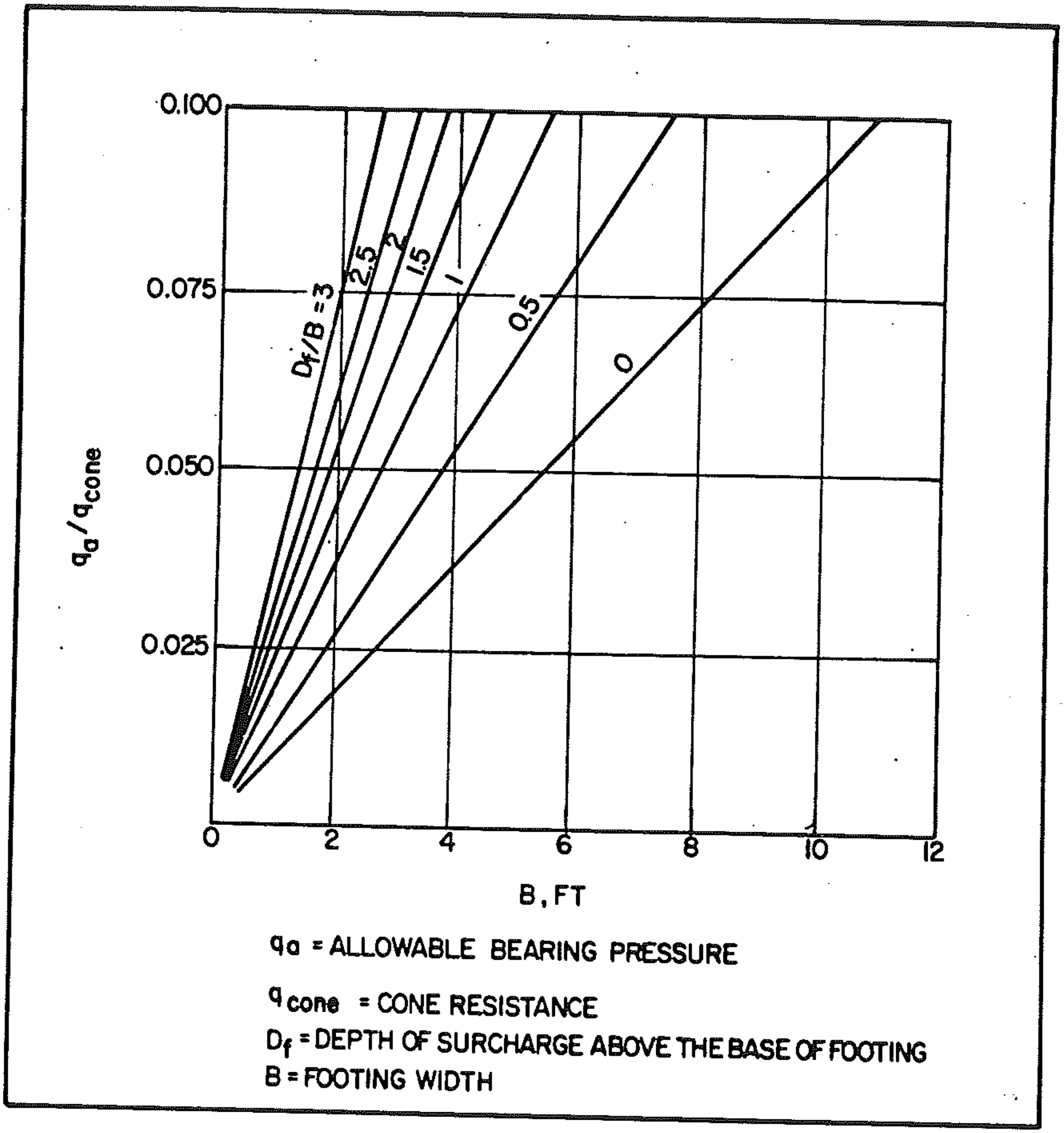


Figure 6.23 Navfac DM7 (1982) Bearing Capacity Chart.

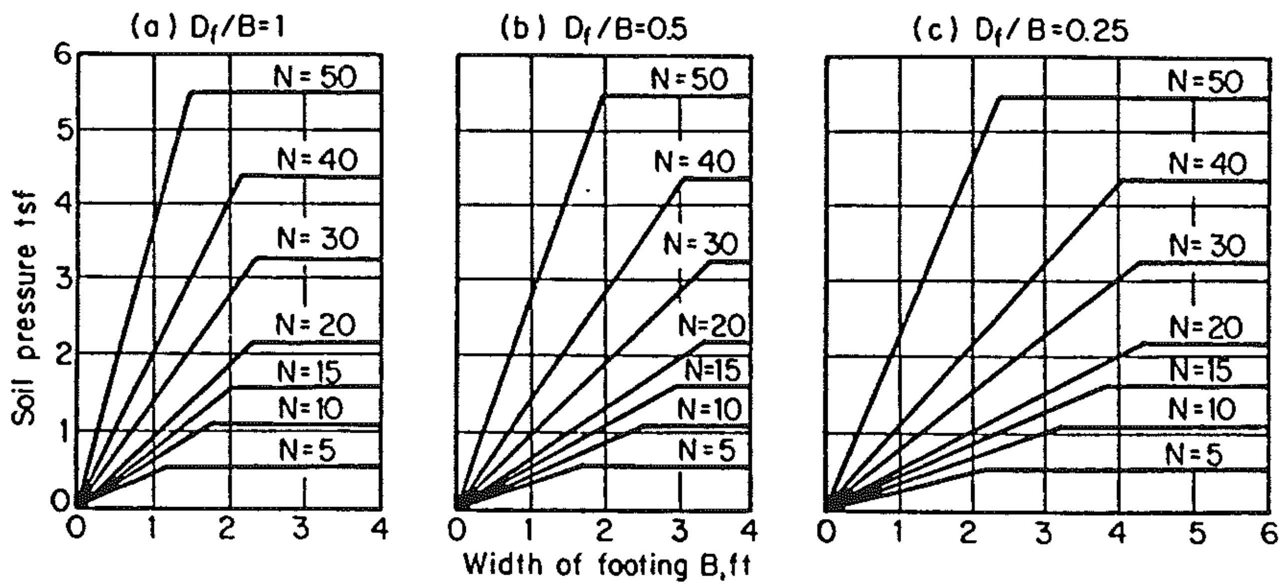


Figure 6.24 Hunt (1986) Allowable Soil Pressure Chart.

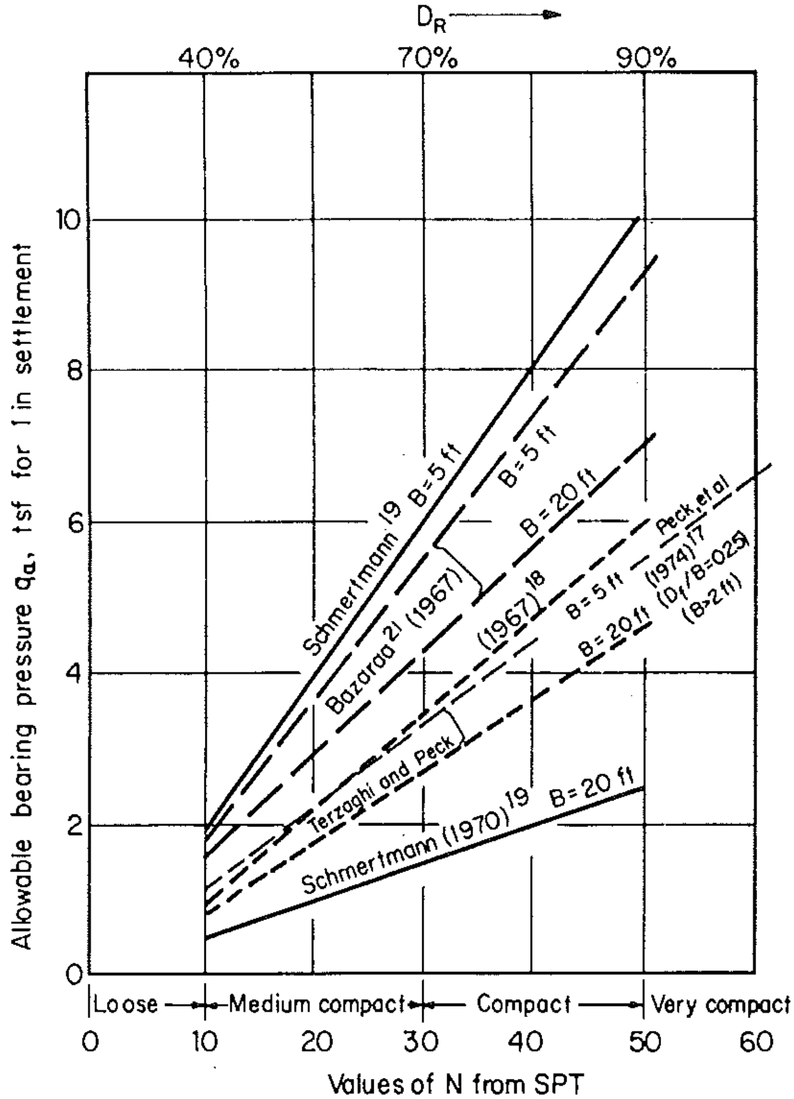


Figure 6.25 Hunt (1986) Comparison of Allowable Soil Pressures.

6.16 van der Vlugt and Rosenthal (1989)

Using the settlement procedure proposed by Schmertmann (1970) and Schmertmann et al. (1978) van der Vlugt and Rosenthal (1989) presented a design chart for allowable soil pressure for surface loaded footings to produce a settlement of 25 mm (1 in.), Figure 6.26 which is similar to the technique used by Spangler and Handy (1982). The form of the settlement curves for different values of cone resistance, q_c, are different than proposed by Terzaghi and Peck (1948) and Peck et al. (1974) but are similar to that shown by Taylor (1948). These curves simply indicate higher allowable pressures for smaller footings and lower pressures for larger footings than do previous curves which is consistent with previous observations made by others. Approximate equivalent SPT values are also shown in Figure 6.26 assuming q_c/N equals 4.

6.17 Comparison of Design Charts

In order to illustrate the difference between the different proposed charts, a comparison can be made of the observed and predicted footing pressure producing a settlement of 25 mm (1 in.). To do this, the results of the FHWA footing load test performed at the Texas A&M NGES are used. These tests represent footing widths of 1, 1.5, 2.5, and 3 m as indicated in Table 6.2. The values of footing pressures were obtained from footing settlement curves presented by Briaud and Gibbens (1994).

