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A MATHEMATICAL ALGORITHM FOR SELECTION OF
SPT RESISTANCE VALUE IN FOUNDATION DESIGN

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A MATHEMATICAL ALGORITHM FOR SELECTION OF SPT RESISTANCE VALUE IN FOUNDATION DESIGN

By

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ABSTRACT

A MATHEMATICAL ALGORITHM FOR SELECTION OF
SPT RESISTANCE VALUE IN FOUNDATION DESIGN

By

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A mathematical algorithm has been developed to aid foundation engineers in the selection of a design SPT resistance or N-value. The algorithm accounts for most factors that affect standard penetration test resistance. It is verified by analyzing a large number of case histories of actual foundations on sand. The algorithm is used to select a design N-value for each case history. That value was compared with those selected by experienced and knowledgeable engineers. Encouraging agreement was obtained between the algorithm's results and those used by foundation engineers. The potential benefits of the proposed algorithm outweighs any difficulty in interpreting the results of SPT because of a wide variation of test results.

Soil stress history expressed by the coefficient of earth pressure (Ko) was found to have a significant effect on the settlement of foundations on sand. This parameter together with the proposed algorithm were used to introduce a new procedure for estimating foundation settlements on sand. The new settlement procedure was compared with the most widely used settlement equations. The new procedure tends to be consistent and accurate in all cases.

To my mother

and

my father

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CHAPTER 1

OBJECTIVE AND SCOPE

1.1 Introduction

Foundation analyses normally begin with an assessment of the soils and rocks at the anticipated site of a structure. Geotechnical engineers have been mainly concerned with evaluation of soil properties and the prediction of foundation performance under a superstructure. They have long recognized the considerable variability of soil materials as exhibited from laboratory tests and field performance.

For shallow foundation design problems, the ultimate bearing capacity of cohesive soils can generally be estimated from bearing capacity theory and the shear strength of undisturbed samples. However, for soils with little or no cohesion, which are the focus of this study, their engineering design parameters are usually determined by field sounding tests such as the Cone Penetration Test (**CPT**) and the Standard Penetration Test (**SPT**). The latter (**SPT**) is still the most widely used test in the United States of America and other parts of the world and will be dealt with in this study.

In design of shallow foundations, for "routine" structures on sand, the soil boring data, SPT test results and a few load values would include the all information available to a designer in the United States, and no additional data would be obtained unless there were special circumstances.

In view of these uncertainties and variability of various soil boring data and standard penetration resistance (N-value) results, an important question to be considered by the engineer is what design N-value should be selected for estimating engineering design parameters such as the soil modulus (**E**) and the angle of internal friction (ϕ). Selection of a design N-value is a pivotal point in the foundation analysis and design, yet there is no rational consistent procedure or mathematical algorithm to guide a designer to select this design value. Some engineers will use a mean of the measured values while others will assume the most conservative of the measured values (Whitman 1984, and Wolff, 1986). Moreover, the same engineer may adopt a different approach for similar jobs. Thus one soil foundation with a reported safety factor of 3 may actually have little margin of safety, while another with the same reported safety factor may be highly safe against failure.

In pavement design, it was shown by Yoder (1969) and Yoder and Witczak (1975) that if the average of measured soil design values is used, about one-half of the roads would be overdesigned and one-half would be underdesigned. On the other hand, if the minimum of measured values were selected, most of the roads would be overdesigned. As a result of this problem, a new method was developed Yoder (1969). The approach depends partly on the variation of soil properties and partly on the design load for the pavement. A similar approach should be applicable to Standard Penetration Test results. In a recent study of uncertainty and engineering judgment in the design of footing on shallow foundation (Wolff, 1988, 1989), it was found that professional engineers who were given same foundation problem and same related design information selected N-values that varied considerably. Consequently, there were substantial differences in the reported values of bearing capacity of the shallow foundation as well as the calculated settlements.

In the interest of safety, to compensate for uncertainties, the geotechnical engineer prudently makes conservative assumptions about soil parameters, applied loads, and allowable settlement, and frequently about all of those (Vanmarcke, 1975).

Different approaches for selecting a design N-value as well as the tendency of foundation engineers toward conservatism in the design, arise from the fact that most designers are aware of numerous factors which affect substantially the results of SPT test,but have not yet been accounted for quantitatively. Recently, foundation researchers have conducted numerous experiments in the field concerning standard penetration test (Kovacs and Salomone,1982, and Robertson et al.,1983). Their findings clearly illustrated that there are many factors beside overburden pressure that are extremely important in the interpretation of SPT results.

In light of new developments and recent available information in the literature about SPT, the intent of this study was to quantify an approach and to lay out a systematic way for selecting a design N-value through developing a mathematical algorithm that takes into account the most significant factors that influence the results of SPT. Doing so undoubtedly would help practicing engineers to make rational decisions about a foundation of the structure, and consequently should lead to more consistency than associated with previous approaches. The latter is desirable since up to the present the SPT is still one of the most commonly used in-situ tests for site investigation (Tavenas, 1986; Chung-Tie 1988) and remains the workhorse of the practicing engineers despite advances in other in-situ tests.

1.2 Objectives of study:

In this study, a mathematical algorithm was established to aid a design engineer in selecting a rational standard penetration resistance number (N-value). This number is the basis for estimating bearing capacity and settlement for shallow foundations on sand. Development of such an algorithm will be based partly on the current available procedures for selecting a design N-value and partly on the knowledge gained from laboratory tests and field data concerning the most significant factors that affect the interpretation of SPT results.

To meet this objective, the following secondary objectives will be identified:

1- Study the current approach in selection of design N value and how designers treat SPT results during analysis and design of shallow foundations. This step will be accomplished by reviewing case histories that involve the design of such foundations based on SPT results.

2- Study the effects of the most important factors that substantially influence the interpretation of SPT results for foundation design. This will be done on the basis of the knowledge gained recently from laboratory experiments and field data.

Thus the developed algorithm should either yield a design N-value that an experienced and knowledgeable designer would select or a value that will lead to increase the accuracy of the SPT-based settlement equations for better prediction of foundation performance. Whether a single algorithm will do both is addressed herein.

1.3 Scope Of Study:

In the following chapter the various factors that influence the standard penetration values are discussed. An attempt has been made to give a rational explanation the of disadvantage of correcting SPT values for the effect of overburden pressure for settlement calculations.

In chapter 3, the different available methods for selecting design N-value are presented. The difficulty of selecting a representative N-value for foundations design is shown. The most common SPT-based methods for estimating settlements of shallow foundations on sand are reviewed, with some discussion of the accuracy of each method. The importance of soil stress history and its effect on settlement prediction is demonstrated. The results of Standard Penetration Tests in several actual foundations with known soil stress history are analyzed. From these and other field data, a new approach for estimating settlements of shallow foundations on sand is proposed.

In chapter 4, a new method for selection of N-design value is developed. The first step in the development was to establish a function that contains the most significant variables affecting selection of N-design value. The second step was to define each variable by examining five hypotheses.

In chapter 5, the reliability of the proposed method is checked. Results of more than 85 SPT-based design case histories of actual structures were used for this purpose. The suggested method is shown to accurately simulate the experience and knowledge of foundation engineers as far as selection of design N-value is concerned. Four different settlement methods, presented in chapter 3, are used together with the newly proposed approach to estimate settlements of numerous case histories and results are compared with measured values. The suggested approach was shown to be more accurate in estimation of settlements.

In the final chapter, summary and conclusions are presented. Suggestions are also made concerning future work which appears warranted as a result of this study.

CHAPTER 2

FACTORS THAT INFLUENCE STANDARD PENETRATION VALUES

2.1 Factors Related To Test Procedures:

The standard penetration test has been used extensively in the United States and the world for estimating the relative density and the angle of shearing resistance of granular soils. A standard split spoon-sampler, about 1 3/8 in. inside diameter and 2 in. outside diameter Figure (1) is driven into the ground by blows from a 140 lb. drop hammer that falls freely for 30 in. The sampler is driven 6 inches into the soil at the bottom of a previously cleaned cased or mud-filled drill hole. The number of blows (N) required to drive the sampler a further 12 in. is then recorded. There are many variables involved in the preparation of the hole and in the performance of the test that can sometimes greatly influence the test results. There are also variations in the equipment and methods of presentation of the test results that can have important effects on the penetration values. These factors have been studied repeatedly (U.S.B.R., 1952; Palmer and Stuart, 1957; Fletcher, 1965; Kovacs et al.,1977; Kovacs and Saloman, 1982; Robertson and Campanella 1983; Skempton, 1986).

a) Ground water conditions: When the empty cased hole is extended below the ground water table, an upward intrusion of water will occur. This disturbance can affect the soil in the bottom of the hole, sometimes producing sand boiling (Terzaghi and Peck, 1948).

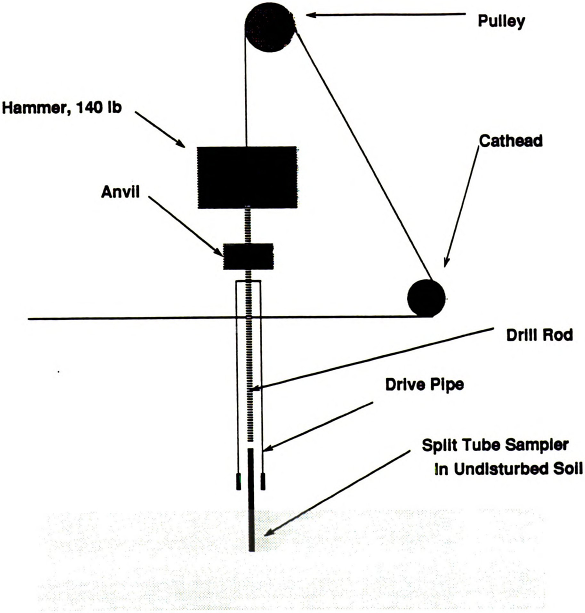


Figure 1. Schematic Diagram of Standard Penetration Test
with Split Spoon Sampler

Upward flow of water into the hole can transform the sand to much looser densities than those corresponding to the natural state. Consequently, the true penetration resistance of the soil may be considerably underestimated (Bazaraa, 1967; Schmertmann, 1978).

