MOVEMENTS OF FOOTINGS AND RETAINING WALLS

by

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(ABSTRACT)

The objectives of this dissertation are: (1) to examine the relationship between the accuracy and reliability of methods of estimating settlements of footings on sand and gravel, (2) to develop a procedure for estimating horizontal movements and rotations of footings without the need of determining soil modulus values, and (3) to develop a simple procedure for calculating movements of retaining walls due to the weight of backfill.

The accuracy and reliability of twelve methods of estimating settlements of footings on sand and gravels were examined by comparing calculated settlements with the measured values. Eleven of the methods are based on Standard Penetration Test Results, while Schmertmann's method is based on Cone Penetration Test Results. The study showed that methods which are more accurate tend to underestimate settlements about half of the time; while those which are more reliable (in the sense that they infrequently underestimate settlements) tend to be less accurate.

The study also indicated that these methods of estimating settlements of footings on sands and gravels involve

approximately the same relationship between accuracy and reliability, regardless of the approach that they use to calculate settlement. The results demonstrate that there is a tradeoff between accuracy and reliability. Any of the methods can be adjusted to achieve approximately the same combination of accuracy and reliability as other method.

A simple procedure is presented to relate horizontal movements and rotations of footings to settlements. The procedure does not require the determination of soil modulus, and its accuracy and reliability can be assessed qualitatively by association with the method used to calculate the settlement.

A simple procedure based on elastic theory was also developed to estimate movements of abutments and retaining walls due to the weight of backfill placed behind them. To avoid the inherent difficulty in determining the soil modulus, a procedure for relating these movements to the settlement of the wall was also developed. The new procedure was applied to a case history, and the calculated movements agree quite well with those calculated using the finite element method, and with field observations.

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late father, who passed away during the course of my study in the United States.

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CHAPTER 1

INTRODUCTION

The design of footings must satisfy two requirements. First, the footings must be able to support the loads they carry safely. The foundations themselves must not suffer structural failure, and the soils beneath them must not be loaded so heavily that their supporting capacities are exceeded. Second, the settlements, horizontal movements and rotations of the footings must be small enough that the structures they support are not damaged by movements of the foundations, and that their functions are not impaired.

In practice, footing design is likely to be governed by consideration of limiting foundation movements (Meyerhof, 1965). Thus, the ability to predict movements of the foundation under service loading conditions and the ability to determine the amount of movements the structure can tolerate are important parts of the foundation design.

Settlements of footings on clay can be predicted with reasonable accuracy using conventional methods (MacDonald and Skempton, 1956; Skempton, Peck, and MacDonald, 1957; Skempton and Bjerrum, 1957; Butler, 1974). Settlements of footings on sand, however, are more widely variable and more difficult to predict accurately. Although many procedures are available, none of them is capable of predicting settlements of footings on sand with high accuracy.

Although structures may be damaged by excessive horizontal movements, little information concerning methods for estimating horizontal movements or their effects on structures. Horizontal movements are usually estimated using elastic theory, but accuraccy for this estimation is yet to be verified with field measurements.

This thesis has three primary objectives. The first is is to evaluate the accuracy and reliability of twelve methods of estimating settlements of footings on sand and gravel. The relationship between reliability and accuracy of the methods are examined closely, with the aim of developing a procedure that allows footing designers to understand the relationship between accuracy and reliability of settlement estimates.

The second objective of the thesis is to develop a procedure for estimating horizontal movements and rotations of footings without the need for evaluating soil modulus. In this new procedure, the magnitudes of horizontal movements and rotations are related to the settlements of footings. An advantage of this approach is that reliability and accuracy of the horizontal movement and rotation calculation can be assessed qualitatively through association with settlement estimates.

The third objective of the study is concerned with developing methods for estimating movements of retaining and abutment walls. For these structures, displacements and rotations are caused by compression of the foundation soils immediately below the footing, and also by deformation of soils beneath the backfill. A simple procedure based on elastic theory is developed to estimate movements of walls and abutments due to the loads on the footing and the weight of the backfill.

In Chapter 2, the characteristics of available methods for estimating settlements of footings on sand and gravel are reviewed. The accuracies and reliabilities of the twelve methods for estimating settlements of footings on sand are evaluated in Chapter 3. The proposed procedure for calculating movements of retaining and abutment walls is discussed in Chapter 4. In Chapter 5, a procedure for relating horizontal movements and rotations of footings and retaininig walls to settlements is presented. The procedures described in Chapters 4 and 5 are applied to a case history in Chapter 6. Finally, in Chapter 7, the ability of structures to tolerate movements without damage is discussed and summarized.

CHAPTER 2

GENERAL BACKGROUND AND PRESENT STATE OF KNOWLEDGE CONCERNING METHODS OF ESTIMATING FOUNDATION MOVEMENTS

2.1 INTRODUCTION

Although engineers had long recognized the problem of settlement, rational methods of settlement analyses were not available until the early nineteenth century, when results of field load tests were first used to evaluate deformation characteristics of foundations (Sowers, 1965). Shortly thereafter, Terzaghi (1923) published his theory of consolidation that marked the beginning of modern soil mechanics. Since then, the problem of settlement has received worldwide attention from geotechnical engineers and researchers, and has been a major theme for many international conferences. Table 2.1 presents a list of major conferences devoted to the problem of settlements.

The problems of horizontal displacement and tilt, however, have not received as much attention as the problem of settlement. Lee and Shen noted in 1969 that prediction of horizontal displacement and tilt was still in its infant stage. Even though nearly two decades have passed since the work of Lee and Shen, there is still relatively little published information in this area.

TABLE 2.1: MAJOR INTERNATIONAL OR SPECIALITY CONFERENCES WITH THEMES RELATING TO SETTLEMENT

Year	Conference Theme	Comments
1964	Design of Foundations for Control of Settlement, held at Evanston, Illinois	Proceedings appear in a special volume, and in Proc., ASCE,JSMFE, Vol. 90. No. 5, and Vol. 91, No. 2.
1965	Bearing Capacity and Settlement of Foundations, held at Durham, North Carolina	Papers include classics by Peck, Brinch Hansen, DeBeer and Lambe.
1972	Performance of Earth and Earth-supported Structures, held at Lafayette, Indiana	Include many interesting settlement case records.
1974	Settlement of Structures, held at Cambridge, England	One of the keystone conferences on settlement problems. Papers include case histories and five excellent state-of-the-art reports.

Table 2.1 (continued)

Year	Conference Theme	Comments
1977	Ninth International Conference on Soil Mechanics and Foundation Engineering, held at Tokyo	Methods for calculating settlements were reviewed by Burland, et al. in Session 2- "Behavior of Foundations and Structures."
1986	Settlement of Shallow Foundations on Cohesionless Soils: Design and Performance, held at Seattle, Washington	Proceedings were published by ASCE as Geotechnical Special Publication No. 5.

In this chapter, methods of estimating settlements, horizontal displacements and tilt of footings, that are have been described in the literature, will be briefly summarized.

2.2 METHODS FOR ESTIMATING SETTLEMENTS OF FOOTINGS ON SANDS AND GRAVELS

Settlements of footings on sands and gravels occur as a result of distortion and volume change within the soil at shallow depth beneath the footing. Because sands have high permeability and low compressibility, settlements in sands and gravels typically take place fairly rapidly and their magnitudes are quite small.

A knowledge of compressibility characteristics of sand deposits is required for settlement estimates. Due to the basic difficulty in obtaining good quality and representative soil samples, compressibility characteristics of sand deposits are seldom determined from the results of laboratory tests. Thus, in practice, predictions of settlements in sands and gravels are either based directly on in situ test results, or are based on elastic theory with value of modulus estimated using empirical correlations. Table 2.2 summarizes several methods of estimating settlements of footings in sands and gravels which have been described in the literature. For the purpose of discussion below, these methods are categorized into four groups, according to the type of test the particular

TABLE 2.2: METHODS FOR ESTIMATING SETTLEMENTS OF FOOTINGS ON SANDS AND GRAVELS

Reference	Method/Formula	Remarks
1. Based on Standard Penetration Test Data		
Alpan (1964)	$\rho = \alpha_o p \left(\frac{2B}{B+1}\right)^2$	- Based on the Terzaghi and Peck approach - α_o is determined from corrected SPT data.
Burland and Burbridge (1984)	$\rho = f_s\ f_1\ f_t \times [(p' - \frac{2}{3}\sigma v_o')B^{0.7} I_c]$	- Based on regression analysis of case records - I_c is a function of SPT blow count.
D'Appolonia, et al (1968)	$\rho = \frac{pB}{M}\mu_o\mu_1$	- Based on elastic theory - M is determined empirically from SPT data.
Duncan and and Buchignani (1976)	$\rho = \frac{5p}{(N-1.5)\ C_B}$	- Based on Terzaghi Peck approach. Gives about twice as much settlement as Terzaghi and Peck method.
Hough (1959,1967)	$\rho = \Sigma\left(\frac{1}{C}\right)\Delta z \log\left(\frac{\sigma v_o' + \Delta r_v'}{\sigma v_o'}\right)$	- Similar to one-dimensional method of calculating consolidation settlement - Bearing capacity index, C, is related to SPT blow count and soil type.

Table 2.2 (continued):

Reference	Method/Formula	Remarks
1. Based on Standard Penetration Test Data (continued)		
Meyerhof (1965)	$\rho = \frac{2p}{N}\left(\frac{2B}{B+1}\right)^2$	- Based on Terzaghi and Peck approach - Correction for water table effect is not required.
NAVFAC (1982)	$q = \frac{4pB^2}{K_{v1}(B+1)^2}$	- Empirical method based on plate load test - K_{vi} can also be determined from SPT data.
Oweis (1979)	$q = \Sigma \frac{pB}{E_i}(F_i - F_{i-1})$	- Based partly on elasic theory - Account for non-linear stress-strain behavior with an iterative procedure - Ei is empirically determined from corrected SPT data.
Parry (1971)	$q = \frac{200pB}{N} C_D C_W C_T$	- Based partly on elastic theory
Peck and Bazaraa (1969)	$q = C_W C_D \left(\frac{2p}{N_c}\right)\left(\frac{2B}{B+1}\right)^2$	- Refined from and Peck approach - Corrected SPT blow count, N_c, is used.

Table 2.2 (continued):

Reference	Method/Formula	Remarks
1. Based on Standard Penetration Test Data (continued)		
Peck, Hanson & Thornburn (1973)	$q = q_a/p$ where $qa = 0.11\ C_w N_1$	- Empirical method on settlement observations - SPT blow count corrected to 1 ton per sq. ft. effective overburden pressure is used.
Schultz and Sherif (1973)	$q = \dfrac{p\ f\ B}{1.71 N_{0.87}\ (B/B_1)^{0.5}} \times \dfrac{1}{(1 + 0.4\ t/B)}$	- Based on statisical study of settlement
Terzaghi and Peck (1948, 1967)	$q = C_w\ (\dfrac{3p}{N})\ (\dfrac{2B}{B+1})^2$	- Empirical method based on settlement observations.
Webb (1969)	$q = \dfrac{\sigma v}{E}\ \Delta z$	- Based partly on elastic theory - E is determined using correlation with SPT blow count.
2. Based on Cone Penetration Test Data		
Buisman -DeBeer (1965)	$\rho = \Sigma (\dfrac{2.3}{C}) \Delta z \log (\dfrac{\sigma v_o' + \Delta\sigma_v'}{\sigma v_o'})$ where $C = 1.5\ (q_c/\sigma v_o')$	- Similar to conventional one-dimensional settlement method - Applies only to normally consolidated sands.

Table 2.2 (continued):

Reference	Method/Formula	Remarks
Schmertmann (1970, 1978)	$\rho = C_1C_2 \, \Delta p\Sigma \, \frac{I_z\Delta z}{E}$	- Based partly on elastic theory - Non-linear strain distribution is accounted for through the use of strain influence facto, I_z - E is obtained through correlation with q_c.Â
3. Based on Pressuremeter Test Results		
Menard and Rousseau (1962)	$\rho = \frac{0.22p}{E_p} B_o(\lambda_d \frac{B}{B_o})^{\alpha} + \frac{0.11p}{E_p} \alpha\lambda_c B$	- Empirical method based on pressuremeter test experience in Europe.
4. Based on Dilatometer Test Results		
Schmertmann (1986)	$\rho = \Sigma \frac{\Delta\sigma v'}{M_t} \Delta z$	- Based on Janbu's one-dimensional compression theory - M_t is determined through correlation with dilatometer test results.
Leonards and Frost (1988)	$\rho = C_1C_2 \, \Delta p\Sigma \, \frac{I_z\Delta z}{E}$	- Equation is the same as Schmertmann's CPT-based method - E is estimated using correlation with dilatometer test results.

method uses as a basis of settlement estimate.

The density and compressibility of sand deposits are often highly erratic, and it is thus unrealistic to imagine that any method would be capable of estimating settlements of footings on sand with perfect accuracy. Many studies have been performed to evaluate the accuracies of the current available techniques for estimating settlements of footings on sands, and the current 'state-of-the-art' appears to be far from satisfactory due to the highly erratic variations of compressibility of sand deposits, the inherent difficulties associated with the interpretation of in situ test results, and the inevitable soil disturbance in boring.

In the following paragraphs, some background information of the methods for estimating settlements of footings on sands and gravels, as given in Table 2.2, will be described. However, the reliability and accuracies of these methods Table 2.2 shall be discussed subsequently in Chapter 3.

<u>Methods Based on Standard Penetration Test (SPT) Results</u>: The Standard Penetration Test was developed in the United States in 1927 as a tool to assess the compactness of soil (Fletcher, 1965). Even today, the test continues to be used worldwide, and is used to a greater extent than any other in situ test. While experience with the use of SPT has accumulated over the last six decades, the number of methods of settlement prediction based on SPT data has also

proliferated. A recent survey by Clayton, et al. (1988) showed that there are currently more than 20 SPT-based methods for estimating settlement. However, only those that are commonly used and relatively widely known are included in Table 2.2. A thorough and comprehensive review of the SPT-based methods can be found in Sutherland (1974), Talbot (1981) and Nixon (1982). The methods included in Table 2.2 are discussed in the following paragraphs.

The first technique for predicting settlements using SPT blow counts was proposed by Terzaghi and Peck (1948, 1967), who recommended a method of estimating settlement that includes bearing pressure, blow count, footing width, location of groundwater table and depth of footing embedment. Their method uses a chart that relates the bearing pressure required to cause one inch of settlement to the SPT blow counts. Settlements of footings are estimated using an assumed linear relationship between pressure and settlement. The chart developed by Terzaghi and Peck is applicable directly to footings at sites where the water table is located at depth greater than two times the footing width below the foundation base. For cases where the water table rises to the base of footing, Terzaghi and Peck recommended that settlements deduced from the chart be increased by 100% to account for the effect of submergence on effective confining pressure and thus the compressibility of the sand. The chart, according to

Terzaghi (1947) and Bazaraa (1967), was prepared on the basis of plate loading test results and on very limited data regarding building settlement observations.

Over the years various changes in the Terzaghi and Peck's method have been suggested by many researchers, including Alpan (1964), Meyerhof (1965), Peck and Bazaraa (1969), Peck et al. (1973) and Duncan and Buchignani (1976). Their modifications include some or all of the followings: (1) correction of SPT blow counts for the effect of overburden pressure, based on either the work of Gibbs and Holtz (1957) or the work of Peck, et al. (1973); (2) reduction of the magnitude of calculated settlement by 33%, which was first proposed by Meyerhof (1965) who concluded that Terzaghi and Peck method was unduly conservative; and (3) ignoring the effect of groundwater table, again advocated by Meyerhof, who suggested that water table effects are reflected in the measured SPT blow counts.

Beginning in the 1970's, many methods based on elastic theory were proposed (D'Appolonia, et al., 1968; Webb, 1969; Parry, 1971; and Oweis, 1971). The soil modulus values for these methods are determined using empirical correlations with SPT blow counts, which were backcalculated using either results of plate loading tests (as in the methods by Webb, Parry and Oweis) or case studies of actual footings (as in the method of D'Appolonia, et al.). In addition, Oweis attempted to account for non-linear stress-strain behavior through the use of an iterative procedure which results in more lengthy calculation time than other methods. The effect of stress history on soil compressibility was recognized in the procedure by D'Appolonia et al., who provided two different correlations between soil modulus and SPT blow count, one for normally consolidated sands and another for preconsolidated sands.

Yet another approach for estimating settlements in sands and gravels was suggested by Schultze and Sherif (1973) and Burland and Burbridge (1984), whose methods are based on statistical evaluation of available case histories. The method of Schultze and Sherif used linear elastic theory as a model and relied on an empirical correlation relating soil modulus to SPT blow count. Unlike the elastic methods discussed above, the relationship between soil modulus and SPT blow counts was derived using statistical correlations from 48 settlement observations. On the other hand, Burland and Burbridge's method is not based on theory. It was derived from regression analysis of over 200 case records of settlement on sands and gravels. Both methods include the effect of footing embedment, although Burland and Burbridge's method also incorporates the influence of time, footing shape and stress history. A study by Burland and Burbridge indicated that the two methods agreed reasonably well for footings up to 5m wide. For larger footings, Burland and Burbridge's method was found to give significantly larger settlements.

<u>Methods Based on Cone Penetration Test (CPT) Results</u>: The most widely used static sounding test is the Dutch Cone Penetrometer test that has been used extensively in Europe since about 1935 (Bazaraa, 1967). In recent years, it has gained acceptance in the United States and elsewhere in the world. The test was originally devised to assess bearing capacity of piles, but is now also used to estimate settlements in sands.

For calculating settlements based on CPT data, the methods of Buisman-DeBeer (1945, 1965) and Schmertmann (1970, 1978) are commonly used. In both procedures the modulus of compressibility index of the sand is estimated using correlation with cone resistance.

As can be seen in Table 2.2, the equation for the Buisman-DeBeer procedure is the same as the equation for estimating primary consolidation settlement of insensitive clay. The Buisman-DeBeer method assumes that distribution of foundation strains relates closely to the induced vertical stresses under the footing due to the applied load. This implies that maximum foundation strain would occur immediately below the foundation, which is found by other investigators not to be correct (Schmertmann, 1970).

Schmertmann's approach is quite different from that adopted by DeBeer. It is based on elastic theory, and the settlement is obtained through integration of foundation strains. Based on studies using elastic theory, models, and the finite element method, Schmertmann (1970) recommended that variation of vertical strain below the foundation may be estimated using a triangular distribution of a dimensional strain influence factor denoted as I_z. The maximum strain influence factor (with a value of 0.6), and thus the maximum vertical strain if the value of q_c does not vary with depth, occurs at a depth equal to one-half the foundation width (B) below the footing base. The foundation strains below a depth of 2B below the footing base are negligible, and are taken as zero. Later, Schmertmann's strain influence factor distribution was revised to account for the effect of the shape of the loaded area (Schmertmann, et al.; 1978). Two different strain influence factor distributions were proposed: one for axisymmetric (or square) footings and the other for plane strain (or long) footings. Also, the maximum value of the strain influence factor for the new distributions is not a constant; it varies with the magnitude of the applied pressure and the overburden pressure at the depth of maximum influence.

<u>Methods Based on Pressuremeter Test (PMT) Results</u>: The pressuremeter test developed by Menard (1956) is widely used in Europe and is gaining some acceptance in the United States

and other countries. Unlike the SPT and the CPT, deformation characteristics of soils are directly measured in the pressuremeter test, and a deformation modulus (known commonly as pressuremeter modulus) can be derived from the test results. However, soil disturbance during boring and installation of the pressuremeter probe can significantly reduce the value of pressuremeter modulus, and direct use of this value in design may lead to overestimated values of settlement. As a result, most of the available design procedures based on PMT results are derived empirically or semi-empirically.

The method developed by Menard and Rousseau (1962) is based on elastic theory, with soil modulus determined using values of pressuremeter modulus that are corrected using an empirical coefficient, α, so that calculated settlements agree reasonably well with measured settlements. Values of a are smaller than or equal to unity, and they vary with the soil type and stress history.

More recently, an alternative approach was proposed by Briaud (1990). The soil modulus used in Briaud's method is estimated using an iterative procedure that can account for the effects of stress level, strain level and time.

<u>Methods Based on Dilatometer Test Results</u>: The flat plate dilatometer was developed in the 1970's in Italy by Marchetti (Schmertmann, 1986). The test involves pushing the

dilatometer to the desired depth, and measuring the horizontal pressures required to "lift off" the attached membrane and the pressure required to inflate the membrane by a displacement of 1.1mm. Pertinent soil properties, such as constrained modulus (M_t), can be estimated using correlations with the dilatometer test results (Marchetti, 1980; Robertson, 1986).

The settlement calculation method proposed by Schmertmann (1986) is based on Janbu's one-dimensional compression theory (1963, 1967), with the constrained modulus deduced from the results of dilatometer tests. To account for the fact that the in situ stress level at which the constrained modulus was determined may be different from the effective stress level under the foundation, Schmertmann proposed a simple procedure for adjusting the modulus to reflect the stress level under the foundation. Recently, Leonards and Frost (1988) proposed a new procedure using the framework of Schmertmann's CPT-based method (1970, 1978), with the soil modulus determined from results of dilatometer tests, taking into consideration the influence of stress path and stress history.

2.3 METHODS FOR ESTIMATING SETTLEMENTS OF FOOTINGS ON CLAYS

Settlements in saturated clays are considered to consist of three components: immediate constant volume settlement due to distortion in the clay, consolidation settlement which occurs as water is squeezed out of the clay, and secondary compression due to the slow continued deformation of the clay under constant effective stress. Of these components, the largest settlements are usually those due to consolidation, and routine practice often neglects the contribution from immediate settlement. However, for shallow foundations supported by relatively stiff clay, immediate settlement can be an important portion of the total. Secondary compression is significant only in highly plastic or organic clays. The presence of these highly compressible materials often results in the use of deep foundations. It is therefore apparent that only immediate and consolidation settlements are of practical interest in estimating settlements of footings on clays.

A major difference between the methods for clays and those for sands is that fairly reasonable quality soil samples may be obtained from clay deposits, and the compressibility parameters determined from laboratory tests are relatively reliable for settlement estimates. In addition, the methods for estimating settlements of footings on clays appears to be more 'standardized' than those for sands and gravels. In practice, relatively few methods have been used to predict settlements of footings on clays. Burland, et al. (1977) concluded that " (these) simple traditional settlement calculations are usually adequate for practical purpose provided the appropriate in situ soil properties have been obtained".

In the following sections, methods for estimating immediate and consolidation settlements of footings on clays are discussed:

Consolidation Settlement: Some of the most widely used methods of estimating consolidation settlements, as reported in the literature, are summarized in Table 2.3. Of these methods probably the most widely applied is the conventional one-dimensional method first developed by Terzaghi in 1923. Based on results from oedometer tests, the method is reported to provide reliable estimates for consolidation settlements of foundations and embankments on normally consolidated clays, and especially for cases where the compressible layer is thin relative to the extent of the loaded area.

The three dimensional nature of the settlements of shallow foundations and the importance of immediate settlements for such conditions were first recognized by Skempton and Bjerrum (1957). They developed a method in which the consolidation settlement was computed from three-dimensional pore pressures obtained from undrained axisymmetric stress conditions. They further showed that, if the settlement is calculated on the basis of oedometer tests, the three-dimensional effect may be incorporated by means of a correction factor, ν. While Skempton and Bjerrum related the correction factor to the geometry of the problem and the value

TABLE 2.3: METHODS FOR ESTIMATING CONSOLIDATION SETTLEMENTS

Reference	Method	Remarks
Terzaghi (1923)	$\rho_T = \rho_{oed} = \Sigma\, m_v \Delta\sigma_v \Delta_z$	- Yields reliable estimates if compressible layer is thin relative to the extent of the loaded area.
Skempton and Bjerrum (1957)	$\rho_T = \rho_i + \rho_c$ $\rho_c = \Sigma\, m_v \Delta u \Delta_z$ where $\Delta u = \Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)$ or $\rho_c = \mu\rho_{oed}$	- Accounts for 3D effect - Includes immediate settlement - Excess pore pressure calculated using elastic theory.
Janbu (1963, 1967)	$\rho = R \dfrac{\Delta\sigma_v'}{M_t} \Delta z$	- M_t can be determined from results of laboratory or field tests.
Lambe (1964, 1967)	Stress path method	- Simulates field stress path in laboratory tests - Integrates strains from laboratory tests to obtain settlement.
Davis and Poulos (1963, 1968)	Elastic method $\rho_c = \rho_T - \rho_i$ where $\rho_T = \dfrac{pB\, I_\rho}{E'}$ and $\rho_i = \dfrac{pB\, I_\rho}{E_u}$	- Success of method depends on values selected for the elastic constants, E' and E_u.

Table 2.3 (continued):

Reference	Method	Remarks
Menard and Rousseau (1962)	$\rho_c = \frac{0.11p}{E_p} \alpha\lambda_c B$	- Based on elastic theory - Makes use of pressuremeter test results.
DeBeer (1965), Mitchell and Gardner (1978)	$\rho_T = \Sigma \frac{\Delta\sigma_v' \Delta z}{\alpha_c q_c}$ where $3 < \alpha_c < 8$ for low plastic clays; $2 < \alpha_c < 6$ for high plastic clays	- Based on cone penetration test results.

where ρ_T = total settlement
ρ_c = consolidation settlement
ρ_i = immediate settlement
ρ_{oed} = consolidation settlement calculated using oedometer test results
m_v = coefficient of volume compressibility
M_t = tangent modulus
E' = drained elastic modulus
E_u = undrained elastic modulus
E_p = pressuremeter modulus
$\Delta\sigma_v'$ = change in effective stress
Δu = change in excess pore pressure
Δz = thickness of sublayer
A = Skempton's pore pressure coefficient
α = empirical correction factor for pressuremeter test
μ = emprical correction factor accounting for 3D effect
λ_c = shape factors
I_q = settlement influence factor
B = foundation width
p = foundation pressure
q_c = cone resistance
α_c = correlation coefficient used in cone penetration test

of pore pressure coefficient, A; Simons and Som (1969) related the correction factor to the incremental stress paths in the foundation soil due to the foundation loading. Duncan and Poulos (1981) demonstrated that values of the correction factor can be determined using elastic theory.

Janbu (1963, 1967) developed a unified approach for estimating settlements of foundations on all type of soils, using the concept of tangent modulus, M_t (see equation in Table 2.3). His method, widely used in Europe, requires estimates of changes in stress due to foundation loading from elastic theory. The tangent modulus is a function of the effective confining stress in normally consolidated clays, but it is independent of confining stress if the clay is overconsolidated. Values of the tangent modulus can be estimated from the results of oedometer or triaxial compression tests, or more recently, from correlation with the results of in situ tests.

The stress path method advocated by Lambe (1964, 1967) is based on the use of elastic theory and triaxial tests. Elastic theory is used to assess the changes in stress on soil elements under the loaded area. These changes in stress are then applied to triaxial test specimens to determine the resulting deformation. The measured strains in the tests are assumed to be those that would occur in the field, and are integrated to predict the surface settlements. The use of this method for practical design is limited because very complex and expensive triaxial tests must be performed. Furthermore, design changes often invalidate the pattern of stresses initially chosen, and therefore necessitate additional laboratory tests.

By using appropriate values of elastic modulus, both total and immediate settlements can be estimated using elastic theory (Davis and Poulos, 1963, 1968). The difference between the total and the immediate settlements can be taken as consolidation settlement. Difficulties involved in selecting the appropriate elastic constants has limited the use of this method.

Results from in situ tests, such as the pressuremeter (Menard and Rousseau, 1962) and cone penetration tests (DeBeer, 1965) can also be used to estimate consolidation settlement. In both methods the compressibility of the clay is estimated through empirical correlation with the results of the relevant in situ tests.

Consolidation settlements of normally or slightly overconsolidated clays may take years to complete, and predictions of rate of consolidation settlement in these soils are of practical significance. The rate can be estimated using the conventional Terzaghi one-dimensional consolidation theory, or other more refined theories that take into considerations of the three-dimensional effects (e.g., Biot, 1941; Gibson and McNamee, 1963; Davis and Poulos, 1965, 1972; Schiffman, et al, 1969), the non-linear strain behavior (Janbu, 1965), and the effect of step loading (Oslon, 1977).

Settlement in less compressible overconsolidated clays occurs fairly rapidly, and it is reasonable to assume that consolidation settlement in these materials takes place immediately after the load is applied.

Immediate Settlement: Immediate settlement is usually calculated using elastic theory. Three commonly used procedures have been summarized by Lambe (1973) as shown in Table 2.4. The undrained Young's modulus, E_u, is used in all three procedures.

The equation suggested by Janbu, et al. (1956), as shown in Table 2.4, gives the average initial settlement for a flexible foundation, with the value of Poisson's ratio of the foundation clay equal to 0.5. The values of the factors $\mu 1$ and $\mu 2$ are functions of the width, length and depth of the loaded area and thickness of the foundation clay. Recently, Christian and Carrier (1978) have reinterpreted the chart for $\mu 1$ and $\mu 2$ originally proposed by Janbu, et al., and have offered a new improved chart for estimating values of these factors.

The elastic strain summation method developed by Davis and Poulos is similar to the stress path method discussed above. Three-dimensional stresses induced by foundation

TABLE 2.4: METHODS OF CALCULATING IMMEDIATE SETTLEMENT (AFTER LAMBE, 1973)

Method	Formula	Reference
Elastic displacement	$\rho_i = \dfrac{pB\mu_o\mu_1}{E_u}$	Janbu, Bjerrum, & Kjaernsli (1956)
Elastic strain summation	$\rho_i = \dfrac{\Sigma[\sigma_z - 0.5(\sigma_x + \sigma_y)]\Delta z}{E_u}$	Davis & Poulos (1968)ï
Modified elastic displacement	$\rho_i = \dfrac{pB\mu_o\mu_1}{S_R E_u}$	D'Appolonia, et al. (1970)

where ρ_i = immediate settlement (in same length units as B)
p = vetical pressure at base of footing (in pressure units)
B = footing width (in length units)
σ_x, σ_y, σ_z = Stresses in the X, Y, and Z directions, induced by foundation loading (in pressure units)
E_u = undrained soil modulus (in pressure units)
Δz = thickness of compressible layer (in same length units as B)
μ_o, μ_1 = dimensionless settlement influence factors
S_R = dimensionless settlement modification factor

loading are estimated using elastic theory.

Janbu's elastic displacement method was refined by D'Appolonia, et al.(1970) to account for the effect of local yielding in foundation soils. Their study indicated that for footings with safety factor against bearing capacity failure of 3, settlements of normally consolidated clay computed using Janbu, et al.'s procedure should be increased by approximately 60%. However, if the same footing is placed on overconsolidated clay, the increase in consolidation settlement as a result of local yielding is reduced drastically to about 5%.

2.4 METHODS FOR ESTIMATING HORIZONTAL DISPLACEMENTS AND ROTATIONS OF FOOTINGS ON SANDS AND CLAYS

Relatively few methods have been developed for estimating horizontal displacements and rotations of footings, and only a handful of these methods have found their ways into textbooks or routine design practice. Predictions of horizontal displacements and rotations are usually made using elastic theory. Tables 2.5 and 2.6 summarize some of the elastic solutions for estimating horizontal displacements and rotations. The major difficulty in the use of these equations is the selection of appropriate value of Young's modulus.

At present, there appear to be no field observations

TABLE 2.5 : ELASTIC SOLUTIONS FOR ESTIMATING HORIZONTAL DISPLACEMENTS OF FOOTINGS

Foundation Type	Formula	Reference
Rigid strip footings on finite layer	$\Delta h = \dfrac{H\mu_h}{EB}$	Milovic, et al. (1970)
Rigid circular footings on semi-infinite layer	$\Delta h = \dfrac{(7-8\nu)(1+\nu)}{16(1-\nu)\ E} H\pi r$	Bycroft (1956)
Rigid rectangular footings on semi-infinite layer	$\Delta h = \dfrac{H\ (1-\nu^2)}{\beta_x \sqrt{BL}\ E}$	Barkan (1962)
Footings	$\Delta h = \dfrac{H}{EB}$	Meyerhof (1953)

where $\Delta\theta$ = horizontal displacement (in same length units as B)
H = horizontal load (in force units)
E = soil modulus (in pressure units)
B = footing width (in length units)
L = footing length (in same length units as B)
r = radius of circular footings (in length units)
μ_h = dimensionless horizontal displacement coefficient
β_x = dimensionless coefficient
ν = Poisson's ratio of soils (dimensionless)

TABLE 2.6 : ELASTIC SOLUTIONS FOR ESTIMATING ROTATIONS OF FOOTINGS

Foundation Type	Formula	Reference
Rigid strip footings on semi-infinite layer	For sands $\theta = \dfrac{0.6\ M}{E\ B^3}$ For clays $\theta = \dfrac{0.45\ M}{E\ B^3}$	Muskhelishvili (1963)
Rigid strip footings on finite layer	$\tan 2\theta = 2\omega_{cm} \cos\delta \dfrac{M}{B^2 E}$	Milovic et al. (1970)
Rigid circular footings on semi-infinite layer	$\theta = \dfrac{3M\ (1-\nu^2)}{4Er^3}$	Borowicka (1943)
Rigid circular footings on finite layer	$\theta = \dfrac{(1-\nu^2)\ M}{4r^3\ \beta_h\ E}$	Yegorov and Nichiporovich (1961)
Rigid rectangular footings on semi-infinite layer	$\theta = \dfrac{(1-\nu^2)\ M}{B^2 L\ E} I_\theta$	Lee (1962); Whitman and Richart (1967)
Rigid rectangular footings on finite layer	$\theta_L = \dfrac{M_L}{(L/2)^3\ E} I_{\theta L}$ $\theta_B = \dfrac{M_B}{(B/2)^3\ E} I_{\theta B}$	Sovinc (1969)

Table 2.6 (continued):

Foundation Type	Formula	Reference
Flexible strip footings on semi-infinite layer	$\tan\theta = \frac{\nu^2-1}{\nu^2} \frac{3}{\pi} \frac{M}{B^2 L_E}$	Tettinek (1965)
Flexible circular footings on semi-infinite layer	$\tan\theta = \frac{\nu^2-1}{\nu^2} \frac{16}{3\pi} \frac{M}{r^3 E}$	Tettinek (1965)
Flexible rectangular footings on semi-infinite layer	$\tan\theta = \frac{\nu^2-1}{\nu^2} \frac{3}{\pi} \frac{M}{B^2 LE} k_M$	Tettinek (1965)

where θ = rotation of footings (in units of radian)
M = applied moment (in units of force-length)
E = soil modulus (in pressure units)
B = footing width (in length units)
L = footing length (in same length units as B)
r = radius of circular footings (in length units)
δ = load inclination (in units of degree or radian)
ω_{cm} = dimensionless coefficient
β_h = dimensionless factor
I_θ, $I_{\theta L}$, $I_{\theta B}$ = dimensionless rotation influence factors
ν = Poisson's ratio of soils (dimensionless)
k_M = dimensionless factor

available for checking the reliability of these elastic procedures, although Meyerhof (1953) has reported reasonable agreement between his analyses and the results of model studies. Some engineers, however, believed that these procedures would overpredict the footing rotation because of the rigidity of retaining walls, redistribution of contact pressure during rotation, and other factors (Gifford, et al, 1987).

Another approach for calculating rotations of foundations has been suggested by Prakash (1981). His method is empirical and is based on the results of model footing tests. The rotation is calculated based on the maximum settlement, S_m, at the edge of footing, and the settlement at the location of the resultant force, S_e. The values of S_m and S_e can be estimated from the empirical formula that relates them to the settlement of the same footing under a concentric vertical load. At present, the method is not widely used. Its reliability and accuracy remain to be evaluated.

CHAPTER 3

EVALUATION OF METHODS FOR CALCULATING SETTLEMENTS OF FOOTINGS ON SANDS AND GRAVELS

3.1 INTRODUCTION

The selection of allowable bearing pressures of footings on sands and gravels is nearly always controlled by considerations of settlement rather than safety against bearing capacity or structural failure. The task of estimating settlements thus plays an important part in the design of footings on sands and gravels. It is important that engineers who design footings on sands and gravels understand the accuracy with which settlements can be estimated, and the reliability of the estimates they make.

Terzaghi and Peck (1967) suggest that, if a large number of identical footings, all equally loaded, were built on the same sand or gravel deposit, the footing that settled the most would settle about twice as much as the footing that settled the least. They also suggest that, if, in addition, the footings were of various sizes and supported loads of different magnitudes, even if all exerted the same bearing pressure on the sand or gravel, the settlements would vary even more. Under these conditions, the largest settlement might be as much as four times the least settlement.