Table 6.2 Observed and Predicted Footing Pressure at 25mm (1in.) Settlement

Method	**1 x 1m A**	**1.5x1.5m B**	**2 x 2m C**	**3 x 3m D**	**3 x 3m E**
Observed	850	670	560	580	480
Terzaghi & Peck (1948)	193	180	158	156	156
Peck, Hanson & Thornburn (1953, 1974)	— 172	192 172	152 172	151 172	151 172
Meyerhof (1956) (1965)	201	194	169	163	163
Sowers (1962)	235	235	235	235	235
Bowles (1968) (1982)	322 336	306 307	257 264	245 256	245 256

Table 6.2 (continuted)

Method	**1 x 1m A**	**1.5x1.5m B**	**2 x 2m C**	**3 x 3m D**	**3 x 3m E**
McCarthy (1977)	—	441	351	319	319
Parry (1977)	1400	933	560	467	467
Spangler & Handy (1982)	582	371	227	193	193
Van der Vlugt & Rosenthal (1989)	85*	130*	200	180	180

Notes: *Bearing capacity controls.
All values of footing pressure in units of kPa.
Observed data from FHWA footing load tests at Texas A&M.

As can be seen, there is a wide range of allowable values of bearing capacity indicated in Table 6.2. The method proposed by Parry (1977) gives the best comparison with the observed values for all footings and gives the correct trend of decreasing allowable pressure with increasing size of footing.

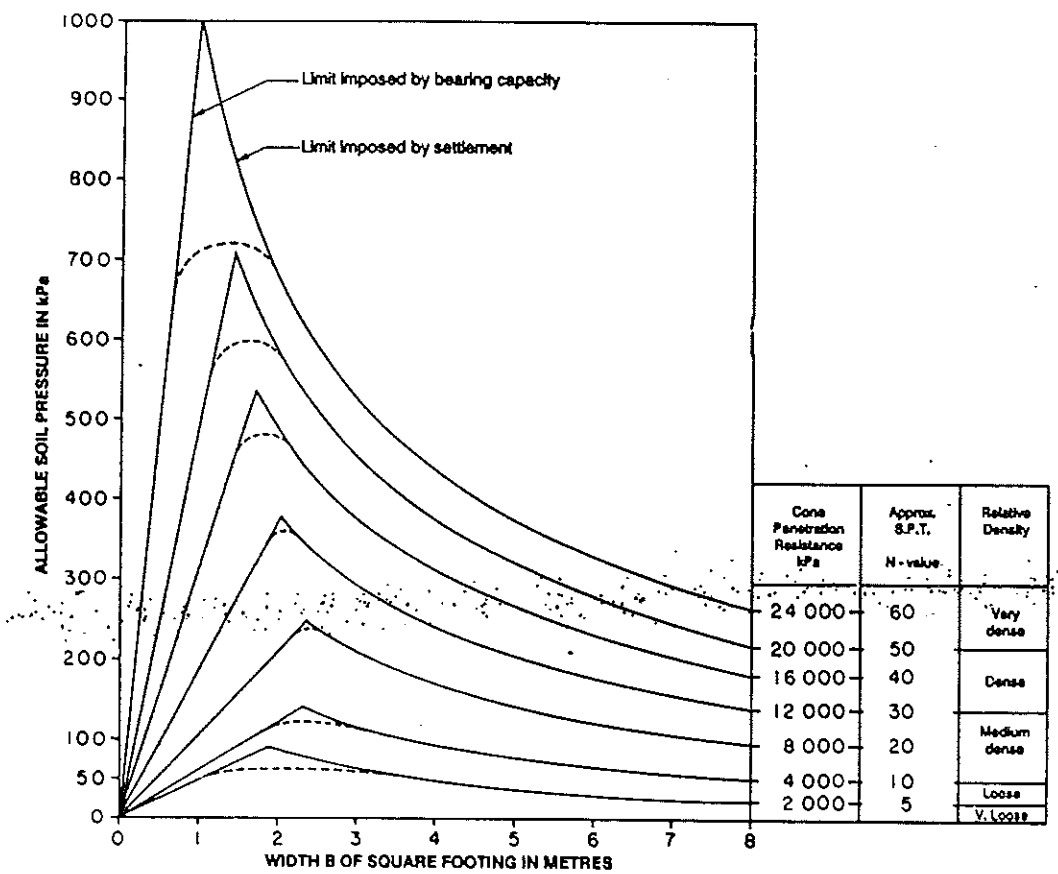


Figure 6.26 van der Vlugt and Rosenthal (1989) Allowable Soil Pressure Chart.

7.0 LOAD-SETTLEMENT BEHAVIOR OF SPREAD FOOTINGS ON GRANULAR SOILS-UNIVERSAL APPROACH

The settlement prediction methods previously described in this report generally suffer from being site specific and are based on a decoupling of the limit equilibrium and deformation behavior of shallow foundations. Because of this, it is useful to approach the problem from a different philosophical viewpoint and search for an alternative approach. It may be possible to link the settlement of shallow foundations resting on granular soil deposits to the limit equilibrium condition, i.e., ultimate bearing capacity. Since most shallow foundations are designed to function at working loads well below the limiting value, which would produce a bearing capacity failure, it is normally the settlement criteria which governs the overall design. This means that while a normal industry factor of safety on the order of 3 against bearing capacity is sought, this level of loading provides sufficient safety from a bearing capacity but generally allows excessive settlements, especially for settlement critical structures. This means that a higher operational factor of safety is usually in effect.

7.1 Background

There is substantial evidence in the literature which suggests that there is a direct connection between load capacity and deformation of foundations for both deep and shallow foundations. For example, Trautmann and Kulhawy (1988) have shown that this link exists in the performance of shallow foundations subjected to axial uplift loading. Based on the results of a large number of full-scale load tests conducted on shallow ($D/B \leq 3$) grillage and cast-in-place foundations, they suggested that the uplift displacement under load could be expressed in terms of the normalized load i.e., any given load divided by the load at failure. In order to express the deformation in a non-dimensional fashion, they suggested that the deformation be normalized with respect to the depth of burial (D) as shown in Figure 7.1 the depth of burial, D, which in effect identifies the active zone of soil where the deformation occurs.

In granular soils, it was found that the failure of the foundation occurred at a displacement corresponding to about 0.005D. When plotted non-dimensionally, as shown in Figure 7.1, the results displayed a general shape expressed as a hyperbolic function by:

$$Q/Q_{ult} = (z/D)/(0.013 + 0.67(z/D)) \qquad [7.1]$$

where:

Q = Current Load
Q_{ult} = Ultimate Load
z = displacement
D = Depth

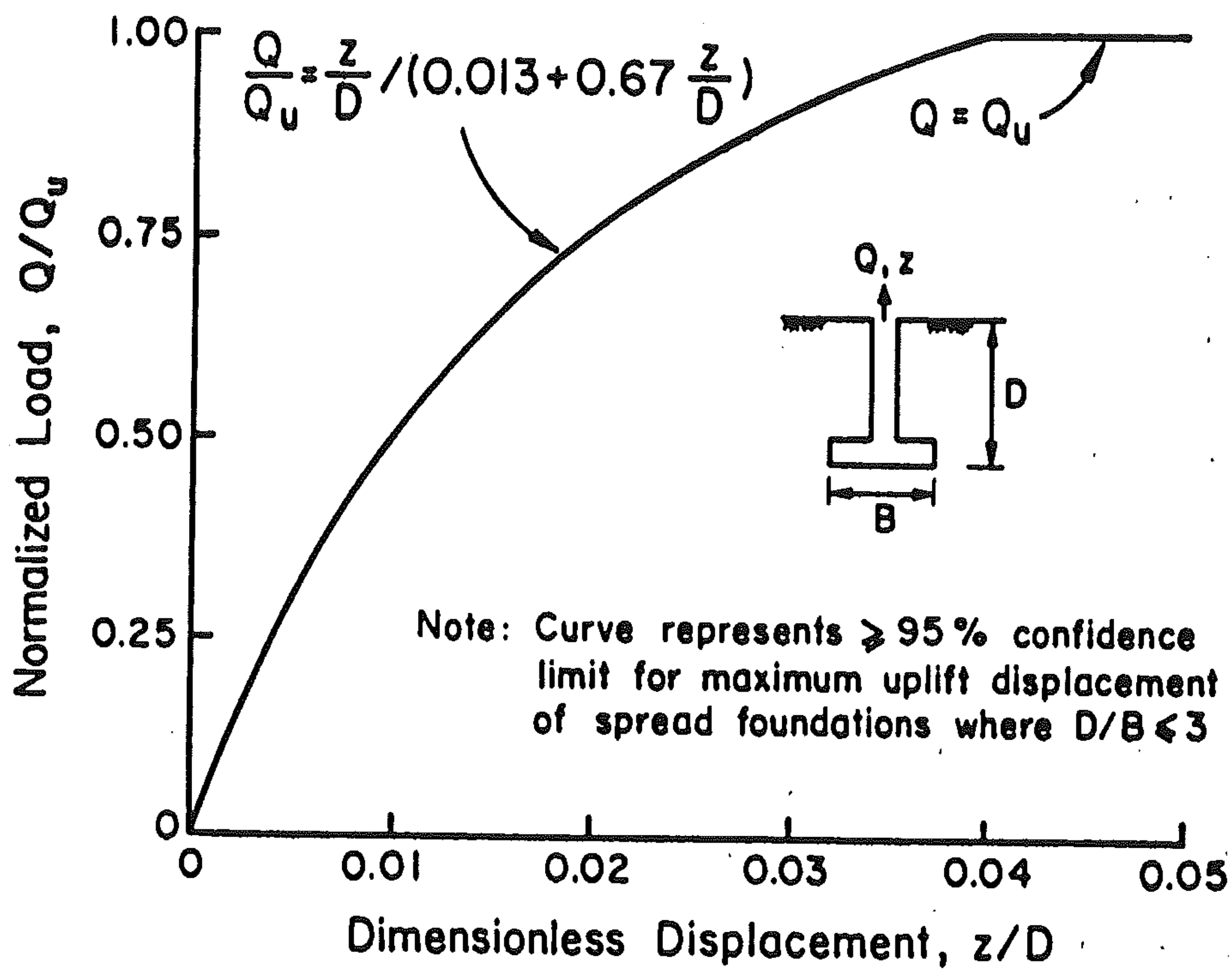


Figure 7.1 Normalized Load-Displacement Behavior of Shallow Foundations in Uplift (from Trautman and Kulhawy 1988).