Sutherland (1963) referred to the results of standard penetration tests at a given site. The average penetration value, obtained from two borings where sand boiling occurred at the bottom of the holes, was found to be 22 to 29. Reference boreholes were made adjacent to the original borings and no sand boiling occurred. The average penetration value from these borings was found to vary between 64 and 94. This demonstrates the importance of preventing sand boiling at the bottom of the hole if a representative sand resistance is to be obtained.

The upward flow of water into the hole can usually be prevented by filling the bore hole with water to the level of the ground water table, thus equalizing the pressure.

b) Cleaning of the bore hole: Inadequate cleaning of the casing hole may cause sludge to be trapped and compressed in the spoon sampler. The penetration test may possibly have been started while the sampler is still above the bottom of the hole. This may greatly increase the number of blow counts needed for one foot of penetration because of the confining effect at the base of the casing hole.

c) Length of hammer drop: The amount of energy that is actually delivered to the drill rod varies with the fall height and number of rope turns around the cathead. It may be rather difficult to attain a free fall of exactly 30 in. if the slip-rope method is used for releasing the hammer. However, an experienced driller using a rope can generally attain a close approach to a 30 in. drop with a reasonable deviation of about 2 in. to 3 in. (Kovacs, 1982; Riggs et al., 1984; Riggs, 1986). A height of 0.8 m is typical for present practice in North America (Kovacs, 1979; Riggs, 1986).

d) Weight and length of the drill rods: Use of drill rods heavier than standard may underestimate the true penetration value of soil. Results of laboratory tests (U.S.B.R., 1952 and 1953; Gibbs and Holtz, 1957) appear to indicate that the apparent effect of a small increase in rod weight alone is not significant. Schmertmann and Palacios, (1979) found that the theoretical maximum ratio of rod energy decreases with decreasing rod length. However, Brown, 1977; Matsumoto and Matsubara (1982) stated that the weight of the rod stem of a given length appears to have little effect.

Fletcher (1965) indicates that the effect of rod length is not important up to depths of 120 ft. to 140 ft. At depths of 200 ft., the standard penetration values may be high and unreliable. Skempton (1986) proposed corrections for rod length based on thorough investigations of researchers in the field in different part of the world and this will be illustrated later in this chapter under energy heading.

e) Use of non-standard split spoon sampler: It is generally recognized that adherence to such details of the test as the weight and drop of the hammer and the diameter of the sampler is important. However, there are many recorded cases where the dynamic penetration test was performed with non-standard sampler. In these cases the sampler's length was arbitrarily changed from the standard value of 22 in. Such a change may seriously affect the recorded penetration values particularly if the length of the sampler used is less than 18 in. Similarly, if the diameter of the standard sampler is changed due to the presence of gravel in the soil, the recorded SPT values can be greatly overestimated. Correlations of this type were made between the results of the Mohr-Geotecnica sampler and the standard penetration values. The Mohr-Geotecnica sampler is 1 5/8 in. O.D. and 1 in. I.D. and is commonly used in Brazil where the penetration value is recorded as the number of blows for the first 30 cm. of penetration. These correlations (Vargas, 1961; De Mello et al., 1960) suggest that the standard penetration N value varies between 1.3 and 1.7 times the Brazilian penetration values, with an average given by:

N = 1.62 x Brazilian Penetration Value ------------- (2.1)

f) Effect of bore hole diameter: In the original procedure, the SPT was performed from the bottom of a 2.5 in. or 4 in. diameter wash boring. The best modern practice still utilizes these dimensions (Skempton,1986). The effect of testing from relatively large bore holes in cohesive soils is minimal. However, in sandy soil the effect of bore hole diameter can be significant. Bore hole diameter has an effect on the stress relief at a depth that the SPT performed. Consequently, a lower N value may be obtained from a relatively large bore holes diameter (Lake, 1974; Sanglerat and Sanglerat, 1982). Bore size correction factors, (Skempton, 1986), based on recent field investigation, have been recommended Table (2.1).

Table 2.1. Recommended correction factors to measured N-values (Skempton, 1986)

Borehole diameter (cm)	Corrections
6.5 - 11.5	1.0
15	1.05
20	1.15

g) <u>The depth range of measured SPT resistance</u>: The standard test procedure involves striking the top of the drill rod with a 140 lb hammer until the sampler penetrates about 6 inches. The sampler is then driven a further 12 inches, and the number of required blows is recorded (Terzaghi and Peck, 1948). The blow count of the first 6-inch represents the penetration resistance of disturbed soil at the base of the hole. The penetration value generally increases in the second and third 6-inch intervals. The results of dynamic penetration tests in Brazil (De Mello et al., 1960) show that the penetration characteristics are continuous over the driven length of sampler, and that the ratios of the number of blows in the first, second, and third 6-inch increments to the total number of blows in the three increments are ideally 0.28, 0.33, and 0.39 respectively. This typically indicates that the sum of the blows in the last two 6-inch increments is somewhat higher than the sum of the blows in the first two 6-inch increments.

2.2 Soil Grain Properties:

The size of soil particles and the distribution of sizes throughout the soil have important effects on SPT resistance (Gibbs and Holtz, 1957). Holubeck and D'Applonia (1972) suggest that the SPT is influenced by the shape and size of particles of granular soil. Soil grains with rough surfaces yield higher penetration resistance than soils with round and smooth grains. Burmister (1962a) presented curves that show a relationship between relative density and standard penetration values. These curves were constructed by direct relative density determinations on relatively undisturbed granular samples. For a given relative density, the curves indicate an increase in the penetration value with an increase in roughness. However, D'Applonia and D'Applonia (1970) concluded that when gravel sizes are not present, the particle size does not seem to have a significant effect on SPT values.

2.3 Submergence:

a) Fine to coarse sand and gravel: Terzaghi and Peck (1948) stated that, in any sand with an intermediate grain size, the penetration value is not significantly different above and below the water table provided that the relative density is the same. This was confirmed by a study of 231 penetration tests in gravels and fine to coarse sands (Schultze and Menzenbach, 1961). The results of their study showed that when the water table was reached, the standard penetration value was reduced by about 16 percent. A similar study was made by Bazaraa (1967) for the standard penetration values of fine to coarse sands and gravels at 11 different sites. Borings that were included in his study were those in which the same soil extended above and below water table.

The results confirm previous suggestions that the submergence of fine to coarse sands and gravels does not have a significant influence on the penetration values.

b) Very fine or silty sands. In very fine or silty sands that have a diameter of 0.1 to 0.05 mm., the effect of submergence on standard penetration values may be significant. Fine sands and silty sands have a rather low permeability, and excess pore pressures might develop in soil under rapid application of dynamic load from standard penetration test (Terzaghi and Peck, 1948). These positive pore water pressures would reduce the soil shearing strength that opposes the penetration of the sampler. Hence the standard penetration value of submerged loose soils decreases. On the other hand, it was suggested that for dense, very fine or silty submerged sand, the penetration test might produce negative pore water pressures that would increase resistance to the sampler, consequently increasing the penetration value. Terzaghi and Peck (1948) considered this effect and suggested that, for very fine or silty submerged sand with a standard penetration value N' greater than 15, the relative density would be nearly equal to that of a dry sand with a standard penetration value N where:

$$N = 15 + 1/2\,(N' - 15) \qquad \text{------------------} \qquad (2.2)$$

Gibbs and Holtz (1957) and Schultz and Melzer (1965) investigate the effect of submergence on the results of dynamic penetration tests in very fine sands. The results of their investigations indicated that submergence caused a noticeable decrease in penetration values. Their findings contradict the suggestions of Terzaghi and Peck (1948). However, Bazaraa (1967) found, from analysis of a large number of SPT results in fine and silty sand within 3 feet above and below water table, that the effect of submergence is generally to increase SPT resistance values. Based on the results of his study it was found that the measured N-values in dense fine and silty sand should be corrected by the formula:

$$N' = 0.6 * N \quad \text{---------------------} \quad (2.3)$$

However, this formula was not recommended in lieu of Terzaghi and Peck's (1948) suggestion. Equation 2.2 is recently recommended by Burland and Burbidge (1985). The results of above-mentioned studies showed a considerable scatter in data which might account for different conclusions concerning the effect of submergence on SPT values.

2.4 Relative Density of Sand

The degree of sand compactness, as suggested by field data, may be one of the primary factors that affects the standard penetration value. It is generally assumed that for very loose sands, the SPT-values will be low whereas for very dense sands, the SPT-values will be high. Terzaghi and Peck (1948) introduced the following correlation between the standard penetration values and the relative density of the sand.