The variations in the settlements of footings on sands or gravels are thus controlled to a significant extent by chance -- by the chance that a footing will be located over the loosest, most highly compressible part of the sand deposit, and by the chance that this footing will be the largest, most heavily loaded footing. It is thus clear that no method of estimating settlements of footings on sand can be expected to predict the settlement accurately in all cases, because there will inevitably be chance variations in the actual settlements.

In this chapter the accuracy of twelve methods of estimating the settlements of footings on sands is examined to determine with what degree of reliability these methods can be used to estimate the maximum likely settlements. Eleven of these methods use the Standard Penetration Test blow count as a basis for estimating settlement. Schmertmann's method uses the results from the Cone Penetration Test. Results of this study are presented below. Similar studies performed previously by other investigators are also reviewed.

3.2 REVIEW OF PREVIOUS WORKS

A considerable number of studies, generally using a fairly restricted database, have been performed to assess the relative accuracy of the methods for estimating settlements of footings on sands and gravels. Most of these studies used the ratio of calculated settlement to observed settlement, referred to hereafter as settlement ratio (R_s), as a basis of evaluation; and they dealt exclusively with the accuracies of the methods. In a recent state-of-the-art report, Nixon (1982) noted that Talbot (1981) had collected 360 published comparisons between predicted and observed settlements. The settlements of these foundations were estimated using methods based on results of SPT or CPT. A summary of the cases reviewed by Talbot is given in Table 3.1.

As can be seen in Table 3.1, the values of settlement ratio for the methods studied vary over a wide range, between 0.1 and 20 for methods based on SPT data, and between 0.1 and 24 for methods based on CPT results. Methods based on CPT results appear to be slightly more accurate on the average than those using SPT data. This may be attributed to the fact that results of cone penetration tests are usually more consistent, and that the CPT provides a continuous record of cone resistances, as compared to discrete measurements in SPT.

Talbot also found that of the 40 methods he reviewed, 15 had not been applied to cases reported in the literature. He further indicated that subsequent applications of a method by others generally gave wider variations in the settlement ratios than was obtained by the person who originated the

TABLE 3.1: PERFORMANCE OF SETTLEMENT PREDICTIONS BY SPT AND CPT METHODS (AFTER TALBOT, 1981)

Base of Settlements Prediction	No. of Methods	No. of Predictions	Values of Settlement Ratio (R_s)			
			Max.	Min.	Average	Std Dev.
SPT	18	213	19.79	0.12	2.01	1.07
CPT	7	147	23.79	0.11	1.71	1.09
Combined	25	360	23.79	0.11	1.92	1.09

R_s = calculated settlement/measured settlement method. Of all the established methods studied, Talbot concluded that the methods of D'Appolonia, et al. and Parry seemed to provide the more consistently reliable results.

Clayton, et al. (1983); Clayton, et al. (1988); Jeyapalan and Boehm (1986); and Gifford, et al. (1987) also examined the accuracy of various methods of estimating the settlements of footings on sand. These studies deserve attention because they were all performed to compare various established methods for estimating settlements, and the database they used was relatively large or contained readings from well instrumented case histories. Some ot the details of these studies are summarized in Table 3.2. The databases used by Clayton, et al. (1988) and Gifford, et al. are also employed in the studies performed during the research descibed in the last part of this chapter.

Using 71 case histories of footings on mostly clean medium sand, Jeyapalan and Boehm evaluated the relative accuracy of nine methods for estimating settlements. Except for Schmertmann's method, all the methods are based on SPT blow count. The average settlement ratios for these nine methods were found to vary between 0.8 and 3.66; however, the computed values of settlement ratio for individual case records varied over a wide range. Assuming a normal distribution for the reults, Jeyapalan and Boehm computed 80 and 90% confidence intervals for meeting a 25mm settlement

TABLE 3.2: SUMMARY OF STUDIES BY CLAYTON, ET AL. (1983, 1988); GIFFORD, ET AL. (1987), AND JEYAPALAN AND BOEHM (1986)

Investigator	Methods Studied	No. of Case Records	Comments
Clayton, et al. (1983)	- Alpan - Burland, et al. - Meyerhof - Schultze and Sherif - Terzaghi and Peck	12	- Case records obtained from literature - Schultze and Sherif's method was considered to be the best.
Clayton, et al. (1988)	- Burland and Burbidge - Schultze and Sherif	90	- Data base is a subset of Burland and Burbidge's. - SPT-based methods were unacceptably inaccurate.
Gifford, et al. (1987)	- Burland and Burbidge - D'Appolonia, et al. - Hough - Peck and Bazaraa - Schmertmann	31	- Data included 21 well instrumented bridges in the northeastern United States - D'Appolonia, et al.'s method is considered to be the most accurate.

Table 3.2 (continued)

Investigator	Methods Studied	No. of Case Records	Comments
Jeyapalan and Boehm (1986)	- Alpan - D'Appolonia, et al. - Elastic Method - Meyerhof - Oweis - Peck and Bazaraa - Schmertmann - Schultze and Sherif - Terzaghi	71	- Case records compiled from literature - Used relative error as a basic for evaluation

criterion by all nine methods. Based on these confidence intervals, the methods of Alpan and Schultze and Sherif appeared to be the most dependable.

In the study by Gifford, et al. (1987), five methods for estimating settlements of footings on sands and gravels were compared using 31 case records of bridge foundations. The average settlement ratios for the five methods range between 0.8 and 1.9. Their study indicated that the methods of Burland and Burbidge, D'Appolonia, et al., and Peck and Bazaraa typically underpredicted settlement, while those of Hough and Schmertmann typically overpredicted. On the average, the method of D'Appolonia, et al. was found to be the most accurate, with Burland and Burbidge next. The method of Hough was found to be the least accurate.

The criteria employed by Clayton, et al. (1983) for assessing accuracy of a settlement prediction method is somewhat different from that employed by Jeyapalan and Boehm, and Gifford, et al. They believed that, from the standpoint of a designer, a good method should " tend to overpredict, and should have increasing accuracy as predicted settlement increases, in addition to having an acceptable range of the (settlement) ratio at any value of predicted settlement". Based on these criteria, they concluded that, of all of the four SPT-based methods they investigated, the method of Schultze and Sherif appeared to be the best suited for estimating settlements of footings on sands and gravels. In a more recent study, Clayton and his co-workers concluded that it is fundamentally incorrect to correlate SPT blow count with the compressibility of sand, and, consequently, that methods which relate settlement to SPT will be unacceptably inaccurate.

3.3 SIMPLE STATISTICAL EVALUATION

As for many previous studies, the methods of estimating settlement were evaluated in the present investigation by comparing calculated and measured settlements of footings on sands. However, in light of the inevitable variability in sand density and compressibility and the limitations on the use of in situ test such as the SPT as a means of estimating sand compressibility, the objective of the present study was somewhat different from the previous investigations. The study examined the relationship between the accuracy and reliability of the methods for estimating settlements, with the aim of developing a means by which a designer may select a method that will yield the best combination of accuracy and reliability.

3.3.1 Methods Examined

The methods examined in this study were selected based on the following criteria:

(1) The methods widely accepted in practice,

(2) Soil parameters for use with the methods should be obtained from routine in situ tests. Methods based on results of pressuremeter and dilatometer tests were excluded.

The twelve methods examined are listed in Table 3.3, which summarizes the variables, the correction factors, and the charts that they use. A brief review of these methods is given in Section 2.2; detailed procedures for each method can be found in the reference cited.

It is important to note that there are considerable differences in the procedures these methods use to determine the N-value for use in estimating the settlement, to correct for the position of water table, and to correct for the effect of footing embedment. In this study each of the methods was employed in careful conformance with the procedures suggested by the originator of the method.

3.3.2 Data Bases

The methods shown in Table 3.3 were used to calculate settlements for a number of cases where the settlements of footings have been measured. The cases used in this study were compiled by Burland and Burbidge (1984), and by Gifford, et al. (1987)

Burland and Burbidge examined more than 200 published

TABLE 3.3: VARIABLES USED IN METHOD OF ESTIMATING SETTLEMENTS OF FOOTINGS ON SAND

Method (reference)	Variables Used											
	N	N_{cor}	q_c	B	D_w	D_f	γ_t	L	T	Soil Type	Str. Hist	Time
Alpan (1964)		✓		✓	✓	✓	✓					
Burland and Burbidge (1985)	✓			✓	✓	✓	✓	✓	✓	✓	✓	✓
D'Appolonia and D'Appolonia (1970)	✓			✓	✓	✓			✓		✓	
Duncan and Buchignani (1976)	✓			✓						✓		✓
Meyerhof (1956)	✓			✓								
NAVFAC (1982)	✓			✓	✓							
Parry (1971)	✓			✓	✓				✓			
Peck and Bazaraa (1969)		✓		✓	✓	✓	✓			✓		
Peck, Hanson, Thornburn (1974)		✓		✓	✓	✓	✓					
Schmertmann (1978)			✓	✓	✓	✓	✓					✓
Schultz and Sherif (1973)	✓			✓		✓			✓			
Terzaghi and Peck (1967)	✓			✓	✓					✓		

N = SPT Blow Count

N_{cor} = SPT Blow Count corrected for overburden pressure

q_c = Cone Penetration Test tip resistance

B = footing width

D_w = depth to water table

D_f = depth of footing below ground surface

γ_t = total unit weight of sand

L = footing length

T = thickness of sand layer below footing

Soil Type = silty or clean sand

Stress Hist. = max prev load

Time = duration of loading

records of settlements on sands. These include bridges, test footings, tanks, and embankments. A number of the structures studied by Burland and Burbidge were founded on mat foundations, with quite large widths. Because the focus of this study was the settlement of spread footings for bridges and buildings, foundations with widths larger than 30 ft were not considered. This left 60 footings for bridges, buildings, test footings, tanks, and embankments in the Burland and Burbidge data base. Table 3.4 summarizes the pertinent information for each case record of the Burland and Burbidge data base. The SPT blow count for these cases ranged from 4 to 60, and the bearing pressure was as large as 6.0 tsf (tons per square foot). The measured settlements ranged from 0.08 in to 3.82 in.

Gifford, et al., in a study sponsored by the Federal Highway Administration, measured the settlements of 21 footings under 9 bridges in the northeastern part of the United States. They also examined 10 published records, for a total of 31 cases. Table 3.5 summarizes the pertinent information for these cases. The widths of the footing in the Gifford, et al. data base ranged from 8 ft to 28 ft. SPT tests were performed in 21 of these cases, and the measured blow counts ranged from 8 to 58. In 15 cases CPT tests were performed, and the measured tip resistances (or cone resistances) ranged from 30 to 185 kg/cm^2. The bearing

TABLE 3.4: BURLAND AND BURBIDGE DATA BASE

Source	Type of Structure	Pressure (tsf)	Field N	Length (ft)	Width (ft)	Measured Settlement (inch)
DeBeer	Bridge	0.54	8	45.9	10.8	0.79
DeBeer	Bridge	0.54	8	45.9	10.8	1.38
DeBeer	Bridge	1.69	30	52.5	19.7	0.41
DeBeer	Bridge	1.69	30	52.5	19.7	0.43
DeBeer	Bridge	0.97	35	52.5	18.1	0.26
DeBeer	Bridge	1.46	38	46.8	9.8	0.12
DeBeer	Bridge	0.97	10	78.7	14.8	0.31
DeBeer	Bridge	1.53	10	72.2	8.5	0.47
DeBeer	Bridge	2.96	60	31.2	8.2	0.12
DeBeer	Bridge	1.26	17	172.2	17.4	0.35
Bjerrum	Embankment	0.81	15	-	2.6	0.28
D'Appolonia	Steel Mill	2.58	20	17.7	11.5	0.32
D'Appolonia	Steel Mill	2.24	20	19.4	12.1	0.48
D'Appolonia	Steel Mill	1.68	20	21.0	13.1	0.41
D'Appolonia	Steel Mill	1.38	20	22.6	14.1	0.30
D'Appolonia	Steel Mill	1.46	20	24.3	15.1	0.28
D'Appolonia	Steel Mill	1.54	20	25.6	16.1	0.26
D'Appolonia	Steel Mill	1.45	20	28.9	18.1	0.34
D'Appolonia	Steel Mill	1.68	20	32.2	20.0	0.37
D'Appolonia	Steel Mill	1.57	20	33.5	21.0	0.40
D'Appolonia	Steel Mill	1.18	20	35.1	22.0	0.57
Garga	Building	2.65	16	7.2	7.2	0.43
Garga	Steel Mill	2.56	16	2.9	2.9	0.39

Table 3.4 (continued)

Source	Type of Structure	Pressure (tsf)	Field N	Length (ft)	Width (ft)	Measured Settlement (inch)
Greenwood	Test Footing	1.57	35	4.9	4.9	0.08
Greenwood	Test Footing	1.57	28	3.9	3.9	0.05
Marivoet	Bridge	0.95	12	100.1	14.8	0.41
Martins	Building	1.28	6	11.5	11.5	3.54
Meigh	Building	0.81	13	3.6	3.6	0.08
Meigh	Building	0.80	13	4.9	4.9	0.08
Meigh	Building	0.80	13	4.9	4.9	0.05
Muhs	Various	1.74	35	88.3	77.4	0.61
Muhs	Various	2.40	25	5.9	5.9	0.13
Muhs	Various	2.40	25	4.6	4.6	0.15
Muhs	Various	2.96	25	7.2	7.2	0.41
Muhs	Various	2.04	35	18.7	14.8	0.15
Muhs	Various	2.36	25	45.9	1.3	0.30
Muhs	Various	2.61	25	41.3	5.3	0.37
Muhs	Various	2.61	25	41.7	3.9	0.39
Muhs	Various	3.07	25	60.4	2.6	0.23
Muhs	Various	2.15	25	79.1	5.9	0.67
Muhs	Various	3.07	40	3.3	3.2	0.20
Muhs	Various	3.17	40	18.7	10.8	0.43
Muhs	Various	3.17	40	18.7	10.8	0.48
Muhs	Various	3.17	40	20.7	11.8	0.50
Muhs	Various	3.17	40	20.7	11.8	0.54
Muhs	Various	3.17	40	22.3	14.8	0.72
Muhs	Building	2.05	25	3.3	3.3	0.24
Muhs	Test Footing	2.30	34	3.3	3.3	0.14

Table 3.4 (continued)

Source	Type of Structure	Pressure (tsf)	Field N	Length (ft)	Width (ft)	Measured Settlement (inch)
Muhs	Test Footing	5.89	45	3.3	3.3	0.17
Muhs	Test Footing	3.54	45	3.3	3.3	0.24
Muhs	Test Footing	2.96	45	3.3	3.3	0.19
Nonveiler	Silos	1.67	23	73.5	36.1	1.57
Basaraa	Building	1.75	12	400.0	7.9	0.62
Bazaraa	Building	1.96	12	400.0	7.9	0.75
Ronan	Concrete Tank	2.51	60	29.2	29.2	0.28
Schultze	Building	3.06	37	35.1	8.5	0.43
Tschebotarioff	Building	4.00	50	157.5	12.5	0.19
Webb	Footing	1.98	7	19.7	19.7	2.91
Wennerstrand	Bridge	1.48	4	45.9	45.9	3.82
Wennerstrand	Bridge	1.03	4	47.6	10.8	1.46

TABLE 3.5: GIFFORD, ET AL. DATA BASE

Source	Type of Structure	Pressure (tsf)	Field N	Length (ft)	Width (ft)	Measured Settlement (inch)
FHWA	Bridge	1.60	30	63.7	17.0	0.35
FHWA	Bridge	1.34	60	63.7	17.0	0.67
FHWA	Bridge	1.16	38	52.5	15.3	0.94
FHWA	Bridge	1.22	21	52.5	16.8	0.76
FHWA	Bridge	0.94	12	41.0	12.5	0.61
FHWA	Bridge	0.85	19	74.6	11.0	0.42
FHWA	Bridge	1.17	22	79.0	18.5	0.61
FHWA	Bridge	1.05	18	21.0	21.0	0.28
FHWA	Bridge	0.75	18	30.4	21.0	0.26
FHWA	Bridge	1.17	17	26.8	16.0	0.29
FHWA	Brdige	1.24	20	18.5	16.0	0.25
FHWA	Bridge	1.65	21	42.9	8.1	0.46
FHWA	Bridge	1.72	8	42.9	8.1	0.34
FHWA	Bridge	1.20	36	76.9	16.8	0.23
FHWA	Bridge	1.17	24	76.1	15.2	0.44
FHWA	Bridge	0.80	30	67.3	15.2	0.83
FHWA	Bridge	1.12	21	28.0	28.0	0.64
FHWA	Brdige	1.50	25	100.8	20.0	0.46
FHWA	Brdige	1.63	28	100.8	20.0	0.66
FHWA	Bridge	1.75	37	44.4	21.8	0.61
FHWA	Bridge	1.69	38	44.7	16.0	0.28
Bergdahl	Bridge	1.90	22	28.0	16.4	0.47
Wennerstrand	Bridge	1.03	5	47.7	10.9	1.46

Table 3.5 (Continued):

Source	Type of Structure	Pressure (tsf)	Field N	Length (ft)	Width (ft)	Measured Settlement (inch)
DeBeer	Bridge	2.41	41	32.9	9.8	0.83
DeBeer	Bridge	0.76	18	78.9	19.0	0.47
DeBeer	Bridge	2.05	7	68.9	8.5	1.30
Levy	Bridge	5.30	38	23.0	13.0	0.47
DeBeer	Bridge	1.65	32	52.5	19.7	0.31
DeBeer	Bridge	2.24	33	52.5	19.7	0.16
DeBeer	Bridge	1.37	34	118.0	23.0	0.47
DeBeer	Bridge	1.00	34	92.0	17.0	0.39

pressures of the footings ranged from 0.75 tsf to 2.25 tsf, and the measured settlements ranged from 0.16 in to 1.46 in.

There is some overlap between the Burland and Burbidge data and the Gifford, et al. data. The combined compilation contains 76 case records. The settlements for all 76 cases were calculated using the methods in Table 3.3 that use SPT blow count, and the settlements for the 15 cases with CPT results were calculated using Schmertmann's method, to provide a basis for examining the accuracy and reliability of these methods for estimating settlements of footings on sand.

3.3.3 Definitions of Accuracy and Reliability

A perfectly accurate method of calculating settlements would be one that resulted in calculated settlements equal to the measured settlement in every case. Because of the erratic nature of the density and compressibility of sand deposits, it is unreasonable to expect that any method can achieve close to perfect accuracy.

A perfectly reliable method of calculating settlements would be one that never resulted in estimated values of settlement that were smaller than the actual settlement. Such a method would be perfectly reliable because engineers who used the method could be fully confident that the actual settlements would not exceed their estimates. The reliability of any method of estimating settlements could be improved by increasing the estimated values of settlement by some arbitrary factor. However, increasing all values of estimated settlement by a factor would be expected to reduce the degree of accuracy of the method.

For purposes of discussion in this study, accuracy and reliability are defined as follows:

Accuracy is defined as the average value of calculated settlement divided by measured settlement. For each of the methods examined, the ratio of calculated settlement divided by measured settlement was calculated for each case. The average value of this ratio for all the cases in the data base was used as a measure of the accuracy of the method. A value of this ratio equal to unity represents the best possible accuracy.

Reliability is defined as the percentage of the cases for which the calculated settlement is greater than or equal to the measured settlement. A value approaching 100 percent represents the most desirable value of reliability.

3.3.4 Inherent Accuracies and Reliabilities of the Methods

The results of the study of the eleven methods that use SPT blow count to estimate settlements are shown in Figure 3.1. Values of "accuracy" vary from 1.0 (the ideal value) for Alpan's method to 3.2 for Terzaghi and Peck's method. Values of "reliability" vary from 34 percent for Schultze and

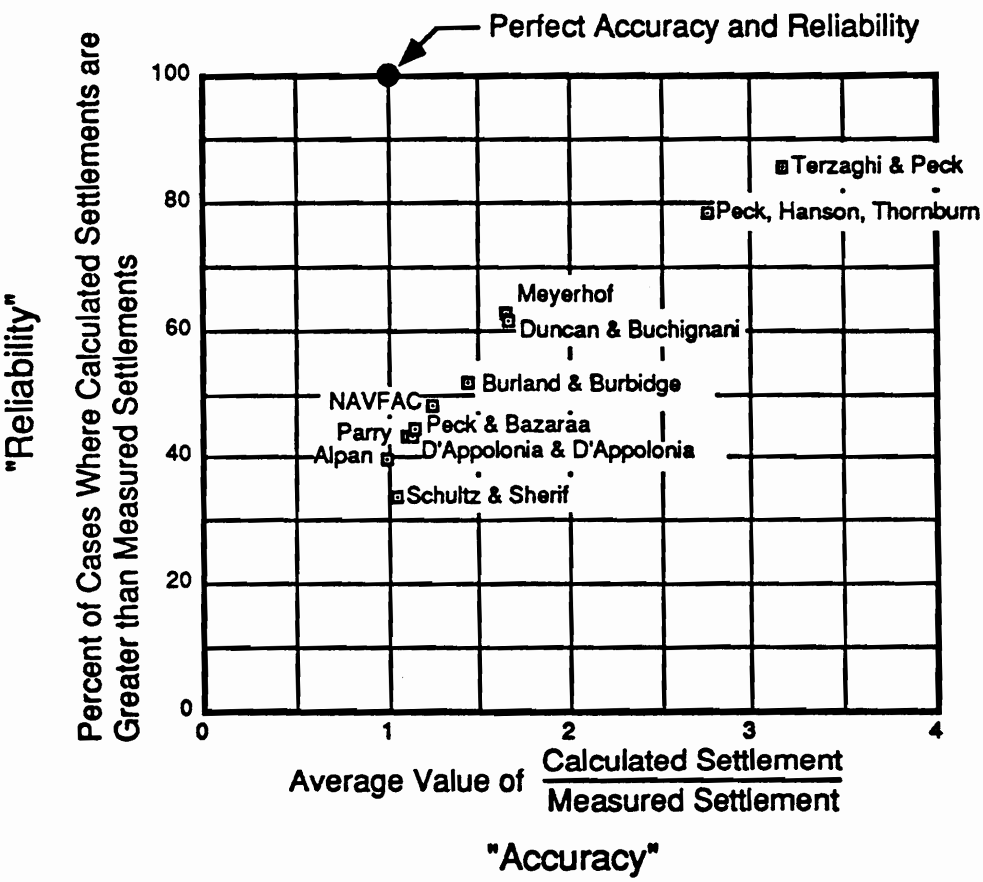


Figure 3.1: Relationship Between Accuracy and Reliability for Eleven Methods Based on SPT Blow Count

Sherif's method to 86 percent (a highly desirable level of reliability) for Terzaghi and Peck's method. It can be seen that, in general, the methods that are less accurate (those with values of "accuracy" farther from unity) are more reliable, in the sense that they underestimate settlement relatively infrequently.

The results in Figure 3.1 appear to indicate that the various methods of estimating settlements were developed to achieve different objectives. Most of the methods appear to have been devised to achieve the best possible accuracy. These include all the methods that have values of "accuracy" between 1.0 and 1.7, ranging from Schultze and Sherif's method at the lower left to Meyerhof's method at the upper right of this group. The methods of Terzaghi and Peck, and Peck, Hanson and Thornburn, on the other hand, appear to have been devised with maximum "reliability" in mind. Terzaghi and Peck's method underestimates settlements of footings on sands only about 14 percent of the time.

The accuracy and reliability of Schmertmann's method were evaluated using the same procedures, based on the set of 15 cases for which CPT test results were available. The computed value of accuracy was 2.0, and the reliability was 70 percent. It can be seen that this falls in line with the trend of results in Figure 3.1 between Meyerhof's and Peck, Hanson, and Thornburn's methods. Although this result may appear to support the conclusion that settlement can be estimated as accurately using SPT test results as using CPT test results, it is important to recognize that the statistics were developed using two different data bases. As discussed subsequently, the statistical results are somewhat dependent on the data base used. If a larger data base of CPT test results was available, analyses using it might support the conclusion that it is possible to estimate settlement more accurately using CPT test results, a conviction shared by many experienced soil engineers.

It appears from Figure 3.1 that methods based on empirical correlations (e.g., Alpan's method) or based partially on elastic theory (e.g., D'Appolonia et al.'s method) are as accurate as those derived using statistical evaluation of case records, such as Schultze and Sherif's method. This apparently does not support the observations of some investigators (Clayton, et al., 1988; and Burland, et al., 1977), who advocated the use of the methods based on evaluations of settlement records of similar structures. These investigators believed that the case-record based approach is more sound than the use of SPT blow count as a measure of sand compressibility, because the correlation between the SPT N-value and compressibility is believed to be rather weak. However, as discussed earlier, in addition to the uncertainties involved in estimating soil modulus values from field test results, the inherent uncertainties associated with the chance variations in the densities of sand deposits may also affect the accuracy of settlement estimates. Since it is difficult to quantify the inherent uncertainties in conventional methods for estimating settlements, it is not surprising that these methods, no matter how they are derived, provide the same range of values of accuracy and reliability, as indicated in Figure 3.1.

It is thus clear that a understanding of the relationship between the accuracy and reliability is important. The results from Figure 3.1 indicate that there is a tradeoff between accuracy and reliability. Improvement in reliability seems always to involve poorer accuracy, and improvement in accuracy seems always to result in lower reliability.

3.3.5 Adjustment of The Level of Accuracy and Reliability

The accuracies and reliabilities of any of the methods of estimating settlements of footings on sands can be changed through the simple process of multiplying the values of settlement calculated using the "standard" method by a factor. An example is shown in Figure 3.2, for Alpan's method. The standard method, with no modification, is characterized by an accuracy of 0.99 and a reliability of 40 percent. If the values of settlement calculated using this standard method is

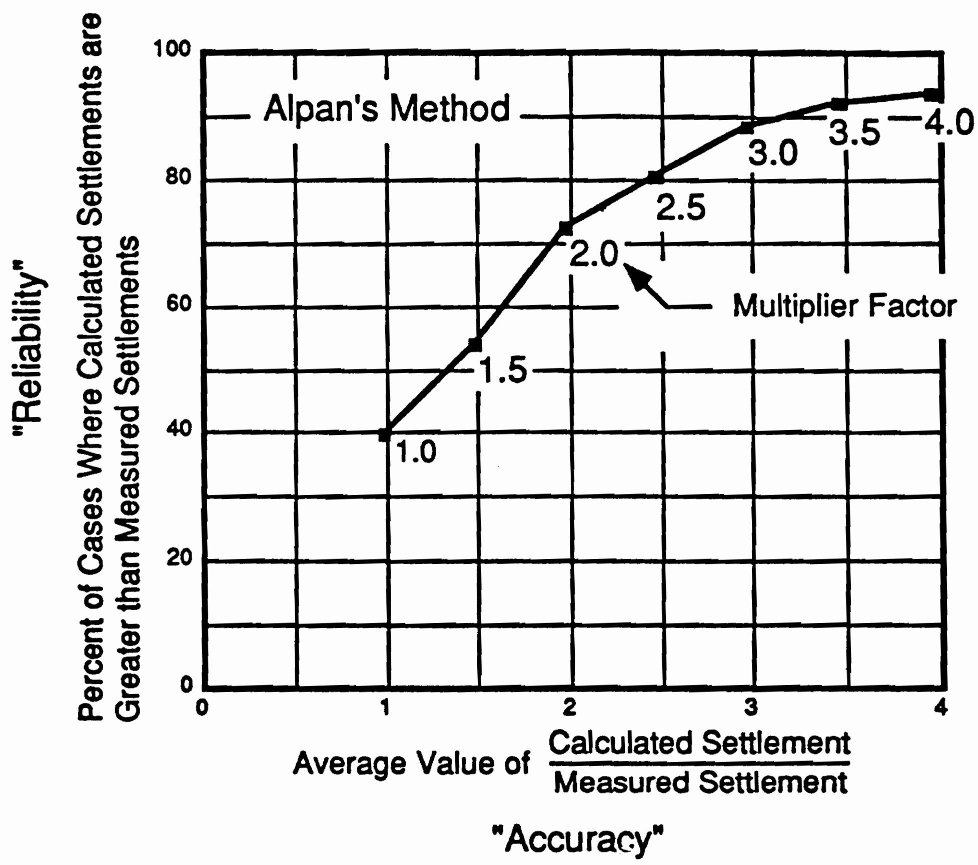


Figure 3.2: Possible Combinations of Accuracy and Reliability with Alpan Method

multiplied by 1.5, the accuracy increases to 1.48 and the reliability increases to 54 percent. Multipliers ranging from 1.0 to 4.0 have been used to develop the relationship between accuracy and reliability shown in Figure 3.2. It may be noted that the Alpan's method could be made 90 percent reliable if the calculated values of settlement were multiplied by 3.23. The corresponding accuracy would be 3.18.

Similar analyses have been made for all of the methods that relate settlement to SPT blow count. Because the data base for CPT results is much smaller, this type of analysis was not done for Schmertmann's method. The results for the SPT-based methods are summarized in Table 3.6 in terms of the multiplier factors needed to achieve 50, 60, 70, 80, and 90 percent reliability for each method, and the corresponding values of accuracy.

The optimum method is one that is characterized by the desired level of reliability and value of accuracy as near as possible to unity. If it was desired to achieve 50 percent reliability, there would be several candidates, all with the same level of accuracy. If the desired level of reliability was 70 percent, D'Appolonia, et al.'s method or Terzaghi and Peck's method would be the best candidates. With no modification (a multiplier of unity), the highest reliability is afforded by Terzaghi and Peck's method.

It should be noted that the results shown in Table 3.6

TABLE 3.6: MULTIPLIER FACTORS AND ACCURACIES CORRESPONDING TO 50, 60, 70, 80, AND 90 PERCENT

	Reliability									
	50 percent		60 percent		70 percent		80 percent		90 percent	
Method	Mult.	Acc.	Mult.	Acc.	Mult.	Acc.	Mult.	Acc.	Mult.	Acc.
Alpan (1964)	1.4	1.3	1.7	1.6	1.9	1.9	2.5	2.4	3.2	3.2
Burland and Burbidge (1985)	0.9	1.3	1.1	1.6	1.3	1.9	1.5	2.1	2.0	2.9
D'Appolonia and D'Appolonia (1970)	1.1	1.2	1.2	1.4	1.4	1.6	1.6	1.8	2.0	2.2
Duncan and Buchignani (1976)	0.7	1.2	1.0	1.6	1.2	2.0	1.5	2.4	2.0	3.2
Meyerhof (1956)	0.7	1.2	0.9	1.5	1.2	2.0	1.5	2.4	1.9	3.1
NAVFAC (1982)	1.0	1.3	1.2	1.5	1.5	1.8	1.8	2.2	2.4	3.0
Parry (1971)	1.2	1.3	1.5	1.7	1.9	2.1	2.4	2.6	3.6	4.0
Peck and Bazaraa (1969)	1.1	1.3	1.3	1.5	1.5	1.7	1.7	2.0	2.1	2.4
Peck, Hanson, Thornburn (1974)	0.45	1.2	0.5	1.4	0.65	1.8	1.0	2.9	1.4	3.7
Schultz and Sherif (1973)	1.3	1.3	1.4	1.5	1.6	1.7	1.9	1.9	2.3	2.4
Terzaghi and Peck (1967)	0.4	1.3	0.45	1.4	0.5	1.6	0.65	2.1	1.2	3.8

Mult. = multiplier required to achieve indicated level of reliability
Acc. = Accuracy

are somewhat dependent on the data base that was employed in the analyses. Similar analyses performed using only the Burland and Burbidge data base indicated that Schultze and Sherif's method would have the best accuracy at 90 percent reliability (about 2.0), and that the value of accuracy for D'Appolonia, et al.'s method at 90 percent reliability would be higher (about 3.8). While it seems reasonable that the most generally applicable results would be achieved using the larger, combined data base, it is clear that the results are dependent on the data base used, and should not be considered as definitive. The results are more sensitive at high values of reliability (like 90 percent), where curves of the type shown in Figure 3.2 are nearly horizontal, and small changes in reliability correspond to a large change in accuracy.

The results in Table 3.6 show that, except at high values of reliability (90 percent or more) where the results are sensitive, the accuracies of the methods for a given reliability, after some simple adjustments, are almost the same. Thus, from the viewpoint of reliability and accuracy, the methods may be considered as being essentially the same.

3.3.6 Selection of Settlement Estimate Methods

Ease of use, as well as accuracy, is of importance in the selection of engineering methods. In this regard it is useful to have some measure of the amount of time and effort required to estimate the settlement of a footing using the methods discussed previously. To develop some information concerning ease of use, the settlement of the same footing, shown in Figure 3.3 was estimated, by manual calculations, using all of the methods. The calculations involved processing the SPT blow count data for four borings, and making the other computations involved in each method. The results are summarized in Table 3.7.

Although the computation times listed in Table 3.7 are reflective of only a single example, they do provide an indication of the relative amounts of effort involved in using the various methods. The methods can be put into two main groups with regard to ease in use -- those that require less than 15 minutes for an estimate of settlement, and those that required 25 minutes or more. The methods that involve the greatest number of calculations (Alpan's, Peck and Bazaraa's, and Peck, Hanson and Thornburn's methods) do so primarily because of the time required to correct the SPT blow count values for overburden pressure (about 12 minutes for four borings).

Considering accuracy, reliability, and ease in use, a method can be selected that provides the best combination of characteristics for a particular purpose. For example, suppose that it is desired to have a method that can be used to estimate settlements with 70 percent reliability, with

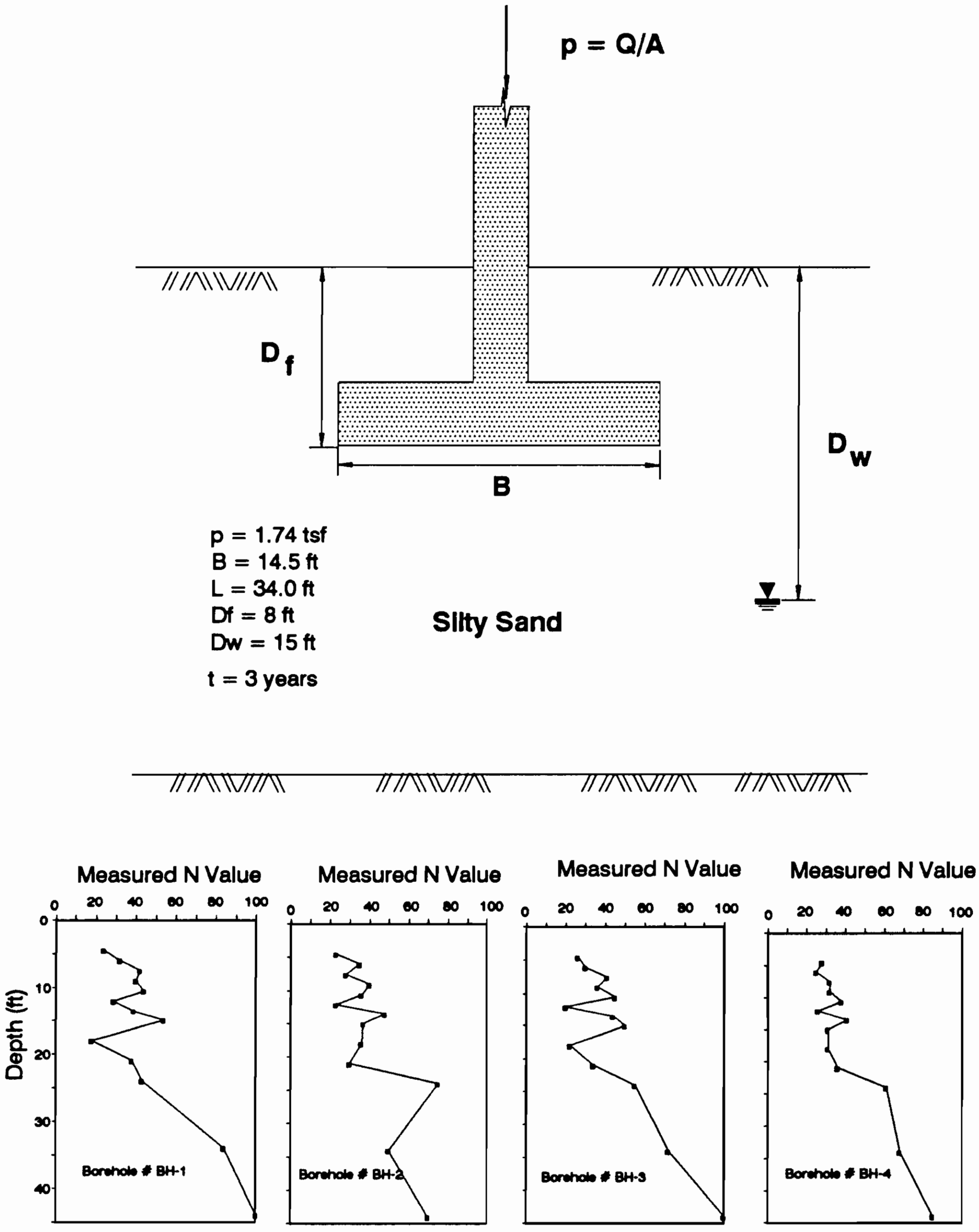


Figure 3.3: The Example Problem

TABLE 3.7: COMPUTATION TIMES FOR METHODS BASED ON SPT BLOW COUNT

Method	Computation Time (minutes)
Alpan (1964)	29
Burland and Burbidge (1985)	14
D'Appolonia and D'Appolonia (1970)	8
Duncan and Buchignani (1976)	9
Meyerhof (1956)	6
NAVFAC (1982)	8
Parry (1971)	9
Peck and Bazaraa (1969)	25
Peck, Hanson, Thornburn (1974)	25
Schultz and Sherif (1973)	6
Terzaghi and Peck (1967)	11

relatively good accuracy, and with little computation effort. The choice might be to use D'Appolonia, et al.'s method or Terzaghi and Peck's method. With appropriate multiplier values, either would result in accuracy values of 1.6 for 70 percent reliability. The data in Tables 3.6 and 3.7 can be used as a basis for choosing methods corresponding to other values of reliability.