The value of Q/Q_{ult} is essentially the inverse of the uplift bearing capacity factor of safety (FS) i.e., 1/FS. Therefore, in order to predict the deformation at any given load, the load at failure must be evaluated and the active soil contributing to soil displacement (in this case the depth of burial) must be identified. Since most shallow foundations resting on granular soils would be expected to have a relatively high design factor of safety i.e., generally > 3 and perhaps as high as 5, this means that displacements would be in the lower approximately linear portion of the curve in Figure 7.1. Use of this nondimensional relationship between normalized load and normalized displacement was presented by Kulhawy and Stewart (1994) for additional grillage foundations subjected to uplift loading.

Meyerhof and Murdock (1953) illustrated that results of deep plate load tests and load/settlement behavior of bored and driven piles in London clay could be expressed in nondimensional normalized form as illustrated in Figures 7.2 and 7.3.

Burland et al. (1966) further demonstrated that the results of plate bearing tests in stiff clays could be expressed in nondimensional form as shown in Figure 7.4. They suggested that this was consistent with laboratory triaxial compression tests on normally consolidated clay in which the results of tests performed under different confining stresses produces a unique stress-strain curve when strain is expressed in terms of q/q_{ult}. They further noted that up to a value of $q/q_{ult} = 1/3$ (i.e. F.S. = 3) the normalized settlement was essentially linear so that the relationship between settlement and load could be expressed as:

$$s/B = K(q/q_{ult}) \qquad [7.2]$$

where:

s = settlement
B = width
K = constant of proportionality

For a number of plate bearing tests at various sites in London, values of K in Equation 7.2 were remarkably similar and only varied between about 0.009 and 0.02.

A similar approach has been used to describe the load deformation response of deep foundations, both driven and drilled, subjected to uplift loading in granular soils by Tucker (1987). For example, results of full-scale pullout load tests to failure on drilled piers installed in granular materials are presented in Figure 7.5. In this case, the vertical deplacements were not normalized, however, this might be done with respect to the depth of embedment, D_1 to see if a single curve resulted. As indicated in Figure 7.5, in this case the exact shape of the curve appears to be directly related to the D/B ratio.

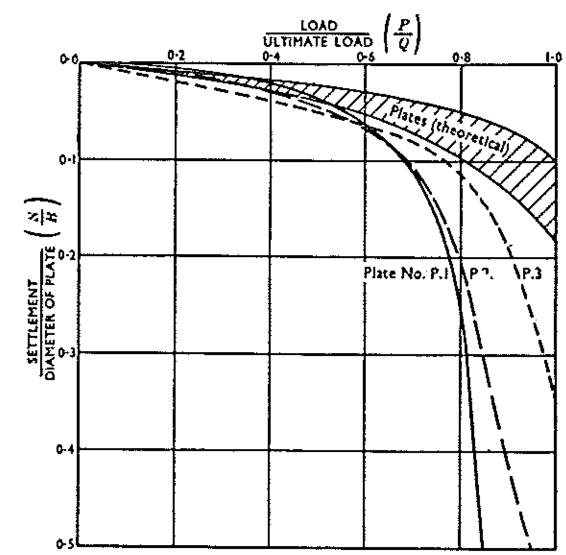


Figure 7.2 Nondimensional Load - Settlement Behavior of Plates (from Meyerhof and Murdock 1953).

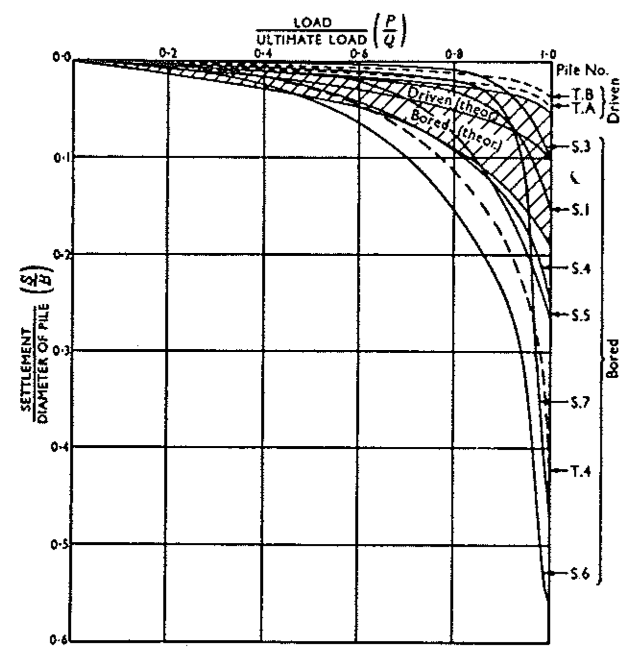


Figure 7.3 Nondimensional Load-Settlement Behavior (from Meyerhof and Murdock 1953).

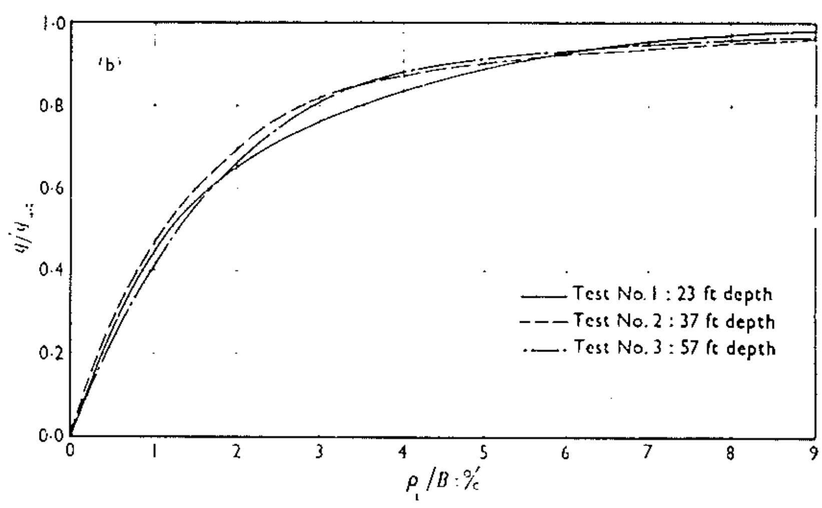


Figure 7.4 Nondimensional Load-Settlement Behavior (from Burland et al. 1966).

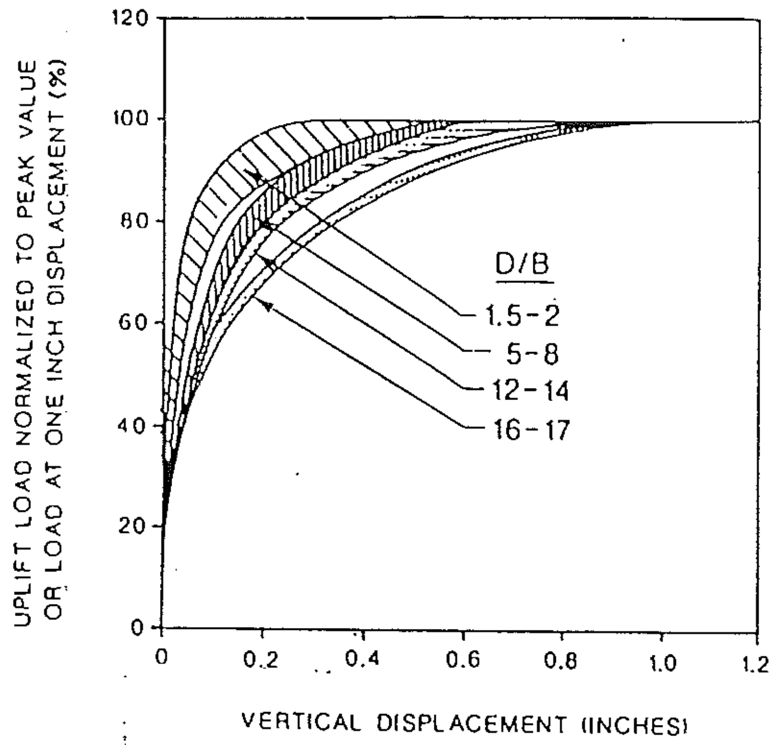


Figure 7.5 Normalized Load vs. Displacement Behavior of Deep Foundations in Uplift (from Tucker, 1987).

Nondimensional results obtained from uplift tests conducted on drilled shafts in sands and gravels have been presented by Rollins et al. (1994) and are shown in Figure 7.6. Additional examples of the use of nondimensional curves to describe the load/deformation behavior of deep foundations have been presented by Moore and LoKuratna (1987). Moore and LoKuratna (1987) demonstrated for small model footing tests loaded in compression, a nondimensional plot provided a linear relationship between settlement and load which appeared to be independent of plate size.

Normalized load-displacement curves have also been used to describe the behavior of drilled shafts in compression (e.g. Matsui 1993) as shown in Figure 7.7. The end bearing behavior of drilled shafts in granular soils has also been described using normalized load-displacement curves by Ghionna, et al. (1993) and Ghionna et al. (1994). As shown in Figures 7.8 and 7.9, the relative displacement needed to mobilize complete failure is relatively high, probably as a result of loosening of the sand during construction.

7.2 Load-Settlement Response of Footing on Sand

The load settlement behavior of shallow footings on sands is predominantly nonlinear throughout the entire range of applied loads up to and beyond the ultimate load or bearing stress, Q_u. This behavior is illustrated in Figure 7.10. Vesic (1973) suggested that for a given sand deposit and a given footing size the load settlement behavior could be described by a single curve provided that the footing displacement and applied load normalized to give a dimensionless curve as shown in Figure 7.11. Assuming that the footing width in some way reflects a function of the zone on influence beneath the footing involved in the deformation, the settlement for any load could be normalized with respect to the footing width. Similarly, the load, Q, could be normalized with respect to the ultimate load or the failure, Q_u. The results presented by Vesic (1973) suggested that the relationship between normalized load and normalized settlement was independent of the relative density of the sand. This is because different relative density produce different ultimate loads and correspondingly different deformation curves. Unfortunately, the data presented by Vesic (1973) was for only three tests performed on small model footings.