N	Relative Density
0-4	Very loose
4-10	Loose
10-30	Medium
30-50	Dense
Over 50	Very dense

In this correlation, it should be noted that the various ranges of relative density were expressed in descriptive terms rather than numerical values. Burmister (1948) realized that assigning numerical values to the relative density was both important and practical. He consequently defined the following various ranges of sand compactness in terms of numerical values of relative density;

Compactness	Relative Density (%)
Loose	Below 40
Medium	40-70
Compact	70-90
Very compact	Over 90

U.S.B.R. (1952) introduced another arbitrary definition of the ranges of compactness of sand. Their correlation is as follows:

Compactness	Relative Density (%)
Very loose	Below 15
Loose	15-35
Medium	35-65
Dense	65-85
Very dense	Over 85

Meyerhof (1956) presented a somewhat different definition of the ranges
of sand compactness. His suggestion is as follows:

Compactness	Relative Density (%)
Very loose	Below 20
Loose	20-40
Medium	40-60
Dense	60-80
Very dense	Over 80

Skempton (1986) suggested that the classification of relative density should be done based on SPT values that are corrected for the effect of energy and the correlation between SPT results that are normalized to energy ratio (ER%) of 60 and relative density may be represented as :

N60	Relative Density	Equivalent Numerical value
0-3	Very loose	0-15 (%)
3-8	Loose	15-35
8-25	Medium	35-65
25-42	Dense	65-85
Over 42	Very dense	Over 85

This correlation is the same as Terzaghi and Peck's correlation except that N-values are normalized with respect to standard rod energy ratio.

The values in these correlations show clearly that, for the same value of the relative density, the N-value can vary within wide limits. Some of the important possible sources for this variation are the grain properties, aging, overconsolidation and energy.

2.5 Overburden Pressure:

Gibbs and Holtz (1957) performed the first controlled laboratory tests which showed that standard penetration tests performed in sands are greatly influenced by overburden pressure. An increase in vertical pressure from 0 to 40 psi caused a significant increase in standard penetration test values. Since then many investigators, (Mansur and Kaufman (1958); Schultz and Menzenbach (1961); Philcox (1962); Zolkov and Wiseman (1965); Schultze and Melzer (1965); Bazaraa 1967; Bieganousky and Marcuson (1976); and Bieganousky and Marcuson,1977), have shown that the penetration resistance N depends not only on relative density, D_R , but also on overburden pressure. This is reasonable since sand strength is mainly from internal friction, which by definition is strength that is greatly affected by confining pressure.

A. Methods for Correcting Standard Penetration Values for the Effect of Overburden Pressure:

The need for correcting the SPT resistance values to account for effect of overburden pressure originated with Gibbs and Holtz (1957). Since then different correction factors have been suggested to account for overburden pressure. The following is a summary of these methods:

1. Teng Method (1964):

$$N_n = N * \left(\frac{50}{10 + P_v} \right) \qquad \text{-----------------} \qquad (2.4)$$

This method is frequently referred to as Gibbs and Holtz (1956) method because it based on their experiment data. Later many investigators have shown that the field data suggest that this equation may be unconservative (Peck and Bazaraa 1969), because its reference stress level is too high, approximately 40 psi. Correction of SPT values, therefore, for tests performed under 40 psi will result in a higher N-values than the ones actually recorded.

2. Bazaraa Method (1967):

Results of his study indicated that at overburden pressure of 40 psi the standard penetration value can be even as high as twice the value indicated by Gibson and Holtz (1957). A correction factor, therefore referenced at 40 psi will overestimate the compactness of the soil deposits. Two correction factors were then suggested as follows :

$$C_n = \frac{4}{1 + 2P_v} \quad \text{where ; } P_v \leq 1.5 \text{ (ksf)} \quad \text{---------} \quad (2.5)$$

$$C_n = \frac{4}{3.25 + 2P_v} \quad \text{where ; } P_v \geq 1.5 \text{ (ksf)} \quad \text{---------} \quad (2.6)$$

3. Peck, Hansen, and Thornburn Method (1974):

Their method is based on the results of Bazaraa (1967) with slight modification that led to somewhat less conservative correction value specially at high overburden pressure and it was presented in the following form:

$$C_n = 0.77 \text{ Log } \frac{20}{P_v} \quad \text{where ; } P_v \text{ in tsf} \quad \text{---------} \quad (2.7)$$

4. Seed Method (1976 and 1979):

In 1976 a very conservative correction method in the following form was presented by Seed :

$$C_n = 1 - 0.25 \log_{10}(Pv) \quad \text{--------------} \quad (2.8)$$

where ; Pv in tsf. Later it was revised based on the experimental data from Marcuson and Bieganousky (1977a ,1977b) in which they showed that the penetration test resistance value is not only a function of overburden pressure but relative density (Dr) as well; consequently two ranges of relative density namely 40-60 % and 60-80% were used (Seed,1979) to construct a correction factor that takes in consideration the effect of both stress level and relative density on the SPT value.

5. Tokimatsu and Yoshimi Method (1983):

The correction factor by this method is somewhat conservative especially at high stress level comparing to the preceding ones. Tokimatsu and Yoshima stated that this is justified because of a small amount of energy that reaches SPT sampler for longer rod length at high depths corresponding to high values of stress level. Their correction factor is given by:

$$C_n = \left(\frac{1.7}{0.7 + P_v} \right) \text{ where ; } P_v \text{ (tsf)} \quad \text{-------------} \quad (2.9)$$

6. Liao and Whitman Method (1986):

Based on a review of previous studies, Liao and Whitman suggested the following method:

$$C_n = \frac{1}{(Pv)^n} \qquad \text{(2.10)}$$

where n is a function of relative density, soil gradation, soil type and more importantly stress history. Jamiolkowski et al. (1985b) found that parameter n equal 0.56 for SPT and 0.72 for cone penetration test for normally consolidated sand. Liao and Whitman (1986) suggested n = 0.5 for the standard penetration test. This is a very simple correlation that is consistent with field data.

B. Should SPT values be normalized for overburden pressure?:

It is not clear how standard penetration test results in general can best be normalized, because in theory any normalization to account for overburden pressure should include the effect of lateral pressure or the coefficient of earth pressure at rest K_o.

In recent years a number of investigators have found (Baldi et al., 1981; Jamiolkowski et al.,1985b; Tokimatsu, 1988) that any increase in lateral earth pressure caused a substantial increase in the recorded SPT values. The results of control chamber test (CC) show that all kinds of penetration resistance are more sensitive to lateral stress than to the overburden pressure (Clayton et al., 1985). From actual measurements of mean effective stress and corresponding penetration test values, Hababa (1984) constructed curves for the correction of penetration resistance values Figure (2). The effect of horizontal stress increase due to K_o- overconsoldation was also included.

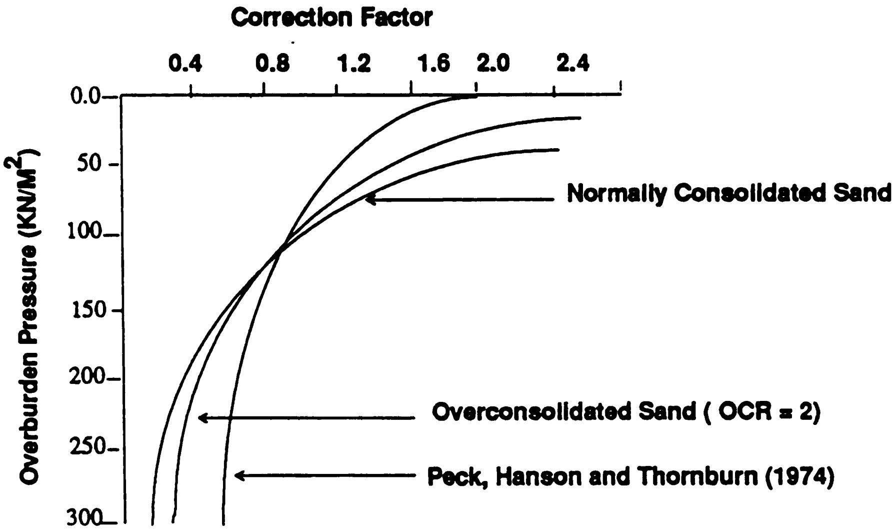


Figure 2. Effect Of Overburden Pressure on Penetration Resistance (after, Hababa (1984).

Figure (2) shows that Hababa correction curves might be more meaningful than the previous methods since they properly account for effect of pressure on the SPT results. This suggests that the effect of horizontal effective stress level cannot reasonably be neglected and that the mean effective stress on the soil deposits may have a rather important effect on the SPT values.

However, it should be emphasized that the previous correction factors are intended primarily to improve settlement prediction of shallow foundation on sand, but as can be seen from Figure (2) that correcting for the effect of the mean effective stress will lead to highly overconservative settlement prediction. Laboratory tests by Daramola (1978) as reported by Burland and Burbidge (1987) indicated that, for a given K_o stress history, the two most important factors influencing the sand compressibility are relative density and stress level. Since these same factors influence the SPT values, it appears eliminating the effect of overburden pressure has an adverse effect on settlement prediction. For example, it has been repeatedly reported that the sand compressibility is linearly related to the square root of vertical effective stress ($E = \alpha$ (Pv)), then it is obvious that eliminating overburden pressure will be of little use in settlement computations. No correction for the overburden pressure, therefore, will be considered in this study.