3.4 PROBABILISTIC EVALUATION

In this section, the reliability and accuracy of the methods of estimating settlements of footings on sand are examined using probabilistic concepts. As noted earlier, from the perspective of designers, a method is considered reliable if the settlements calculated using the method exceed or equal the observed settlements. Thus, in the context of probability theory, reliability of the settlement estimate can be defined as the probability that the calculated values exceed or equal the measured values, as given below:

$$\text{Reliability} = P\,(\rho_{calc} \geq \rho_{meas}) \qquad (3.1)$$

where P ( ) = probability

ρ_{calc} = magnitude of calculated settlement (in length units)

ρ_{meas} = magnitude of measured settlement (in length units)

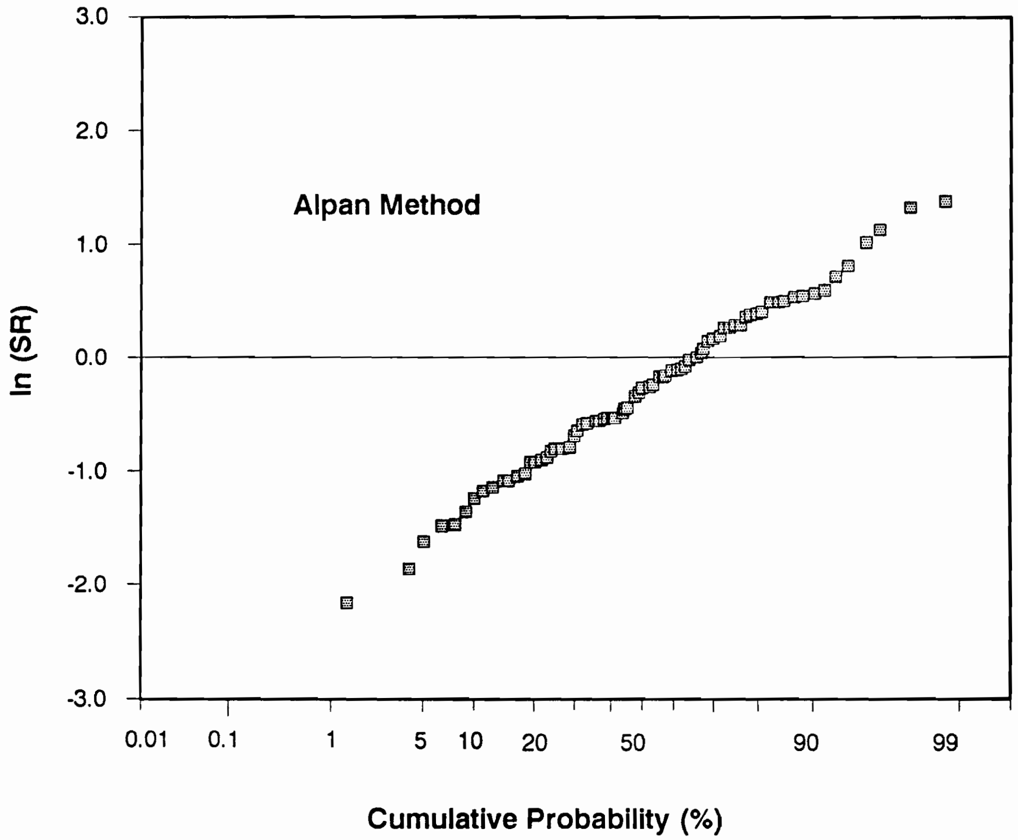
In terms of the settlement ratio (SR = ρ_{calc}/ρ_{meas}), reliability of a method can be written as:

$$\text{Reliability} = P\left(\frac{\rho_{calc}}{\rho_{meas}} \geq 1\right) \tag{3.2}$$

The accuracy of the method, on the other hand, can be measured in terms of the mean value of the settlement ratio. A perfectly accurate method will have a mean settlement ratio of 1.0.

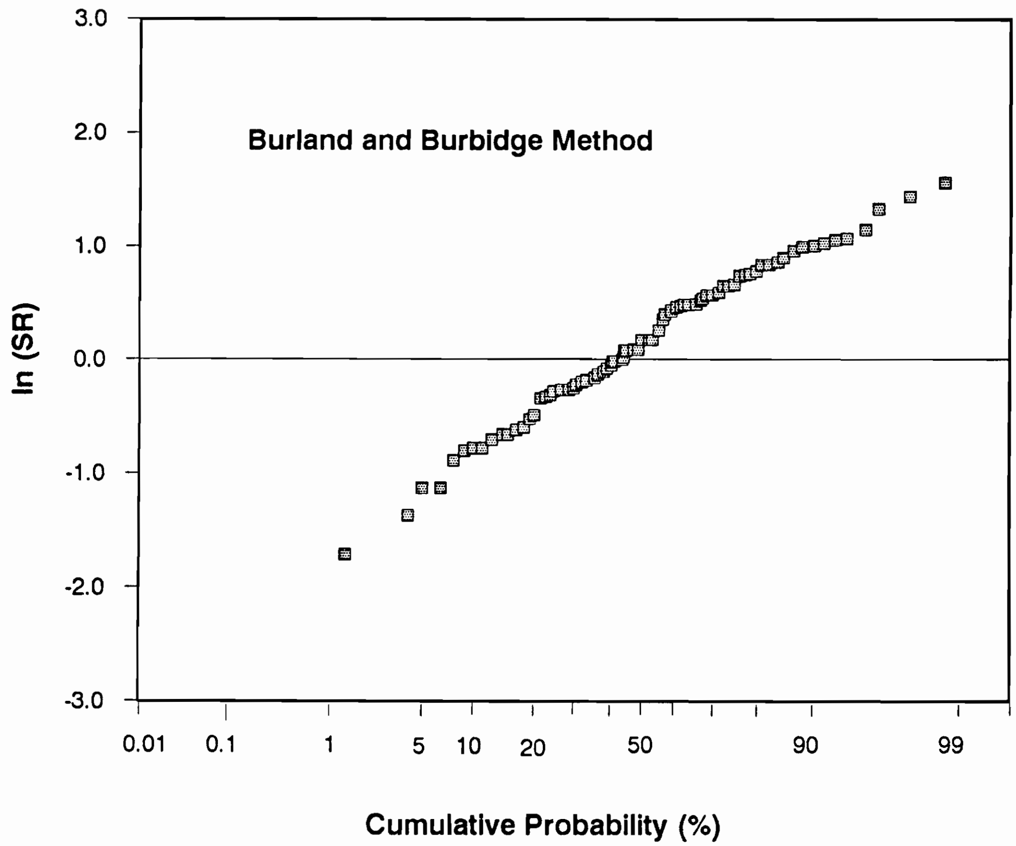
In order to determine the accuracy and the reliability, a knowledge of the probability density function of the settlement ratio is required. The cummulative probability for the logarithms of settlement ratios for the eleven SPT-based methods are plotted on normal probability papers, as shown in Figure 3.4. These plottings indicates that the probability density function for settlement ratio can be closely fitted by a lognormal distribution. Similar analysis was not performed for Schmertmann's method because the data base for CPT results was much smaller.

For a lognormal distribution, the reliability of the method defined in Equation 3.2 can be written in terms of the standardized normal variable, as presented below:



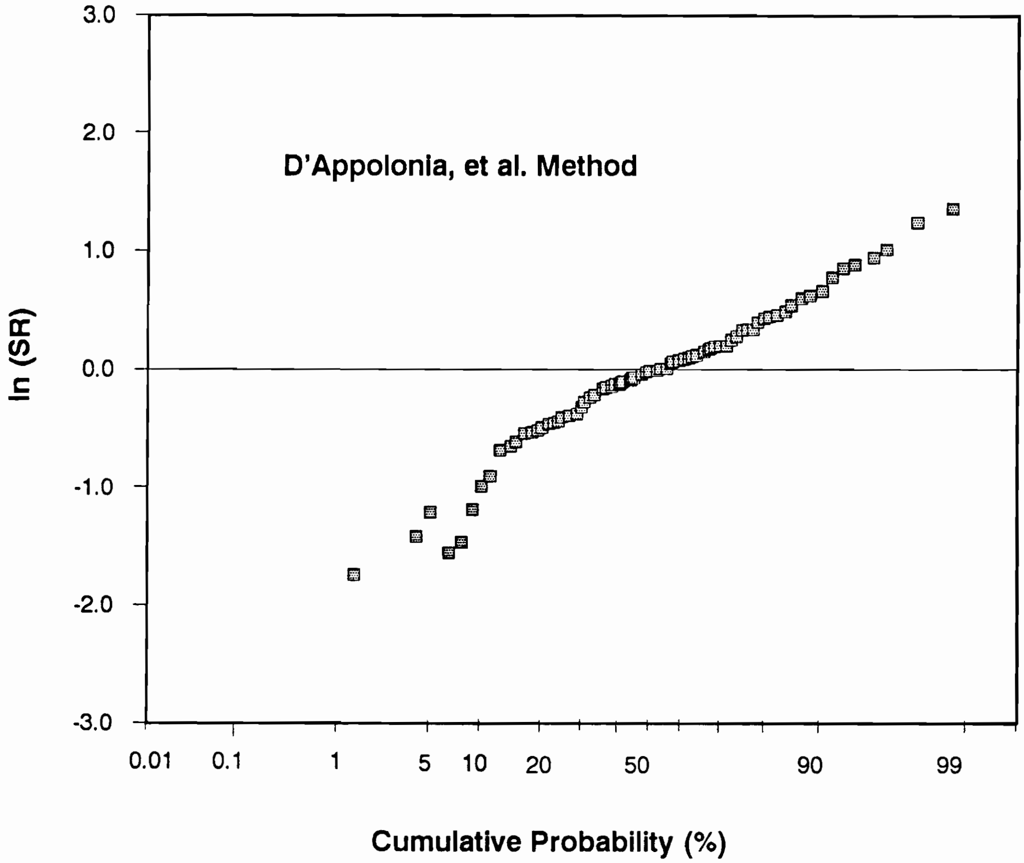
Note: SR = calculated settlement/measured settlement

Figure 3.4 a: Logarithms of Settlement Ratio on Normal Probability Paper - for Alpan Method -



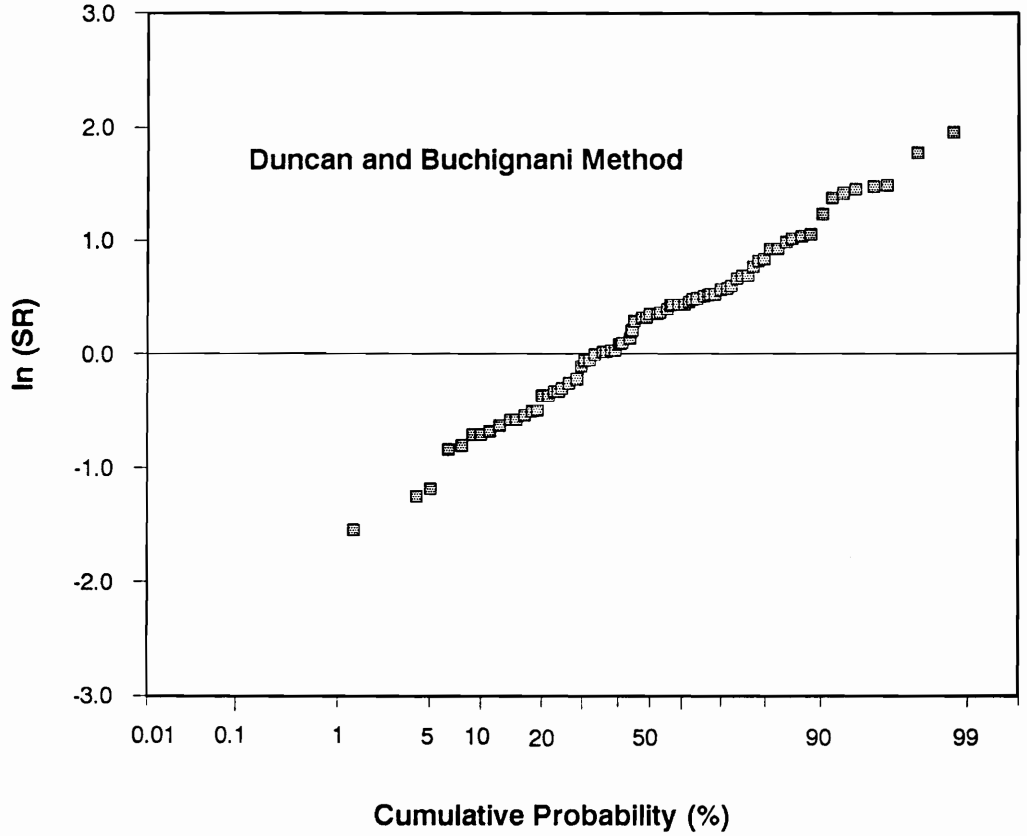
Note: SR = calculated settlement/measured settlement

Figure 3.4b: Logarithms of Settlement Ratio (SR) on Normal Probability Paper - for Burland and Burbidge Method -



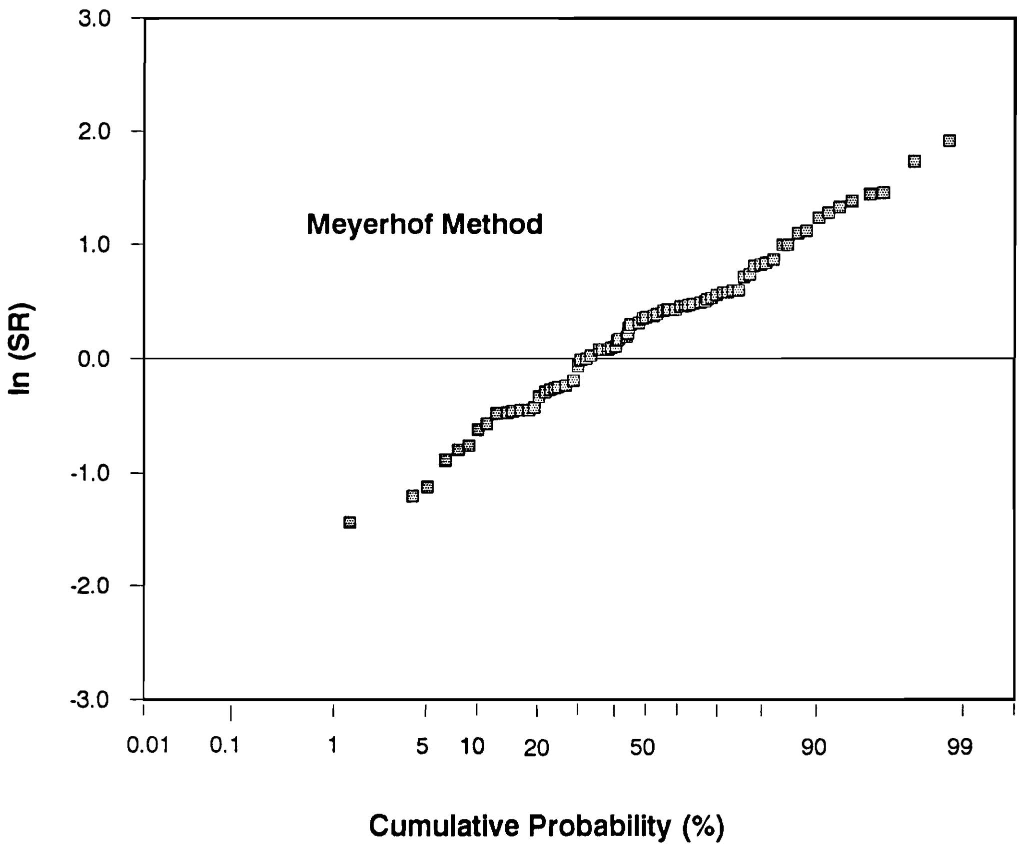
Note: SR = calculated settlement/measured settlement

Figure 3.4c: Logarithms of Settlement Ratio (SR) on Normal Probability Paper - for D'Appolonia, et al. Method -



Note: SR = calculated settlement/measured settlement

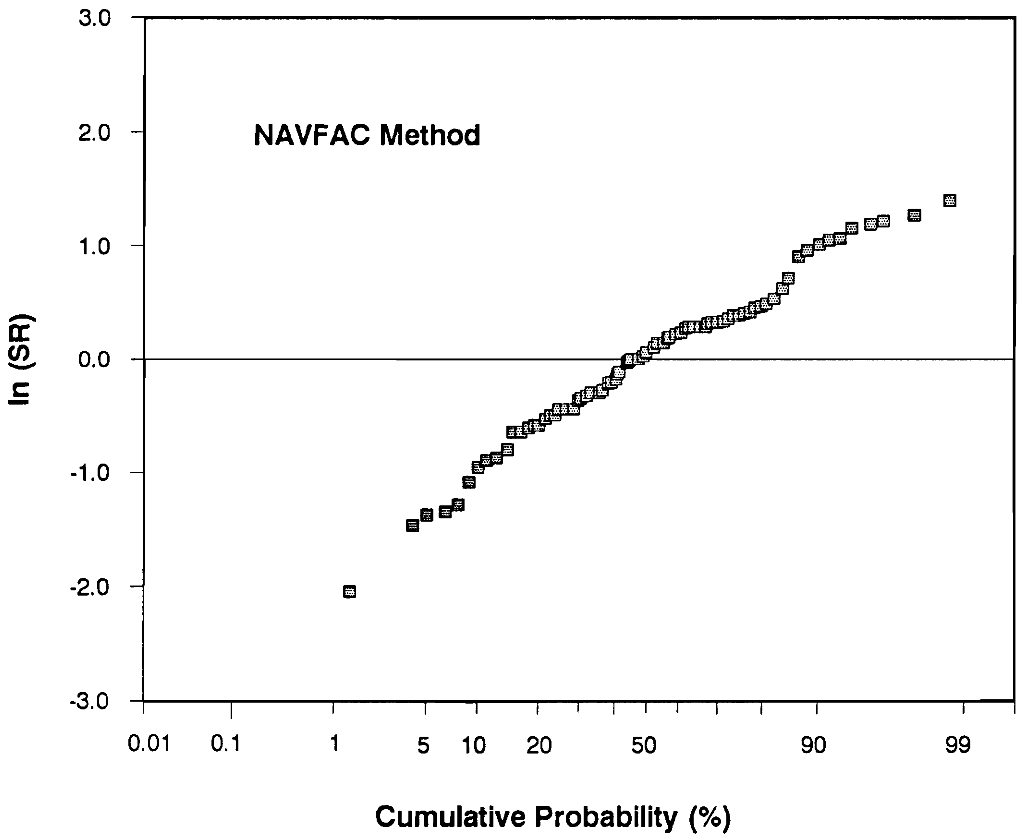
Figure 3.4d: Logarithms of Settlement Ratio (SR) on Normal Probability Paper - for Duncan and Buchignani Method -



Note: SR = calculated settlement/measured settlement

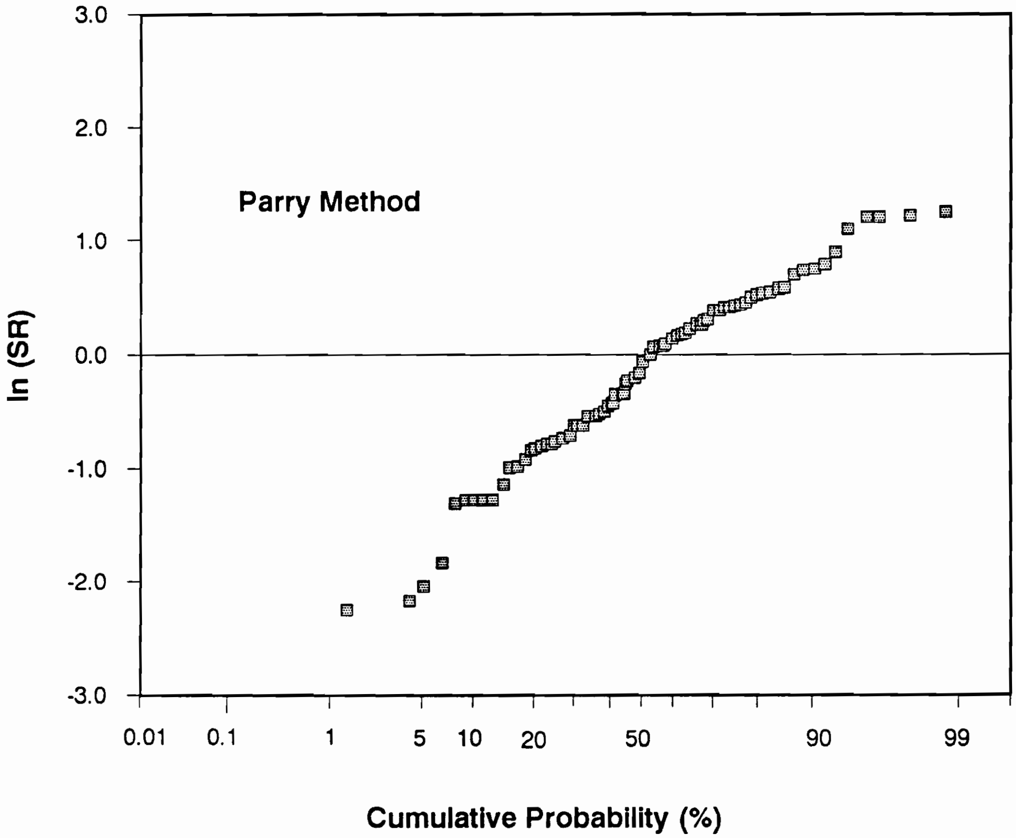
Figure 3.4e: Logarithms of Settlement Ratio (SR) on Normal Probability Paper

- for Meyerhof Method -



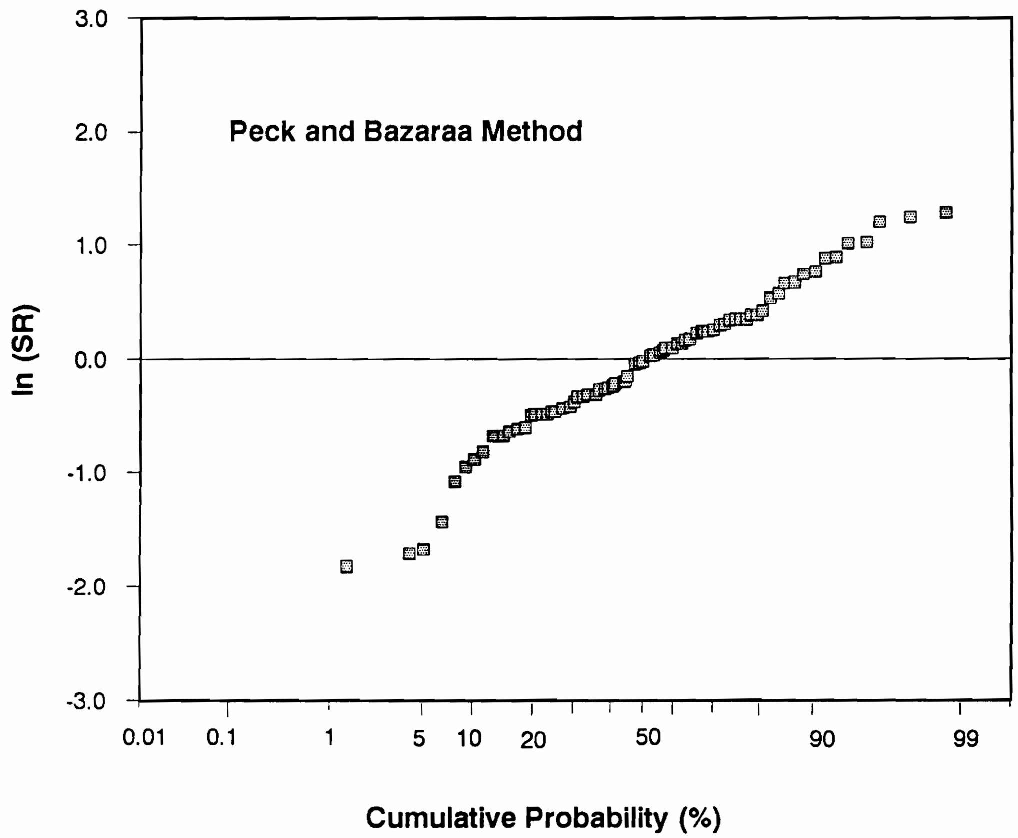
Note: SR = calculated settlement/measured settlement

Figure 3.4f: Logarithms of Settlement Ratio (SR) on Normal Probability Paper - for NAVFAC Method -



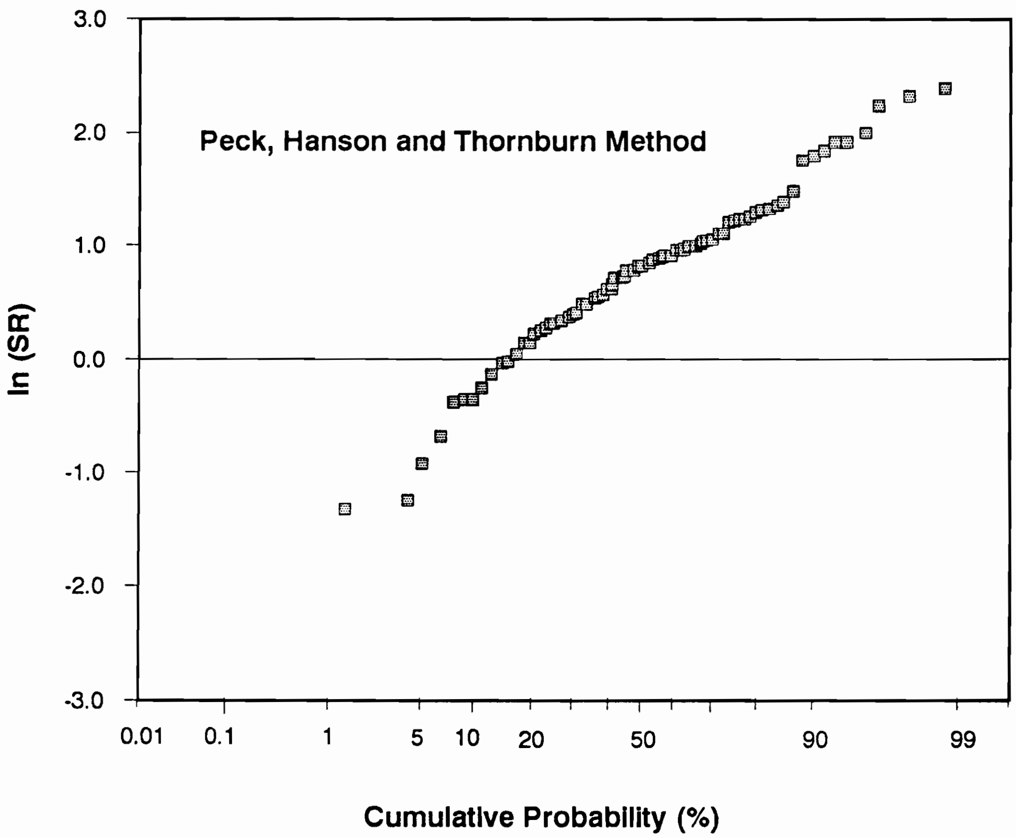
Note: SR = calculated settlement/measured settlement

Figure 3.4g: Logarithms of Settlement Ratio (SR) on Normal Probability Paper
- for Parry Method -



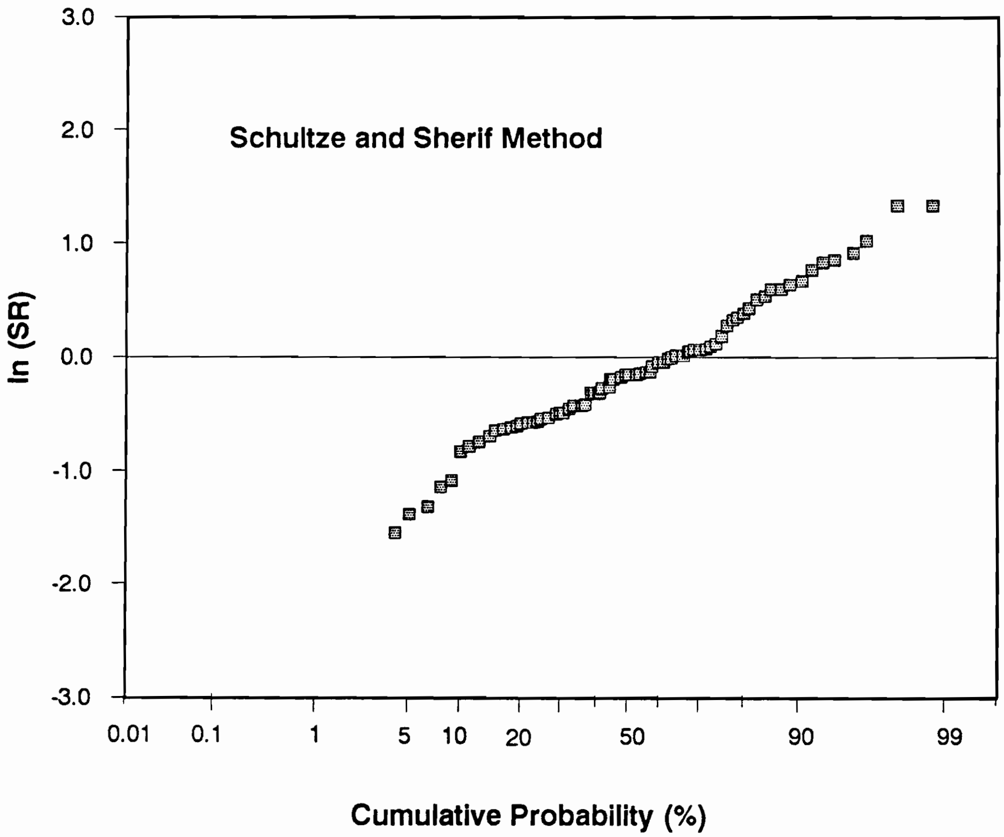
Note: SR = calculated settlement/measured settlement

Figure 3.4h: Logarithms of Settlement Ratio (SR) on Normal Probability Paper - for Peck and Bazaraa Method -



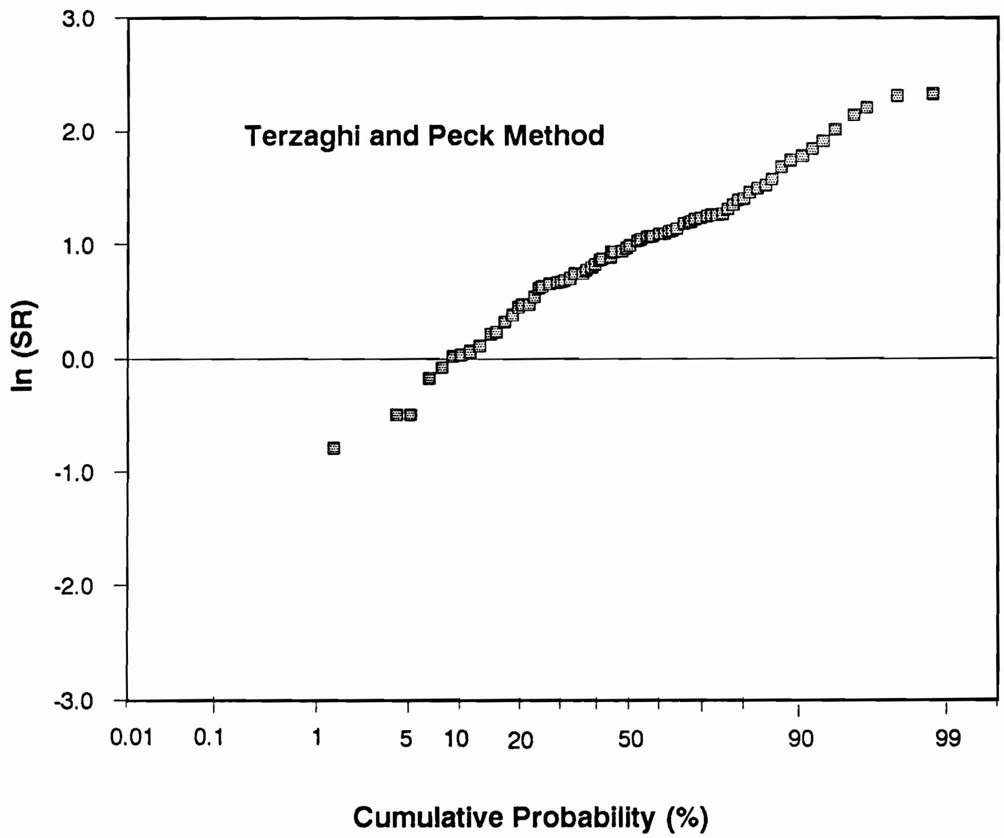
Note: SR = calculated settlement/measured settlement

Figure 3.4i: Logarithms of Settlement Ratio (SR) on Normal Probability Paper - for Peck, Hanson and Thornburn Method -



Note: SR = calculated settlement/measured settlement

Figure 3.4j: Logarithms of Settlement Ratio (SR) on Normal Probability Paper - for Schultze and Sherif Method -



Note: SR = calculated settlement/measured settlement

Figure 3.4k: Logarithms of Settlement Ratio (SR) on Normal Probability Paper - for Terzaghi and Peck Method -

$$\text{Reliability} = P\left(Z \geq -\frac{\overline{\log SR}}{\sigma_{\log SR}}\right) \tag{3.3}$$

where Z = standardized normal variable defined as:

$$Z = \frac{\log SR - \overline{\log SR}}{\sigma_{\log SR}} \tag{3.4}$$

in which logSR = logarithm of settlement ratio

$\overline{\log SR}$ = mean of variable logSR

$\sigma_{\log SR}$ = standard deviation of logSR

It is thus clear from Equations 3.3 and 3.4 that values of the mean and the standard deviation of the logarithms of the settlement ratio are required in the calculation of the reliability. These values were estimated using the statistics from the combined data base described previously in Section 3.3.2, and were used subsequently in conjunction with Equations 3.3 and 3.4 to determine the reliabilities of the eleven methods, which are summarized in Table 3.8. The reliabilities calculated using the probabilistic approach agree quite well with those shown in Figure 3.1.

As described in Section 3.3.6, the reliability of a method can be adjusted to any desired level. The multiplier

TABLE 3.8: RELIABILITIES OF METHODS OF ESTIMATING SETTLEMENTS OF FOOTINGS ON SAND CALCULATED USING PROBABILISTIC CONCEPTS

Method	Reliability (%)
Alpan (1964)	38
Burland and Burbidge (1984)	67
D'Appolonia, et al. (1970)	50
Duncan and Buchignani (1976)	64
Meyerhof (1956)	65
NAVFAC (1982)	52
Parry (1977)	44
Peck and Bazaraa (1969)	48
Peck, Hanson, Thornburn (1974)	85
Schultze and Sherif (1973)	42
Terzaghi and Peck (1967)	92

factor required for such adjustment may be determined using the following procedure:

1. Calculate the standardized normal variable, Z, corresponding to the desired level of reliability from Equation 3.3;

2. Assuming the standard deviation of the "standard" method is the same as that of the adjusted method, determine the mean of the logarithms of settlment ratio for the adjusted method;

3. Calculate the value of the multiplier factor by dividing the adjusted mean estimated in Step 2 by the mean of the logarithms of the settlement ratio for the "standard" method.