The ultimate bearing capacity of shallow foundations on a granular soil may be obtained using traditional bearing capacity theory presented by Terzaghi or by one of a number of modifications suggested by Meyerhof. For a centrally vertically loaded footing resting some distance below the ground surface the ultimate load may be determined as:

$$Q_{ult} = cN_c + 1/2\gamma BN_\gamma + D_f\gamma N_q \qquad [7.3]$$

where:
c = cohesion
γ = soil unit weight
B = footing width
D_f = footing depth
N_c, N_γ, N_q = bearing capacity factors

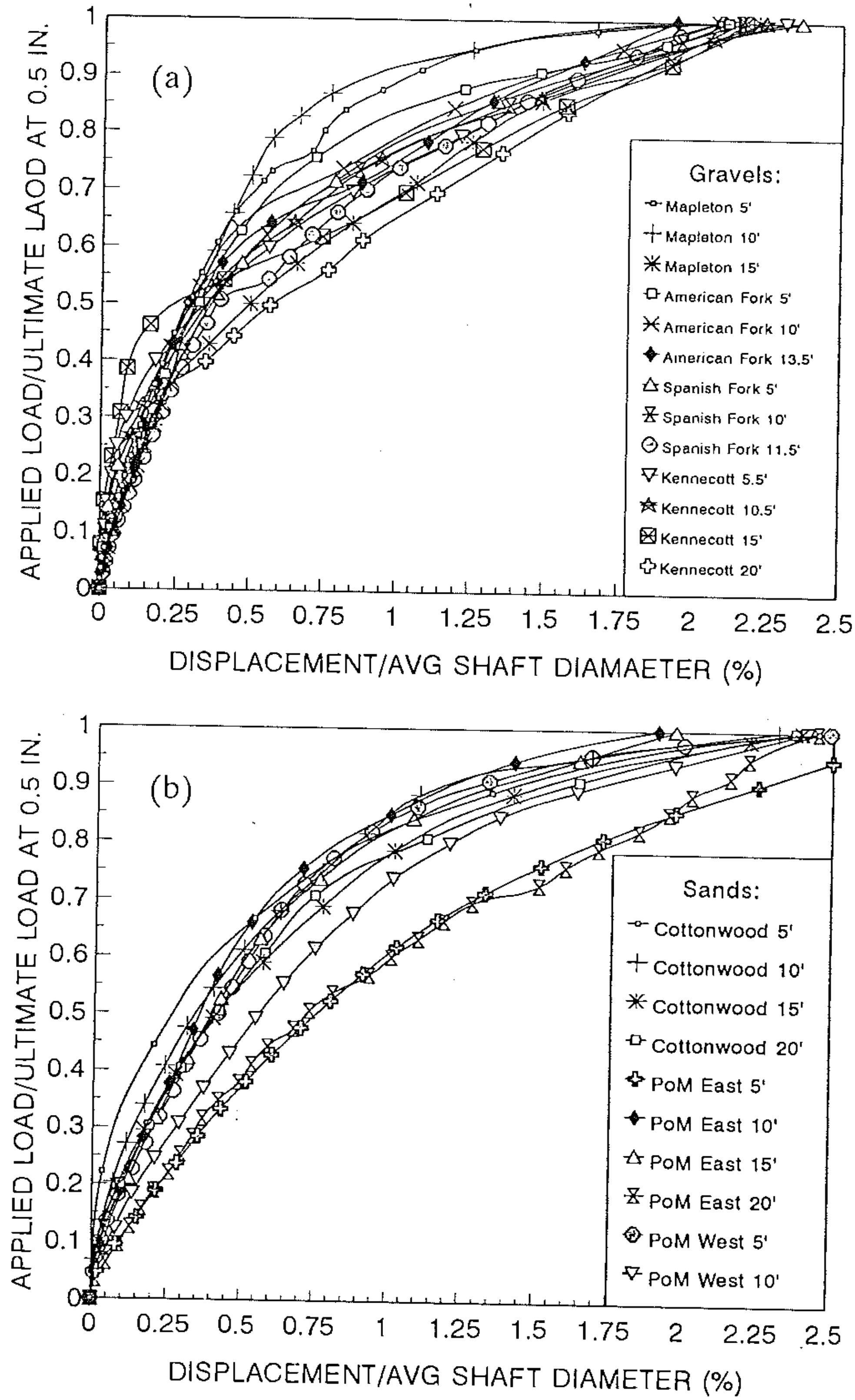


Figure 7.6 Normalized Load vs. Displacement Behavior of Drilled Shafts in Granular Soils (from Rollins et al. 1994).

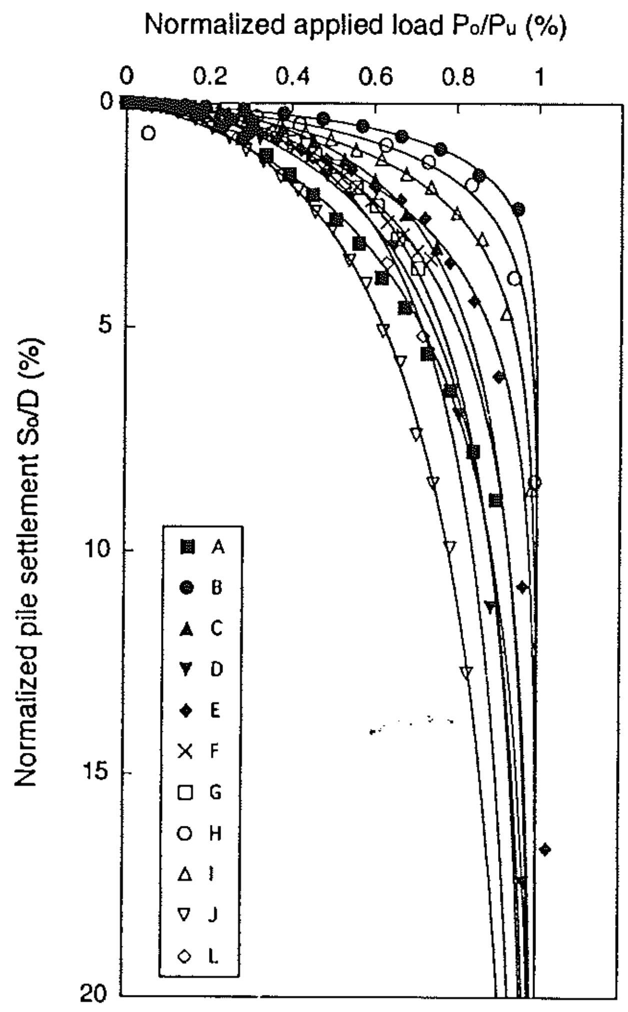


Figure 7.7 Normalized Load-Settlement Behavior of Drilled Shafts (after Matsui 1993).

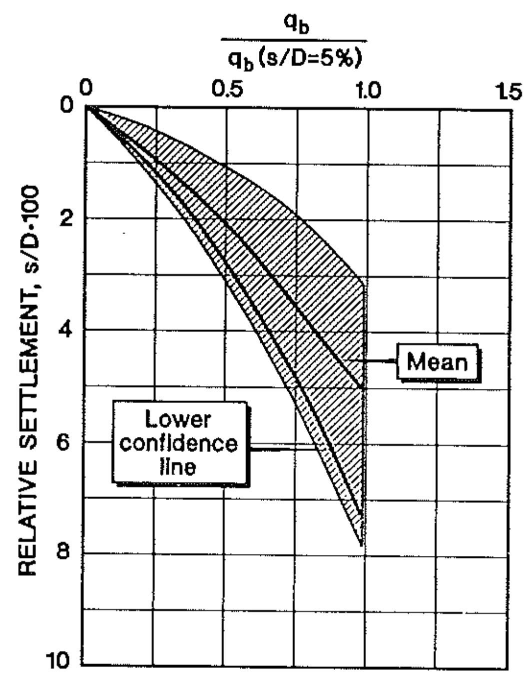


Figure 7.8 Normalized Load-Settlement Behavior of End Resistance of Drilled Shafts (Ghionna et al. 1993).