2.6 Energy Effect:

a) Data Regarding The Effect Of Method Of Releasing Hammer On SPT Resistance Value:

Generally there are four methods of releasing the hammer in performing SPT (Skempton 1986):

1. A trigger method, such as the Japanese Tombi.
2. A trip hammer method, such as a pilcon or dando hammer.
3. Manual method and release of the rope passing over the crown sheave of the drilling rig.
4. The slip-rope method. It is usual to have two turns of rope on the cathead.

b) The number of turns of the rope around the cathead

Kovacs (et. al 1981) have shown that the energy delivered to the rods during a SPT can vary from about 30%-80% of the theoretical maximum. When using the rope and cathead procedures with two turns of the rope the typical energy delivered from a standard donut type hammer is about 50%-60% of the theoretical maximum (Kovacs and Salomone 1982). Schmertmann (1976) has suggested that based on limited data an efficiency of about 55% may be the norm for which it can be assumed that many North America correlations were delivered.

c) Data Regarding The Effect Of Methods Of Releasing Hammer On Velocity Energy Ratio; And Consequently On SPT Resistance Value:

Kovacs (1979) measured the impact velocity of a Borros trip hammer. The results showed that the velocity energy ratio ER_v is about 0.99. This indicates that such a mechanism of releasing the hammer is nearly a free fall. In methods 3 and 4 there is some retardation even though the rope appears to be completely freed.

Frydman (1970) reported the results of alternate tests in drilled holes, carried out either with trigger release or by the two-turn slip-rope method using the same hammer. His data showed that ER_v was about 0.77. Similar tests with a trip release or with a two-turn slip-rope in a sand fill at San Diego showed that ER_v is about 0.66, Douglas (1982). Diameters of the cathead in both cases were probably about 8 in., a typical dimension in America and America-influenced practice (Skempton 1986).

In Japan at Niigata, a thorough investigation was carried out to compare a donut hammer released by the Tombi trigger and by a two-turn slip-rope (Yoshimi and Tokimatsu, 1983). The results showed that the ER_v was on the average, for both hammers was about 0.83. Similar value was obtained by Ohoka (1984). The higher velocity ratio of the Japanese slip-rope method, compared with the America method is reasoned by the smaller cathead diameter (5 in.) used in the Niigata tests and partly by the thinner manila rope (12-17 mm.) diameter compared with (19-25 mm.) in America (Skempton 1986).

In China, Shi-Ming (1982) reported a value of ER_v as high as 0.87 from the results of comparing the traditional Chinese manual operation of donut hammers with trip release hammers of the same form. The value is in agreement to what was reported by Seed et al. (1984) when they performed some tests in Japan in which the rope was thrown sideways completely off the cathead. Kovacs et al. (1977) and Kovacs and Salomone (1982) performed comparative tests with trigger release and the America two-turn slip-rope. Their results indicate that the ER_v was around 0.71. Skempton (1986) stated that from experience at Kalabagh, there was no difference in average N-values obtained by manual release and one turn of rope on a small (80 mm) cathead.

d) Data Related To The Effect Of Rod Energy Ratio On SPT Resistance Value

Schmertmann and Palacios (1979) used dynamic load cells which were inserted in the rod stem to determine the energy delivered to the rods and consequently the rod energy ratio. Their results showed that the blow count in a given sand is inversely proportional to rod energy ratio ER. In 5-6 tests in adjacent borings at a depth between 10 ft. and 30 ft., the average N-values results for two different hammers and drill rods were in a close agreement when the rod energy ratios were taken into account. For example the average N-value for S-hammer AW rods was 8.8 with ER about 0.52, while the average N-value for F-hammer N rods was 14.0 with ER about 0.31. Hence, if the average N-value of each hammer is multiplied by its corresponding ER, the results would be almost similar. The effect of rod energy ratio on the SPT resistance value for two different hammers (donut and safety) was reported in alternate tests in a site on Tibury Island in Canada (Robertson et al., 1983). The standard penetration resistance values were made in the same borehole, using the same rig and a two-turn slip-rope method. Considerable variation in measured SPT N-values were observed using two different hammers. The average overall energy ratio levels for the hammers were 43% and 62% for the donut and safety hammers, respectively. When both hammer N-values are corrected to an energy level chosen as 55%, the apparent variation in SPT is decreased and a consistent pattern of SPT values was observed. The error in energy measurement was $\pm$ 5. This means that different hammers have different efficiency and it is important,therefore, to normalize the recorded standard penetration values to a common energy level.

e) <u>Drill Rod Length Effect On The Efficiency Of The SPT Hammer</u>:

Fairhurst (1961) showed theoretically what ideally happens when the hammer impacts a rod stem of infinite length. A compression wave travels with the same velocity both down the rod and up the hammer. Due to the short length of the hammer the compression wave reaches the end of hammer rather quickly. At the end of hammer it returns as a tension wave which cancels out the upcoming compression wave. This cycle continues in progressively reduced level of energy that was originally developed at the time of impact. This theory was adopted by Schmertmann and Palacios (1979) in their research of the effect of energy on the measured N-values with one modification that involved a finite rod length stem. The compression wave that is generated during impact travels a distance equivalent to the length of rod (L) and it reflects at the end of rod and travels for the same distance as a tension wave. The time required for this process is equal to 2L/C in which C is the theoretical sound velocity of the wave in the rod (5120 m/s). As the rod length (L) increases, the time required for tension cut off of the hammer energy input to the rod increases and consequently more energy will transfers to the rod. On the other hand, the shorter the rod length (L) the shorter the time required for tension wave to reach the contact point of the hammer and rod. Consequently, this tension wave will cause losses in energy from that part of compression wave of the hammer that would otherwise transfers to the rod between time 2L/C and time equales infinity. Schmertmann and Palacios (1979) measured the time required for a tension wave to reach the end of the rod by load cells which were placed at a distance L from the top of the rod. Such a reduction in the rod length will have effect on the time travel for a tension wave and consequently on the hammer efficiency. Correction factors, therefore, were suggested for such an adjustment. However, Seed et al.,(1985) suggested somewhat less conservative values as shown in Table (2.2).

Table 2.2. Recommended corrections for effect of rod length on hammer efficiency.

Rod length (m)	Seed et al. (1985)	Schmertmann and Palacios (1979)
3	0.75	0.69
3-4	1.0	0.850
4-6	1.0	0.893
6-10	1.0	0.988
Over 10	1.0	1.00

Robertson et al. 1983, on the other hand, did not recommend for any corection regarding the effect of rod length because they are already reflected in the measured values of rod energy ratio. If such an effect is taken in consideration, then the suggested values by Seed et al. will be adopted since Schmertmann an Palacios, (1979) assumed that the hammer rod impedance ratio (r) to be zero as the ideal case in wave theory as a basis for their correction which in practice the value of (r) cannot be zero.

f) <u>Standard Rod Energy Ratio</u>

SPT energy measurements that have been done either in laboratory or field were obtained by instrument system that consisted of load cells to measure the stress wave generated from hammer impact. A measure of kinetic energy in the drill rod after impact was obtained from the force-time equation that is used in the one dimensional wave-equation. The general form of this equation is:

$$Er = K \int F(t)^2 \, dt \quad \text{------------------} \quad (2.11)$$

where, Er is the energy in the drill rod, K is constant and F(t) is wave force as a function of time in the drill rod. The integration process is done automatically by a digital processing Oscillocope. The energy in the drill rod then divided by the theoretical maximum energy of (4200 lb-in.), and the result is expressed as the ERr% (the rod energy ratio). The maximum theoretical energy (4200 lb -in) is obtained based on the kinetic energy equation:

$$E = 1/2\ (M\ V^2) \quad \text{---------------} \quad (2.12)$$

Where M is the mass of the hammer 140 lb, and V is the velocity of free fall hammer $(2gh)^{0.5}$ for a distance (h) of 30 inches. The ratio of ERr% depends on the type of hammer and mechanism of release. The most widely used hammers in U.S.A are the donut hammer and safety hammer which were used by researchers in the field experiments. The results of rod energy ratio (ERr%) from field data of those previous mentioned studies clearly showed that in all cases the donut hammer tends to be less efficient than Safety hammer; consequently the measured N values that obtained by this type of hammer are higher than the values obtained by Safety hammer even though both were used alternatively in the same soil with same releasing mechanism (i.e., two wraps of ropes around the cathead). Under recent practice and test conditions, significant variations in N values would be expected when different drill rig systems are used at the same site (Kovacs et al. 1982). To eliminate the variabilties between different tests the same authors suggested the development of a National Average Energy (NAE). Once established the NAE could be used for standardization of SPT or correcting blow counts data, or both for a common energy. Schmertmann et al. 1979 has suggested an efficiency of 55% may be used to normalize the rod energy ratio (ERr%)) for hammers that have been used in SPT in North America. Similar value is suggested by Robertson et al. 1983. On the other hand, Seed,1983, Seed et al.,1985 and Skempton (1986) suggested somewhat a conservative value of 60% to be used for normalization.

The question is what value of rod energy ratio (ERr) for both, Donut hammer and Safety hammer be normalized to a common energy? and then what is the value of a common energy, 55% or 60%?

Any normalization of ERr should be done in a way that be comparable with the past in order not to obviate all existing empirical correlations with SPT value (Kovacs et al. 1982). The typical field value of ERr for both Donut hammer and Safety hammer is 45 and 55 respectively. If these values are normalized at 60%, then the following expressions would be used for normalization of each hammer:

A. For Safety hammer

$$N_{norm} = N * \frac{ERr}{60} = N * \frac{55}{60} = N * 0.92 \quad \text{---------} \quad (2.13)$$

B. For Donut hammer

$$N_{norm} = N * \frac{ERr}{60} = N * \frac{45}{60} = N * 0.75 \quad \text{---------} \quad (2.14)$$

where, N_{norm} is the normalized N at 60%. However, if the N value is normalized at 55% then the following expression would be used:

A. For Safety hammer

$$N_{norm} = N * \frac{ERr}{55} = N * \frac{55}{55} = N * 1.0 \quad \text{---------} \quad (2.15)$$

B. For Donut hammer

$$N_{norm} = N * \frac{ERr}{55} = N * \frac{45}{55} = N * 0.82 \quad \text{---------} \quad (2.16)$$

The value of 60 % to be a common factor for normalization seems to be more reasonable than the value of 55% for the following reason; All researchers indicated that the hammer assembly such as the anvil size may be the primary cause of the energy differences. Kovacs et al. (1982) stated that such an observation requires experimental verification. Similarly, Skempton (1986) emphasized the importance of anvil size on the hammer efficiency and he did also call for verification of such an effect on the variation of energy between both types of hammers.

g) Effect of Anvil:

For American donut hammer, the anvil weight is 26 lb, while for safety hammer, the Anvil weight is 2.5-3 lb. Each hammer has 140 lb weight falling for a distance of 30 inch. For free fall of hammer, the equation of energy can be expressed as:

$$E = 1/2\ (M\ V^2) \qquad \text{-----------------} \qquad (2.17)$$

The velocity V of the hammer before impact on the anvil is:

$$V = (2gh)^{0.5} \quad \text{-----------------} \quad (2.18)$$

where, g, is the acceleration due to gravity and, h, is height of falling hammer.