Following this procedure, values of the multiplier factor for 70, 80, and 90 percent reliabilities were calculated for the eleven SPT-based methods of estimating settlements of footings on sand. The results are summarized in Table 3.9. The values agree quite well with those listed in Table 3.6.

TABLE 3.9: VALUES OF ADJUSTMENT FACTOR, k, FOR ACHIEVING 70, 80, AND 90 PERCENT RELIABILITY

Method	Value of k for Reliability of 70 %	80 %	90 %
Alpan (1964)	2.0	2.7	3.7
Burland and Burbidge (1984)	1.3	1.6	2.2
D'Appolonia, et al. (1970)	1.4	1.6	2.0
Duncan and Buchignani (1976)	1.1	1.4	2.0
Meyerhof (1956)	0.9	1.4	1.9
NAVFAC (1982)	1.4	1.8	2.5
Parry (1977)	1.8	2.4	3.4
Peck and Bazaraa (1969)	1.4	1.8	2.4
Peck, Hanson, Thornburn (1974)	0.7	0.9	1.2
Schultze and Sherif (1973)	1.6	2.0	2.7
Terzaghi and Peck (1967)	0.6	0.7	1.0

CHAPTER 4

PROCEDURE FOR ESTIMATING MOVEMENTS OF FOOTINGS AND RETAINING WALLLS

4.1 INTRODUCTION

Design considerations for horizontal movements and rotations of footings are as important as for vertical movements (settlements). Studies of damage to bridges and buildings caused by ground movements (Walkinshaw, 1978; Bozozuk, 1978; and Geddes, 1984; Moulton, et al., 1985) have shown that horizontal movements tend to be even more damaging than vertical movements. Thus, especially for footings that carry significant horizontal loads and/or bending moments, as do the footings of bridge abutment, and retaining walls, consideration of likely horizontal movement and rotation is an important aspect of design.

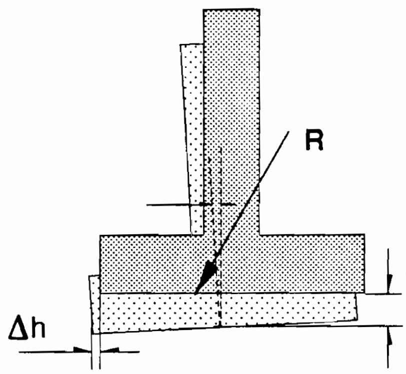
For most structures, displacements of footings arise primarily from the weights of the structure and its foundation, and from other superimposed loadings. In the case of abutments and retaining walls, additional movements can also occur as a result of the compression of the foundation material due to the weight of backfill behind the wall.

This chapter is concerned with the procedures used to estimate movements of individual footings that are subjected to inclined and eccentric loads. In addition, a simple procedure for estimating the additional movements of abutments and retaining walls, resulting from the weight of the backfill, is also presented.

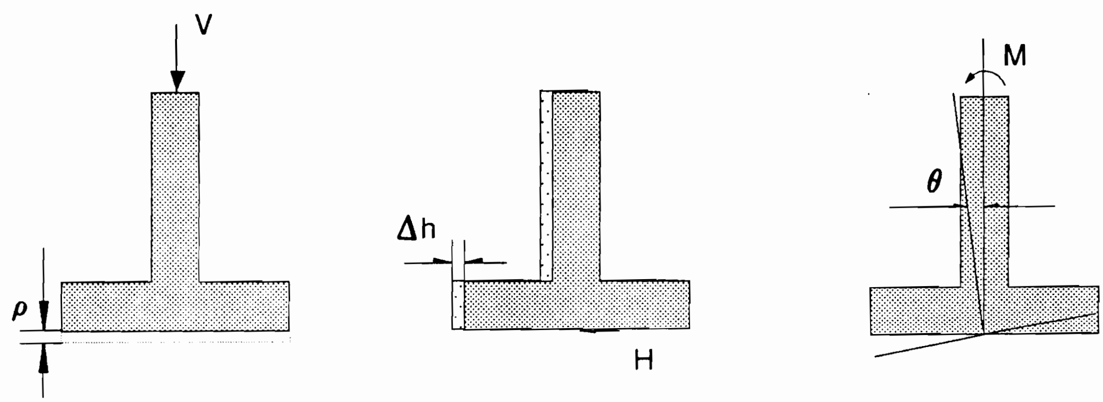
4.2 COMPONENTS OF MOVEMENTS FOR FOOTINGS SUBJECTED TO ECCENTRIC AND/OR INCLINED LOADINGS

The movements of footings under eccentric and inclined loads may be described in three different modes: vertical displacement (settlement), horizontal movement (translation), and rotation. These three modes of displacements can be combined to provide the overall response of the footing.

Inclined and eccentric loads can be resolved into vertical and horizontal resultant loads, plus a moment, as shown in Figure 4.1. The settlements, horizontal movements, and rotations can then be computed based solely on the vertical resultant load, the horizontal resultant load, and the moment loadings, respectively. As described in Chapters 2 and 3, settlements of footings can be estimated using several established procedures. Most of these procedures have been studied extensively, and their limitations and reliabilities are well documented. Estimations for horizontal movements and rotations, on the other hand, are usually made by using



(a) Combined Movements



(b) Components of Movements

Figure 4.1: Movements of Footings Subjected to Inclined and Eccentric Loads

elastic theory. Although estimations of horizontal movements and rotations have been compared to the results of model studies, very few field observations are available.

4.3 PROCEDURE FOR ESTIMATING HORIZONTAL DISPLACEMENTS AND ROTATIONS OF FOOTINGS

A footing with a width, B, and supported by an elastic material of thickness, H_t, is shown in Figure 4.2. According to elastic theory, the maginitudes of the settlement, horizontal movement, and rotation of the footing are given by:

$$\rho = \frac{q_v B}{E} I_\rho \tag{4.1}$$

$$\Delta h = \frac{q_h B}{E} I_h \tag{4.2}$$

$$\theta = \frac{M}{ELB^2} I_\theta \tag{4.3}$$

in which ρ = settlement in same length units as B)

Δh = horizontal displacement (in same length units as B)

θ = rotation (in radians)

q_v = uniform vertical pressure (in pressure units)

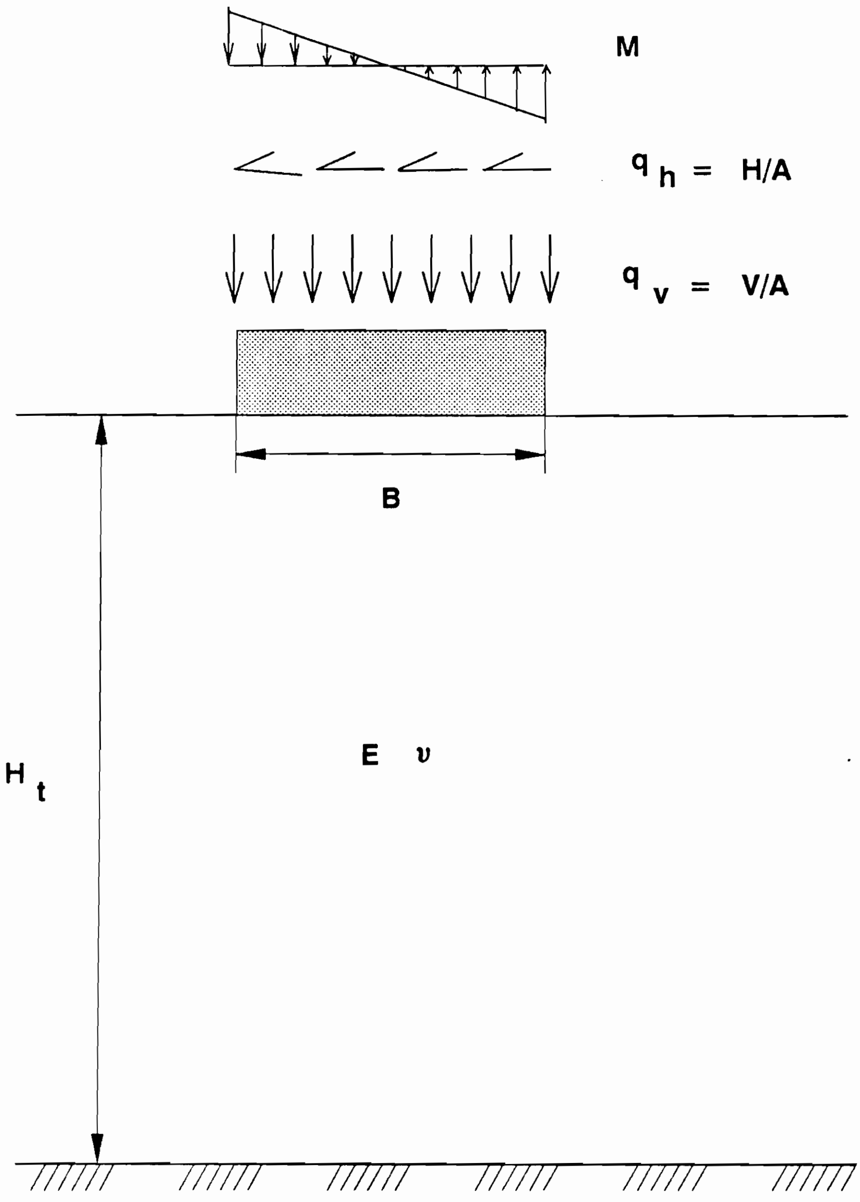


Figure 4.2: Simple Elastic Model for Estimating Movements of Individual Footings Subjected to Inclined and Eccentric Load

q_h = uniform horizontal pressure (in pressure units)

M = applied moment (in units of force x length)

B = footing width (in length units)

L = footing length (in same length units as B)

E = soil modulus (in pressure units)

I_ρ, I_h, and I_θ = dimensionless influence factors for estimating settlement, horizontal movement, and rotation, respectively.

In general, values of the displacement influence factors, I_ρ, I_h, and I_θ, are dependent upon the geometry of the loading area, and the values of Poisson's ratio for the soil. Even though real soils are typically non-homogeneous and anisotropic, they are usually assumed to be homogeneous and isotropic. In engineering design the effect of inhomogeneity is accounted for by the use of an appropriate "equivalent" soil modulus.

Many solutions for displacement influence factors have been published, and have been compiled by Poulos and Davis (1974). For estimating immediate settlements of foundations, the charts originally derived by Janbu, Bjerrum, and Kjaernsli (1956), and subsequently modified by Christian and Carrier (1978), and Taylor and Matyas (1983) are widely used. Alternatively, the chart developed by Butler (1974) and Meigh (1976), for soils whose modulus increases linearly with depth, can also be used. These charts are presented in Figures 4.3

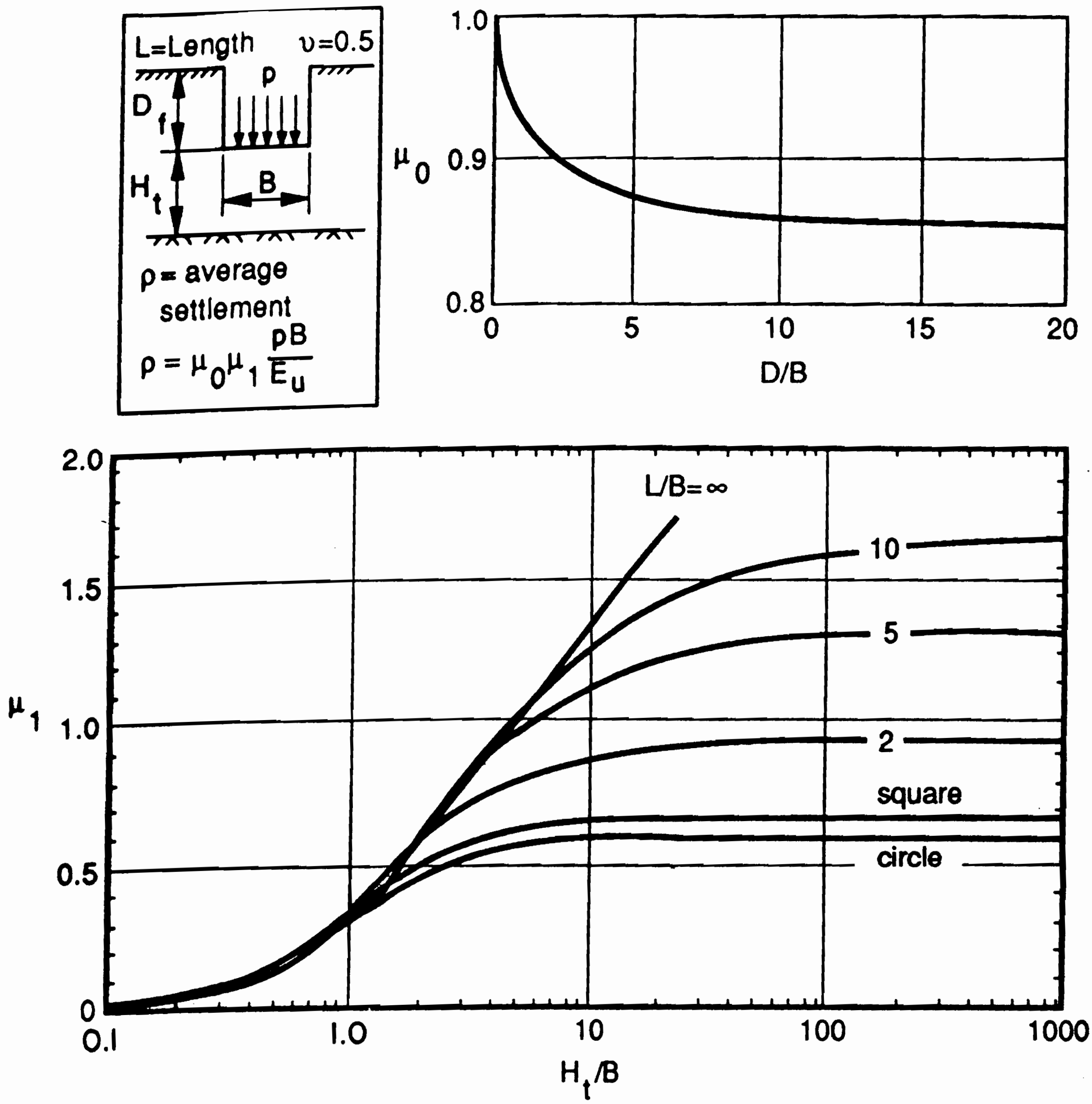


Figure 4.3: Settlement Influence Factors μ_0 and μ_1 (After Christian and Carrier, 1978)

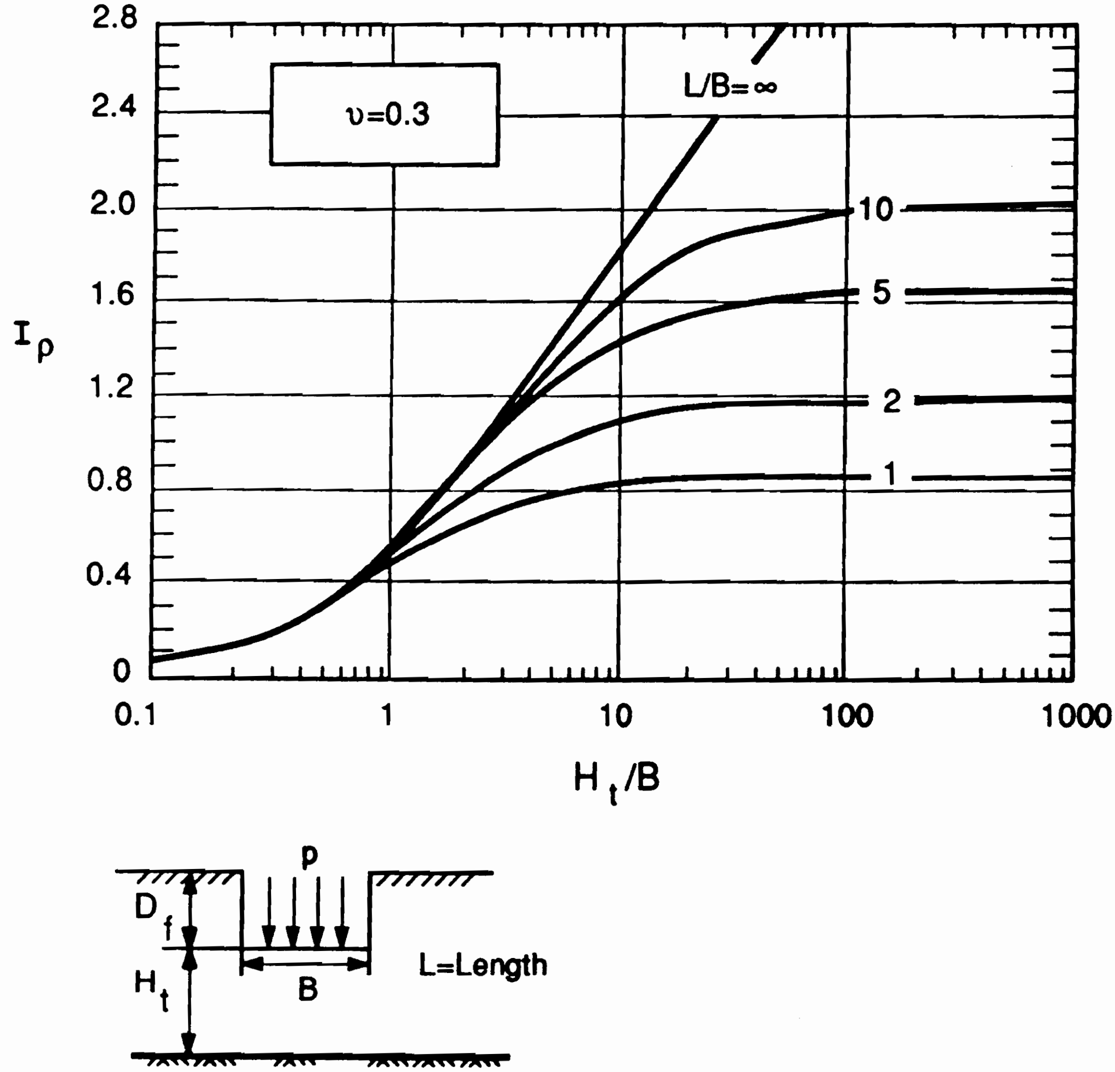


Figure 4.4: Settlement Influence Factor from Steinbrenner's Approximation Method for $\nu = 0.3$ (After Taylor and Matyas, 1983)

through 4.5.

Values of horizontal displacement and rotation influence factors computed, using the solutions collected by Poulos and Davis (1974), are summarized in Tables 4.1 and 4.2. The values given can be used directly with Equations 4.2 and 4.3. It should be noted that in some of the solutions collected by Poulos and Davis, the original expressions for estimating horizontal movements and rotations are different from those presented in Equations 4.2 and 4.3. For consistency, these expressions have been rearranged so that they are compatible with Equation 4.2 and 4.3.

4.4 COMPONENTS OF MOVEMENTS FOR RETAINING WALLS

Field observations (Tschebotarioff, 1958) and finite element studies (Clough and Duncan, 1971) have shown that movements of retaining walls are caused by the loadings from the weight of the structure and its backfill, as well as from surcharge loadings and earth pressures.

The types of movement induced by the various loadings are illustrated in Figure 4.6. The inclined and eccentric load at the base of the footing tend to produce a rotation of the wall away from the backfill (see Figure 4.6a). In the case of bridge abutments, these movements may reduce bridge clearances; and, if their magnitudes are large, they may

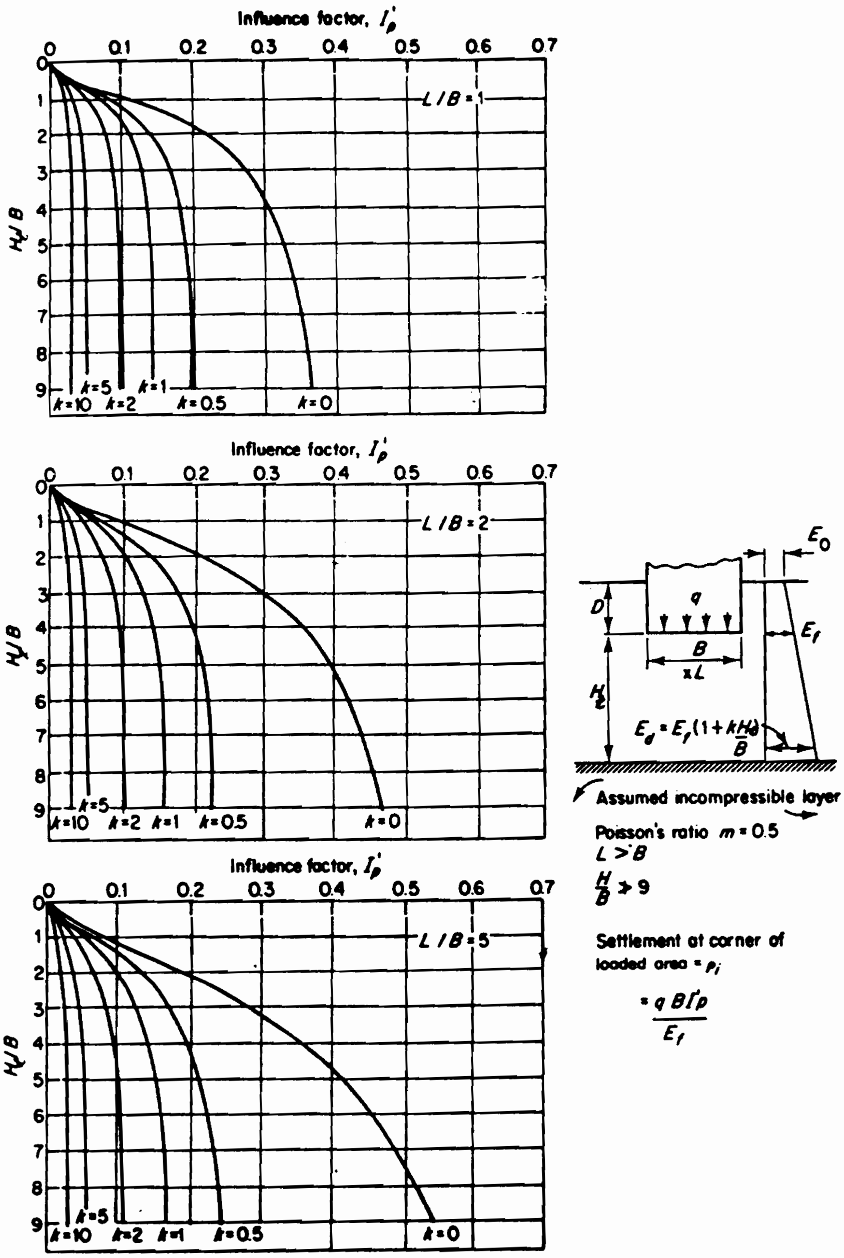


Figure 4.5a: Value of Influence Factor, I_q, for Deformation Modulus Increasing Linearly with Depth and Poisson's Ratio of 0.5 (After Butler, 1974)

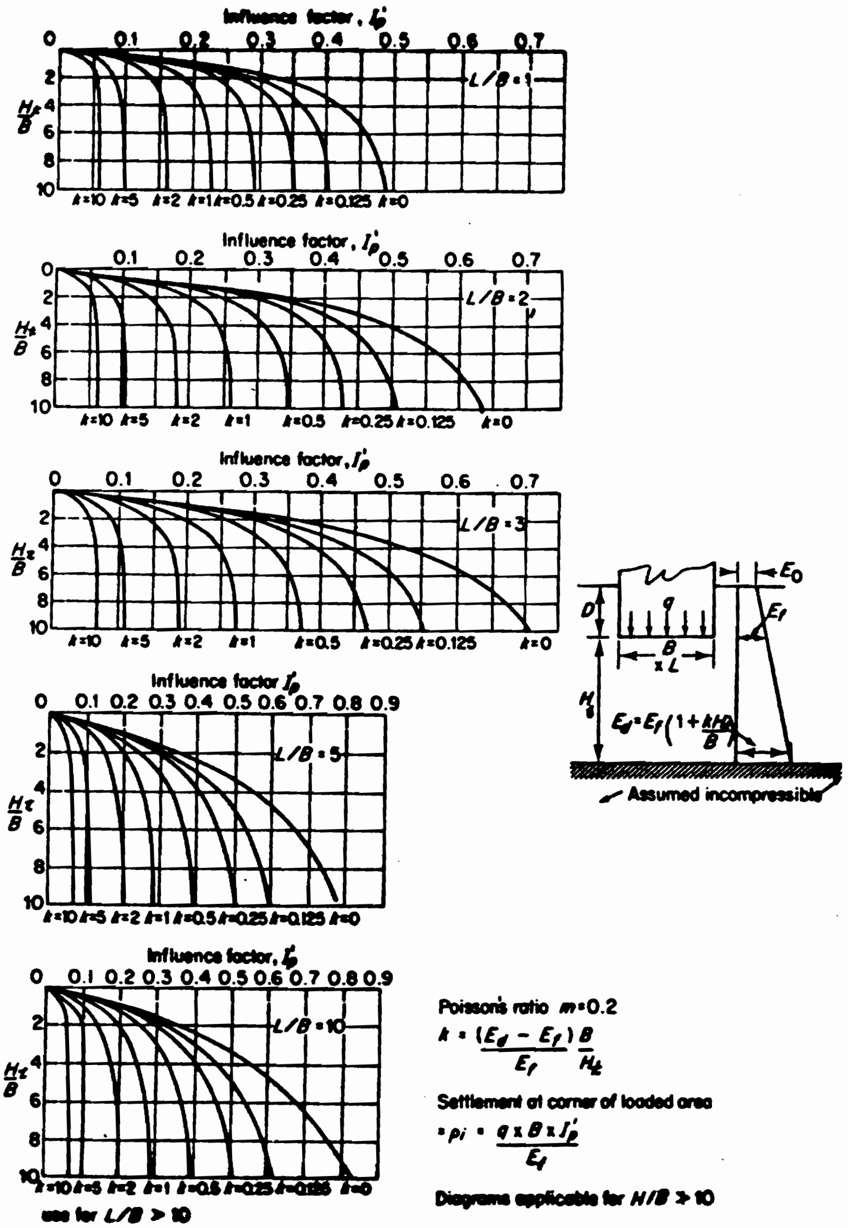


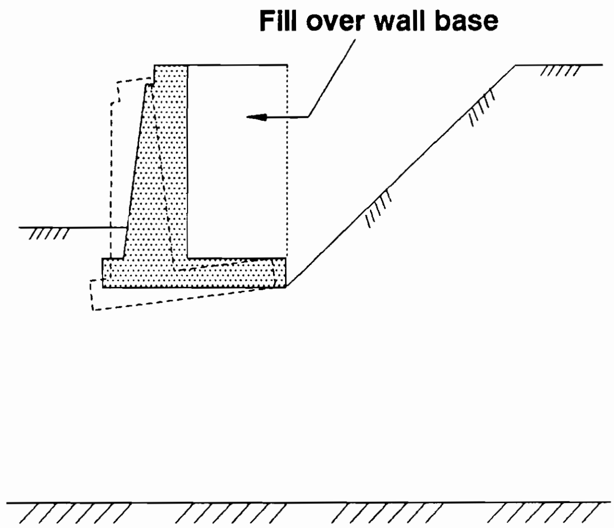
Figure 4.5b: Value of Influence Factor, I_q, for Deformation Modulus Increasing Linearly with Depth and Poisson's Ratio of 0.3 (After Meigh, 1976)

TABLE 4.1: VALUES OF HORIZONTAL MOVEMENT INFLUENCE FACTORS, I_h

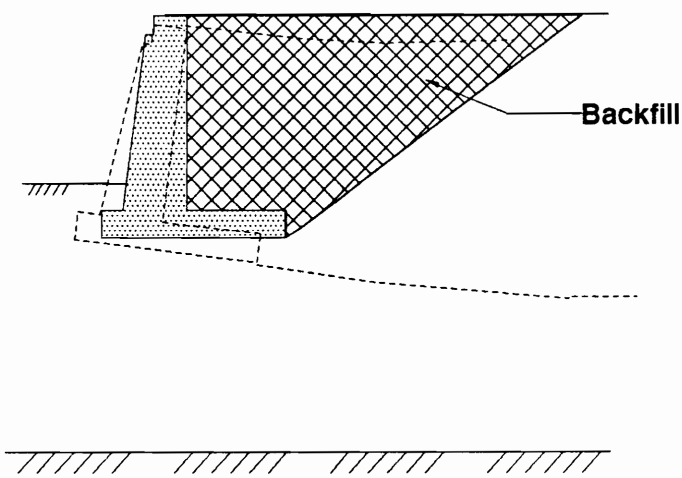
Footing Shape	Elastic Layer Thickness	Values of I_h For $\nu = 0.3$	For $\nu = 0.5$	Source
Square or	Infinite	1.68	1.77	Bycroft
Circle				(1956)
L/B = 5	Infinite	2.14	2.29	Barkan
L/B = 10	Infinite	2.77	2.82	(1962)
L/B =	$H_t/B = 3$	2.23	2.22	Milovic,
L/B =	$H_t/B = 2$	1.85	1.89	et al.
L/B =	$H_t/B = 1$	1.46	1.49	(1970)

TABLE 4.2: VALUES OF ROTATION INFLUENCE FACTORS, I_θ

Footing Shape	Elastic Layer Thickness	Values of I_h For $\mu = 0.3$	For $\mu = 0.5$	Source
Square or Circle	Infinite	3.37	2.78	Lee (1962); Whitman & Richart (1967)
L/B = 2	Infinite	3.94	3.29	Same as above
L/B =	$H_t/B = 3$	6.38	5.50	Milovic,
L/B =	$H_t/B = 2$	6.02	5.22	et al.
L/B =	$H_t/B = 1$	5.54	4.49	(1970)



(a) Wall Movements Due to Wall Loads



(b) Wall Movements During Placement of Backfill

Figure 4.6: Behavior of Abutment and Retaining Walls

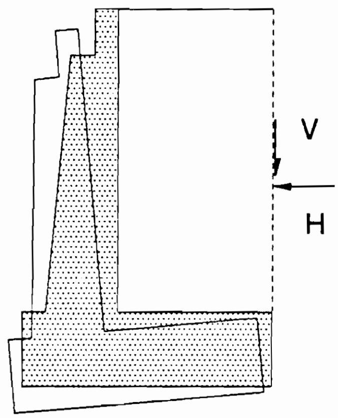
produce significant overstresses in the superstructure.

During placement of the backfill, a different pattern of movements occurs. The 'seat' of the movements is usually deeper; and, because settlements due to the backfill are larger at the heel than at the toe, the wall will tend to tilt backward in this stage (see Figure 4.6b). If the magnitude of these movements is excessive, the bridge deck may lose support and fall off the bridge seat.

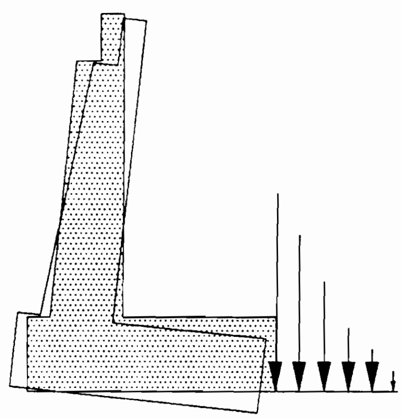
The complex nature of the wall movements discussed above may be divided into six components, as shown in Figure 4.7. The first three components are the settlement, horizontal movement and rotation of the footing due to the inclined and eccentric loads of the footing. The second group includes the settlement, horizontal movement, and rotation of the foundation soil beneath the footing due to the weight of the backfill. The net effect of these components, as depicted in Figure 4.7c, is usually a rotation of the wall toward the backfill, plus a horizontal translation away from the backfill.

The deformation of the foundation soil caused by the inclined and eccentric load is the same as that for individual footings described in Section 4.2. Thus, for estimates of such deformation, the appropriate procedures discussed previously for individual footings can be applied.

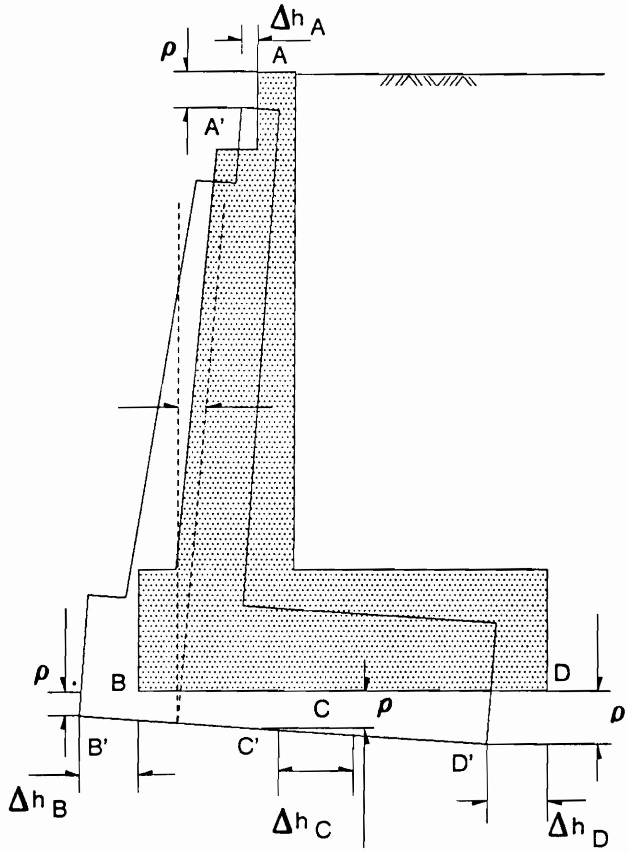
The displacements induced by the weight of the backfill



(a) Movements due to Vertical & Horizontal Loads on Wall, including Weight of Fill Over the Footing



(b) Movements due to Weight of Backfill



(c) Total Movements of Wall

Figure 4.7: Components of Movements for Abutment and Retaining Walls

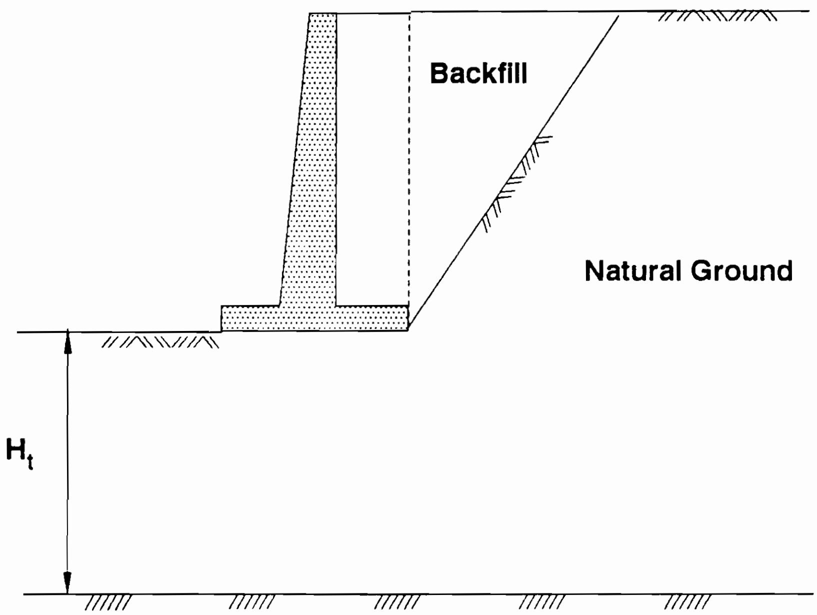
are quite different from those induced by the loads on the footing. It is apparent then that different procedures are required for estimating the displacements caused by the weight of the backfill. At present, there are no simple hand-calculation procedures for estimating such displacements. A simple procedure has been developed, based on elastic theory, and is discussed in the subsequent sections.