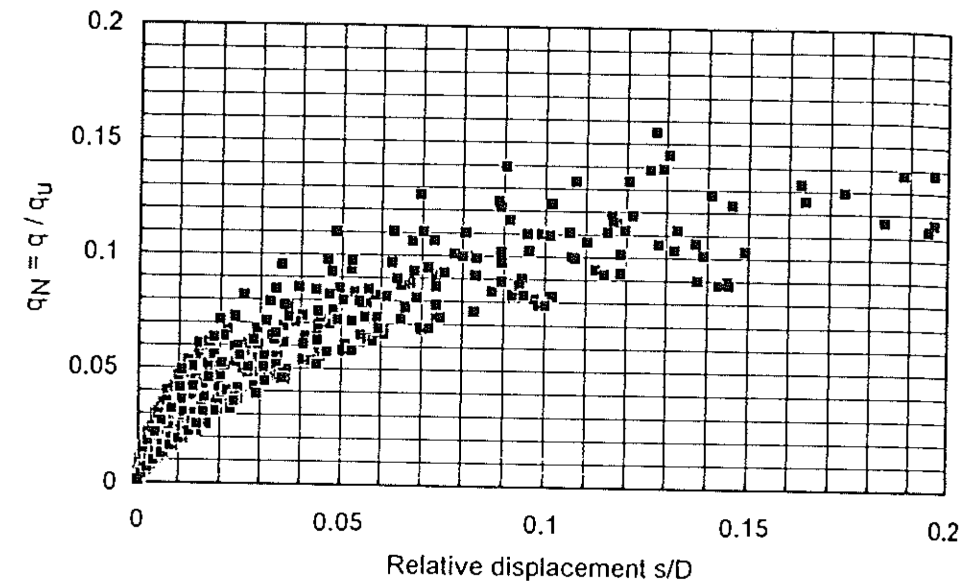


FIG. 8. Variation of q_N with Relative Displacement s_R

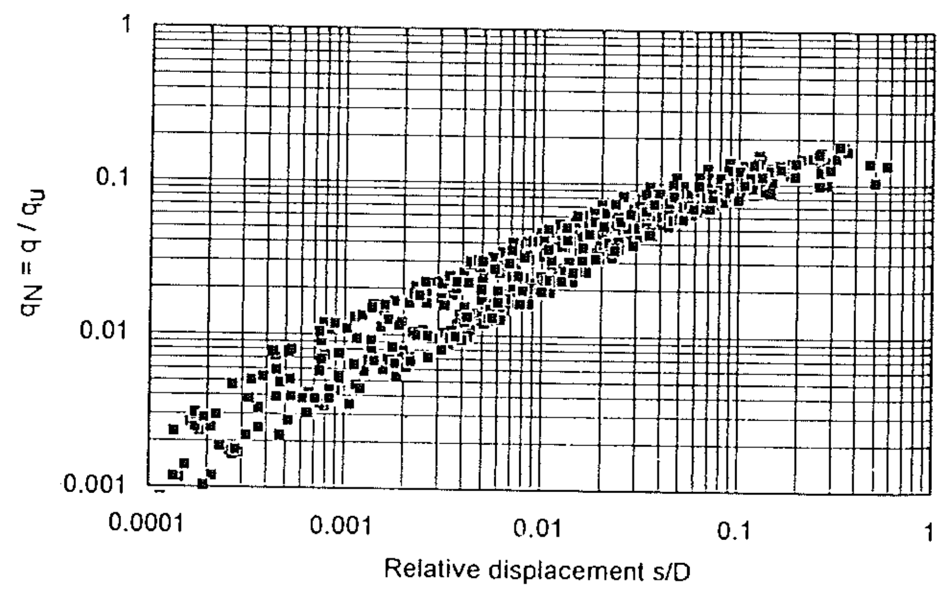


Figure 7.9 Normalized Tip Resistance of Drilled Shafts in Sand (Ghionna et al. 1994).

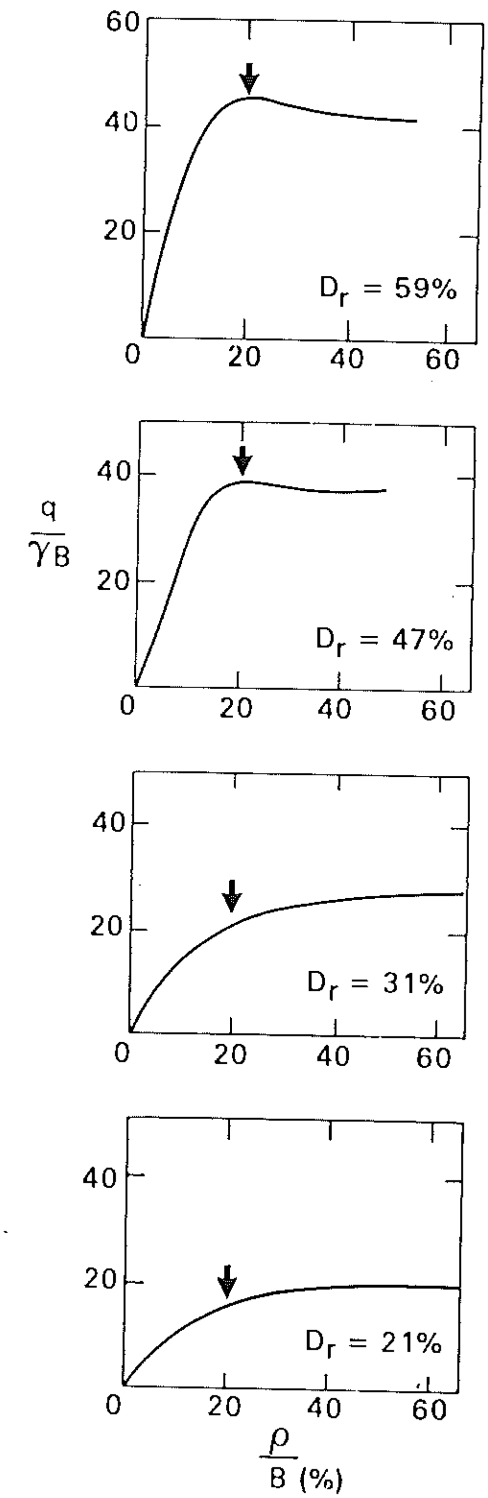


Figure 7.10 Settlement of Shallow Foundations on Sand (after Vesic 1973).

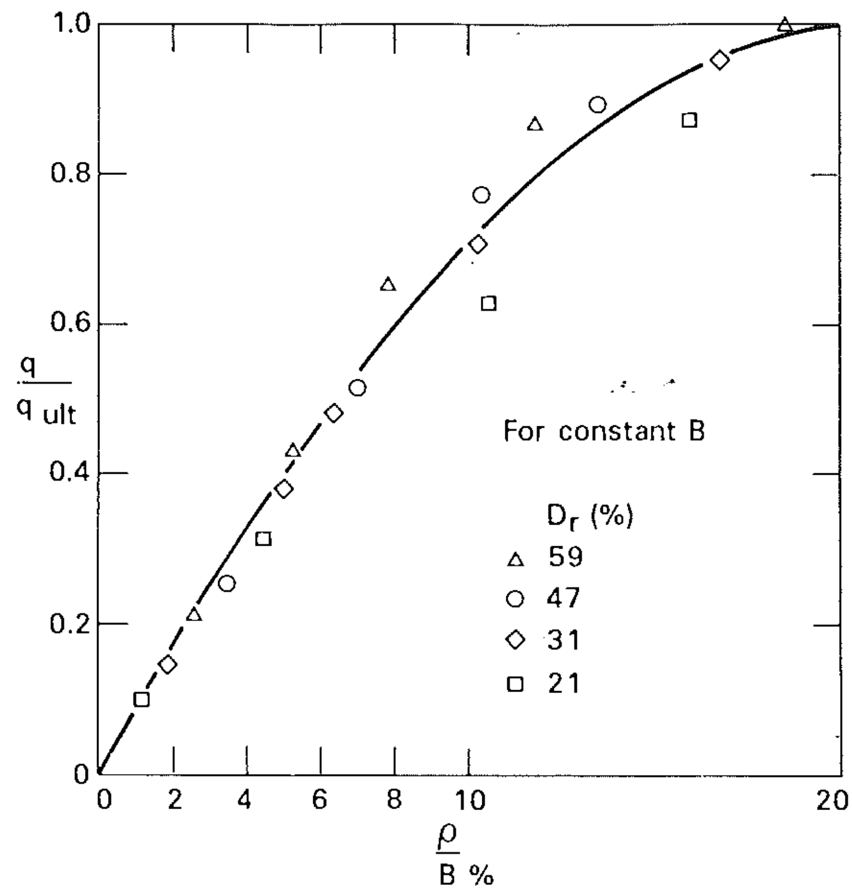


Figure 7.11 Normalized Load-Displacement Behavior of Shalow Foundations on Sand (after Vesic 1973).

The bearing capacity factors are a function of the soil friction angle only. The cohesion term is usually taken as zero which is a conservative approach and generally assumes that the soil is clean and free of any fine material, cementation, or apparent cohesion from moisture which would provide some component of strength at zero normal stress. For saturated clean sands this assumption is probably valid, however for partially saturated sands, especially with an appreciable amount of fines, this assumption provides a conservative estimate of Q_u.

Results of load tests may be evaluated to determine the failure load and displacement characteristics. A number of methods may be used to interpret the failure load, however, Trautmann and Kulhawy (1988) found that the "slope tangent" method worked reasonably well to determine the ultimate load of shallow foundations subjected to uplift load. As shown in Figure 7.12 in this method the failure load is taken as the intersection of the tangents to the initial and final portion of the load-displacement curve and the failure displacement is taken as that corresponding to the failure load. Clearly, this method is somewhat dependent on the range of displacements achieved in the test since the shape of the curve never really approaches an asymptotic valve even at large displacements. In general, provided that a total displacement is in excess of 5% of the footing width, this method should provide reasonable estimates for granular soils. The value given by this method may not be the actual "ultimate" load representing complete plastic failure, but it can be considered a "failure" load.

In the case of a shallow foundation in uplift, the displacement may be normalized by the depth of burial which represents the entire zone of soil participating in the foundation movement. However in the case of a footing under an applied compressive load, the zone of influence contributing to deformation is not actually known and hence, another length term must be chosen. It is reasonable to expect that the width of the footing may be used to reflect in some fashion the zone of influence and therefore the settlements may be normalized by footing width.

Using results of laboratory model footing tests, prototype footing tests and a few tests performed on full-scale footings available in the literature, the load-displacement curves of shallow foundations under axial compression loading may be reevaluated.

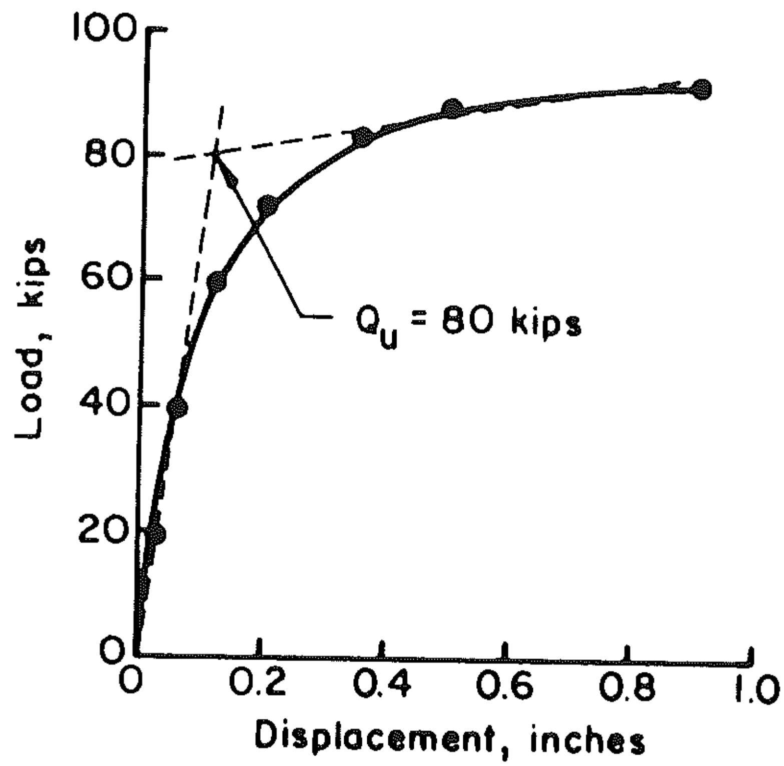


Figure 7.12 Slope Tangent Intersection Method of Determining Failure Load.