Substituting equation (2.18) into equation (2.17), we get,

$$E = 1/2\ M\ (2gh) = 1/2\ w/g\ (2gh) = wh \quad \text{------------------} \quad (2.19)$$

where w is 140 lb and h is 30 in.

Equation (2.19) is the energy before impact.

The velocity of the hammer and the anvil at the impact is:

$$V = Mh\ /\ (Ma + Mh) * (2gh)^{0.5} \quad \text{--------------------} \quad (2.20)$$

where Mh is a mass of the hammer and Ma is a mass of the anvil.

The energy at impact, therefore, is:

$$E = 1/2\ (Wh/g)\ [\ (Wh/g)\ /\ (Wa/g + Wh/g)^2\ ] * 2gh \quad \text{-----------} \quad (2.21)$$

simplifying this equation, the energy E is:

$$E = Wh\ (Wh\ /\ Wa + Wh)^2 * h \quad \text{----------------} \quad (2.22)$$

Equation (2.22) is the energy after impact while equation (2.19) is the energy before impact. Using both equations, the efficiency of each hammer can be determined as follows:

For donut hammer, $W_h = 140$ lb, $W_a = 26$ lb and $h = 30$ in. Substituting these values in both equation, the efficiency of donut hammer is:

Efficiency = equation (2.22)/ equation (2.19) = 0.71

For safety hammer, $W_h = 140$ lb, $W_a = 6.6$ lb and $h = 30$ in. Substituting these values in both equations, the efficiency of safety hammer is:

Efficiency = equation (2.22) / equation (2.19) = 0.91

If the mass of the anvil is negligible then the efficiency of the hammer is 1.

Comparison between efficiency of both hammers that was suggested by different researchers and the efficiency by theoretical solution;

Table 2.3. Recommended rod energy ratios for two types of hammer with the two-turn slip-rope method

References **	Suggested experimental value
Schmertmann et al. (1979)	0.82 (Donut hammer) 1.0 (Safety hammer)
Kovacs et al. (1983)	0.82 (Donut hammer) 1.0 (Safety hammer)
Robertson et al. (1983)	0.82 (Donut hammer) 1.0 (Safety hammer)
Seed (1983)	0.75 (Donut hammer) 0.92 (Safety hammer)
Skempton (1986)	0.75 (Donut hammer) 0.90 (Safety hammer)
Theoretical value based on this analysis	0.71 (Donut hammer) 0.91 (Safety hammer)

**The results are based on the assumption that the typical rod energy ratio for donut and safety hammers are 45 and 55 respectively. These two values are recommended by Skempton (1986).

The theoretical solution shows that the anvil size, indeed, has pronounced effect on the efficiency of the hammer. The standardization of the energy with respect to 60% appears to be more reasonable that 55%.

CHAPTER 3

SPT-BASED DESIGN OF SHALLOW FOUNDATIONS ON SAND

3.1 Introduction

Foundation engineers are concerned with two main properties of soil: shear strength, and compressibility. The shear strength of a soil is defined as the maximum or ultimate shear stress the soil can sustain. It is considered to be one of the most important engineering properties, since most foundation and earth work failures result from excessively large shear stresses. Therefore, a measure of soil strength is needed to ensure that there is an adequate factor of safety against failure. A measure of compressibility is also needed to ensure that the deformations (settlements) under working conditions are not so great as to damage the structure or cause it to become unserviceable. It should be noted that settlement occurs under working conditions, when the stresses in the ground may be well below those which would cause failure of the soil. Design under working conditions would thus depend on the soil settlement (Terzaghi 1948; Dann 1980; Milligan and Houlsby 1984). This is particularly true for sandy soils which have good bearing capacity. Hence, in sandy soils, settlement rather than bearing capacity usually controls the design of shallow foundations. The determination of shear strength and settlement of sands, when used as foundation materials, depend on the measurement of angle of internal friction (ϕ) and the modulus (E) of soil respectively.

The later two soil parameters (ϕ and E) can be estimated using both laboratory and field tests. Laboratory tests are performed on representative soil samples and must be done in a way that simulates the conditions that will exist in field. Considering the great difficulties that are associated with recovering and preparing undisturbed samples of sandy soils, it has become a common practice to use in-situ tests for estimating both parameters. The most commonly used in-situ field test is the Standard Penetration Test (**SPT**) where the sampling procedure was standardized by (ASTM,1967). Numerous correlations between results of SPT and both ϕ and E have been developed either in the form of families of curves or in the form of empirical equations that are of great importance for foundation design engineers. Thus, it will be found that the value selected for N often has a crucial effect on the outcome of foundation design calculations. Moreover, it will be shown that the available recommended procedures that are being used for selecting a design N-value for the purpose of estimating either ϕ-value or E-value are not always consistent, and in many cases lead to very conservative design. The procedures for selection of N-value have been criticized before (Wu, 1967; DeMello, 1971) but no comprehensive attempt appears to have been made to evaluate the validity of such procedures; and a more appropriate criterion should be developed to serve design engineers to come up with a more rational decision about N-value for more efficient and economic designs.

3.2 <u>Design Requirements of Shallow Foundation on Cohesionless Soils</u>:

a) <u>Bearing Capacity</u>:

The ultimate load that may be applied to footings resting on or near the surface of ground may be determined by considering independently the contributions to bearing capacity of surcharge pressure on the soil surface and the weight of the soil. Based on conclusions reached by Terzaghi the total ultimate pressure could be obtained to a good approximation by adding the two contributions to give:

$$q_u = \gamma D N_q + 0.5 \delta N_\gamma$$

in which γD is the surcharge at the level of the base of footing due to the depth embedment of the footing. N_q and N_γ are bearing capacity factors. The last two factors are functions of the angle of soil internal friction (ϕ). The latter can be related to standard penetration test results and is often selected from a published correlation between N and ϕ. There are different such correlations, however, the most widely used is the correlation by Peck, Hanson and Thornburn (1974) and this was shown in the recent study of geotechnical judgment in foundation design Wolff (1988,1989).

b) Soil Settlements :

Accurate estimate of soil settlement is necessary if the foundation engineer is to design a foundation that will efficiently meet the allowable settlement requirements of the superstructure. The prediction of settlements of shallow foundations on cohesionless soil involves the use of an empirical equation in many cases. The empirical equations are mainly developed from the observational field data between settlements and soil in-situ tests. Of those in-situ tests, the Standard Penetration Test is still the most commonly used in soil investigation. At the present time there are more than 15 SPT-based settlement methods available to foundation engineers. Such a large number of methods leads to the suggestion that there is a poor agreement between SPT and soil compressibility. However, it will be shown later that the empirical equations might be unsatisfactory and this might be one of the reasons for the poor prediction of actual settlements of foundations on sands.

3.3 Moduli of Soil Deformation for Sand:

The compressibility or stiffness of sand can be described by various parameters that include Young's modulus (E), shear modulus (G) and constrained modulus (M). All these moduli represent stress per unit strain. In evaluation of the compressibility of sand by plate loading tests, the modulus of subgrade reaction (K) is sometimes used. It is the ratio of the pressure on the plate to the corresponding settlement. The constrained modulus is usually determined from oedometer test in the laboratory. However, the values of both M and E are usually inferred from the results of in-situ tests such as cone penetration tests and standard penetration tests. Over a past 20 years there have been numerous correlations developed for estimating modulus of soil elasticity (E) from results of soil in-situ tests. The following are a number of such correlations:

a) D'Applonia et al. Method (1970):

D'Appolonia et.al (1970) established a relationship between modulus of elasticity (E) and the averages of standard penetration test results for normally loaded sands from seven case histories. The relation is as follows:

$$E = 432 + 21 (N) \quad \text{in ksf} \qquad (3.1)$$

b) 3.2 Parry Method (1971):

A series of plate bearing tests were conducted on sands (1971) in an attempt to develop a relation between E and N-values. In the vicinity of the tests a series of standard penetration tests were conducted and the following relationship was established:

E = 100 (N) in ksf ------------- (3.2)

c) Yoshida and Yoshinaka Method (1972):

In 1972, both authors published a correlation between E and N-values based on plate load tests and lateral pile load tests. The correlation is as follows:

E = 42 (N) in ksf ------------- (3.3)

d) Schmertmann Method (1970,1978):

A series of plate load tests toghether with cone penetration tests were conducted on soil that consisted mostly from fine sand. The results of these two in-situ tests were correlated through the equation E = 2qc where E is soil modulus of elasticity that was determined from results of plate load and qc is the the static cone bearing capacity from the results of cone penetration tests. This correlation was then extended to the results of standard penetration tests in the form of the following equation:

E = 8 (N) to 20 (N) in ksf ------------- (3.4)

There was a wide variation in the test results and the correlation is conservative as it was indicated by the author.

e) Bowles Method (1987):

Bowles (1987) established a correlation between E and standard penetration test results that takes the following form;

$$\mathbf{E = 10\ (\ N + 15\ )\quad in\ ksf} \qquad \text{---------------} \qquad (3.5)$$

It is shown that the interpretation of N-values has a major impact on outcomes of a soil modulus of elasticity and consequently on the predicted settlement of soil foundation if the elastic method or any other method that utilize this soil parameter is used in the settlement calculations. The results of the forementioned correlations combined with the findings of other investigators are plotted and presented in Figure (3). It is shown that there is no unique relationship between E and SPT results. A foundation engineer, therefore, should be extremely cautious when selecting E is based on any of these methods for foundation settlement prediction. Moreover, all these correlation were developed from highly scattered data, and the scatter of test results was one of the prime reasons that led to different interpretations.