4.5 PROPOSED PROCEDURE FOR ESTIMATING MOVEMENTS OF RETAINING WALLS DUE TO THE WEIGHT OF BACKFILL

The elastic model used for developing the proposed procedure for estimating movements of retaining walls due to the weight of backfill is shown schematically in Figure 4.8. The pressure imposed by the triangular wedge of backfill is represented by a triangular loading, with a maximum intensity of q_{bf} at the heel of the wall. The compressible elastic layer, with a thickness of H_t, is assumed to be homogeneous and isotropic.

4.5.1 Settlements of Retaining Walls Due to Backfill

The average settlement of the retaining wall due to the weight of the backfill may be taken as the mean of the settlements at the toe and at the heel of the wall. According to elastic theory, settlements at the toe and heel of the wall can be calculated using the following expressions:



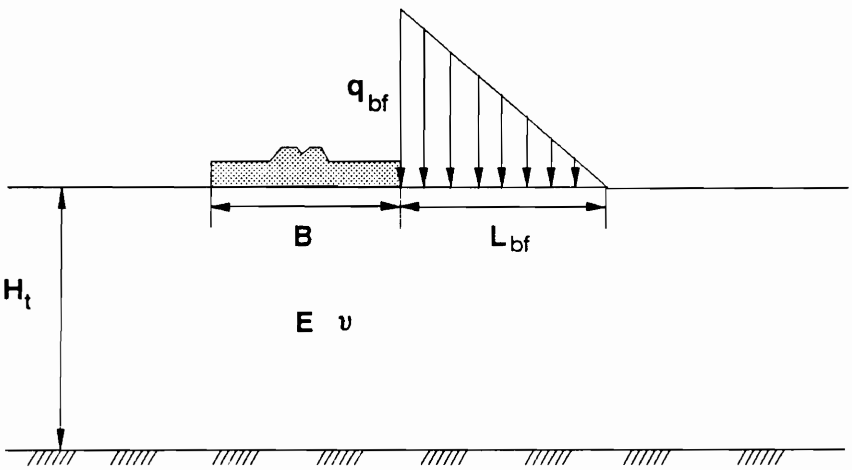


Figure 4.8: Schematic Drawing for the Elastic Model Used to Estimate the Components of Movements of Abutment and Retaining Walls Due to the Weight of Backfill

$$\rho_{heel} = \frac{q_{bf} \, L_{bf}}{E} I_{heel} \tag{4.4}$$

$$\rho_{toe} = \frac{q_{bf} \, L_{bf}}{E} I_{toe} \tag{4.5}$$

where ρ_{heel} = settlement at heel of the wall due to the weight of backfill (in same length units as L_{bf}),

ρ_{toe} = settlement at toe of the wall due to the weight of the backfill (in same length units as L_{bf}),

$q_{bf} = \gamma H$ = maximum intensity of the pressure imposed by the backfill (in pressure units),

γ = unit weight of the backfill (in units of force/length^3),

H = height of the retaininig wall (in same length units as L_{bf}),

L_{bf} = lateral extent of the backfill, measured from the heel of the wall (in length units),

I_{heel}, I_{toe} = dimensionless settlement influence factors due to the weight of backfill,

E = soil modulus (in pressure units).

It thus follows from Equations 4.4 and 4.5 that the expression for estimating the average settlement of the wall due to the backfill, ρ_{bf}, can be written as:

$$\rho_{bf} = \frac{q_{bf}\ L_{bf}}{E}\ I_{\rho bf} \tag{4.6}$$

where the average settlement influence factor, $I_{\rho bf}$, is given by

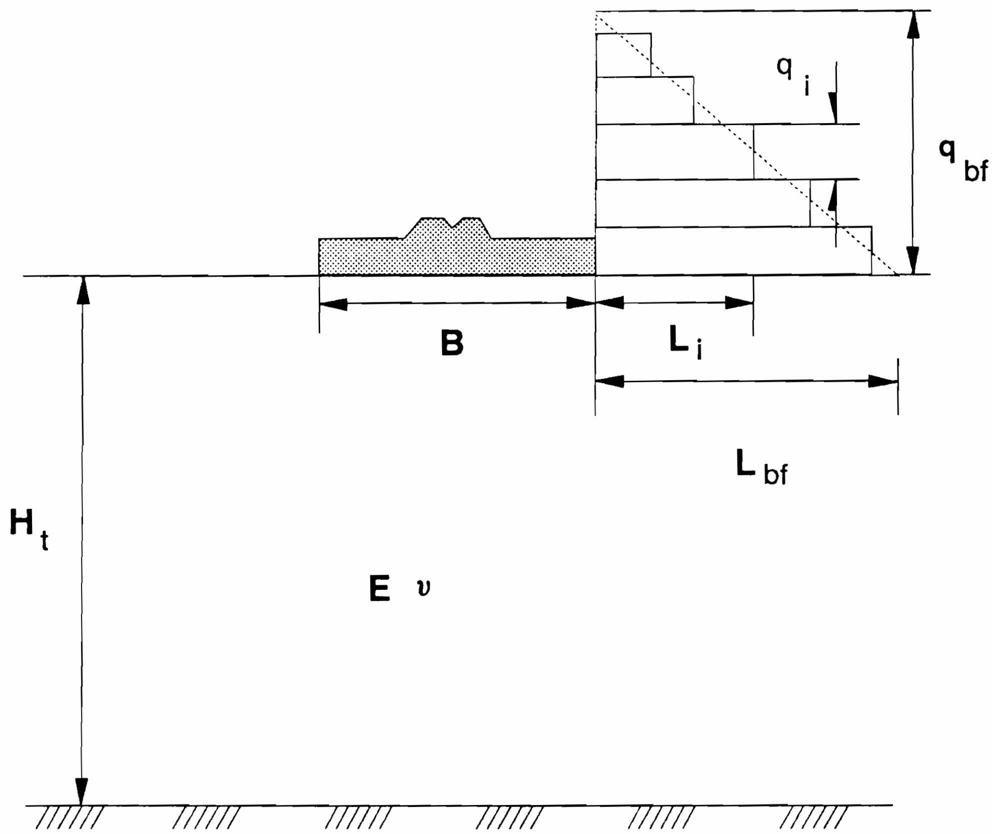
$$I_{\rho bf} = \frac{(I_{heel} + I_{toe})}{2} \tag{4.7}$$

However, no analysis has been published which provides settlement influence factors for the elastic problem considered in Figure 4.8. An approximate analysis can be made by treating the triangular loading as a series of finite strip loadings whose solution has been published by Ueshita and Meyerhof (1968) (see Figure 4.9). The total settlement at the heel due to the backfill is obtained by summing the settlements from the strip loads, as indicated by the equation:

$$\rho_{heel} = \Sigma\ \frac{q_i\ L_i}{E}\ I_i \tag{4.8}$$

where q_i = uniform pressure for the i^{th} strip loading,

L_i = width of the i^{th} strip loading,



$$\rho_{heel} = \frac{q_{bf} \; L_{bf}}{E} \, I_{heel}$$

where $$I_{heel} = \frac{\Sigma \, L_i \, I_i}{N \; L_{bf}}$$

and $$I_i = \text{function } (L_i / H_t, \upsilon)$$

Figure 4.9: Development of Influence Factors for Displacement Due to the Backfill at Heel of The Wall

I_i = settlement influence factor from Ueshita and Meyerhof (1968).

For convenience, the triangular loading may be divided into a number of strip loadings such that each strip load will have the same uniform pressure, q', which is equal to q_{bf}/N, where N = number of strip loadings. For this configuration, the settlement influence factor, I_{heel}, can be related to I_i by combining Equations 4.4 and 4.8, as follows:

$$I_{heel} = \frac{\Sigma\, L_i\ I_i}{N\ L_{bf}} \tag{4.9}$$

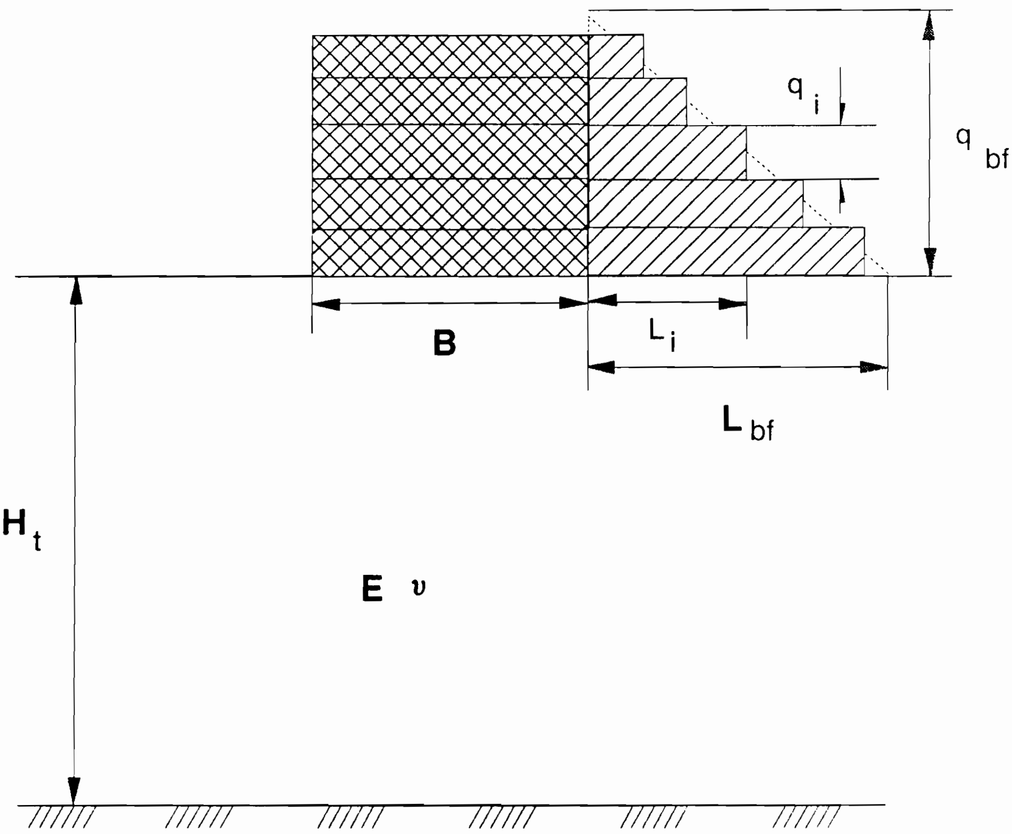
Similarly, settlement at the toe of the wall is given by (see Figure 4.10):

$$\rho_{toe} = \frac{q_1(L_1+B)}{E} I_1 - \frac{q_1 B}{E} I_1' + \frac{q_2(L_2+B)}{E} I_2 - \frac{q_2 B}{E} I_2'$$

$$+ \frac{q_3(L_3+B)}{E} I_3 - \frac{q_3 B}{E} I_3' + \cdots \tag{4.10}$$

Thus, for $q_1 = q_2 = \ldots = q_{bf}/N$, Equation 4.10 becomes

$$\rho_{toe} = \frac{q_{bf}}{EN} [\Sigma(L_i+B) I_i - N\ B\ I'] \tag{4.11}$$



$$\rho_{toe} = \frac{q_{bf}\ L_{bf}}{E}\ I_{toe}$$

where $$I_{toe} = \frac{1}{N\ L_{bf}}\left[\ \Sigma\ (L + B)\ I_i\ - N\, B\, I_i'\ \right]$$

and $$I_i = \text{function}\ (L_i / H_t,\ B/H_t\ ,\ \upsilon\ )$$

$$I_i' = \text{function}\ (B/H_t\ ,\ \upsilon\ )$$

Figure 4.10: Development of Influence Factors for Displacement Due to the Backfill at Toe of The Wall

Note that in Equation 4.11, $I' = I_1' = I_2' = \cdots$.

The approximate solution for I_{toe} can thus be obtained from Equations 4.5 and 4.11, as given below:

$$I_{toe} = \frac{1}{L_{bf}\overline{N}} [\Sigma(L_i+B)I_i - N\ B\ I'] \qquad (4.12)$$

where L_i, B, and N are defined by the geometry of the problem. Values of I_i can be obtained from the charts developed by Ueshita and Meyerhof (1968).

Values of I_{heel}, I_{toe}, and I_{bf}, determined using Equations 4.7, 4.9, and 4.12 are presented in Figures 4.11 through 4.13. These values vary with the geometry of the loading area, as well as with the Poisson's ratio of the soil.

4.5.2 Horizontal Movements of Retaining Walls Due to Backfill

The horizontal movements of retaining walls due to the load imposed by the backfill on the footing can be estimated in a similar manner. Their magnitudes may be calculated using the following expression:

$$\Delta h_{bf} = \frac{q_{bf}\ H_t}{E} I_{hbf} \qquad (4.13)$$

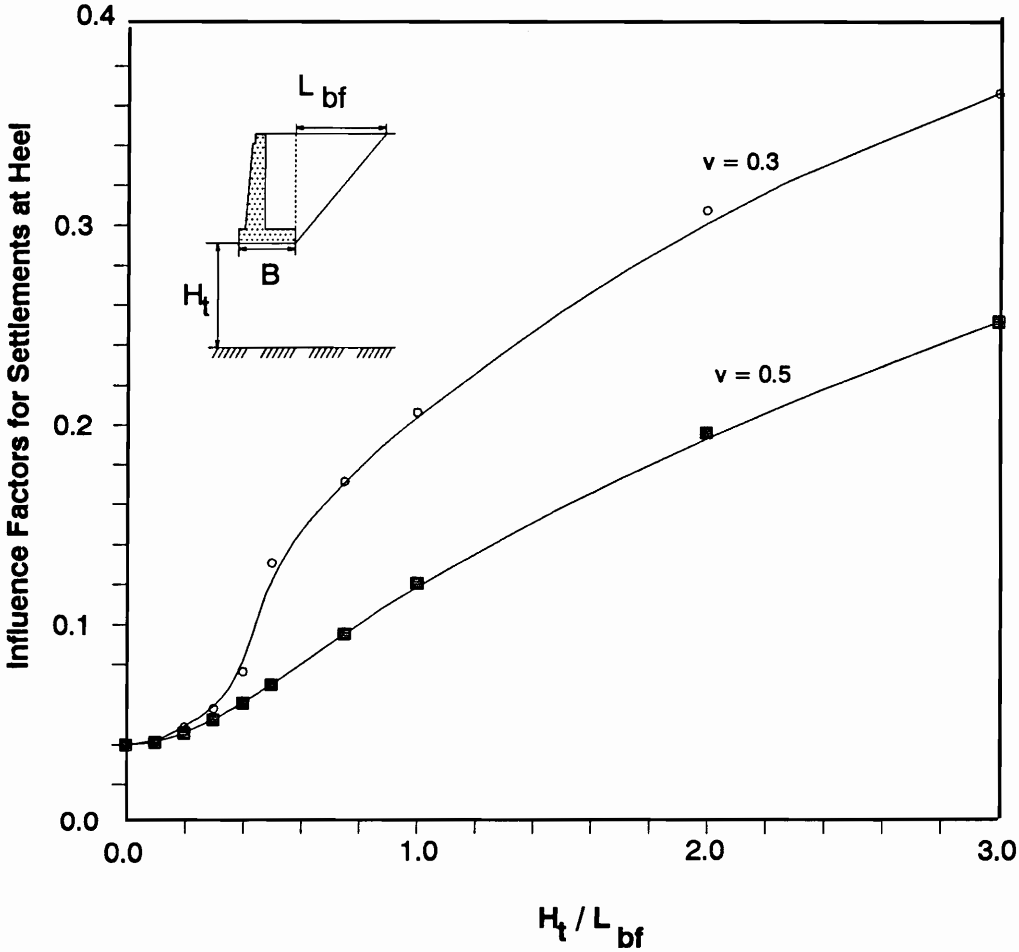
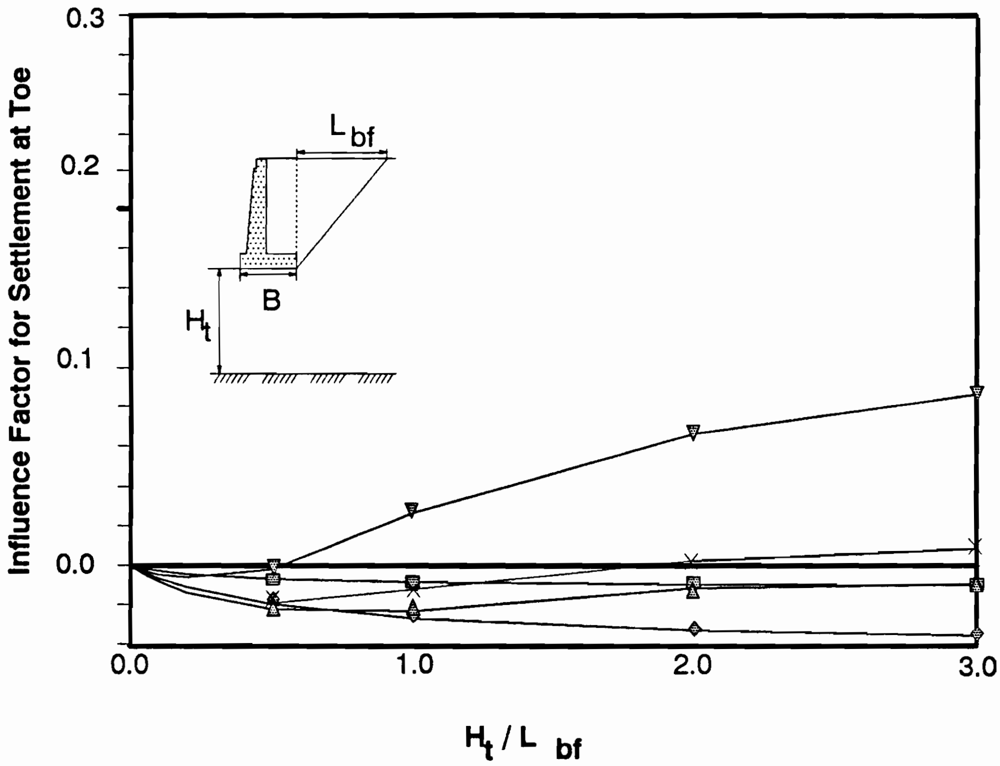


Figure 4.11: Values of Influence Factors for Settlements at Heel Due to the Backfill



Legend:

- B/Ht = 2.0
- B/Ht = 1.0
- B/Ht = 0.5
- B/Ht = 0.25
- B/Ht = 0.1

Figure 4.12a: Values of Influence Factors for Settlements at the Toe, Due to the Backfill (For Poisson's Ratio = 0.5)

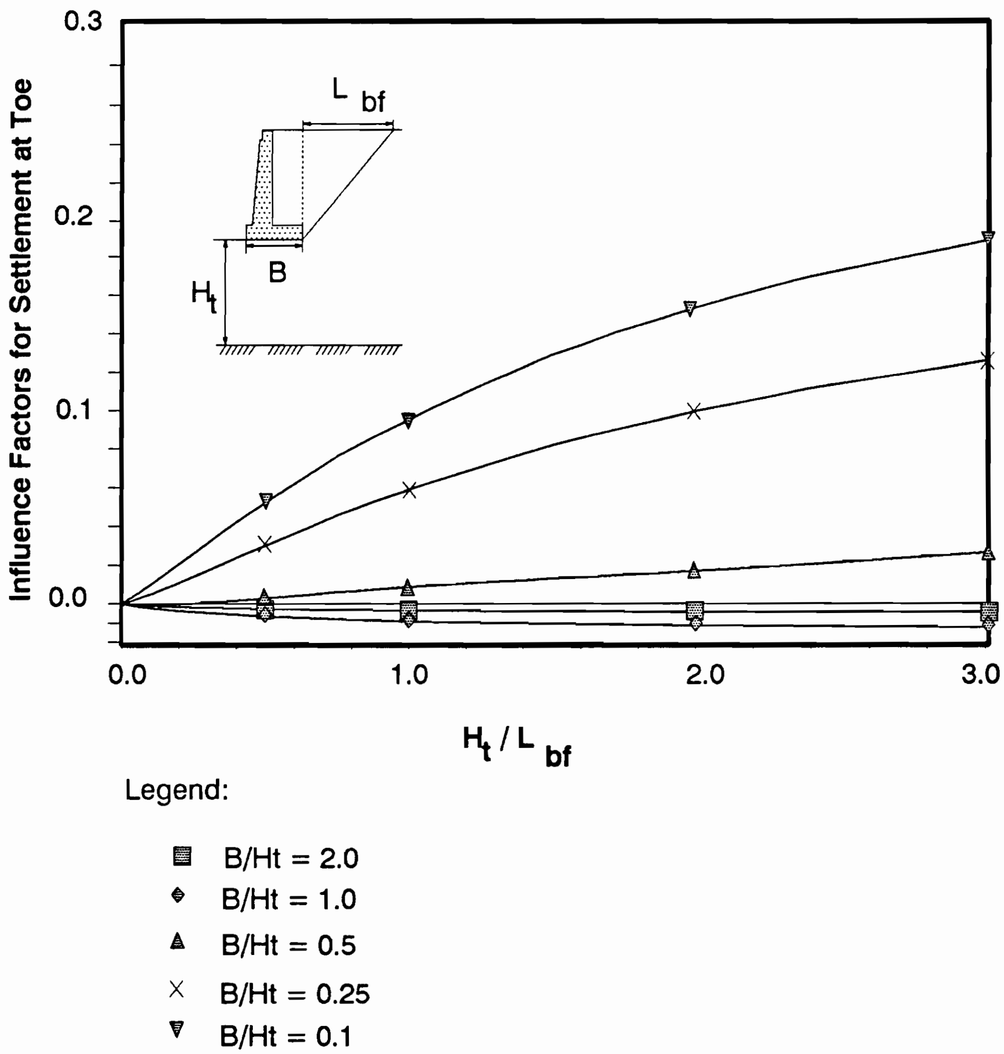


Figure 4.12b: Values Of Influence Factors for Settlements at the Toe, Due to the Backfill (For Poisson's Ratio = 0.3)

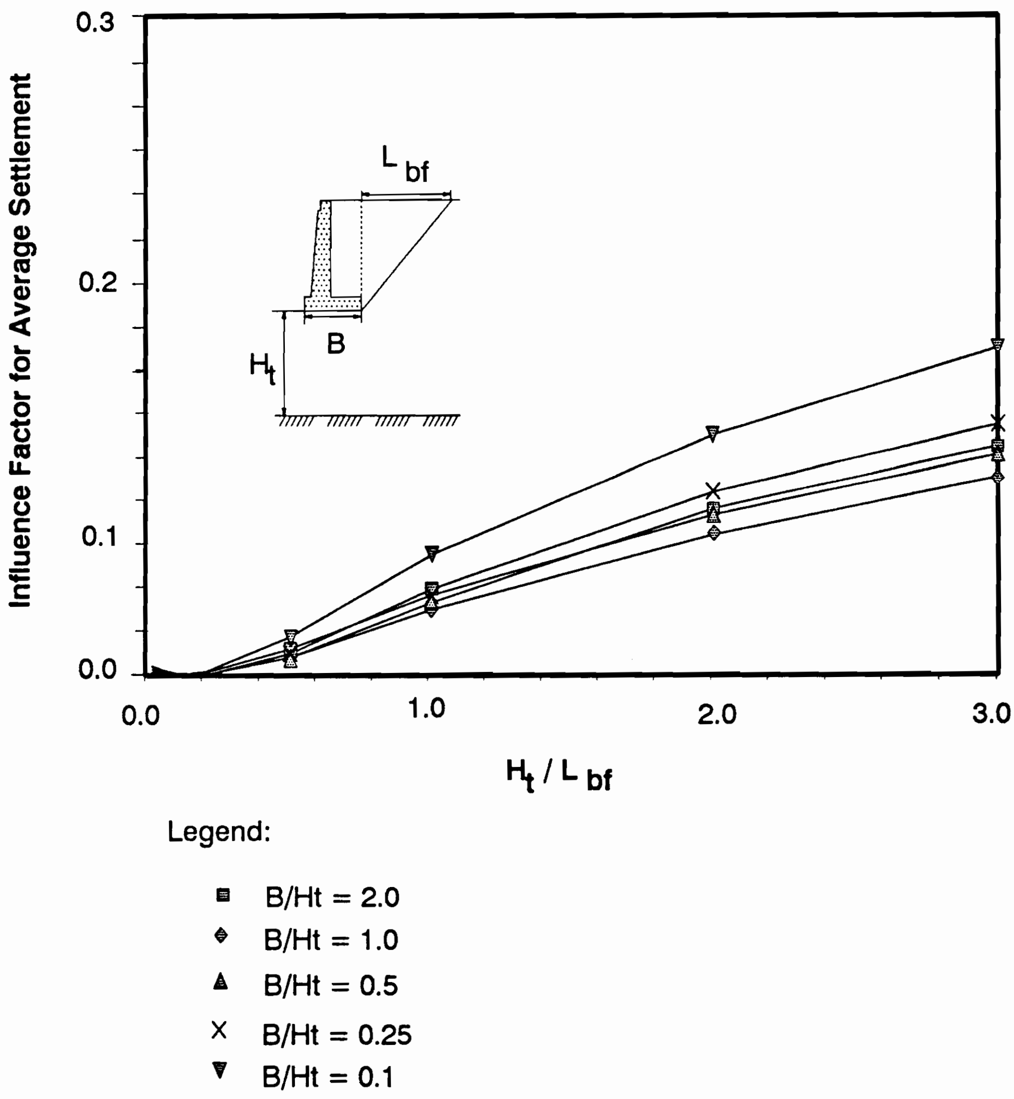


Figure 4.13a: Values of Influence Factors for Average Settlements Due to the Backfill (For Poisson's Ratio = 0.5)

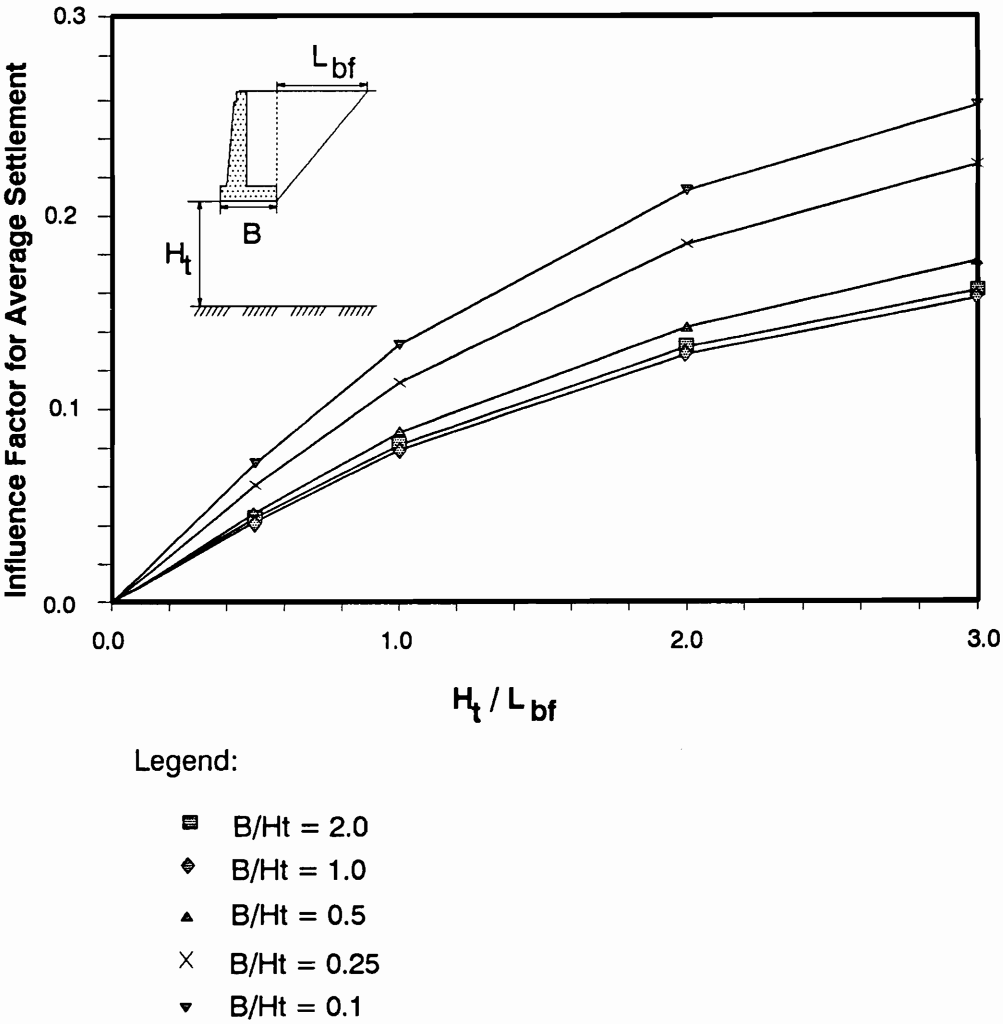


Figure 4.13b: Values of Influence Factors for Average Settlements Due to the Backfill (For Poisson's Ratio = 0.3)

in which Δh_{bf} = magnitude of horizontal movement due to backfill (in the same length units as H_t), I_{hbf} = dimensionless horizontal movement influence factors due to backfill, and other terms are as previously defined.

The horizontal movement influence factors can be determined using the same principles discussed in Section 4.5.1. In these analyses, the pressure imposed by the backfill is considered as a series of strip loadings, as given in Figures 4.9 and 4.10. Values of horizontal movement influence factors for the strip loadings, needed for the determination of I_{hbf}, were obtained from the charts developed by Davis and Taylor (1962). The approximate solutions for I_{hbf} are plotted against H_t/L_{bf} and B/H_t in Figure 4.14.

4.5.3 Rotations of Retaining Walls Due to Backfill

The rigid body rotations of the walls due to the backfill can be related to the settlements at the heel and at the toe as follows:

$$\theta_{bf} = \frac{\rho_{heel} - \rho_{toe}}{B} \tag{4.14}$$

From Equations 4.4 and 4.5, Equation 4.14 can be rewritten as:

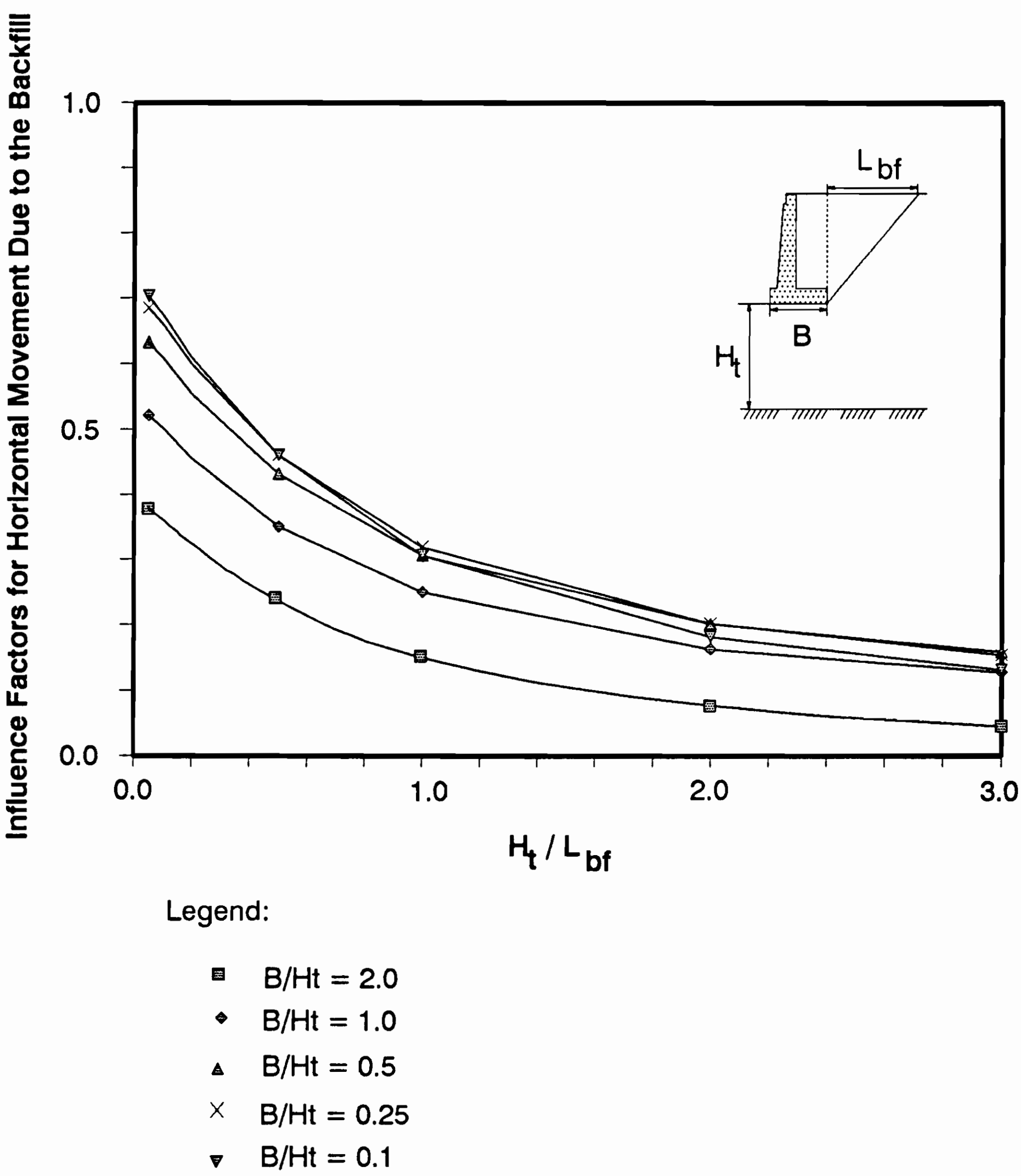
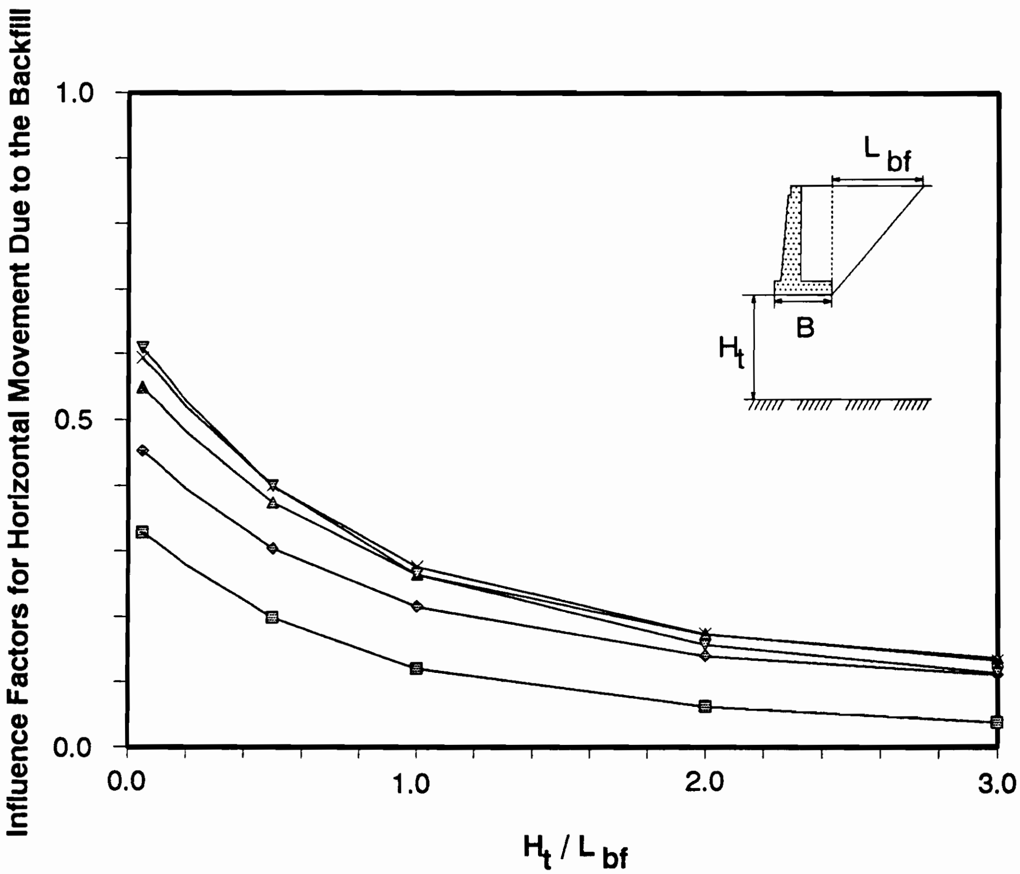


Figure 4.14a: Values of Influence Factors for Average Horizontal Movements Due to the Backfill (For Poisson's Ratio = 0.5)



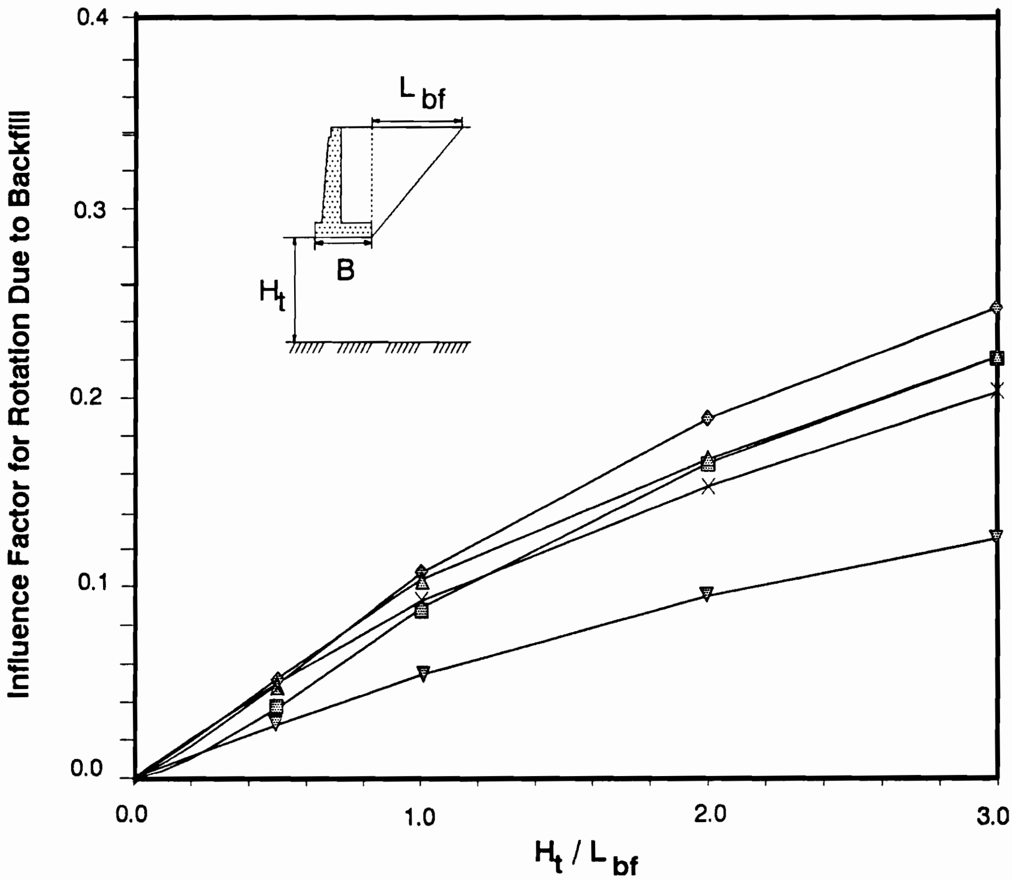
Legend:

- B/Ht = 2.0
- B/Ht = 1.0
- B/Ht = 0.5
- B/Ht = 0.25
- B/Ht = 0.1

Figure 4.14b: Values of Influence Factors for Average Horizontal Movements Due to the Backfill (For Poisson's Ratio = 0.3)

$$\theta_{bf} = \frac{q_{bf}}{E} \frac{L_{bf}}{B} I_{\theta bf} \qquad (4.15)$$

in which $I_{\theta bf}$ = dimensionless rotation influence factors due to backfill, whose values can be obtained by subtracting I_{heel} from I_{toe}. The values of $I_{\theta bf}$ are plotted against H_t/L_{bf} and B/H_t in Figure 4.15.