7.3 Review of Load Tests

Initially, data were gathered from the literature representing reported case histories of load/settlement behavior of shallow foundations in which sufficient results were provided to describe the ultimate bearing capacity from the actual test results, using the slope tangent method, or in which sufficient soil properties were given to allow the ultimate bearing capacity to be calculated. These cases were then subdivided into four categories based on footing size as: (1) small scale laboratory load tests ($B \leq 0.305$m); (2) small footing tests ($B < 0.5$m); (3) medium footing tests ($0.5\text{m} < B < 1.5\text{m}$); and (4) large footing tests ($B > 1.5$m). Unfortunately, there was not the same number of tests available for each category and in fact, for large footing tests, there was a clear lack of high quality tests available. Tables 7.1 to 7.4 summarize the case histories used in each category to develop the nondimensional curves. The resulting curves are presented in Figures 7.13 to 7.16.

The results presented in Figures 7.13 to 7.16 suggest that there is a definite scale effect with progressively larger size footings producing smaller relative settlements at the same relative loading levels. An upper bound curve is shown on each curve which describes the maximum observed settlement behavior from all test cases and provides a conservative upper bound limit of estimated settlement.

To illustrate the use of this approach, the observed settlements of a number of bridge abutments as reported by Gifford et al. (1987) will be used. The bearing capacity of several of these abutments was evaluated by Briaud (1989) using the results of prebored pressuremeter tests. Since the working loads on the abutments and the foundation widths are provided by Gifford et al. (1987), the relative settlement may be determined. Table 7.5 presents results of the calculated factors of safety and relative settlement for cases where sufficient data are available. The average factor of safety under working loads for the 5 abutments is 6.9 corresponding to $q/q_{ult} = 0.14$. The average relative settlement based on observations is 0.30. Based on Figure 7.16, the predicted relative settlement for this level of loading would be on the order of 0.10. While this estimate of settlement is obviously lower than the observed settlements for these abutments, adjustments to the predicted settlement for creep should be made and it should be remembered that the estimate of ultimate bearing capacity obtained from the PMT is usually much larger than by conventional means and therefore the value of q/q_{ult} is too low. This in turn makes the estimate of S/B too low.

Table 7.1 Normalized Load-Settlement Analysis - Model Test Footings

Reference	Test Conditions	Other Information
DeBeer (1970)	• Dry quartz sand • D = 15 cm, $D_f = 0$ • Lab Test • Given Q_{ult}	D_r = 32.7, 65.0, 67.8, 82.5, 84.8, 86.5,87.5, and 89.0%
Siraj-Eldine and Bottero (1987)	• B = 5cm, L/B = 1, $D_f = 0$ • Dry unit wt. = 15.8 kN/m^3 • Lab Test • Found Q_{ult} (log Q vs. log S)	H/B = 0.50, 0.75, 1.00, and 1.80
	• B = 5 cm, L/B = 1, $D_f = 0$ • Dry unit wt. = 16.7kN/m^3 • Lab Test • Found Q_{ult} (sharp break in loading curve)	H/B = 1 and 2
Cooke (1988)	• Diameter = 0.305m • Loose Sand • Lab Test • Q_{ult} Given (Meyerhof)	H/D = 0.50, 0.75, 1.00, 1.50, 2.00, and 3.00
	• Diameter = 0.305m • Dense Sand • Lab Test • Q_{ult} Given (Meyerhof)	H/D = 1, 2, and 3
Poulos et al. (1994)	• Compressible calcareous sand • Dry unit wt. = 1.47 t/m^3, D_r = 47% • D = 2.5 cm, $D_f = 0$ • Lab Test • Found Q_{ult} (log Q vs. log S)	σ'_{vo} = 35, 69, 103, 103, 138, 172, and 207 kPa

Table 7.2 Normalized Load-Settlement Analysis - Small Footings

Reference	Test Conditions	Other Information
Meigh and Nixon (1961)	• Site - Motherwell, Lanarkshire • Fine, Silty Glacial Sand • SPT $(N)_{60} = 65$ • $D_f = 1.22$m, $D_w = 6.10$m • Field Load Test • Q_{ult} Calculated (Stroud)	B = L = 0.31, 0.31, 0.31, 0.31, 0.46, and 0.46 m
	• Site - Sizewell, Suffolk • Medium Dense, Fine to Medium Sand • Field Load Test • Q_{ult} Calculated (Stroud)	B = L = 0.31 m, D_f = 3.1 m B = L = 0.31 m, D_f = 7.6 m
Rodin (1961)	• Site One - Coarse Sand and Gravel • Diameter = 0.305 m • Field Load Test • Q_{ult} Given	D_f = 3.20, 3.96, 4.72, 5.49, and 6.40 m
	• Site Two - Gravel • Diameter = 0.46 m • Field Load Test • Q_{ult} Given	D_f = 4.72 and 5.94 m
Deb (1963)	• Loose Sand Deposit • N avg. = 5 • $D_f = 0$ • Field Load Test • Found Q_{ult} (log Q vs. log S)	Diameter = 0.152, 0.229, 0.305, and 0.457 m
Ismael (1985)	• Well-graded cohesionless Sand • SPT N = 20, $D_f = 1.0$ m, $D_w = 2.8$ m • Wet Density = 1.625 Mg/m^3 • Field Load Test • Q_{ult} Calculated (Stroud)	B = L = 0.25 m B = L = 0.50 m
	• Diameter = 0.5 m • Tests performed at different sites in Kuwait • Field Load Test • Q_{ult} Calculated (Stroud)	SPT N = 30, D_f = 1.0 m SPT N = 25, D_f = 1.0 m SPT N = 15, D_f = 1.0 m SPT N = 12, D_f = 1.0 m SPT N = 10, D_f = 2.6 m SPT N = 10, D_f = 2.0 m SPT N = 20, D_f = 2.8 m

Table 7.2 Cont'd

Reference	Test Conditions	Other Information
Kusakabe et al. (1992)	• Dense, granular, volcanic material • SPT N > 70 • $D_w = 7$ m • Field Load Tests • Tests performed within a Caisson's Chamber • Found Q_{ult} (log Q vs. log S) • Failure due to particle crushing	B = L = 0.4 m, D_f = 23.5 m (I) B = 0.4m, L=1.2m, D_f =23.5m (II) B = 0.4m, L=2.0m, D_f=23.5m (III) B = L = 0.3 m, D_f = 23.5 m (VIs) B = L = 0.3 m, D_f = 19.5 m (VIIs) B = L = 0.3 m, D_f = 27 m (VIIIs)
FHWA Unpublished Data (1994)	• ϕ (BST) =28.8° • Dry Unit Weight = 94 pcf • Field Load Tests • Found Q_{ult} (log Q vs. log S)	B=L=0.31m,D_w=0.31m (1195W) B=L=0.31m,D_w=0.15m (1295W) B=L=0.46m,D_w=0.31m (15195W) B=L=0.46m,D_w=0.31m (15295W) B=L=0.31m,D_f=D_w=0 B=L=0.31m,D_f=D_w=0.31m B=L=0.31m,D_f=0.31m,D_w>2B

Table 7.3 Normalized Load - Settlement Analysis - Medium Footings

Reference	Test Conditions	Other Information
Deb (1963)	• Loose Sand Deposit • N avg. = 5 • $D_f = 0$ • Field Load Test • Found Q_{ult} (log Q vs. log S)	D = 0.610 and 0.762 m
Ismael (1985)	• Well-graded cohesionless Sand • SPT N = 20, D_f = 1.0 m, D_w = 2.8 m • Wet Density = 1.625 Mg/m^3 • Field Load Test • Q_{ult} Calculated (Stroud)	B = L = 0.75 and 1.00 m
Shiraishi (1990)	• Dense Sand • SPT N = 49 • $\phi = 42°$ • D_f = 24 m, D_w = 15.2 m • Field Load Test • Q_{ult} given	Performed one test, diameter = 1.2 m
Kusakabe (1991)	• Dense, granular, volcanic material • SPT N > 70 • D_w = 7 m • Field Load Tests • Tests performed within a Caisson's Chamber • Found Q_{ult} (log Q vs. log S) • Failure due to particle crushing	B=L=0.7m, D_f= 19.5m (IV) B=L = 1.3m, D_f = 27m (IV)

Table 7.3 (continued)

Reference	Test Conditions	Other Information
Briaud and Gibbens (1994)	• Medium dense silty sand • D_r = 55%, D_f = 4.5 m • Very hard shale at D = 11 m • Field Load Test • Found Q_{ult} (log Q vs. log S)	B = L = 1.0 m
FHWA Unpublished Data (1994)	• ϕ (BST) = 28.8° • Dry Unit Weight = 94 pcf • Field Load Tests • Found Q_{ult} (log Q vs. log S)	B=L=0.61m,D_w=0.31m (2195W) B=L=0.61m,D_w=0.23m (2295W) B=L=0.92m,D_w=0.31m (3195W) B=L=0.61m,D_f=0 m B=L=0.92m,D_f=0 B=L=0.61m,D_f=0.61m B=L=0.92m,D_f=0.92m B=L=0.61m,D_f=0.61m,D_w>2B B=L=0.92m,D_f=0.92m,D_w>2B

Table 7.4 Normalized Load - Settlement Analysis - Large Footings

Reference	Test Conditions	Other Information
Lockett (1992)	• Two load tests performed • Field Load Test • Q_{ult} given	B = L = 3.66 m
Briaud and Gibbens (1994)	• Medium dense silty sand • D_r = 55%, D_f = 4.5 m • Very hard shale at D = 11 m • Field Load Test • Found Q_{ult} (log Q vs. log S)	B = L = 1.5, 2.5, 3.0, and 3.0 m
Sutterer (1994)	• assumed $\phi = 27°$ • D_f = 1.07 m, D_w > 2B • Field Load Tests • Q_{ult} given	B = L = 1.98 and 2.74 m

Table 7.5 Relative Settlement of Bridge Abutments

Bridge No.	Abutment No.	F.S.	B (m)	S (mm)	S/B (%)	q/q_{ult}
2 Cheshire	1	5.2	5.18	23.9	0.46	0.19
3 Providence	West	7.5	3.35	10.7	0.32	0.13
7 Manchester	2 East	6.2	8.53	16.3	0.19	0.16
8 Manchester	2 East	6.1	6.10	16.8	0.28	0.16
9 Manchester	1 West	9.5	6.63	15.5	0.23	0.11