There are many factors influencing soil compressibility such as relative density, grain properties, stress level, fabric bond, applied state of stress, and stress history. The latter is considered to be the most important factor influencing the soil compressibility (Schmertmann 1974; Leonards and Frost, 1987). The relations between SPT results and E, therefore, will remain poor until stress history can be known with some confidence.

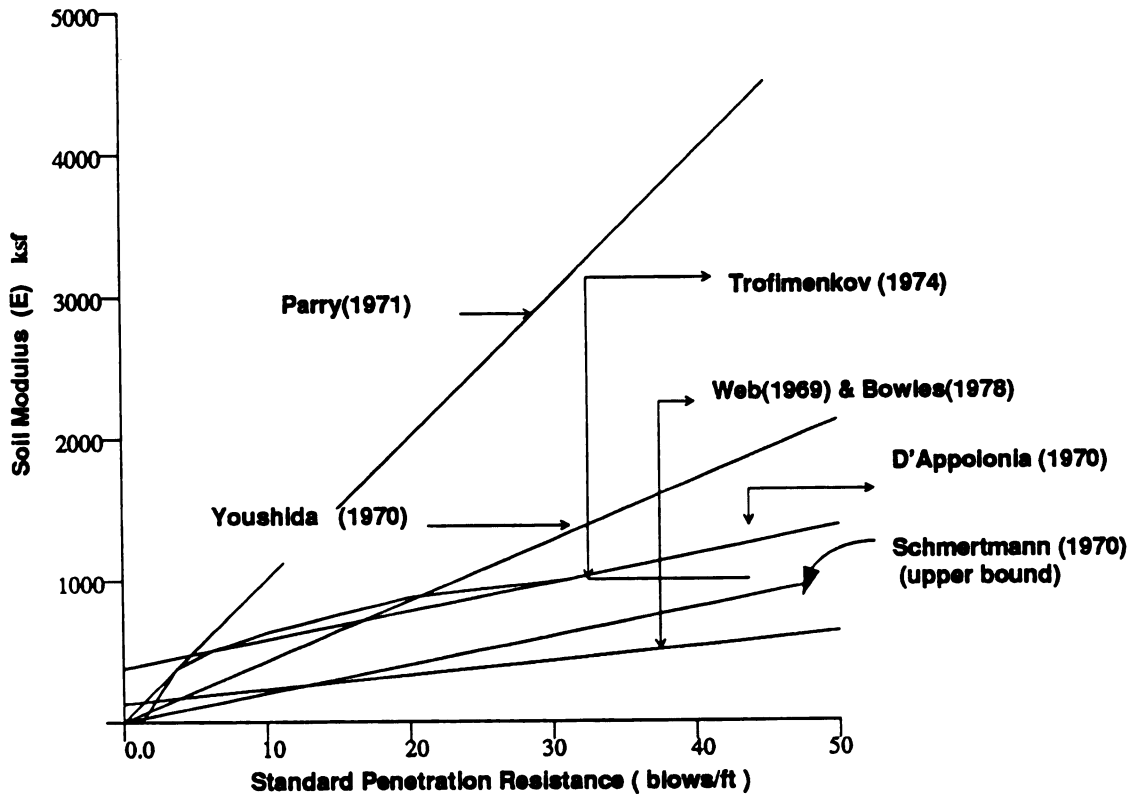


Figure 3 . Relationship between Modulus of Elasticity and Standard Penetration Value for Different Methods

In general, the compressibility of sand is not strongly related to SPT results. The latter depends on current effective stress level, but compressibility is strongly related to soil stress history and can be significantly affected by minor changes in stress history (Clayton, et al., 1985). Clayton et al.,(1988) showed that the relationship between SPT and soil compressibility can assume different forms, as shown in Figure (4), so that soils with the same penetration resistance cannot

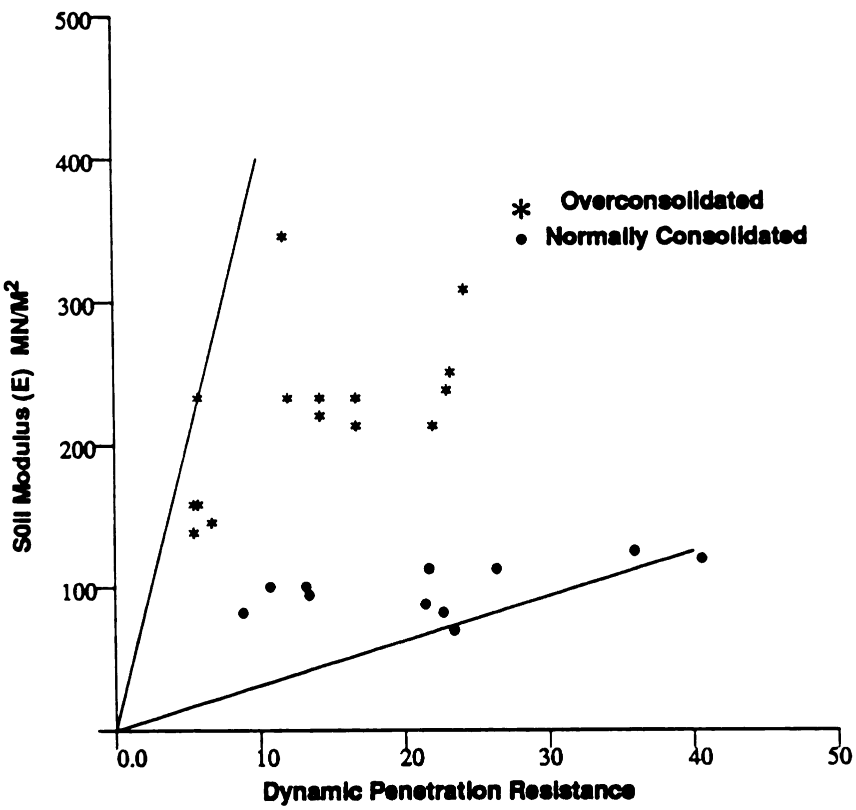


Figure 4. Relationship between Tangent Modulus and Dynamic Penetration Values for Normally consolidated and Overconsolidated Sands (After, Clayton et al. 1988)

be expected to have the same compressibility. Normally consolidated soil, for example, shows slight variation of tangent modulus but very wide variation in penetration resistance. This suggests that even a modest variation in stress history might cause a significant change in the E value. Any correlation, therefore, between E and SPT should be treated with considerable caution (Wroth 1988). This emphases the approximate nature and limitation of the SPT when related to E. Such a poor correlation no doubt will limit the capability of those settlement equations that utilize soil modulus (E) for predictions of foundation settlements.

3.4 Available Procedures for Estimating Settlements of Shallow Foundations on Sand:

There are several empirical equations in foundation engineering, however, the most commonly used are those summerized below:

a) Terzaghi and Peck Method (1948):

Based on load displacement relationships that were obtained by load testing a one ft. square plate on the surface of a sand layer , Terzaghi and Peck (1948) developed the following empirical expression for settlement ratio ;

$$\frac{Sb}{S1} = \frac{(\ 2 * B\ )^2}{B + 1} \qquad \text{----------------} \quad (3.6)$$

where; S1 is the settlement of a standard plate 1 ft in width.

Sb is the settlement of a foundation with the same bearing pressure.

B is the foundation width in feet.

based upon the results of field plate load tests, field penetration tests (N-values) and observations of field behavior of footings, a correlation was obtained, (according to Parry,1978), between test plate settlement Sb and SPT values N,

$$Sb = \frac{3P}{N}$$

where P is the applied bearing pressure (t/ft^2) and Sb is the settlement in inches. The settlement of a full size foundation was then found using the combination of the forementioned expessions:

$$S = \frac{3 * P}{N} \left(\frac{2 * B}{B + 1} \right)^2 \quad \text{--------------} \quad (3.7)$$

The water table is at depth greater than 2B below the base of the footing.

b) <u>Bazaraa's and Peck Method</u>:

Their equation is a modified version of Terzaghi and Peck's equation. The modifications include two effects one for overburden pressure and the other for water table effect. The equation is

$$S = X_B * \frac{2 * P}{N} \left(\frac{2 * B}{B + 1} \right)^2 \quad \text{----------------} \quad (3.8)$$

where X_B is the ratio of the effective overburden pressure at depth B/2 below the base of the footing when the water table is present to that without the water table.