Legend:

- B/Ht = 2.0
- B/Ht = 1.0
- B/Ht = 0.5
- B/Ht = 0.25
- B/Ht = 0.1

Figure 4.15a: Values of Influence Factors for Rotations Due to the Backfill (For Poisson's Ratio = 0.5)

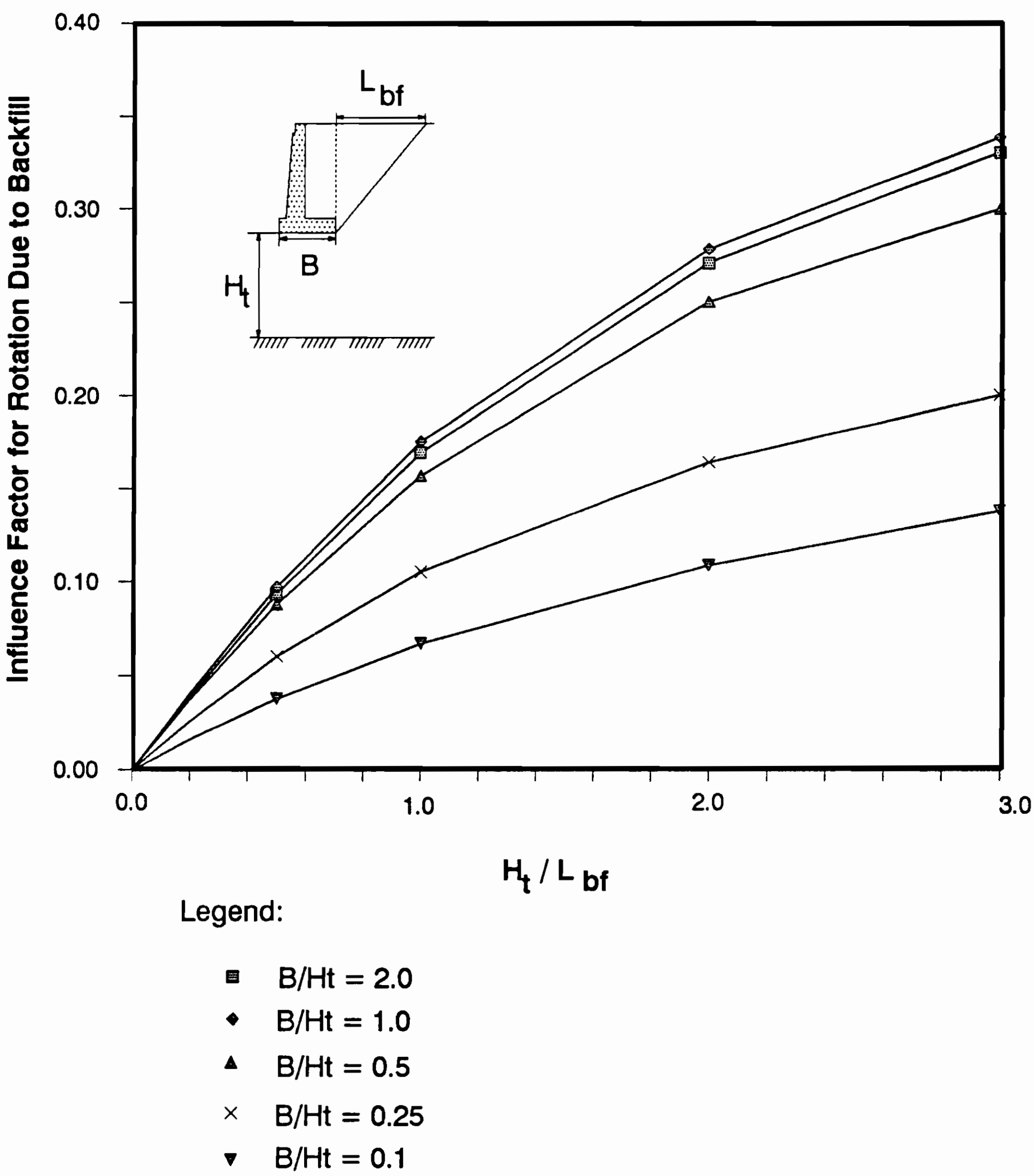


Figure 4.15b: Values of Influence Factors for Rotations Due to the Backfill (For Poisson's Ratio = 0.3)

CHAPTER 5

EVALUATION OF SOIL MODULUS FOR ESTIMATING MOVEMENTS OF FOOTINGS AND WALLS

5.1 INTRODUCTION

The magnitude of movement of a footing or wall under a given load depends to a large extent on the stiffness of the foundation soil. Thus, if displacements are to be estimated with reasonable accuracy, deformation characteristics of soils used in the calculations should be representative of those in the field. Because of the complex nature of the stress-strain relations of soils, selection of these compressibility parameters is a difficult task, requiring the exercise of good judgement.

In this chapter methods for estimating soil compressibility parameters will be discussed. In addition, a procedure for estimating horizonatal displacements and rotations, by relating them to values of settlement estimated using conventional empirical methods, will be described.

5.2 METHODS OF ESTIMATING SOIL MODULUS VALUES

In principle, values of soil modulus may be determined from either laboratory or field tests. They may also be estimated indirectly through empirical correlations, or backcalculated from field performance. These methods are discussed below:

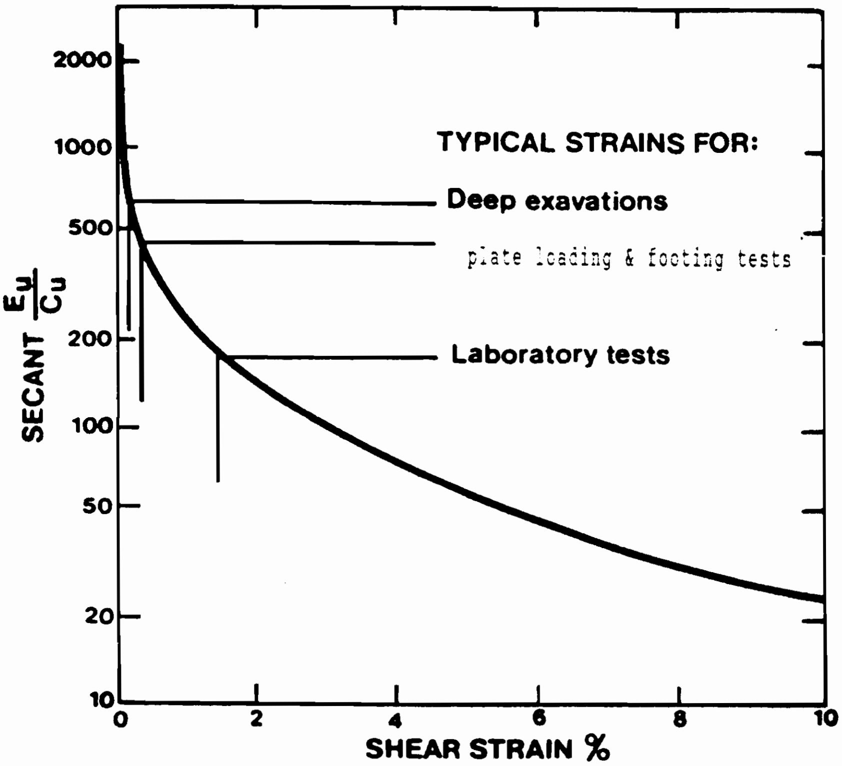
(a) Laboratory Stress-Strain Test

Reliable values of soil modulus can be determined directly from laboratory tests, only if soil sampling and testing are both of the highest quality. This precludes the use of laboratory tests to determine modulus values for cohesionless soils for which high quality sampling is extremely difficult. Relatively good quality samples, on the other hand, can sometimes be obtained from clay deposits. However, the process of sampling involves large stress cycles, possibly irreversible straining, and may open up fissures in the case of stiff clays. It is possible, therefore, that the specimens used in laboratory tests have either been changed irreversably, or will only provide useful results if subjected to a loading sequence designed to return them to the original state. For example, Ladd (1964) has recommended consolidated-undrained (CU) tests in clays in preference to unconsolidated-undrained (UU) tests, because there are fewer sample disturbance effects in CU tests on some types of clay.

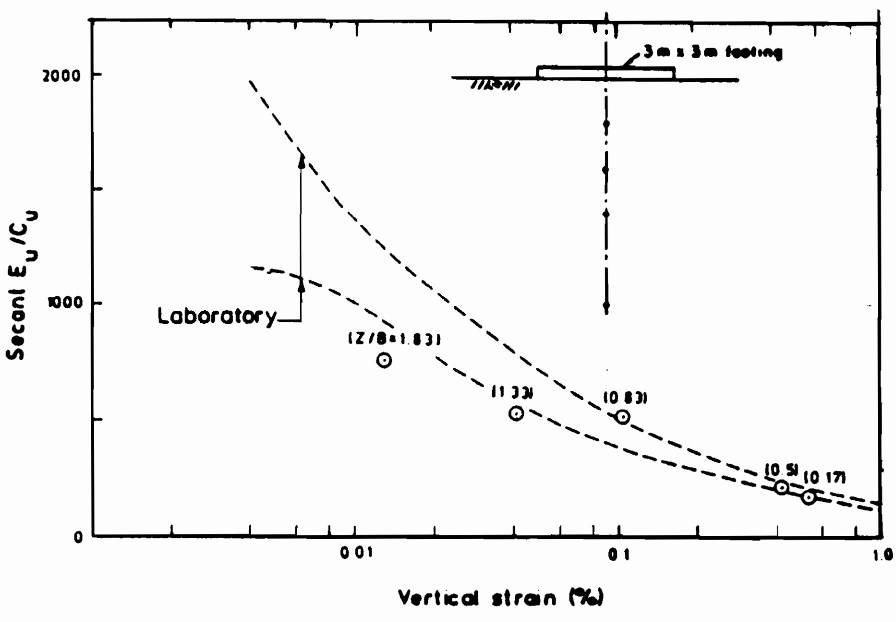
However, there may still be considerable discrepancy between the modulus determined from standard laboratory tests and those back-calculated from field observations. For example, Cole and Burland (1972) found that deduced values of E_u of London clay from back analysis were at least five times greater than those derived from careful laboratory tests. As shown in Figure 5.1a, the discrepancy arises mainly because the modulus obtained from conventional laboratory tests usually corresponds to strains that are larger than those induced by foundation loadings, which seldom exceed 0.1 % (Burland, 1989). The response of soils is highly nonlinear; and, as depicted in Figures 5.1, the normalized secant modulus decreases drastically with increasing strains. Thus, in order to model the field behavior correctly, values of soil modulus at strains less than 0.01% must be used (see Figure 5.1b). Unfortunately, determination of these small-strain modulus values can only be obtained in fairly sophisticated laboratory tests in which strains of such small magnitudes can be measured accurately. It has been suggested that the difficulty of determining these high stiffness at small strains from standard laboratory tests may be overcome by subjecting the soil to a cycle of unloading and reloading. The higher value of the reloading modulus is then used in design calculations.

(b) Field Tests

In situ testing such as plate load and pressuremeter tests may be used to determine the deformation characteristics of a soil in the field. Because the soil is relatively undisturbed, these tests are capable of yielding results that are compatible with those obtained from back analyses at least



(a) Typical Variation of Normalized Soil Modulus with Shear Strain with London Clay



(b) Deduced Stress-Strain Characteristics from Results of Footing Test and Small Strain Laboratory Test

Figure 5.1: Variation of Normalized Soil Modulus with Strain (After Simpson and Sommer, 1985; Jardine, et al., 1985)

under favorable conditions. Where their use is justified by the importance of the project, they provide a logical approach to the evaluation of soil modulus values.

(c) Empirical Correlations

Many empirical correlations between soil stiffness and other parameters obtained from laboratory or field tests have been reported in the literature. In cohesionless soils, the correlations may be the only means of estimating soil modulus values. In some other cases, especially for cohesive soils, the correlations may serve as an independent means of obtaining soil parameters for comparison with values determined using other procedures.

Several authors have correlated the modulus of cohesionless soils with standard penetration and cone penetration resistances. Some of these correlations are presented in Figures 5.2 and 5.3. It is noted that the correlation suggested D'Appolonia, et al. (1968) shown in Figure 5.2 requires only uncorrected SPT blow count. The empirical correlation of Menzenbach (1967), shown in Figure 5.3 does take the effect of overburden pressure into account. This plot shows that the soil that has the same deformation modulus would have a higher value of SPT blow count if it is subjected to higher overburden pressure.

In cohesive soils the undrained modulus is often correlated to undrained shear strength. One correlation is

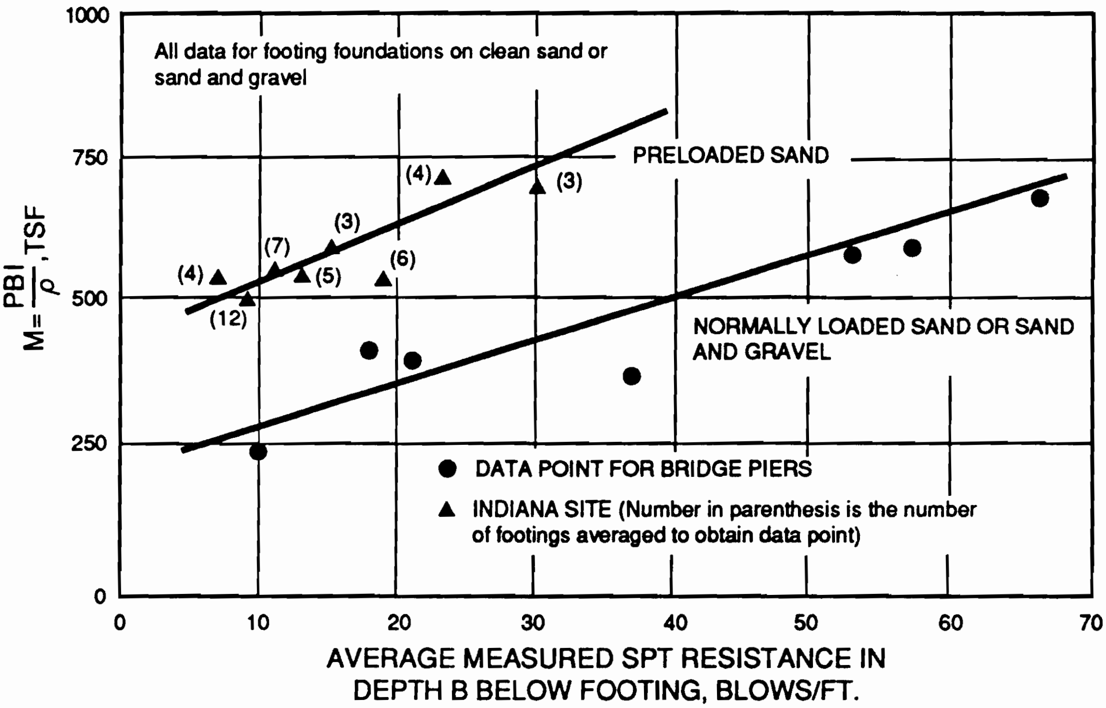


Figure 5.2: Correlation Between Modulus of Compressibility and Average Value of SPT Blow Count (After D'Appolonia, et al., 1970)

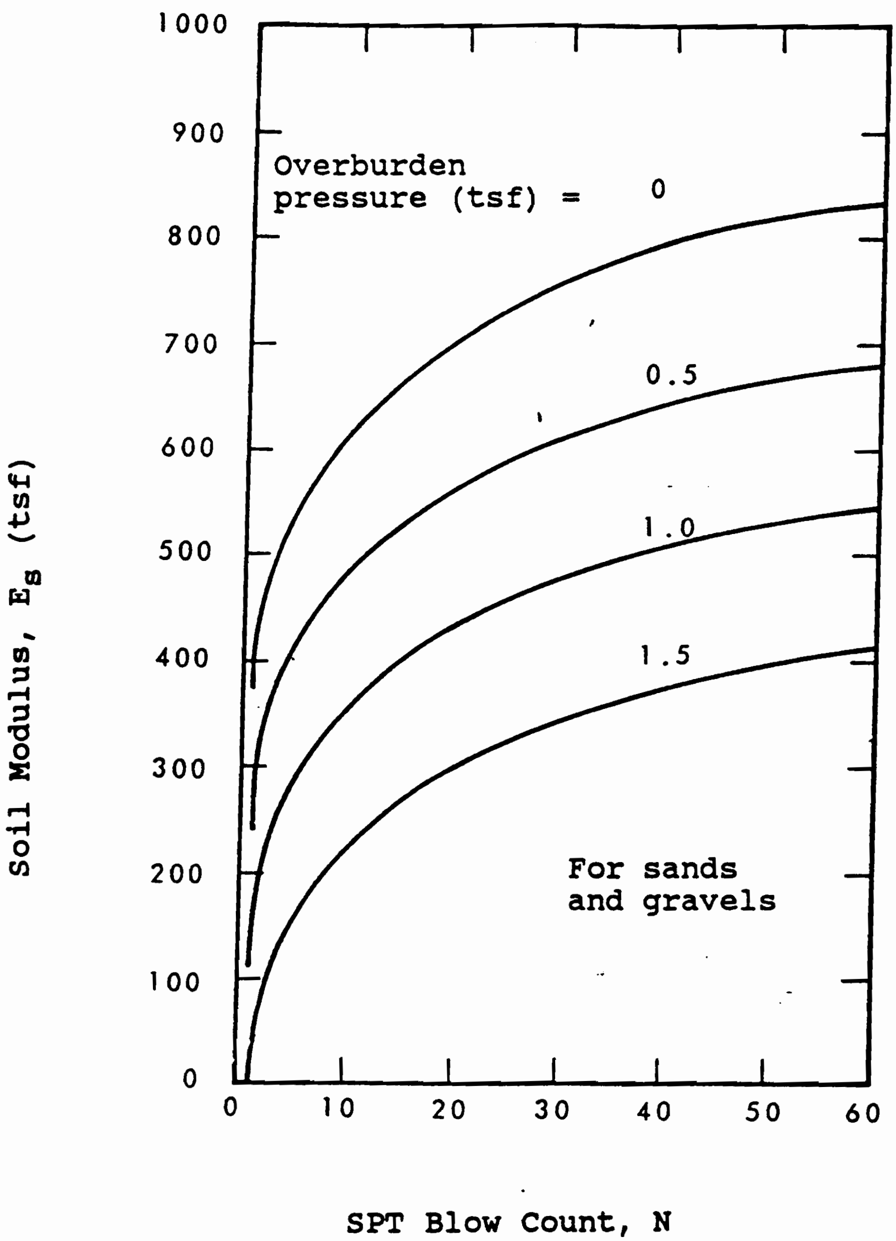


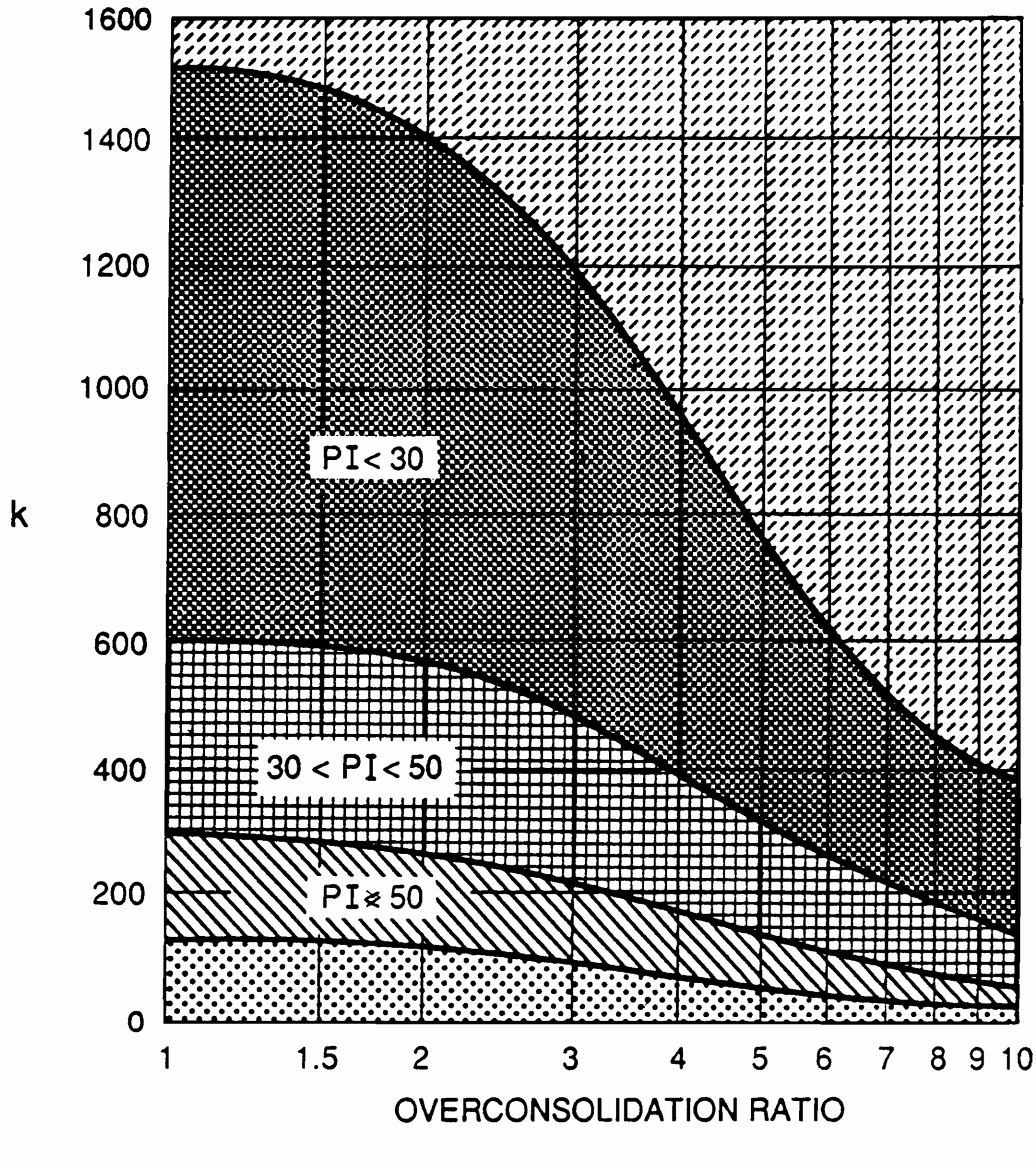
Figure 5.3: Relationship between Soil Modulus and SPT Blow Count (After Schultze and Melzer, 1965)

shown in Figure 5.4, which is based on field measurements of immediate settlement. Callanan and Kulhawy (1985) emphasized that E_u/s_u correlations should be used cautiously. Strengths of fissured clay may vary from strength of the intact clay to strength of the fissured mass, and, as a result, the value of E_u/s_u may vary widely.

5.5 METHOD FOR ESTIMATING HORIZONTAL DISPLACEMENT AND ROTATION BASED ON SETTLEMENT ESTIMATES, WITHOUT EVALUATING SOIL MODULUS

In the previous chapter procedures for using elastic theory to estimate horizontal movements and rotations were described. A fundamental difficulty with the elastic approach is the need for estimating values of soil modulus, which, as indicated by the above discussions, is a difficult and uncertain undertaking.

To avoid some of the problems inherent in estimating values of soil modulus, it is possible to relate horizontal movements and rotations of footings to estimated settlements of the footings, which can be calculated using the well-established procedures discussed in Chapters 2 and 3. In a similar manner the movements of abutment and retaining walls, resulting from the weight of backfill, can also be related to the estimated settlement.



$E_u = k\ S_u$

E_u = Undrained Modulus of Clay

k = Factor from Chart Above

S_u = Undrained Shear Strength of Clay

Figure 5.4: Chart for Estimating Undrained Modulus of Clay (After Duncan and Buchignani, 1976)

The expression for relating horizontal displacements to settlements of footings may be obtained by combining Equations 4.1 and 4.2, as presented below:

$$\Delta h = \frac{q_h}{q_v} \frac{I_h}{I_\rho} \rho \qquad (5.1)$$

where Δh = horizontal movement of footing (in same length units as ρ),

ρ = settlement of footing (in length units),

q_h = horizontal pressure (in units of force^2/length),

q_v = vertical bearing pressure (in units of force^2/length),

I_h, I_ρ = dimensionless influence factors for horizontal movement and settlement, respectively.

Likewise, rotations of footings, θ, can be related to their settlements using the following expression:

$$\theta = \frac{M}{q_v L B^3} \frac{I_\theta}{I_\rho} \rho \qquad (5.2)$$

where θ = rotations of footings (in radians), B = width of footing (in length units), L = length of footing (in same length units as B), I_θ = dimensionless influence factor for rotation; and other terms are as defined previously.

In Equations 5.1 and 5.2 the value of soil modulus does not appear, because the magnitudes of horizontal movement and rotation are related to the magnitude of the settlement. Ratios of I_h/I_ρ and I_θ/I_ρ determined from elastic theory are summarized in Tables 5.1 amd 5.2.

Expressions for relating movements of abutments and retaining walls due to the weight of the backfill to the estimated settlements of the footings of the walls may be derived by combining Equation 4.1 with Equations 4.6, 4.13 and 4.15, and they are given below:

$$\rho_{bf} = \frac{q_{bf}}{q_v} \frac{L_{bf}}{B} \frac{I_{\rho bf}}{I_\rho} \rho \tag{5.3}$$

$$\Delta h_{bf} = \frac{q_{bf}}{q_v} \frac{H_t}{B} \frac{I_{hbf}}{I_\rho} \rho \tag{5.4}$$

$$\theta_{bf} = \frac{q_{bf}}{q_v} \frac{L_{bf}}{B^2} \frac{I_{\theta bf}}{I_\rho} \rho \tag{5.5}$$

TABLE 5.1: VALUES OF THE RATIO I_h/I_ρ FROM ELASTIC THEORY

Footing Shape	Elastic Layer Thickness	Values of I_h/I_ρ For ν = 0.3	For ν = 0.5
Square or Circle	Infinite	2.0	2.7
L/B = 5	Infinite	1.3	1.7
L/B = 10	Infinite	1.4	1.7
L/B = ∞	H_t/B = 3	2.0	2.7
L/B = ∞	H_t/B = 2	2.1	3.0
L/B = ∞	H_t/B = 1	2.7	4.1

Note : Values of I_h are taken from Table 4.1; while those for I_ρ are taken from Figures 4.3 and 4.4.

TABLE 5.2: VALUES OF THE RATIO I_θ/I_ρ

Footing Shape	Elastic Layer Thickness	Values of I_θ For ν = 0.3	For ν = 0.5
Square or Circle	Infinite	3.9	4.3
L/B = 2	Infinite	3.3	3.4
L/B = ∞	H_t/B = 3	5.7	6.6
L/B = ∞	H_t/B = 2	6.7	8.2
L/B = ∞	H_t/B = 1	10.4	12.5

Note: Values of I_θ and I_ρ are taken from Table 4.2 and Figure 4.3 respectively.

where ρ_{bf} = settlement due to the weight of the back fill (in the same length units as ρ),

Δh_{bf} = horizontal movement due to the weight of the backfill (in the same length units as ρ),

θ_{bf} = rotation due to the weight of the backfill (in radians),

L_{bf} = lateral extent of the backfill, measurd from the heel (in the same length units as B),

B = width of the footing for the wall (in length units),

H_t = thickness of the elastic compressible layer (in same length units as B),

q_{bf} = soil pressure at the heel, due to the weight of the backfill (in units of force^2/length),

$I_{\rho bf}$, I_{hbf}, $I_{\theta bf}$ = dimensionless influence factors for settlement, horizontal movement, and rotation due to the weight of the backfill, respectively,

Other terms are as defined previously. Values of the influence factor ratios, $I_{\rho bf}/I_{\rho}$, I_{hbf}/I_{ρ}, and $I_{\theta bf}/I_{\rho}$, determined from elastic theory, are shown in Figures 5.5. through 5.7.