Data from Gifford et al. (1987)

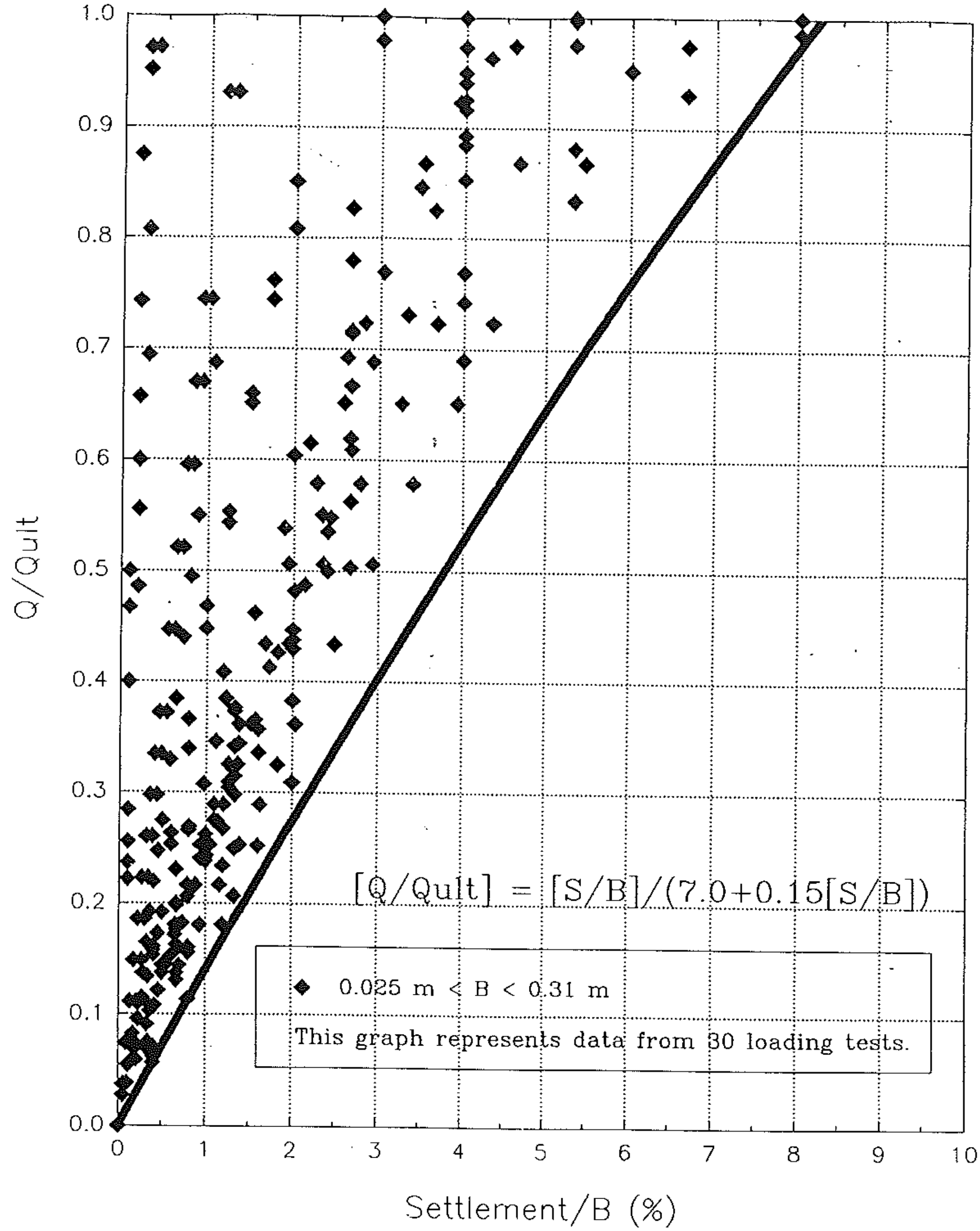


Figure 7.13 Normalized Load-Settlement Data for Small Scale Laboratory Load Tests.

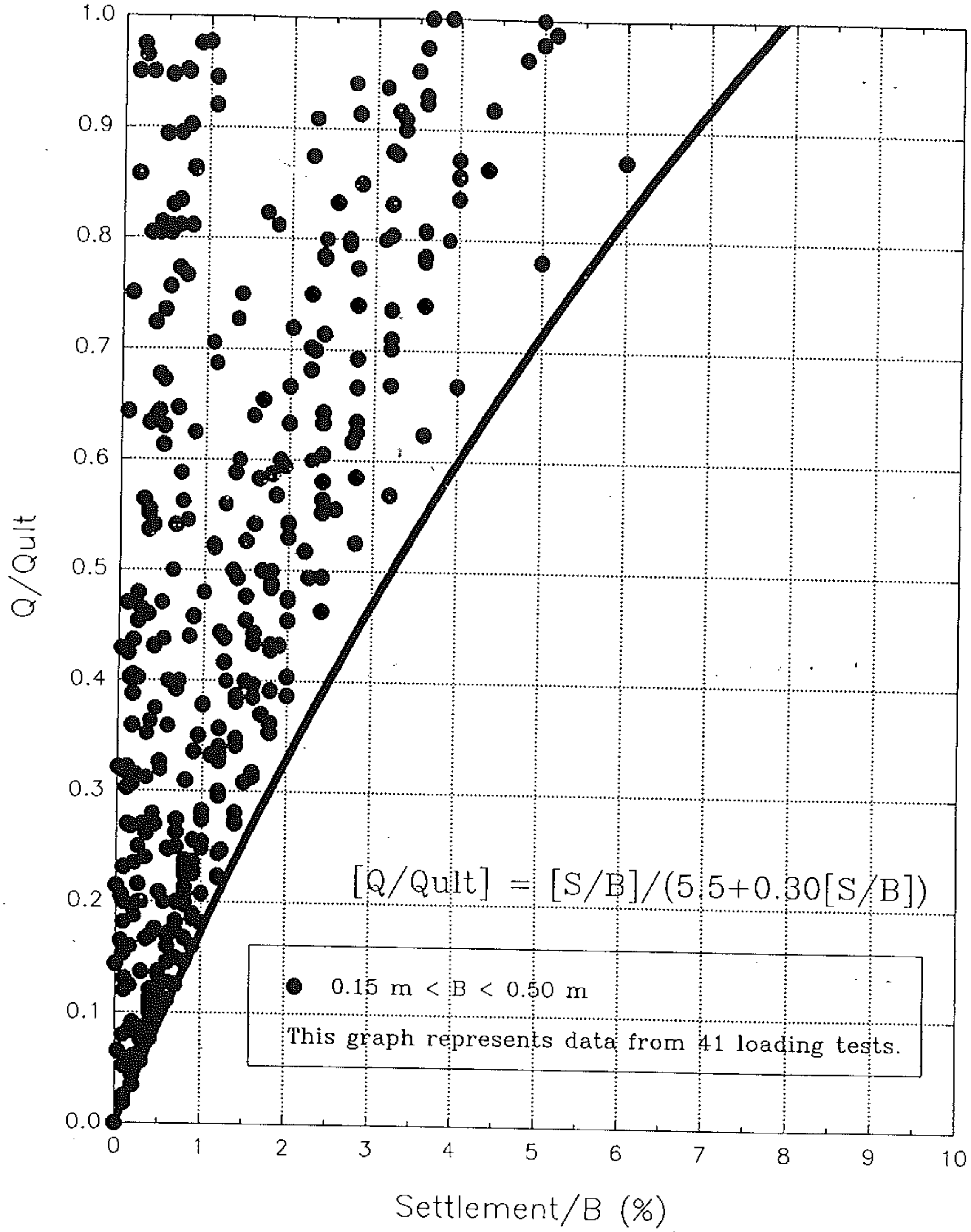


Figure 7.14 Normalized Load-Settlement Data for Small Footing Load Tests.

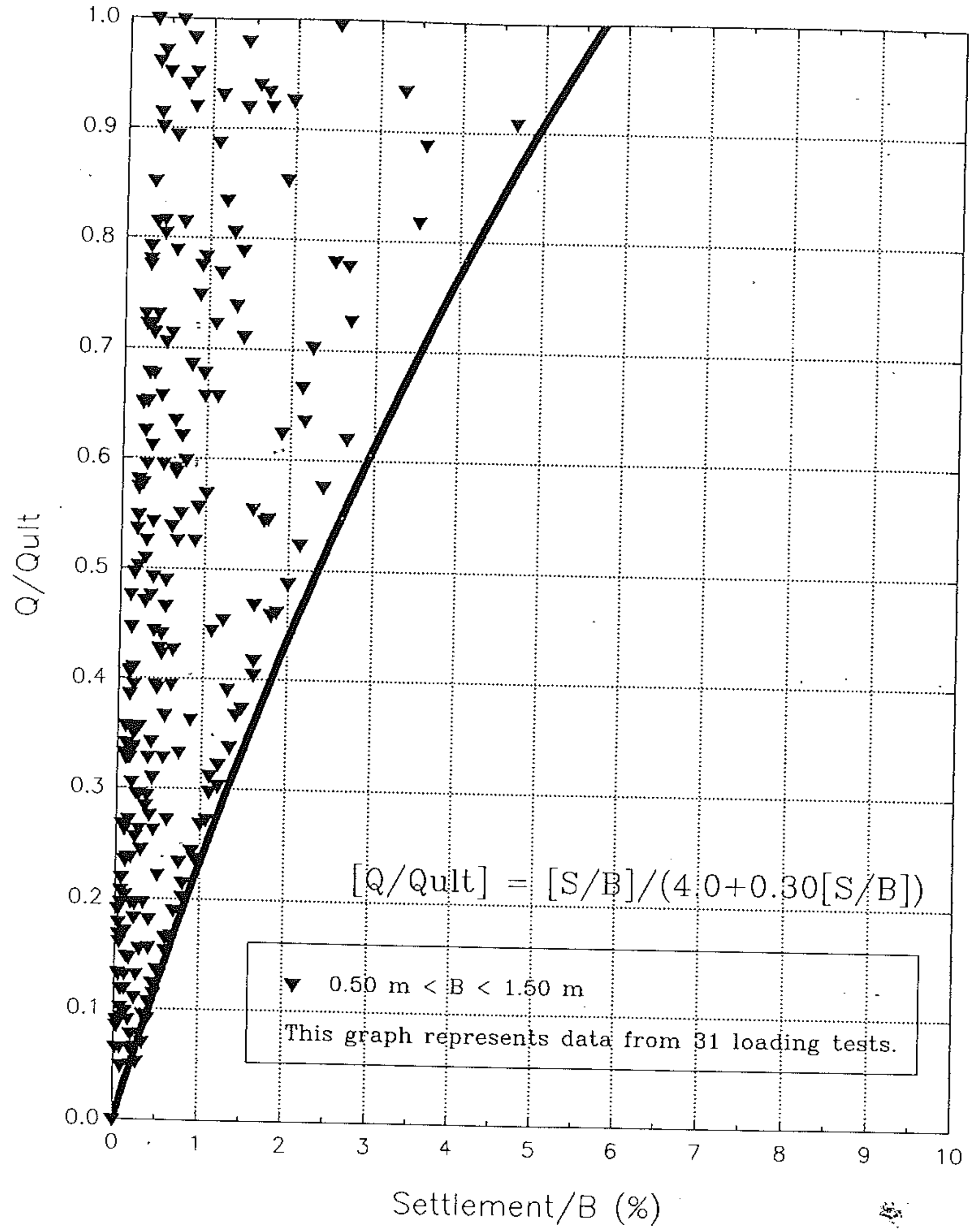


Figure 7.15 Normalized Load-Settlement Data for Medium Footing Load Tests.