N is corrected N-value for effect of overburden pressure.

c) <u>Peck et al. Method (1974)</u>:

In disscussing settlements, Peck et. al., (1974) state that the settlement chart of 1948 was developed based on a limited number of measured performance of actual structures. The settlement chart was, therefore, interpreted conservatively, so that the actual settlements would be less than 1 in. and that subsequent field experience has shown this to be true. Hence settlement chart of 1948 could be considered to give a conservative estimate to the settlement. Moreover, the penetration values upon which the charts were based are associated with an average overburden pressure (1 t/f^2) and hence N-values used in that equation should be normalized to N_1 according to the following equation:

$$Cn = 0.77 * \log (20 / Pv) \qquad \text{--------------} \qquad (3.9)$$

and the equation for settlement should take the following form:

$$S = \frac{P}{0.22 * N_1 * Cw} \qquad \text{---------------} \qquad (3.10)$$

where; S = footing settlement, inches

P = bearing pressure (kips/ft2)

N = corrected N-values within a depth B (footing width) below the base of footing.

Cn = correction factor for the position of the water table.

d) Meyerhof Method (1965):

Meyerhof (1965), in examining footing settlements, used an equation identical to Terzaghi and Peck (1948) equation for B > 4 ft. He found that their correlation is overly conservative and he recommended that the allowable bearing pressure which depends on N value to produce a settlement of less than 1 in. could be increased by 50% over that suggested by Terzaghi and Peck (1948). He also recommended that the presence of water table should be ignored, because it is already reflected in the measured SPT data. Meyerhof's equation is as follows:

$$S = \frac{2 * P}{N} \left(\frac{2 * B}{B + 1} \right)^2 \qquad \text{---------------} \qquad (3.11)$$

where;

N is field N-value i.e. no modification for effect of overburden pressure.

e) D'Applonia et al. Method (1968):

D'Appolonia et al., (1968) reported on actual settlements of a large number of footings on sand. Some footings were founded on natural sands and other compacted sands. The footings vary in width (B) from 8 ft. to 26 ft. The average SPT value within a depth B below the base of footing was about 15. After four years of observation on footings, the actual settlements did not exceed 0.75 in. This was considerably less than those predicted by Terzaghi and Peck or Meyerhof equations. The average settlements were about 1/2 the values computed from Meyerhof method and were well approximated by;

$$S = \frac{0.25 * P}{N_1} \left(\frac{2 * B}{B + 0.3} \right)^2 [\, 1 - 0.25 \, (D / B) \,] \qquad \text{----} \qquad (3.12)$$

where ;

D = depth of footing below the ground surface in meters.

N = the corrected N-values for effect of overburden pressure.

B = footing width in meters.

S = settlement in mm.

P = bearing pressure in KN/M^2. ($1KN/M^2 = 0.011\ t/ft^2$).

Actual values of settlements were about half to twice the value predicted by D'appolonia et al. equation. They stated that the sands were overconsolidated and this could account for lower than expected observed settlements.

f) Elastic Method (1987):

The concepts of elasticity theory have been used to estimate the immediate settlements of shallow foundation on sands. All the elastic equations take almost similar form with identical parameters. The most recent published elastic equation for settlement calculation on sands is the following equation by Bowles (1987);

$$S = \frac{4P * B}{E} \left(\frac{1 - m^2}{B + 0.3} \right) * I_s * I_f \quad \text{-------------} \quad (3.13)$$

where;

S = settlement at the center of foundation in feet ;

P = net applied static bearing pressure in ksf;

B = B/2 = half the width of the foundation in feet;

m = poisson's ratio;

E = soil modulus in ksf;

I_s= influence factor depending on the shape of foundation;

I_f= influence factor depending on the foundation embedment.

Direct measurments of modulus of elasticity and Poisson's ratio are extremly difficult in laboratory from samples of sand soils since recovering of undisturbed samples of such soils is impractical. Therefore, determination of modulus of elasticity is frequently correlated to the SPT values and the value of Poisson's ratio is assumed for sand to be between 0.3 to 0.40. A typical value is 0.35.

g) <u>Schmertmann's Method</u>:

This method depends on the elastic theory concept to estimate the foundation settlement on sand by integrating the strain distributions within each soil layer. Based on results of finite element analysis and tests on sand, Schmertmann proposed a simplified distribution of strain influence factor (Iz) beneath a foundation. Based on this simplified strain-influence digram, He suggested that the immediate settlement of a foundation be obtained as given by :

$$S = C_1 C_2 q \sum_{0}^{2B} \frac{Iz}{Es} * \Delta z \qquad \text{-----------------} \quad (3.14)$$

where:

C_1 = a correction factor for the depth of embedment;

C_2 = a correction factor for soil creep effect;

Δz = a soil layer thickness (increment);

Es = Sand modulus;

B = Footing width;

q = footing pressure

values of E in the above equation are originally inferred from cone penetration test results and it was set to 2.5qc for square or circular footings and to 3.5qc for strip footings having a ratio of length to width greater than 10. To make this settlement equation applicable to SPT, Schmertmann suggested an expression of the form

$qc = 4N\ (t/ft)^2$. The use of SPT results instead of cone penetration test results can be expected to introduce an additional uncertainty in a prediction of settlements of foundations on sand.

3.5 Accuracy of Available Procedures for Settlements Prediction on Sand:

Accuracy of the available methods for estimating settlements of foundations on granular soils has frequently been shown to be very low. Jeyapalan and Boehm (1986) assessed the relative accuracy of a number of settlement methods based on the actual performance of foundations on sand and gravels. Their study clearly showed the inability of the current methods to predict foundation settlements within acceptable accuracy. The results of their study are replotted and presented in Figures 5 to 10. In more recent study of the accuracy of the available settlement methods Clayton et al., (1988) concluded that no significant improvement in the prediction of settlement of foundations on sand can be noticed despite introduction of new methods. They attributed the poor performance of these methods to the lack of correlation between soil compressibility and standard penetration tests. However, discrediting the SPT just because the poor performance of the settlement methods is not the best approach to the problem (Parry, 1978). The improvement in estimating settlements cannot, however, be made without knowledge of the previous state of soil history (Schmertmann, 1985). Leonards and Frost (1987) proposed a new method for settlement prediction that takes the influence of stress history into account. Their method may represent a good approach for estimating settlements of foundations on sand but its implementation requires results of both dilatometer and Cone penetration tests. Moreover, verification of their method is based on only one case history which can not be considered conclusive confirmation of the method. Nevertheless, their method can be expected to be more accurate than other settlement methods because the effect of stress history is introduced.

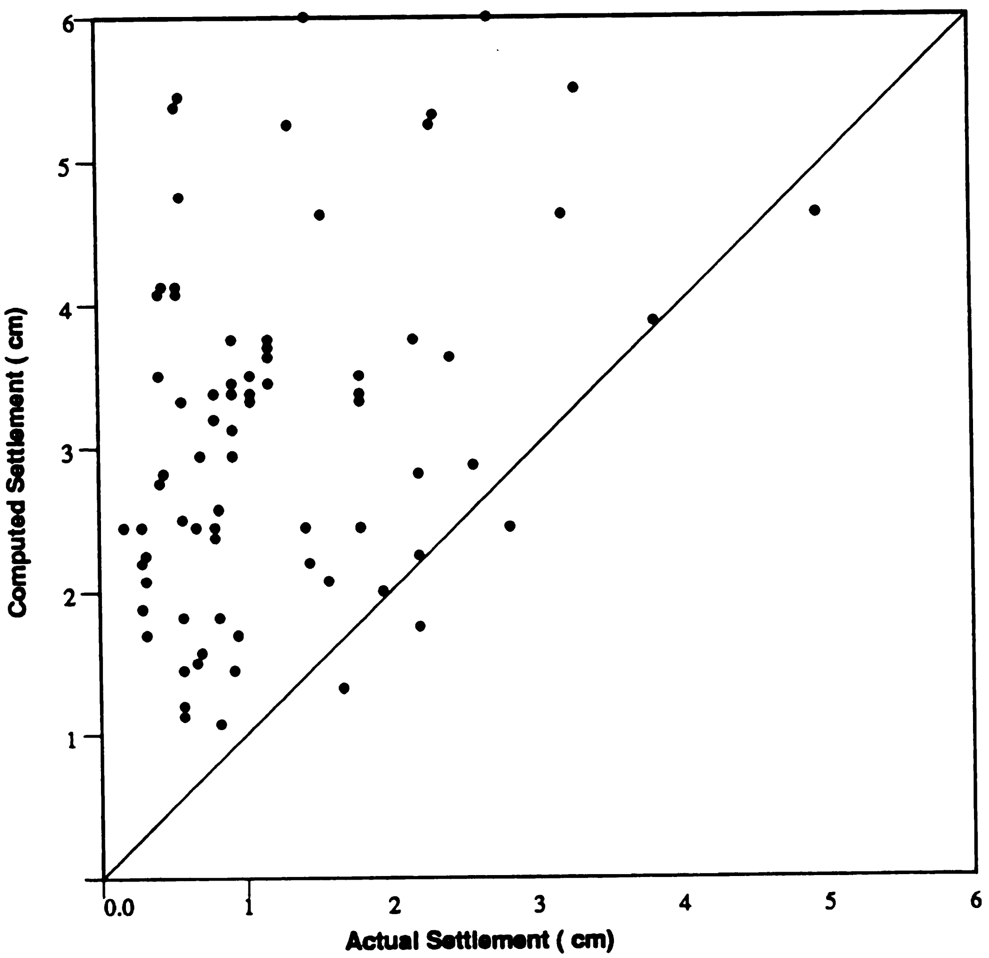


Figure 5. Comparison Of Measured Settlements with Values Computed by Meyerhof's Method (after, Jayapalan and Boehm, 1986)