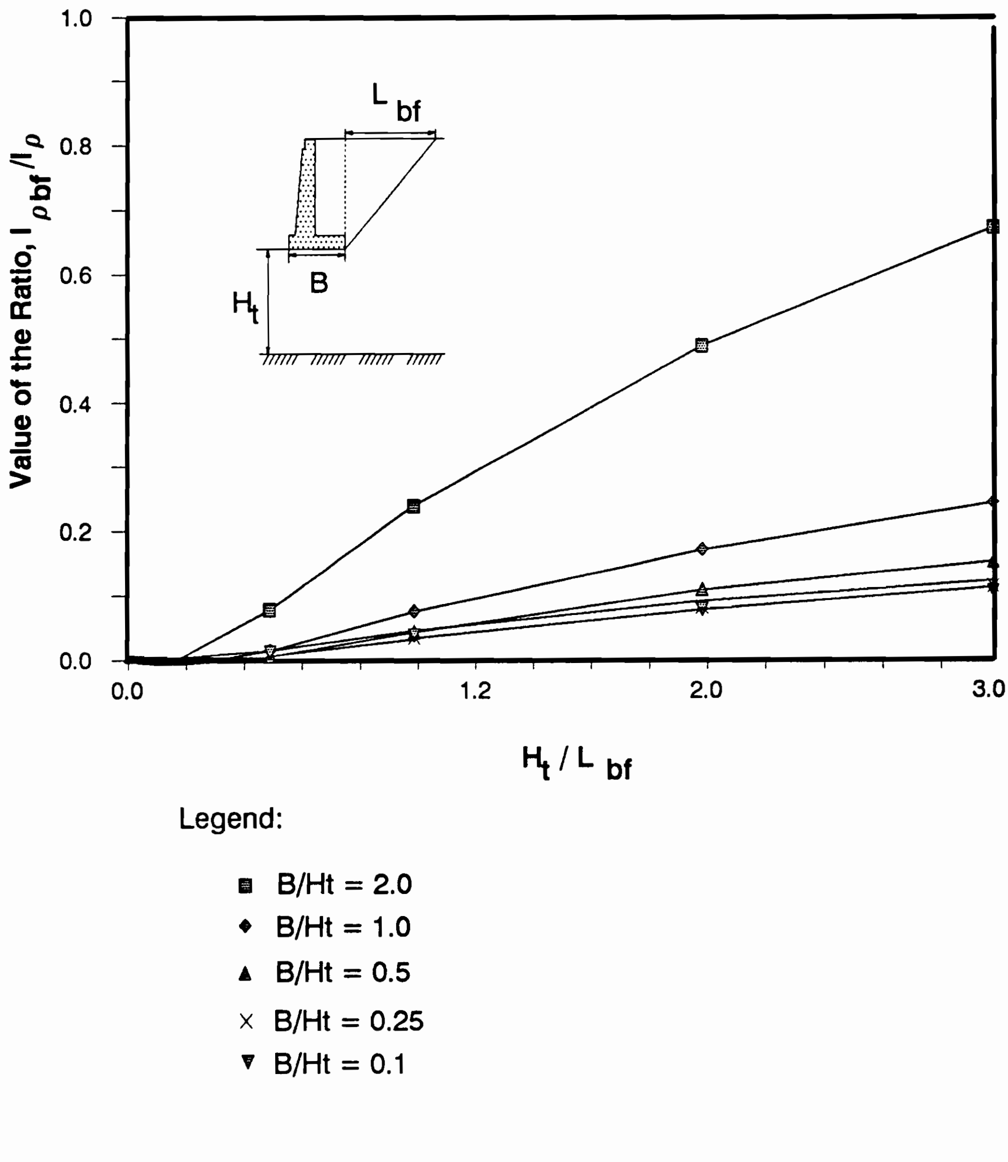


Figure 5.5a: Value of Influence Factor Ratio, $I_{\rho bf}/I_\rho$ (For Poisson's Ratio = 0.5)

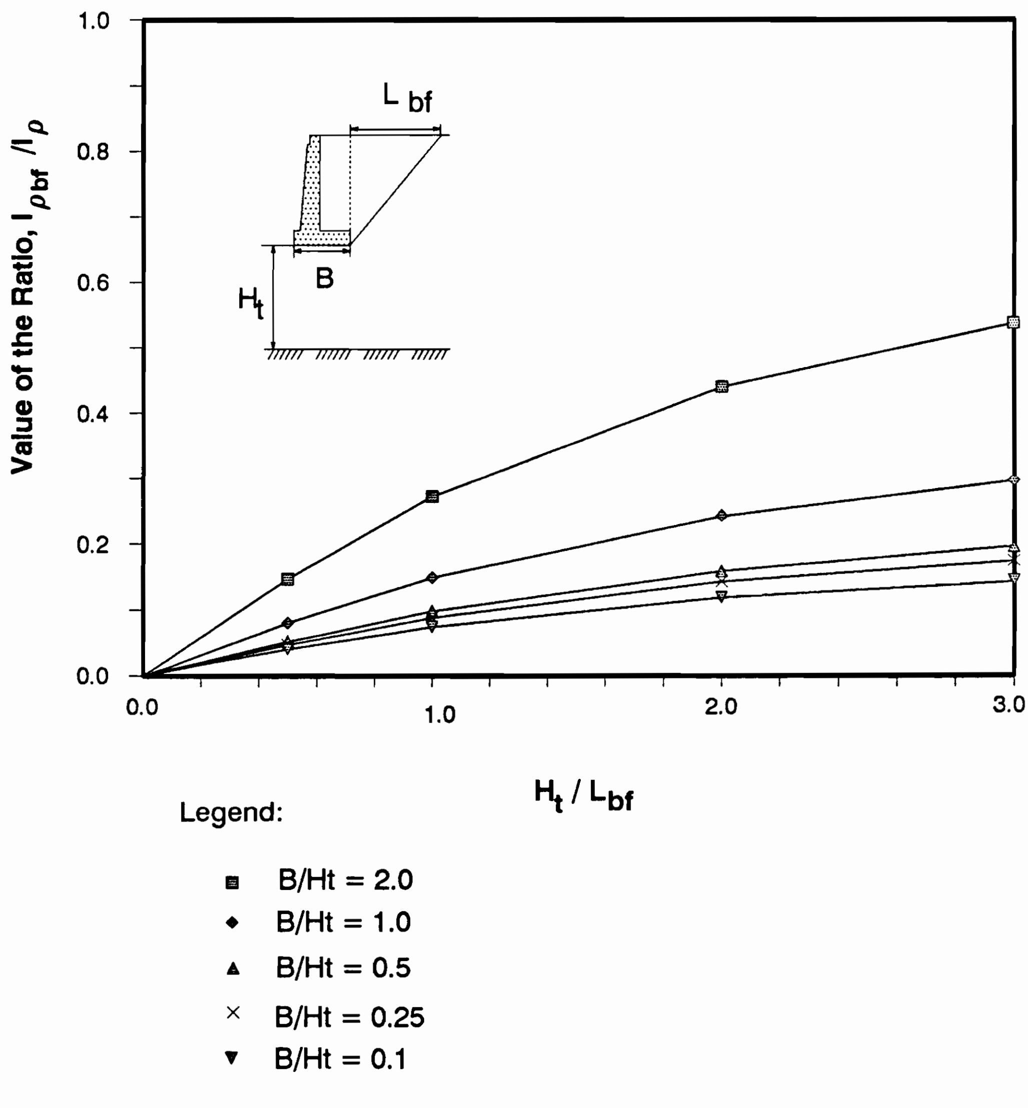
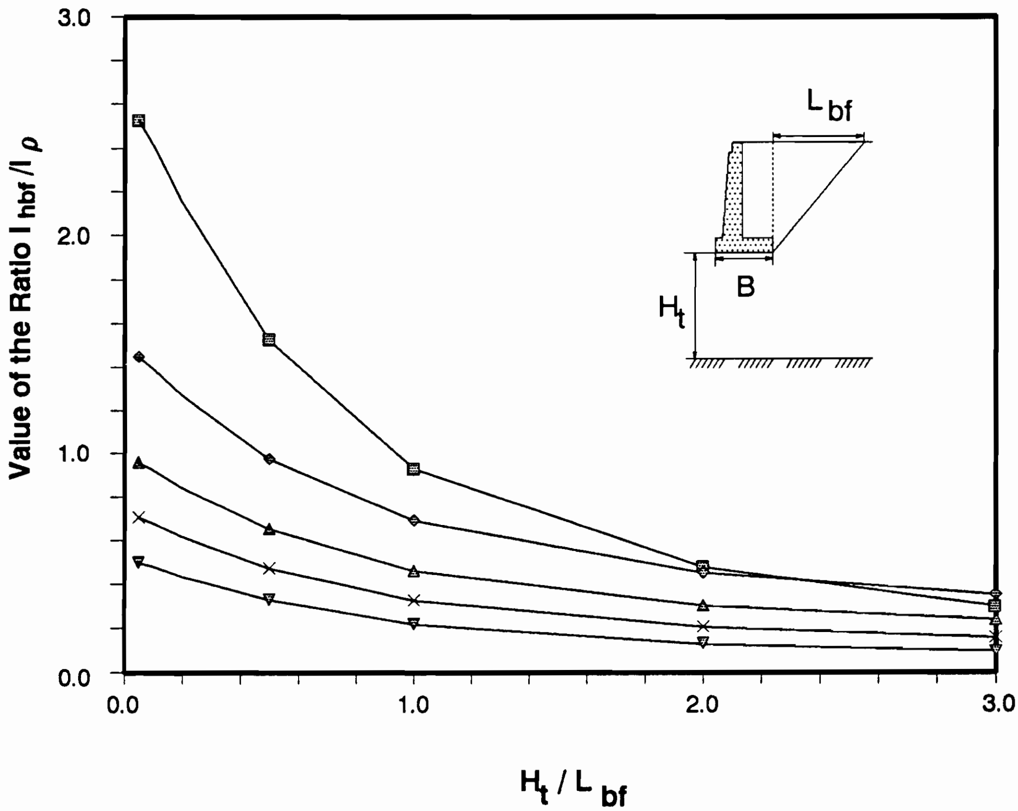


Figure 5.5b: Values of Influence Factor Ratio, $I_{\rho bf}/I_{\rho}$ (For Poisson's Ratio = 0.3)

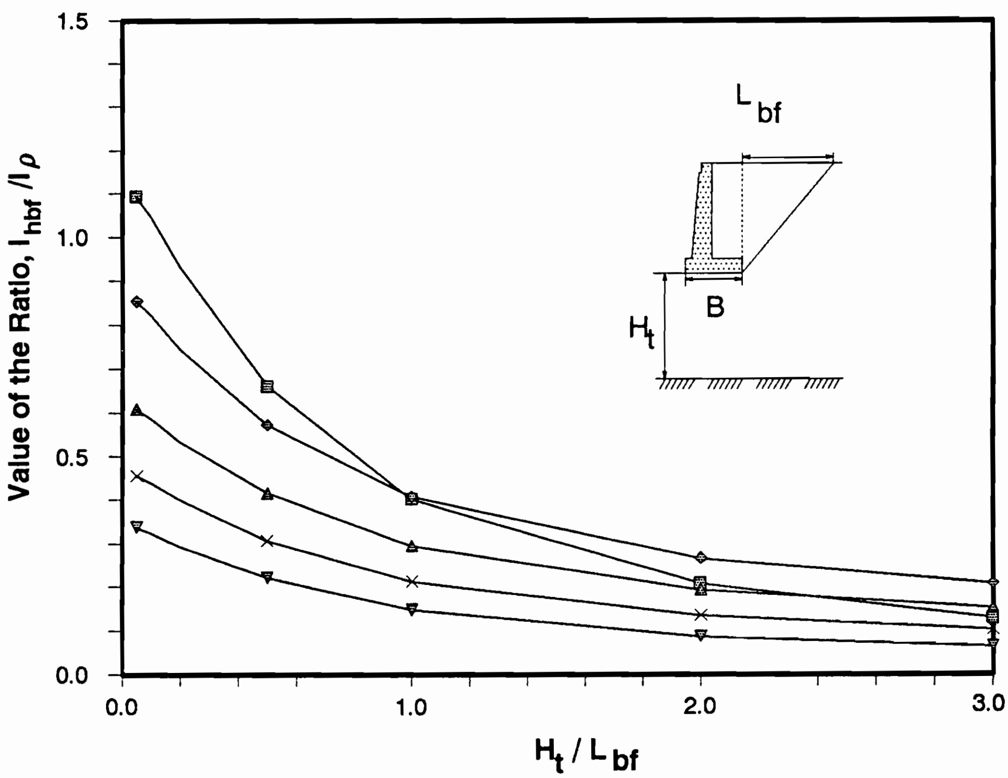


Legend:

- B/Ht = 2.0
- B/Ht = 1.0
- B/Ht = 0.5
- B/Ht = 0.25
- B/Ht = 0.1

Figure 5.6a: Values of Influence Factor Ratio, I_{hbf}/I_{ρ}

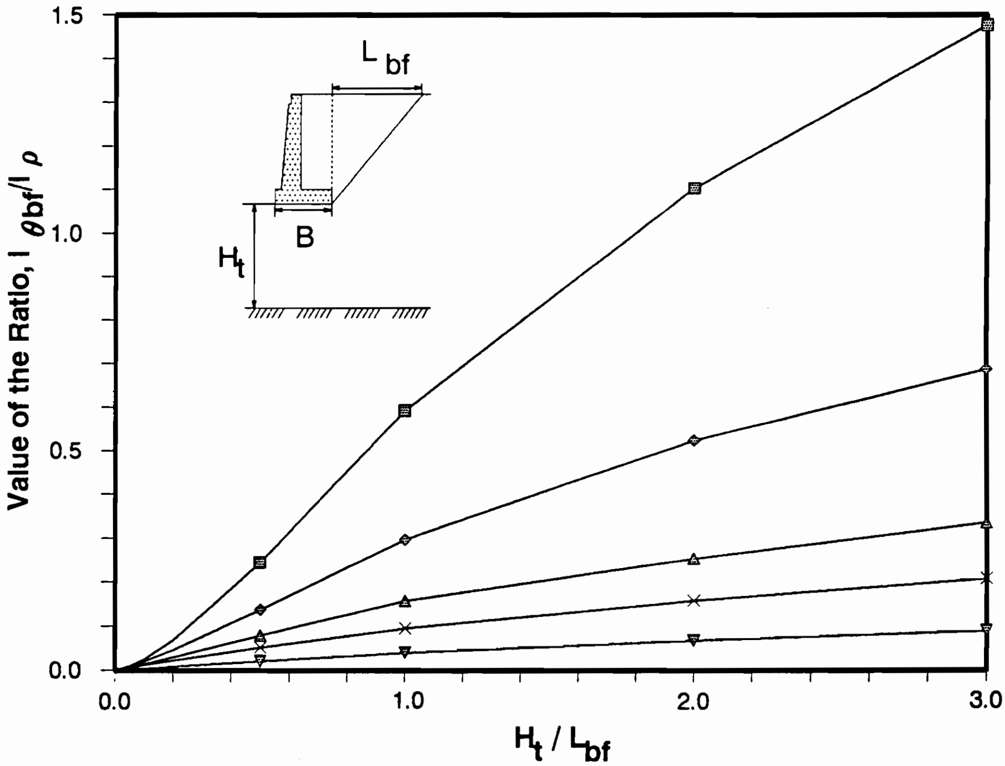
(For Poisson's Ratio = 0.5)



Legend:

- B/Ht = 2.0
- B/Ht = 1.0
- B/Ht = 0.5
- B/Ht = 0.25
- B/Ht = 0.1

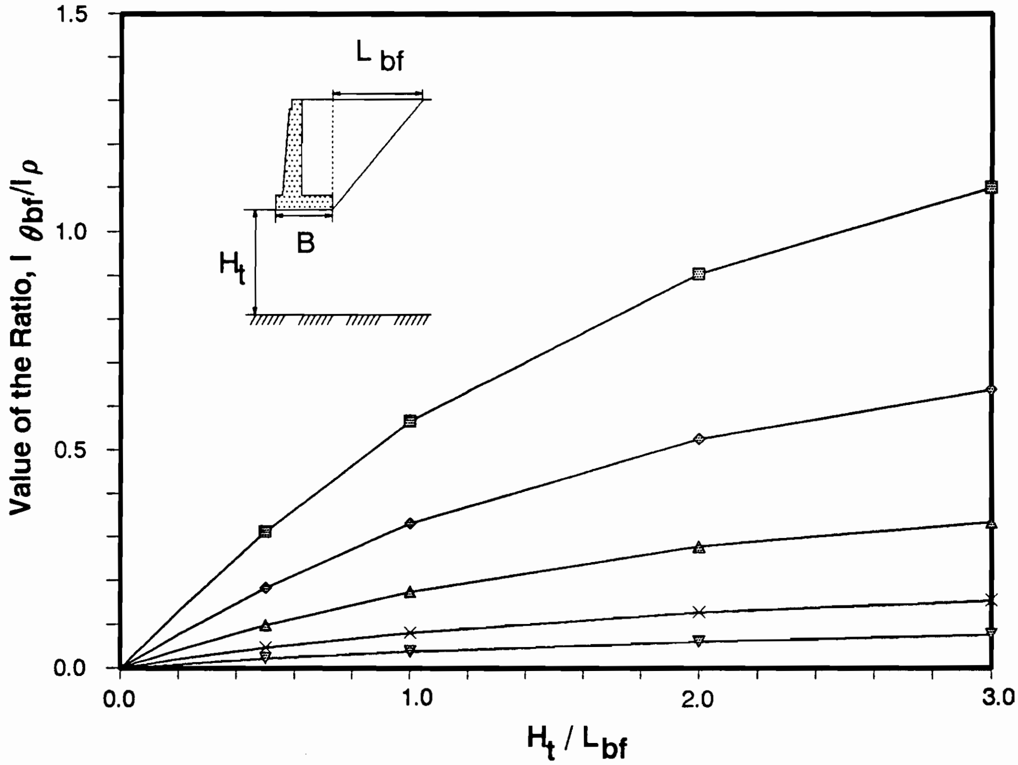
Figure 5.6b: Values of Influence Factor Ratio, I_{hbf}/I_{ρ} (For Poisson's Ratio = 0.3)



Legend:

- B/Ht = 2.0
- B/Ht = 1.0
- B/Ht = 0.5
- B/Ht = 0.25
- B/Ht = 0.1

Figure 5.7a: Values of Influence Factor Ratio, $I_{\theta bf}/I_\rho$
(For Poisson's Ratio = 0.5)



Legend:

- B/Ht = 2.0
- B/Ht = 1.0
- B/Ht = 0.5
- B/Ht = 0.25
- B/Ht = 0.1

Figure 5.7b: Values of Influence Factor Ratio, $I_{\theta bf}/I_{\rho}$
(For Poisson's Ratio = 0.3)

CHAPTER 6

APPLICATION TO A CASE HISTORY

6.1 INTRODUCTION

The procedures developed in Chapter 4 are illustrated by application to a case history in this chapter. The case studied is an abutment wall at Providence, Rhode Island. The abutment wall is supported by a spreading footing on sand (Gifford, et al., 1987).

Movements of the abutment wall were computed using the proposed procedures and the finite element technique. Results from these analyses were compared with the field measurements.

6.2 BACKGROUND INFORMATION

The abutment wall considered is located at the west end of a four-span steel bridge in Providence, Rhode Island. As shown in Figure 6.1, the wall is about 15 ft high, and is supported by a spread footing with plan dimensions of about 75 ft by 11 ft. The footing was constructed on engineered fill. Native soils, consisting of fine to medium grained outwash sands, were used as backfill. Variation of SPT blow count with depth at the site are shown in Figure 6.2. The water table is located at a depth of about 30 ft below the bottom of the footing. Settlements of the wall were recorded by optical

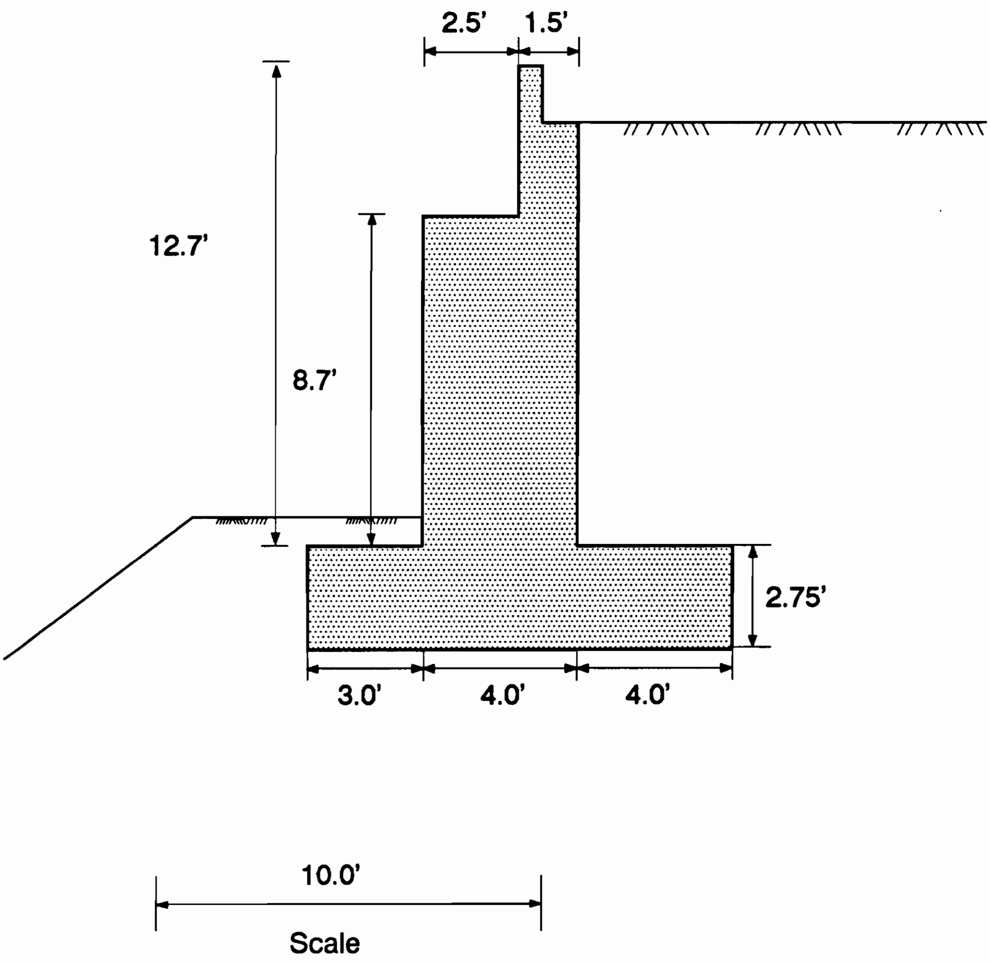


Figure 6.1: Dimensions of The Abutment Wall

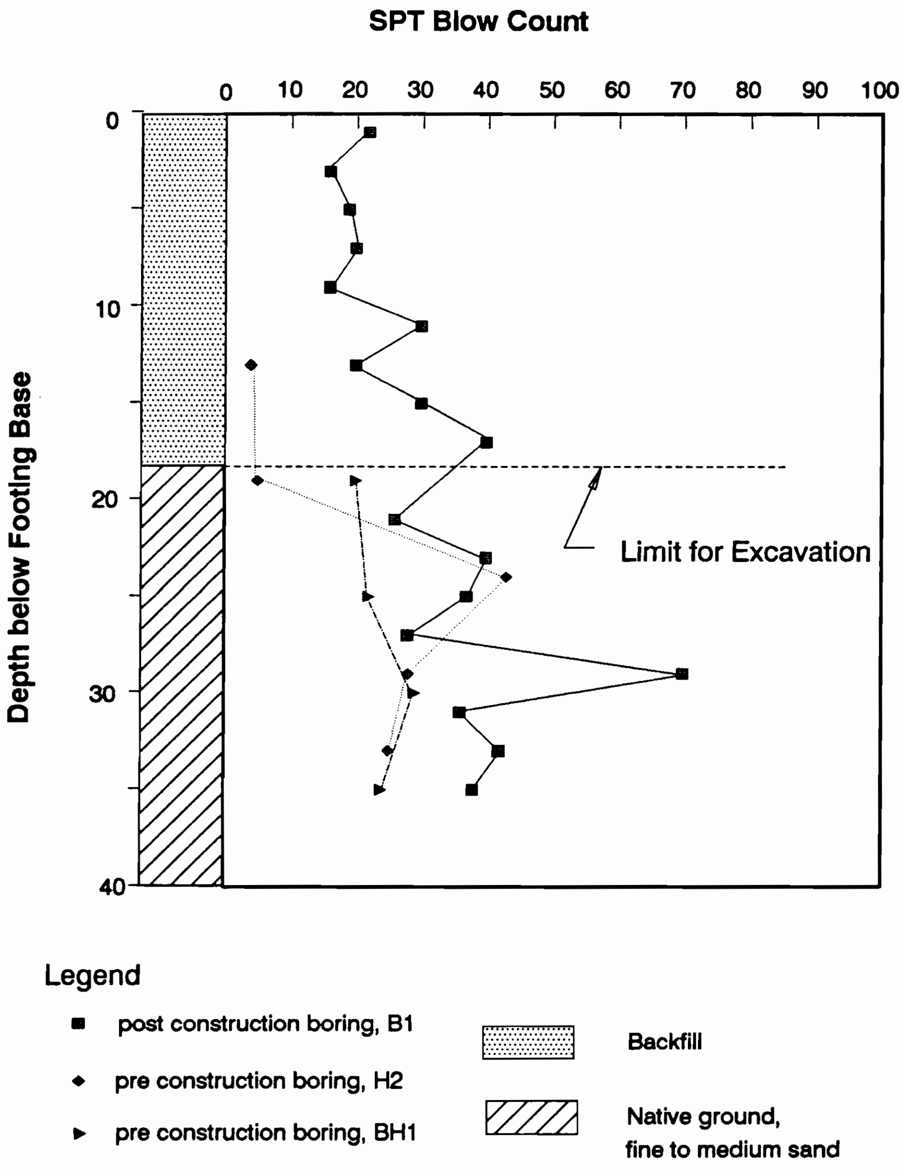


Figure 6.2: Soil Conditions at The Site for The Abutment Wall in Rhode Island

settlement points installed on the stem of the abutment. These settlement points, however, were installed after the footing had been constructed. A chronology of the wall construction is provided in Table 6.1.

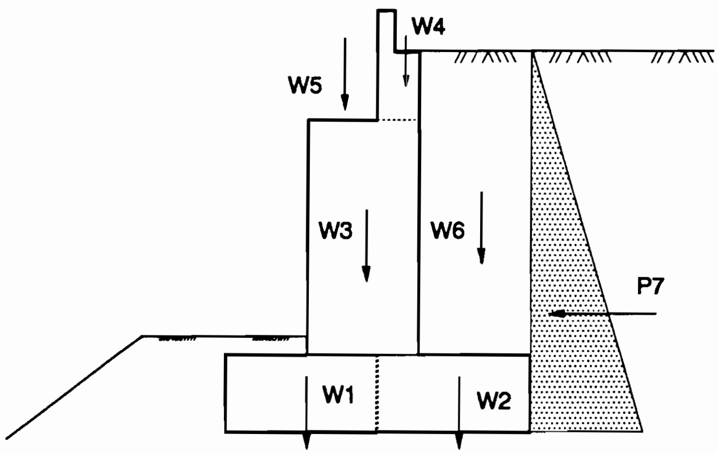
6.3 ESTIMATION OF WALL MOVEMENTS

Simple Elastic Analysis: Design loadings for the abutment wall are shown in Figure 6.3. The lateral load imposed by earth pressure, P7 = 3.72 kip/ft wall, was estimated using Rankine theory. The weight of the footing (W1 + W2) was not included in settlement calculations, so that the calculated value could be compared directly with the field measurements, which began after the footing had been constructed. In addition, the excavation for the wall construction was assumed to slope back from the heel at an inclination of 1:1.

The settlement of the footing was calculated using the procedure of Terzaghi and Peck (1967). Based on this calculated settlement, the horizontal displacement and rotation of the abutment wall and its footing were estimated following the procedures described in Section 5.4. The movements calculated for the top of the wall (Point A) and for the toe, heel, and center of the base (Points B, C, and D) are summarized in Table 6.2.

TABLE 6.1: CHRONOLOGY OF THE CONSTRUCTION OF THE WEST ABUTMENT WALL IN PROVIDENCE, RHODE ISLAND

Date	Activity
Nov 12, 1983	Footing in place, initial instrument readings taken.
Dec 19, 1983	Stem of wall poured.
Mar 5, 1984	Backfill placement completed.
Mar 22, 1984	Bridge girders set.
May 11, 1984	Bridge deck poured.
Nov 19, 1984	Bridge open to traffic.



W1 = 2.27 kip/ft wall
W2 = 2.06 kip/ft wall
W3 = 5.16 kip/ft wall
W4 = 0.90 kip/ft wall
W5 = 7.08 kip/ft wall
W6 = 5.38 kip/ft wall
P7 = 3.72 kip/ft wall

Figure 6.3: Design Loadings for The Abutment Wall

TABLE 6.2: DISPLACEMENTS AND ROTATIONS OF THE WEST ABUTMENT WALL CALCULATED USING SIMPLE ELASTIC THEORY

	Location			
Item	A	B	C	D
Settlement (in)				
Wall Load	0.40	0.40	0.40	0.40
Backfill	0.07	0.05	0.07	0.09
Total	0.47	0.45	0.47	0.49
Horizontal Displacement (in)				
Wall Load	0.47	0.40	0.47	0.40
Backfill	0.28	0.50	0.43	0.50
Total	0.85	0.90	0.90	0.90
Rotation (degree) for the Wall				
Wall Load	-0.00026			
Backfill	+0.016			
Total	+0.016			

Note: 1. Downward settlement is positive,
2. Horizontal movement away from the backfill is positive
3. Rotation toward backfill is positive

Finite Element Analysis: The behavior of the wall was also analyzed using the finite element method. The computer program SOILSTRUCT developed by Clough and his coworkers (Filz, et al., 1990) was used in the analysis. In this plane strain finite element computer program, the stress-strain behavior of the soil is characterized by the hyperbolic model developed by Duncan and Chang (1970).

The nonlinear behavior of the soils was approximated using the hyperbolic stress-strain relationships. The analysis was performed in a series of increments, and the values of soil modulus were varied in each increment in accordance with the calculated stresses. As explained by Duncan and Chang (1970), the modulus value (E_t) assigned to each element is linked to the stresses in the element through the following relationship:

$$E_t = \left[1 - \frac{R_f\ (1-\sin\phi)\ (\sigma_1-\sigma_3)}{2c\ \cos\phi + 2\sigma_3\ \sin\phi}\right]^2 K\ p_a \left[\frac{\sigma_3}{p_a}\right]^n \qquad (6.1)$$

where E_t = tangent modulus

σ_1 = major principal stress

σ_3 = minor principal stress

p_a = atmospheric pressure

The parameters K, n, R_f, c, and ϕ, collectively known as the hyperbolic parameters, are defined in Table 6.3, in which the function played by each of these parameters are also described. A comprehensive review of the hyperbolic stress-strain relationships of soils is provided in the report by Duncan, et al. (1980).

The values of the hyperbolic stress-strain and strength parameters used in the finite element study are listed in Table 6.4. It can be noted from the table that the analyses were performed using ranges of parameter values. These values were estimated on the basis of data from similar soils, using the information provided by Byrne, et al. (1987) who suggested a list of typical values of hyperbolic parameters for cohesionless soils based on back analyses of settlement observations and model tests. In the recommendation by Byrne, et al., the hyperbolic parameters are related to SPT blow count.

Holmes (1990) also analyzed the west abutment wall using the computer program SOILSTRUCT. The parameter values used by Holmes were somewhat different than the values used in the analyses described in the following paragraphs.

The finite element mesh shown in Figure 6.4 was used in the analysis. The mesh was developed by Holmes. Having the mesh available greatly simplified the analyzes described in the following paragraphs. The concrete abutment wall was

TABLE 6.3: DEFINITIONS AND FUNCTIONS OF HYPERBOLIC STRESS-STRAIN PARAMETERS (AFTER DUNCAN, 1990)

Parameter	Function
K = modulus number (dimensionless) and n = modulus exponent (dimensionless)	Relate the initial tangent modulus soil (E_i) to the minor principal stress (σ_3) by means of the equation $E_i = K\, p_a \left[\frac{\sigma_3}{p_a}\right]^n$ p_a = atmospheric pressure
c = cohesion intercept (stress units) and ϕ = friction angle (degrees)	Together relate the compressive strength of the soil to the minor principal stress $(\sigma_1-\sigma_3)_f = \frac{2c\cos\phi + 2\sigma_3\sin\phi}{1 - \sin\phi}$
R_f = failure ratio	Relates the asymptote of the hyperbolic stress-strain curve, $(\sigma_1-\sigma_3)_{ult}$, to the compressive strength, $(\sigma_1-\sigma_3)_f$. $(\sigma_1-\sigma_3)_f = R_f(\sigma_1-\sigma_3)_{ult}$

TABLE 6.4 HYPERBOLIC STRESS-STRAIN PARAMETERS USED IN THE ANALYSIS OF THE ABUTMENT WALL IN PROVIDENCE, RHODE ISLAND

Pameter	Low Limit	High Limit
ϕ (degrees)	40	40
K	450	900
n	0.50	0.50
R_f	0.80	0.80

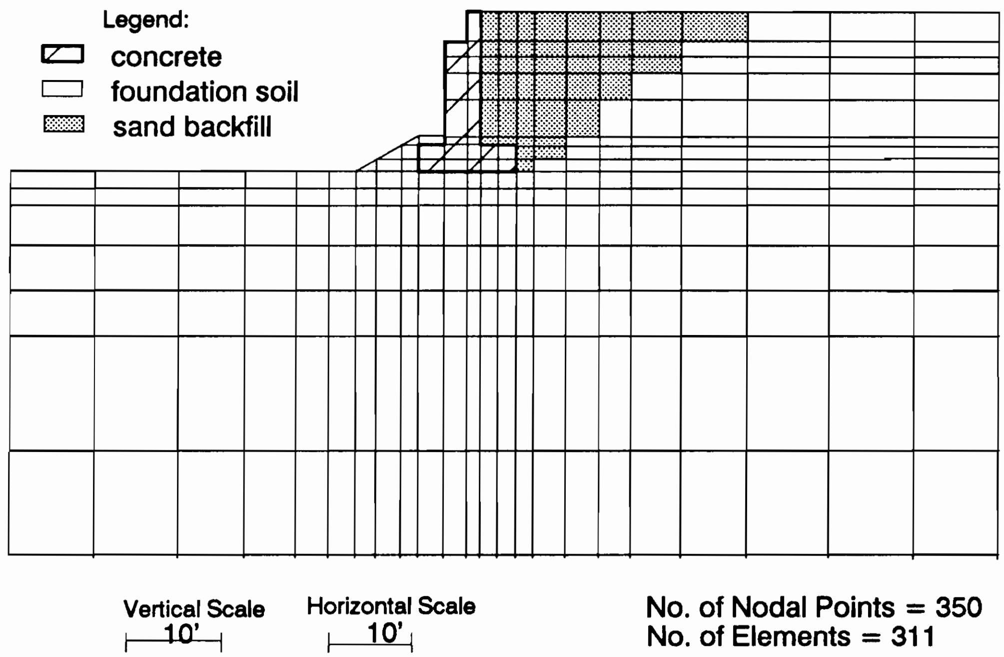


Figure 6.4: Finite Element Mesh Used for the Analysis of the Abutment Wall in Rhode Island

modelled as a linear elastic body with high stiffness value. Interface elements were used between the structure and the backfill, to allow for the possible slip on the interface. Again, as in the simple elastic analysis, the slope of the excavation required for the wall construction was assumed to be 1:1.

The results of the analyses are summarized in Table 6.5, which lists the displacements at the top of the wall (Point A), at the toe (B), the heel (C), and the center of the base (D). Displacements at the end of footing construction, and at the end of the backfill placement are included in the table.

6.4 COMPARISONS OF RESULTS

Figure 6.5 presents values of calculated and measured settlements at the base of the footing. The measured values shown in the figure were calculated based on the observed settlements of 0.37 to 0.46 in, assumed to be measured at Point C; and on the measured tilt of 0.021 to 0.044 degrees toward the backfill. It can be seen that agreement between the calculated and measured values is quite good.

Comparisons between the calculated and measured rotation of the wall are shown in Table 6.6. Results from both simple elastic analysis and the finite element method are in agreement with the field observation that the abutment wall

TABLE 6.5: DISPLACEMENTS AND ROTATIONS OF THE WEST ABUTMENT WALL CALCULATED USING FINITE ELEMENT METHOD

Item	Location A	B	C	D
Settlement (in)				
K = 450	+0.19	+0.36	+0.43	+0.50
K = 900	-0.01	+0.28	+0.30	+0.32
Horizontal Displacement (in)				
K = 450	+0.02	+0.22	+0.22	+0.22
K = 900	+0.42	+0.50	+0.50	+0.50
Rotation for the Wall (degree)				
K = 450	+0.076			
K = 900	+0.024			

Note: 1. Downward settlement is positive,
2. Horizontal movement away from backfill is positive,
3. Clockwise rotation is positive.

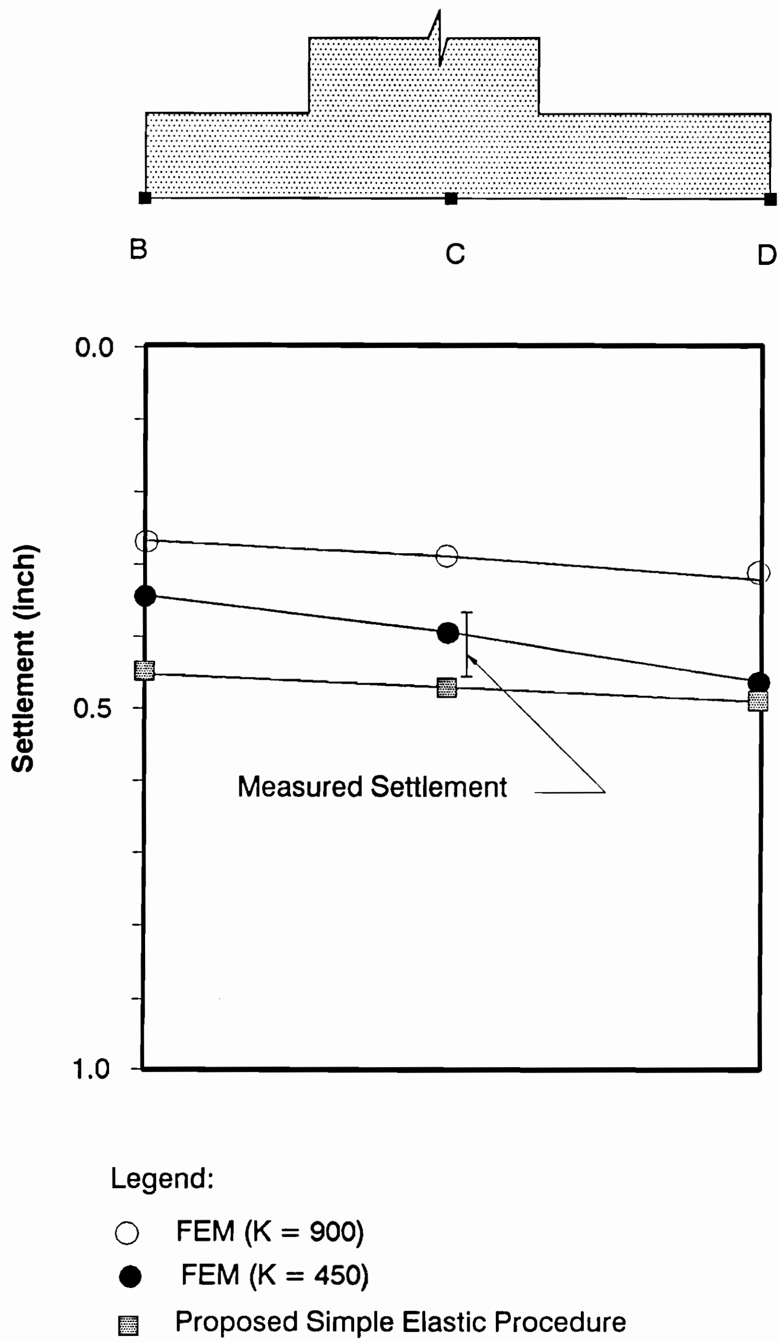


Figure 6.5: Comparison of Settlements at Base of Footing for The Abutment Wall in Rhode Island

TABLE 6.6: CALCULATED AND MEASURED ROTATIONS FOR THE WEST ABUTMENT WALL

Method	Value (degree)
Simple Elastic Analysis	+0.016
Finite Element Method	
K = 450	+0.073
K = 900	+0.022
Field Measurement	+0.021 to +0.044

Note: Clockwise rotation is positive.

rotated toward the backfill. Values calculated using the simple elastic procedure and those calculated using the finite element anlayses agree quite well with the measured values.

CHAPTER 7

TOLERABLE MOVEMENT OF STRUCTURES

7.1 INTRODUCTION

Foundations should be designed so that the movements they undergo will not exceed values that can be tolerated by the structures they support. It is thus important to understand the factors that control the magnitudes of movements that structures are able to tolerate, and to design foundations so that movements do not exceed tolerable values.