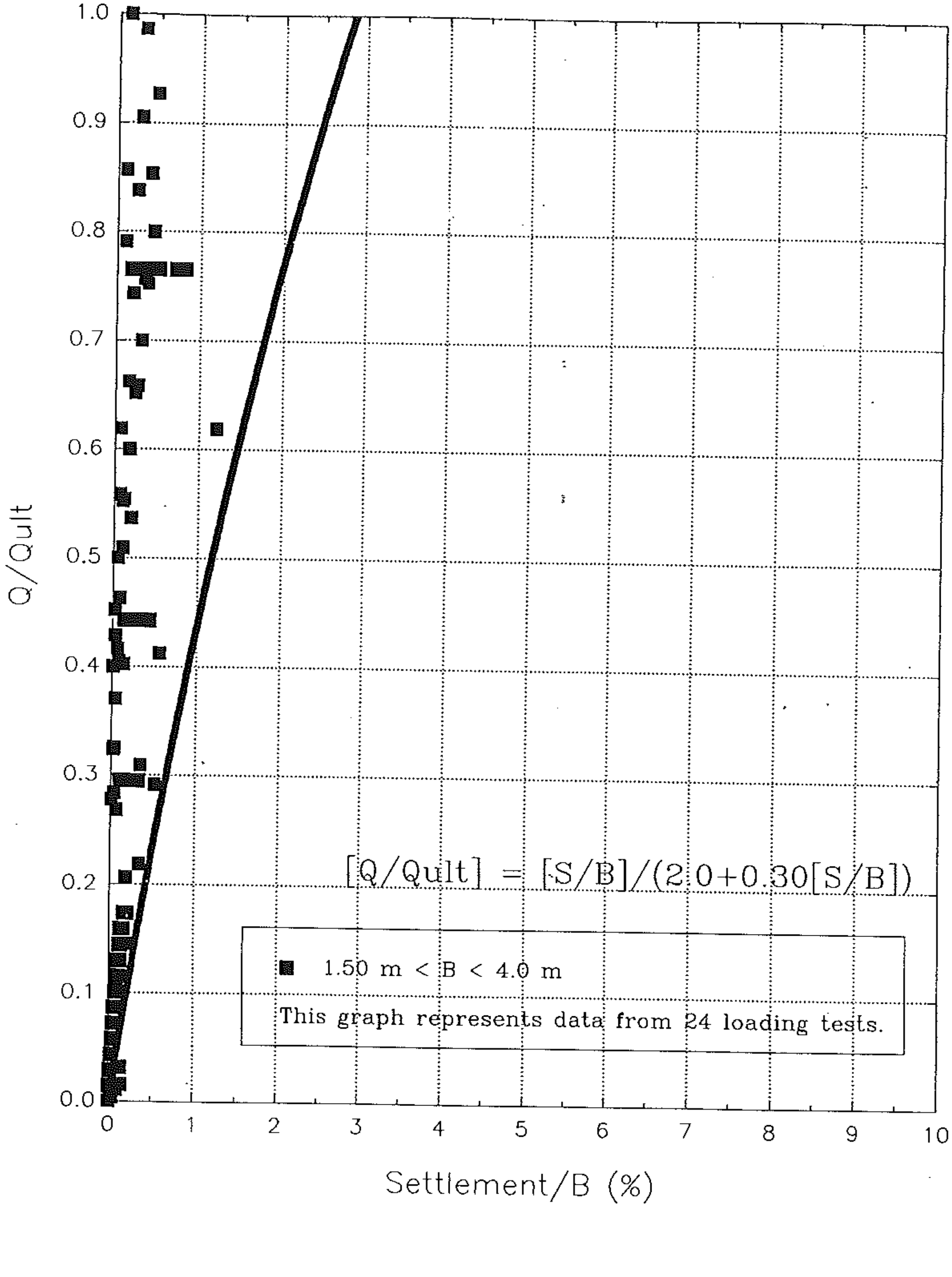


Figure 7.16 Normalized Load-Settlement Data for Large Footing Load Tests.

8.0 SETTLEMENTS FROM CYCLIC LOADING

Although it was not an integral part of this project, there were some questions which arose during the course of the work regarding the additional settlement which might occur in a shallow foundation resulting from cyclic loading and/or dynamic effects of earthquakes. While no detailed review of this subject was performed for this project, a number of references were identified throughout the course of the project relating to these issues. These references are identified in this section and include references related to both seismic loading and repetitive loading on shallow foundations. The interested reader should refer to specific references for more detailed information on calculating settlements resulting from earthquakes or the effects of repeated loading.

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9.0 RECOMMENDATIONS

Based on the results of the extensive and exhaustive review of previous work performed in the area of estimating settlement of shallow foundations on granular soils deposits it is obvious that a number of significant improvements have been made in the last ten years on the understanding of the interaction between foundations and granular soil deposits. It appears that there are two primary sources of error associated with the ability to accurately predict the settlement performance of foundations resting on granular soil deposits. These are: (1) lack of a complete site investigation to provide sufficient test results on the nature and variability of granular deposits at a particular site; and (2) the use of simplistic, empirical and generally outdated methods of analysis which tend to give erratic results which are not of a sufficiently general nature and generally do not recognize the important factors contributing to the deformation characteristics of granular soils under foundation stresses imposed by shallow foundations.

As a result of this work, a number of recommendations are suggested to improve the approach of MHD to predicting the settlement of transportation related structures, (e.g., bridge piers and abutments) supported by shallow foundations on granular deposits.

9.1 Improvements in Site Characterization

A major attempt must be made to improve the scope of site investigations for proposed bridge structures in which shallow foundations may be a viable option to significantly more expensive deep foundations. The potential cost savings associated with using a shallow foundation system will in most cases significantly offset the additional expense associated with an increase in effort during the site investigation. The number of test boring or test locations and the frequency of field tests performed within the zone of most significant influence of the foundation needs to be increased substantially (on the order of double the current amount). Additionally, the type of field tests performed needs to be modified. In light of recent advances in the availability of various in situ tests that may be used to improve the quality of settlement predictions, the use of tests such as the pressuremeter and plate load test should be implemented as routine tools to supplement the more conventional approach using test boring with Standard Penetration Tests. Detailed recommendations regarding the various in situ tests used in practice are given in the Appendices of this report.

The use of the CPT is not considered a particularly significant advantage in the approach to a site investigation, provided that the SPT practice used by all contractors for MHD can be standardized and rigidly enforced. The only advantage to using the CPT is to provide a more continuous sounding, however it is felt that in this case this would only be achieved at considerable expense of equipment and manpower to the state and could actually create a slowdown in data reduction and interpretation for typical projects. Additionally, there may be significant problems associated with deploying the CPT in typical granular deposits located throughout the state.

The use of the Drive Cone Test to supplement the SPT is however, considered an appropriate test to use to help rapidly and economically identify site variability. As with the SPT, the DCT test

results are available immediately at the end of the test boring and may be used immediately in the analysis. The test is considered applicable for the majority of granular soil deposits in the Commonwealth.

A typical site investigation at a single bridge pier or abutment location at a site might be as follows:

1- Four (4) SPT boring with continuous SPT's conducted throughout the anticipated zone of influence of the foundation.

2- Four (4) DCT profiles conducted continuously from the ground surface to the anticipated depth of influence of the foundation.

3- Two (2) pressuremeter borings with six to eight (6 to 8) PMT's performed in each boring;
or
Two (2) plate load tests conducted on a 1 ft. square rigid plate at the anticipated depth of the foundation.

This proposed site investigation appears to represent a significant commitment of time, resources, and cost, however, it is estimated that this work could be performed in two working days with a crew of two drillers (driller & helper) and two technicians. There is no doubt that such an investigation represents an increase in overall cost of the site investigation however, as previously indicated the overall cost savings, averaged out over a complete project, is considered to be siginficant.

9.2 Improvements in Settlement Analysis

Serious consideration should be given to adapting more advanced and appropriate methods of predicting settlements based on the discussions presented in Section 5 of this report. Preference should be given to those techniques which recognize the variation is stiffness parameters with increasing levels of stress and strain and which recognize that the deformation behavior of granular soil is intimately linked to the ultimate bearing capacity. Modifications to the elastic approach of calculating settlements appear to be the most desirable. Methods which are almost entirely based on empirical observations between penetration resistance and measured settlements have limited appeal and should generally be discontinued unless a study is undertaken to establish correlations for specific geologic deposits in a relatively limited geographic area.

The following settlement methods are generally considered to have the highest probability of success:

1- Stroud (1989) Method (SPT)
2- Wahls and Gupta (1994) Method (SPT)
3- Menard and Rousseau (1962) Method (PMT)

4- Martin (1987) Method (PMT)
5- Parry (1978) Method (PLT)

In addition, initial, preliminary estimates of settlement should be made using the upper-bond general load-settlement relationship developed as a part of this project and described in Section 7 for appropriately sized footings and by using the extrapolation approach summarized by Burland et al. (1977).

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