In principle, the effects of settlement can be evaluated in two ways -- through structural analysis to examine the consequences of support movements, or through field surveys of existing structures. Both procedures are discussed in the following sections.

In practice, conventional structural analyses frequently result in excessively conservative settlement criteria. Most real structures are able to withstand settlements that are greater than would be inferred from the results of conventional structural analyses. Consequently, in most cases, field surveys of the behavior of real structures provide the most reliable means of establishing upper limits for the amounts of settlement that can be tolerated by structures.

As explained in Sections 7.4 and 7.5, an effective method of establishing the magnitude of the settlement that can be tolerated by a structure is to (1) use the results of field surveys of real structures to estimate the amount of settlement that can be tolerated by the structure, and (2) compare this tolerable settlement to the estimated settlement of the foundation, to determine if its behavior will be acceptable with regard to settlement.

There is relatively little information concernig horizontal displacements and their effects. It is known, however, that horizontal displacements can be damaging to structures, and in some cases horizontal displacements are even more damaging than settlements of equal magnitude. Tolerance criteria for horizontal displacements are scarce, and they are often borrowed from the mining industry.

Sections 7.2 through 7.6 are concerned with the following aspects of the tolerable movements of structures:

- the nature of movements and the types of damage that they cause;
- the use of structural analysis techniques to evaluate the possible consequences of settlement;
- experience-based limits for the amount of angular distortion that can be tolerated by bridges, buildings and steel tanks;
- experience-based limits for the amount of total

horizontal displacements that can be tolerated by structures;

- correlations between angular distortion and settlement based on field surveys.

7.2 MOVEMENTS AND MOVEMENT PROBLEMS

To establish a basis for tolerable movement criteria, it is useful to examine the nature of foundation movements and the types of problems that result when movements become excessive.

7.2.1 Settlements

7.2.1.1 Components of Settlements

The settlements of structures can be divided into three components -- rigid body vertical displacement, tilt or rotation, and distortion, as shown in Figure 7.1.

Rigid body vertical displacement (uniform settlement) does not cause distortion of structures, and so does not cause problems like cracking and misalignment of windows and doors. Excessive settlement in this mode can result in problems with drainage or disruption of utilities where they are connected to the structure, but these problems are less frequent than the problems that are associated with nonuniform settlement.

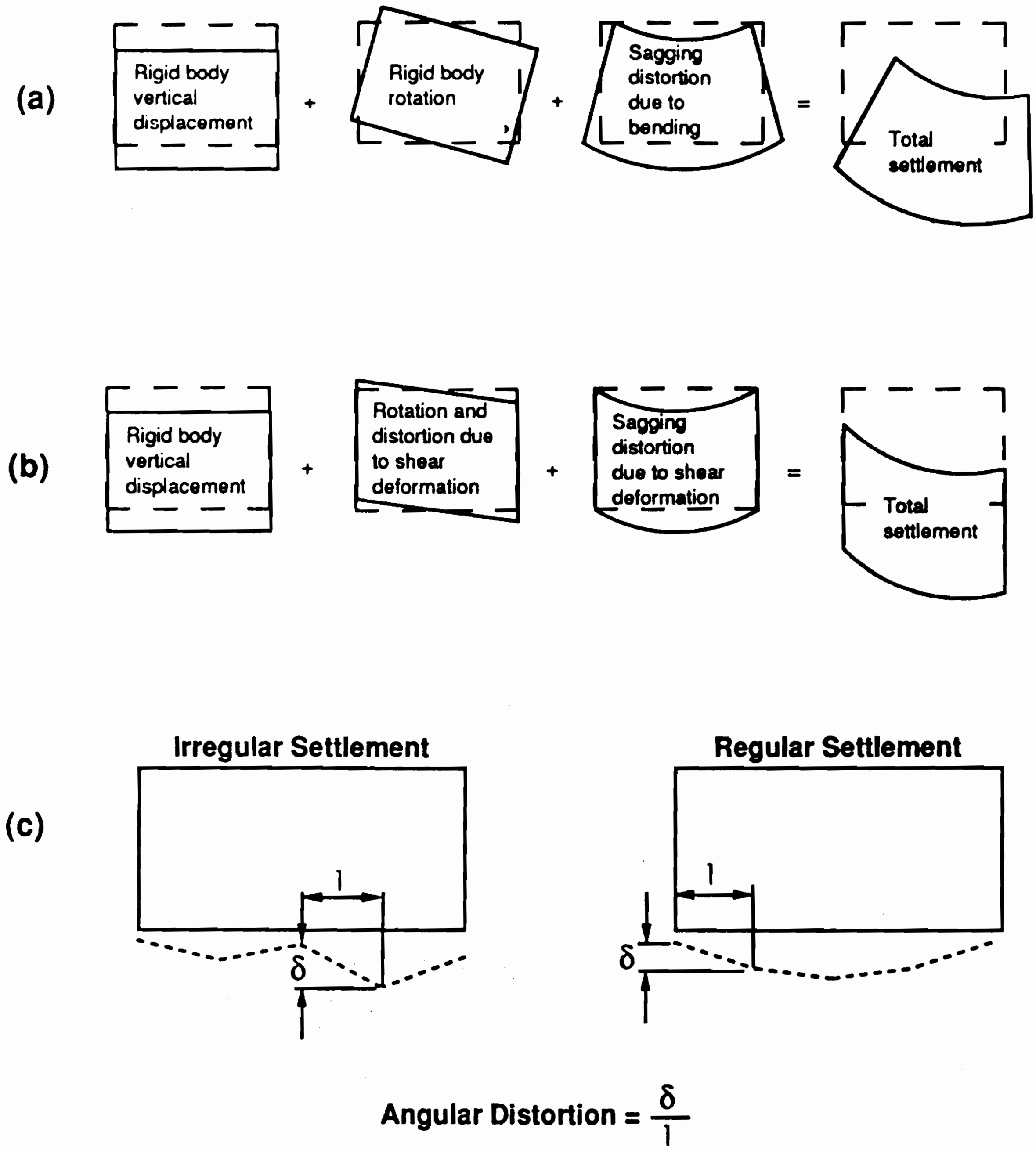


Figure 7.1: Components of Settlement and Angular Distortion

In some cases nonuniform settlements may involve only rigid body rotation (rotation with no deformation) as shown in Figure 7.1(a). This mode of settlement would be expected for a very stiff structure. Although this type of settlement does not cause cracking or misalignment problems, rotation (or tilt) by itself may be a problem: When a tall structure like a stack or silo tilts through an angle of 0.004 radians (about 0.25 degrees), it can be perceived to be out of plumb, and people in or around the structure may feel uneasy about its safety.

In most cases nonuniform settlements (differential settlements) involve deformation of the structure. Three different types of structural deformation are shown in Figure 7.1(a) and (b). These are distortion due to bending, rotation with shear deformation, and distortion due to shear deformation. Distortion of structures as they settle is the cause of cracking and misalignment problems, the most frequent problems caused by settlement.

In addition to the modes of deformation shown in Figure 7.1, other modes of deformation are also possible. The edges of some structures have been known to settle more than the center. This is called "hogging" as opposed to the "sagging" patterns of settlement shown in Figure 7.1. Field studies suggest that hogging settlements are more damaging than sagging settlements, at least in some cases.

In real structures the various components of settlement -- rigid body vertical displacement, rotation, and distortion, all occur at the same time. Two possible scenarios are shown in Figure 7.1(a) and (b). Figure 7.1(a) illustrates a condition in which rigid body vertical displacement and rigid body rotation are combined with bending deformation, resulting in a complex pattern of rigid body movement and distortion. Figure 7.1(b) illustrates a second condition, in which rigid body vertical deformation, rotation accompanied by shear, and distortion due to shear are combined, resulting in a slightly different, but equally complex end result. The number of possible conditions in real structures is practically unlimited.

<u>7.2.1.2 Measures of the Severity of Settlement</u>

Due to the complexity of settlement patterns, no single measure of settlement or distortion affords a perfect indicator of whether a given amount of settlement will cause damage to a structure. Skempton and MacDonald (1956) found that the magnitude of angular distortion caused by settlement provided an effective index of the potential for settlements to cause cracking and misalignment problems in buildings. Most of the buildings they studied were frame structures.

The definition of angular distortion used in this study is shown in Figure 7.1(c). It is the differential settlement

(δ) between two locations in a structure, divided by the distance between them (l). If the settlement pattern is regular, the maximum angular distortion occurs at the edge of the structure. If the settlement is irregular, the maximum angular distortion can occur anywhere within the structure.

Skempton and MacDonald, and also Grant, et al. (1974), subtracted the component of differential settlement that is due to rotation, or tilt of the structure. They chose to exclude the effects of rotation because, if only rigid body rotation is involved, as shown in Figure 7.1(a), rotation causes no distortion of the structure. However, Leonards (1975) pointed out that, if the columns and footings in a structure do not rotate along with the base of the structure, rotation does involve shear deformation. This type of rotation is illustrated in Figure 7.1(b).

Thus it is not clearly preferable to eliminate rotational components of movement from δ/l. Also, it is difficult to estimate the magnitude of the rotational component at the design stage. It is therefore preferable, for the purpose of establishing practical limits of tolerance for differential settlement and angular distortion, to include the rotational component. The result is a simpler and more easily useable criterion, which is probably no less precise in practice.

Polshin and Tokar (1957) summarized Russian experience with damage to structures caused by settlements. Their experience was in substantial agreement with Skempton and MacDonald's. The main difference between the work of Skempton and MacDonald and that of Polshin and Tokar was related to the behavior of buildings with load-bearing walls, with regard to which Polshin and Tokar had collected a considerable amount of data. They found that these types of buildings are more likely to deform in bending than in shear, and that hogging deformations were more damaging in some cases than were sagging deformations.

Polshin and Tokar suggested the use of a quantity they called "relative deflection" as a criterion of possible damage for buildings with load-bearing walls. The deflection ratio is equal to the maximum vertical offset of any point within a building, measured from a line joining the ends of the building, divided by the length of the building

Burland and Wroth (1975) showed that the observations made by Polshin and Tokar agreed well with a beam analogy they used to model bending deformations in buildings with load-bearing walls. They showed that damage to this type of structure correlates well with the development of a critical value of tensile strain in the building, which results in the development of cracks. Although Polshin and Tokar's studies and Burland and Wroth's studies indicate that it may be preferable to use relative deflection (or "deflection ratio" as it was called by Burland and Wroth) as a damage criterion for buildings with load-bearing walls, comparisons with available field measurements indicate that observed damage correlates about equally well with angular distortion.

As noted previously, Skempton and MacDonald found that angular distortion correlated well with damage to frame buildings. Moulton et al. (1985) found that damage to bridges could also be correlated with angular distortion. Thus, angular distortion as defined in Figure 7.1(c) appears to be a reasonable measure of the severity of settlements, and a suitable basis for estimating tolerable settlements of buildings and bridges.

7.2.1.3 Types of Damage Caused by Settlement

Various types of damage that may be caused by settlement are listed in Table 7.1. These include aesthetic damage, functional damage, and structural damage.

Aesthetic damage, which was called architectural damage by Skempton and MacDonald, involves damage that mars the appearance of structures. Damage in this category includes fine cracks, minor tilt in walls, and slight deviations from levelness in floors.

Functional damage involves types of distress that diminish the usefulness of structures, or interfere with the comfort of occupants or users. In bridges these effects are concerned with jamming of components, tilt and misalignment.

TABLE 7.1: TYPES OF DAMAGE CAUSED BY SETTLEMENT

		Typical Problems		
Type	Basis	Bridges	Buildings	Tanks
Aesthetic damage	Appearance	- Hairline and fine cracks, less than 5 mm wide in abutment walls, parapets, and concrete at bearings - Minor tilt of abutment walls, piers, and parapets	- Hairline and fine cracks, less than 5 mm wide - Minor tilt of columns or walls - Out-of-level floors	- Usually not a problem
Functional damage	- Usefulness - Human comfort	- Jamming of beams against abutment walls - Noticeable tilt of rockers, bearings, and parapets - Misalignment of bearing shoes and rockers - Poor riding quality	- Jamming of windows or doors - Disruption of drainage - Floors sloping noticeably - Walls leaning or bulging noticeably	- Ponding of product on the bottom of the tank, failure to drain properly - Disruption of connected piping - Jamming of floating roofs - Loss of seal of floating roofs
Structural damage	- Structural integrity - Structural stability	- Noticeable cracks in beams, girders, struts, and diaphragms - Excessive cracking in bridge decks - Badly leaning abutment walls that require shoring	- Noticeable cracks in beams, columns, and load-bearing walls - Loss of bearing for beams - Badly leaning walls that require shoring for stability - Danger of instability	- Rupture of tank bottom - Rupture of tank shell - Buckling of fixed roofs

These can result in poor riding quality. Functional damage in buildings usually involves doors or windows that jam, and changes of slope that disrupt drainage. Functional damage in tanks involves changes in slope of the tank bottom that causes ponding of the product on the bottom and makes it difficult to drain the tank completely, damage to connected piping, and floating roof problems.

Structural damage is the most severe and dangerous consequence of settlement. In all types of structures, structural damage involves serious deterioration of structural capacity or stability, and, in the extreme, actual collapse of the structure.

7.2.2 Horizontal Displacements

As in the case of settlements, rigid body horizontal displacement (uniform horizontal displacement) does not cause distortion of structures, but excessive horizontal displacement in this mode can result in functional damage, as described in Table 7.1.

Relative (or differential) horizontal displacements can cause distortions to structures, and they may thus lead to structural cracking and misalignment problems. Potential for such damage is frequently measured in terms of the magnitude of horizontal ground strain.

7.3 USE OF STRUCTURAL ANALYSIS TO EVALUATE THE CONSEQUENCES OF SETTLEMENT

It would appear logical that structural analysis methods could be used to evaluate the effects of settlements on structures. In principle, the shear forces and bending moments caused by settlements could be added to those due to imposed loads, and compared with the shear and flexural capacities of the structural elements to determine if the structure could withstand the expected settlements without damage. It has been found, however, that this procedure usually leads to excessively conservative results, indicating that very large overstress would be caused by rather small settlements, even though experience indicates such settlements could be tolerated with no distress.

One such study was performed by Meyerhof (1947). He analyzed the effect of foundation settlement on a five-story, three-bay reinforced concrete building. He found that 0.315 inches of settlement at one of the columns (corresponding to an angular distortion of 0.001 in bays that were 25 ft long) corresponded to a calculated increase of 74 percent in the moment in the most severely loaded beam in the building. This magnitude of increase in bending moment would certainly be expected to cause structural damage. However, field data compiled by Skempton and MacDonald (1956), Polshin and Tokar (1957), and Grant, et al. (1974), which are summarized in

Table 7.2, indicate that framed structures can withstand twice as much angular distortion as considered by Meyerhof without suffering even minor cracking or any other form of damage. It is thus clear that real frame buildings are able to withstand considerably greater differential settlements than would be inferred from the results of structural analyses.

It can be demonstrated readily that the same is true of bridges. Consider a case where the center support of a two-span bridge settles by an amount δ relative to the abutments. The bending stress induced in a simple rectangular reinforced concrete bridge deck by this settlement can be expressed as

$$f_{max} = \frac{\delta}{S} \; \frac{3E}{2} \; \frac{t}{S} \tag{7.1}$$

in which f_{max} = maximum bending stress induced by settlement (same units as E)

δ = differential settlement between the center pier and the abutment (same units as S)

E = Young's modulus (stress units)

t = thickness of deck (same units as S)

S = span length (length units)

To prevent cracking, the maximum tensile bending stress should not exceed the tensile strength of the material, f_t. The value of δ/S corresponding to $f_{max} = f_t$ can be expressed as:

$$\frac{\delta}{S} = \frac{2}{3}\frac{f_t}{E}\frac{S}{t} \tag{7.2}$$

For properies appropriate to concrete with f'_c equal to 4000 psi (f_t = 475 psi, E = 3.8×10^6 psi), S/t = 30, the calculated value of δ/S is 0.002. Thus, this simplified analysis of the consequences of differential settlement in a multiple-span bridge would indicate that cracking would occur if the angular distrtion (δ/S) reached a value of 0.002.

The survey of damage to bridges caused by settlements reported by Moulton, et al. (1985) shows clearly that continuous-span bridges can withstand much larger values of δ/S without damage. The results of Moulton, et al.'s survey indicate that continuous-span bridges, of either steel or concrete, can sustain angular distortions of 0.004 without damage. This is about two times as great as the values inferred from the simplified structural analysis.

Thus, for both bridges and buildings, field experience shows that real structures can tolerate considerably greater magnitudes of angular distortion than would be inferred from the results of simple structural analyses. The reasons for this include the fact that building materials like concrete (especially concrete while it is curing) are able to undergo a considerable amount of stress relaxation when subjected to deformations, and under conditions of very slowly imposed deformations, the effective value of the Young's modulus of concrete is considerably lower than the value for rapid loading. Thus, inferences about the stresses induced by deformations that are based on analyses that ignore the beneficial effects of stress relaxation, and the consequences of the rate of deformation, will be overly conservative for these materials. Another important factor is that part of the settlement of real structures occurs as they are built. Therefore, the portions of the structures that are constructed last do not experience all of the settlement, and cannot be damaged by it.

It is therefore preferable in most case to use field studies as the basis for tolerable settlement criteria, rather than structural analyses. If, in unusual cases it is necessary to use structural anlyses as the basis for evaluating maximum tolerable settlements, the analyses should include consideration of important factors such as stress relaxation, the rate of deformation, and the construction sequence.

7.4 ALLOWABLE VALUES OF ANGULAR DISTORTION FOR BRIDGES, BUILDINGS, AND TANKS

Studies of settlement-induced damage of buildings, bridges, and tanks provides a basis for establishing criteria for the amounts of settlement that can be tolerated by these structures without damage. These criteria are discussed in the following paragraphs.

7.4.1 Buildings

Table 7.2 shows values of allowable angular distortion (δ/l) based on the studies conducted by Skempton and MacDonald (1956), Polshin and Tokar (1957), Bjerrum (1963), Grant, et al. (1974), Burland and Wroth (1975), and Wahls (1981). The values shown are allowable in the sense that larger values than these would be required to induce cracking in buildings. For example, the survey of settlement-induced damage by Skempton and MacDonald indicated that the value of δ/l that best represented the threshold of damage for frame buildings was 0.003. Thus the value of $\delta/l = 0.002$ for frame buildings listed in Table 7.2 includes a margin of safety against the development of cracks due to settlement.

The value of $\delta/l = 0.004$ for tall, slender structures does not refer to the liklihood of cracking. This value applies to stiff structures that would be likely to undergo

TABLE 7.2: ALLOWABLE VALUES OF ANGULAR DISTORTION FOR BUILDINGS
(After Skemptom and MacDonald, 1956; Polshin and Tokar, 1957; Bjerrum, 1963; Grant, et al., 1974; Burland and Wroth, 1975; and Wahls, 1981)

Type of Building	Allowable Value of Angular Distortion (δ/l)
Steel frame with flexible siding	0.008
Steel or reinforced concrete frame with dry wall, plaster, stucco, brick, block	0.002
Load-bearing walls of brick, tile or concrete	0.001
Tall, slender structures such as stacks, silos and water tanks, with rigid mat foundations	0.004

Note: These allowable values of δ/l can occur without cracking or other types of distress or loss of functionality. The limit of δ/l = 0.004 for tall, slender structures is the threshold of human perception of a structure being out of plumb.

rigid body rotation, with little internal deformation. Even though they may not crack, the angular rotation of these structures should be limited to 0.004 so that they will maintain a stable appearance.

7.4.2 Bridges

Moulton, et al. (1985) conducted a survey of about 300 bridges that had suffered damage due to settlement. The study showed clearly that the liklihood of damage increased with the amount of angular distortion (δ/S), and that single-span bridges are able to tolerate more angular distortion without damage than are multiple-span bridges.

The allowable values of δ/S listed in Table 7.3 are based on the results of Moulton's study. These values were established based on the 95th percentile of damaged bridges. That is, of all the damaged bridges included in Moulton's survey, 95 percent had undergone greater amounts of angular distortion than the values listed in Table 7.3. Only 5 percent had undergone less angular distortion.

The use of a 5 percent threshhold appears to be a reasonable basis for establishing the criteria for allowable angular distortion, because it was not established conclusively that all of the observed damage was due to settlement. Some could have been due to shrinkage, temperature effects, or other influences. Especially in the

TABLE 7.3: ALLOWABLE VALUES OF ANGULAR DISTORTION FOR BRIDGES
(Based on the 95th Pecentile of Damage in the Data Collected by Moulton, et al., 1985)

Type of Bridge	Allowable Value of Angular Distortion (δ/S)
Multiple Span (Steel or Concrete)	0.004
Single Span (Steel or Concrete	0.007

Note:

δ = differential settlement (same units as S)

S = span length (unit of length)

cases where the settlements were smallest, these other factors are likely to play important roles.

Moulton, et al.'s study revealed a clear difference between the tolerance of single-span and multiple-span bridges to withstand settlement without damage. As can be seen in Table 7.3, single-span bridges can withstand nearly twice as much angular distortion without damage as can multiple-span bridges.

The data showed that distress in the superstructure was reported more frequently for concrete structures than for steel structures, indicating that steel structures are able to undergo somewhat greater magnitudes of settlement without damage. However, distress in the superstructures of concrete bridges was considered to be "tolerable" more frequently than was distress in the superstructures of steel bridges. This apparent conflict between the quantitative data regarding damage and the qualitative evaluation of whether it was acceptable or not makes it difficult to draw conclusions regarding the relative tolerance of concrete and steel bridges to settlement. Thus the values of angular distortion given in Table 7.3 are the same for concrete and steel bridges.

7.4.3 Steel Tanks

D'Orazio and Duncan (1987) developed the criteria shown in Table 7.4 for steel tanks. These structures are not subject to aesthetic damage, and are able to withstand quite large settlements because of their flexibility.

A survey of the behavior of steel tanks showed that they may settle in any of the three profile shapes shown below Table 7.4. Which profile is more likely depends on the thickness of the compressible layer in relation to the effective diameter of the tank, and the value of the minimum factor of safety with respect to shear failure in the foundation. Many tanks constructed on clay have quite low factors of safety with respect to undrained failure in the foundation, and shear strains make an important contribution to the differential settlements they experience. Thus it is necessary to consider the value of the undrained factor of safety in order to be able to anticipate the shape of the settlement profile.

For the same difference in settlement between the center and the edge of the tank, tanks with settlement profile A undergo the least angular distortion, and those with settlement profile C undergo the most. Thus different amounts of differential settlement are required to cause structural damage in the three cases.

Values of $\Delta\rho/D$ smaller than those shown in Table 7.4 may result in loss of drainage, depending on the initial slope of the tank bottom, and they may cause problems with attached piping or floating roofs, depending on the

TABLE 7.4: ALLOWABLE VALUES OF $\Delta\rho/D$ for STEEL TANKS (After D'Orazio and Duncan, 1987)

D_e/T	F_{min}	Settlement Profile Shape	Allowable $\Delta\rho/D$
≤ 4	≥ 1.1	A	0.025
> 4	≥ 1.1	B	0.015
Any Value	< 1.1	C	0.005

Note: These values of $\Delta\rho/D$ can occur without structural damage.

ρ = settlement

$\Delta\rho = \rho_{center} - q_{edge}$ = center settlement - edge settlement

D = Tank diameter

T_p = Thickness of granular foundation pad

T = Thickness of compressible clay layer

F_{min} = Minimum factor of safety with respect to shear failure in the foundation details of construction. These possibilities need to be considered when the design details for tanks are developed.

Table 7.4, and the previous discussion, are concerned with differential settlements across the tank diameter. This type of differentaial settlement has the potential for causing the problems listed in Table 7.1, including rupture of the tank. Tanks are also subject to differential settlements around their perimeters. Differential settlement of tank walls causes lateral deflections of the walls, and can result in floating roof problems, but does not have the potential to cause rupture of the tank. This type of settlement has been discussed by D'Orazio, et al. (1989).

7.5 RELATIONSHIP OF ANGULAR DISTORTION TO SETTLEMENT MAGNITUDE

In the case of bridges, values of angular distortion (δ/S) can be calculated using conventional settlement analysis techniques and compared to the tolerable values in Table 7.3. The value of δ is the maximum difference between the estimated values of settlement at the two ends of the span.

In the case of tanks, a similar procedure is followed. Conventional settlement analysis techniques are used to estimate the settlement at the center of the tank and at the edge. The value of $\Delta\rho$ is the difference between the center and edge settlements based on the settlement estimates, and the estimated value of $\Delta\rho/D$ is compared to the values given in Table 7.4 to determine if settlement-induced damage is likely.

In the case of buildings, a somewhat different procedure is desirable, because it is difficult to estimate the differential settlements in buildings accurately, particularly when the building is supported by spread footings on sand. In such cases it may be possible to estimate an upper limit for the maximum settlement, but very difficult to estimate the magnitude of the differential settlement, or to anticipate which point in the building will settle most and which will settle least.

In these cases use can be made of the type of information shown in Table 7.5. The values in this table, which are based on field observations, establish a correlation between the maximum settlement and the amount of angular distortion. The values shown in the table are the values of angular distortion corresponding to a maximum settlement of 1.0 inch. If the maximum settlement is 2.0 inches, the angular distortion would be expected to be twice as great as the tabulated values. If the maximum settlement was 3.0 inches, the angular distortion would be expected to be three times as great, and so on. Thus, once the maximum settlement has been estimated, the angular distortion can be estimated by multiplying Δ_{max} (expressed in inches) by the values in the table.

TABLE 7.5: CORRELATION BETWEEN MAXIMYM ANGULAR DISTORTION, $(\delta/l)_{MAX}$, AND MAXIMUM SETTLEMENT, Δ_{max} (AFTER SKEMPTON AND MACDONALD, 1956; GRANT, ET AL., 1974; AND GOULD AND PARSONS, 1975)

Soil Type	Foundation Type	Approximate Value of $(\delta/l)max$ for Δmax = 1 in
Sand	Isolated footings (piles, piers, or spread footings)	0.0017
	Mat foundations	0.0013
Varved Silt	Isolated footings (piles, piers, or spread footings)	0.0017
	Mat foundations	0.0005
Clay	Isolated footings (piles, piers, or spread footings)	0.0010
	Mat foundations	0.0008

The primary use of Table 7.5 is for foundations on sand, where settlements vary erratically, and differential settlements are virtually impossible to estimate with accuracy. Many silt and clay deposits have more uniform properties than natural sand deposits, and settlements vary more systematically from place to place within buildings constructed on sites underlain by silt and clay. Thus the settlement profile is more likely to be of the regular type shown in Figure 7.1, and differential settlements can be estimated with reasonable accuracy based on conventional settlement analysis techniques.

There is, however, some randomness in the settlement patterns on silt and clay, and there are at least rough correlations between angular distortion and maximum settlement for foundations on these materials, as shown in Table 7.5. The correlations between Δ_{max} and δ/l for silt and clay that are given in Table 7.5 provide a simple alternative to the use of detailed analyses to estimate differential settlements and angular distortion values. Once the maximum settlement has been estimated, the angular distortion can be estimated roughly using the same procedure as for foundations on sand.

7.6 ALLOWABLE HORIZONTAL DISPLACEMENT

The effects of horizontal displacement on structures have been studied by Burland and Wroth (1975); Bozozuk (1978); Walkinshaw (1978); Moulton, et al. (1985); and Boscardin and Cording (1989). It has been found that bridges can tolerate 1.0 in to 2.0 in of horizontal movement without suffering unacceptable damage. For buildings, damage of structures is related to horizontal ground strain, as shown in Figure 7.2. The curves in this figure shows that a structure 30 to 40 m long may suffer appreciable to moderate damage (for example, sticking of doors and windows), if horizontal strains exceed 0.15 percent.

In the case of retaining and abutment walls, a further consideration is the fact the top of retaining and abutment walls must be free to move about 1 in for each 20 ft of height if they are designed to withstand only active earth pressure. With movements at the top of these magnitudes, active pressures can be developed. Whether the base of the wall translates does not affect the earth pressures. Thus the wall may either rotate or translate, provided the movement at the top of the wall is large enough to develop active pressure. If movements of the magnitude required to develop active earth pressure are not tolerable, the retaining and abutment walls must be designed to withstand earth pressures that are higher than the active values.

Based on limited field data, Meyerhof (1977) suggested that abutment and retaining walls can rotate as much as 0.007

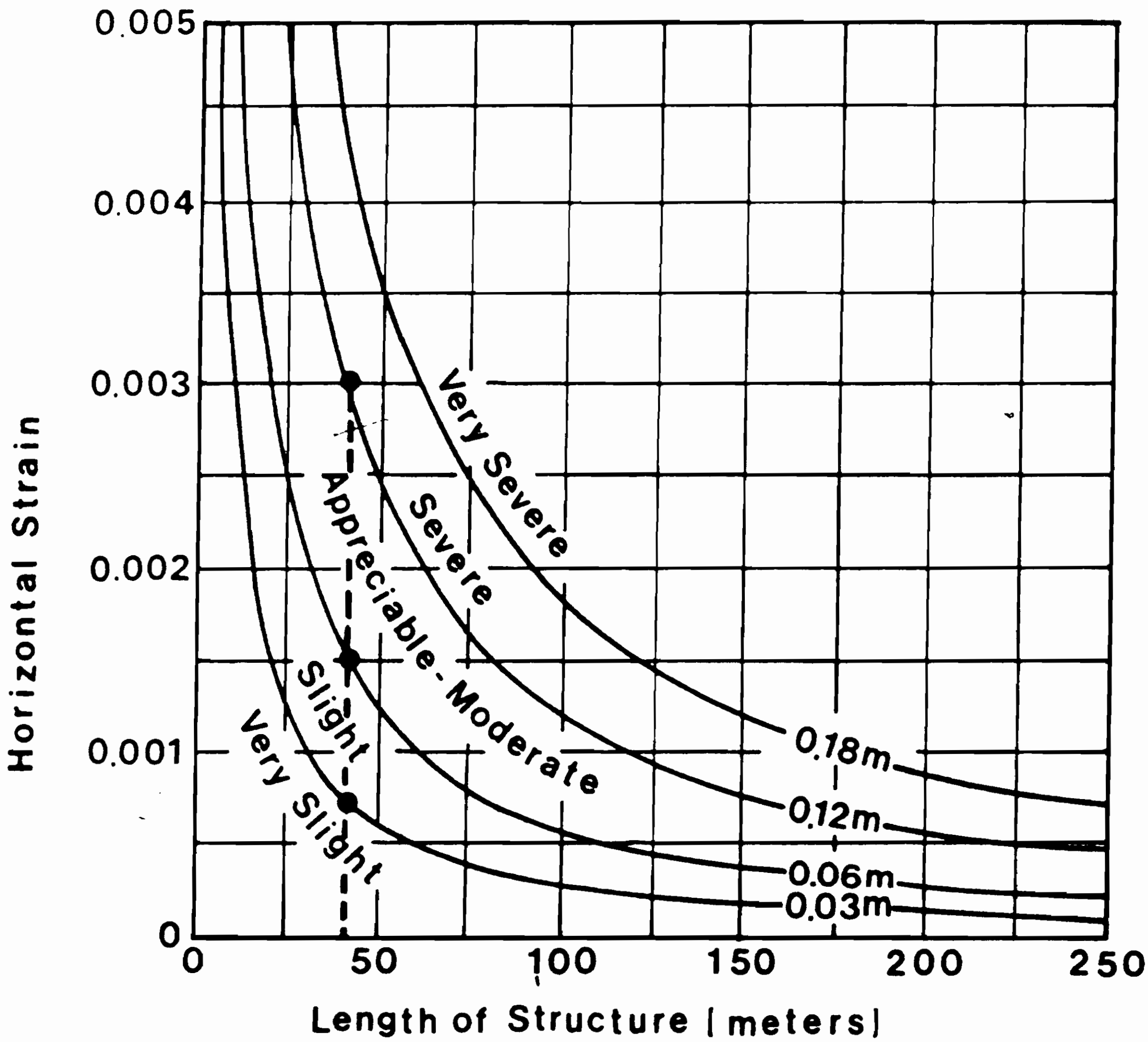


Figure 7.2: Relationship of Damage to Length of Structure and Horizontal Ground Strain (After The National Coal Board, 1975)

without suffering serious consequences. This means that a wall of 20 ft high can tolerate a relative horizontal movement of about 1.7 in at the top of the wall, although the base may translate even more than this amount.

CHAPTER 8

SUMMARY AND CONCLUSIONS

The objectives of the study presented in this dissertation were (1) to evaluate available methods for estimating settlements of footings on sand, (2) to develop a simple procedure for estimating movements of retaining and abutment walls, and (3) to develop a procedure for relating the magnitudes of horizontal movements and rotations of footings and walls to the magnitudes of their settlements.

Twelve methods of estimating settlements of footings on sand were examined. Eleven of these use the Standard Penetration Test as a basis for estimating settlements. Schmertmann's method uses the Cone Penetration Test (CPT). The methods were evaluated by comparing calculated and measured settlements of 76 footings on sands. Standard Penetration Tests results were available for all 76 cases, and CPT available for 15. The methods were evaluated in terms of accuracy, reliability, and ease in use. Accuracy is defined as the average value of calculated settlement divided by measured settlement. Reliability refers to the percentage of cases in which the calculated settlement equals or exceeds the measured settlement. The ease of use of the method was measured by the time required to apply the method.

The results of the evaluation indicate that the various methods of estimating settlements that were studied were developed to achieve different objectives. Most of the methods appear to have been devised to achieve the best possible accuracy. Terzaghi and Peck's method, and Peck, Hanson, and Thornburn's method, on the other hand, appear to have been devised with maximum reliability in mind. It is clear from the results of the study that there is a tradeoff between accuracy and reliability. Improvement in reliability seems always to involve poorer accuracy, and improvement in accuracy seems always to result in lower reliability.

The accuracy and reliability of any of the methods of estimating settlements of footings on sand can be changed through the simple process of multiplying the values of settlement calculated using the "standard" method by a factor. The optimum method is one that is characterized by high reliability, and the best possible accuracy. If it was desired to achieve 90 percent reliability with the best possible accuracy, the results of the study indicate that a reasonable procedure would be to use D'Appolonia, et al.'s method, and to multiply the value of settlement by 2.0. The results of the study are, however, somewhat dependent on the data base employed in the analyses, and somewhat differing results might be achieved if another data base was used.

The study also shows that the adjustment factors for different combinations of accuracy and reliability can be determined using probability theory. The settlement ratio, defined as the ratio of calculated to measured settlement, was shown to be lognormally distributed. Given the probability density function of the settlement ratio, the required value of the adjustment factor can be determined using the values of the mean and the standard deviation of the settlement ratio. The adjustment factors calculated using the simple procedure agree quite well with those from the probability theory.

The methods suggested by D'Appolonia, et al. and Terzaghi and Peck were found to afford reasonable combinations of accuracy and reliability with fairly low amounts of time required for the analyses.

The simple procedure for estimating movements of retaining and abutment walls due to the weight of backfill, as presented in this dissertation, is based on elastic theory. In developing the influence factors for settlement, horizontal movement, and rotation, the soil was assumed to be isotropic and homogeneous, and the loading imposed by the backfill was approximated by a series of strip loadings. Values of the influence factors are functions of the width of the wall base, the thickness of the compressible layer, the lateral extent of the backfill at its topmost elevation, and the Poisson's ratio of the foundation soil.

In order to avoid the problem inherent in the elastic methods for estimating foundation and wall movements, a procedure was developed to relate horizontal movements and rotations to settlements which can be estimated using well-established conventional methods. The proposed procedure eliminates the need for selecting values of soil modulus in order to calculate movements and rotaions.

The procedures for estimating wall movements developed in this study were applied to a case history involving a well-instrumented bridge abutment in Rhode Island. Behavior of the abutment wall was also studied using the finite element method, with the computer program SOILSTRUCT. The movements calculated using the simple elastic procedures are in fairly good agreement with those evaluated using the finite element method and the measured values.

It can be concluded on the basis of previous work that tolerable settlement criteria are best established using semi-empirical procedures. The allowable settlement depends, among other factors, on the type and stiffness of the structure. Tolerance criteria can be expressed in terms of angular distortion, and allowable values of angular distortion have been proposed for buildings, bridges, and steel tanks.

Tolerable criteria for horizontal movements are scarce, and most in use are based on studies by the mining industry. Studies by Moulton, et al, (1985) concluded that bridges can tolerate 1 to 2 in of horizontal movement without suffering damage. In buildings, the degree of damage resulting from excessive horizontal movements is usually related to the magnitude horizontal ground strains.

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