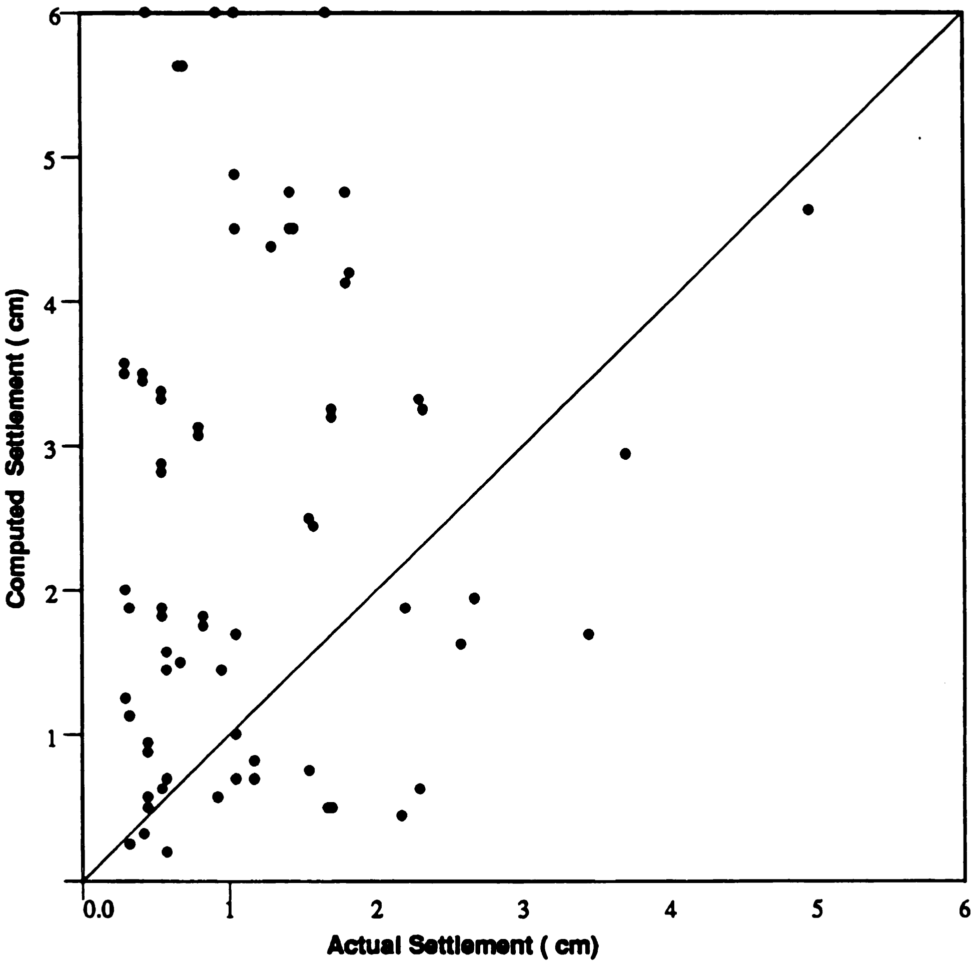


Figure 6. Comparison of Measured Settlements with Values Computed by Owelse's Method (after, Jayapalan and Boehm, 1986)

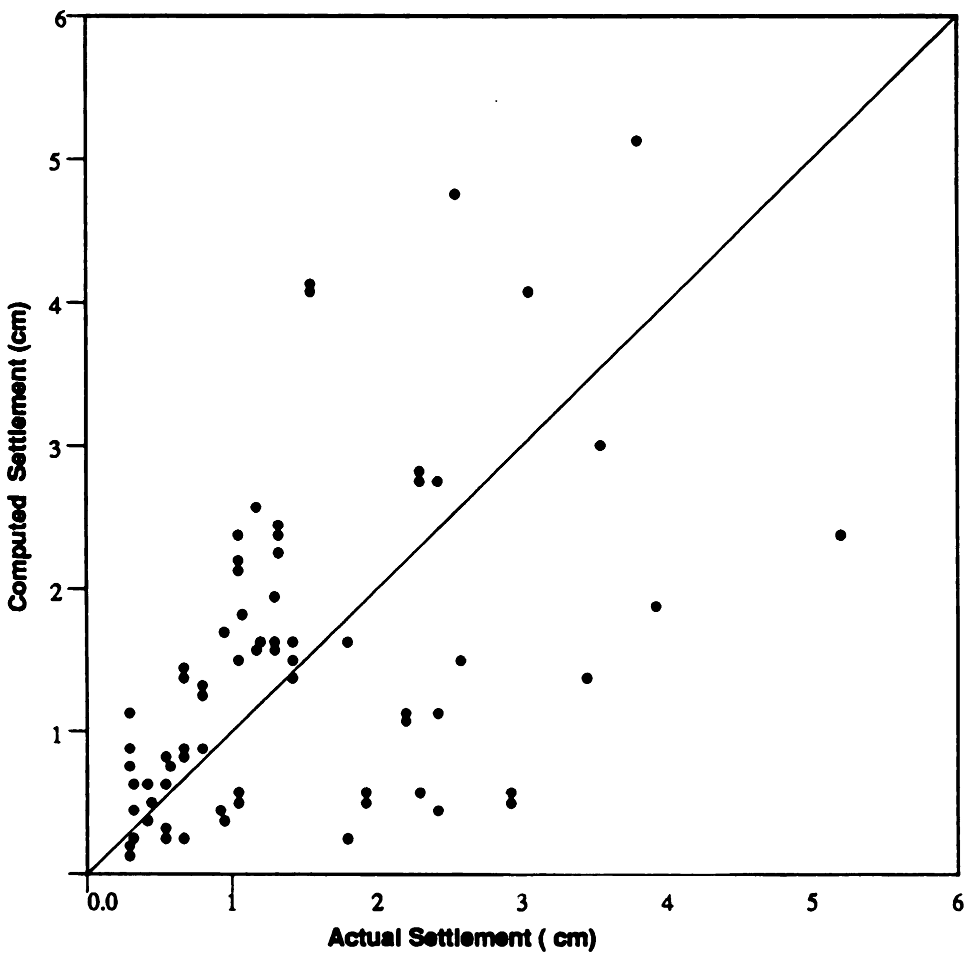


Figure 7. Comparison of Measured Settlements with Values Computed by Bazaraa's Method (after Jayapalan and Boehm , 1986)

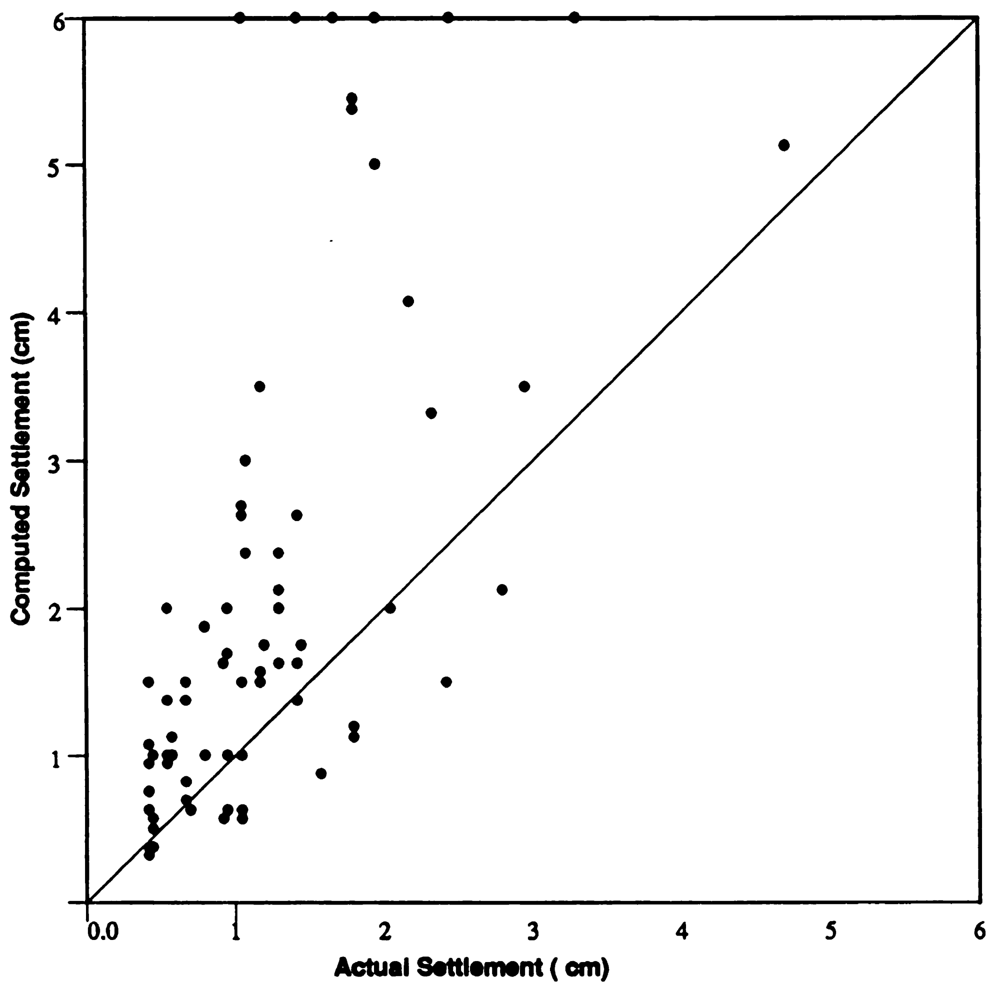


Figure 8 . Comparison of Measured Settlements with Values Computed by Schmertmann's Method (after, Jayapalan and Boehm, 1986)

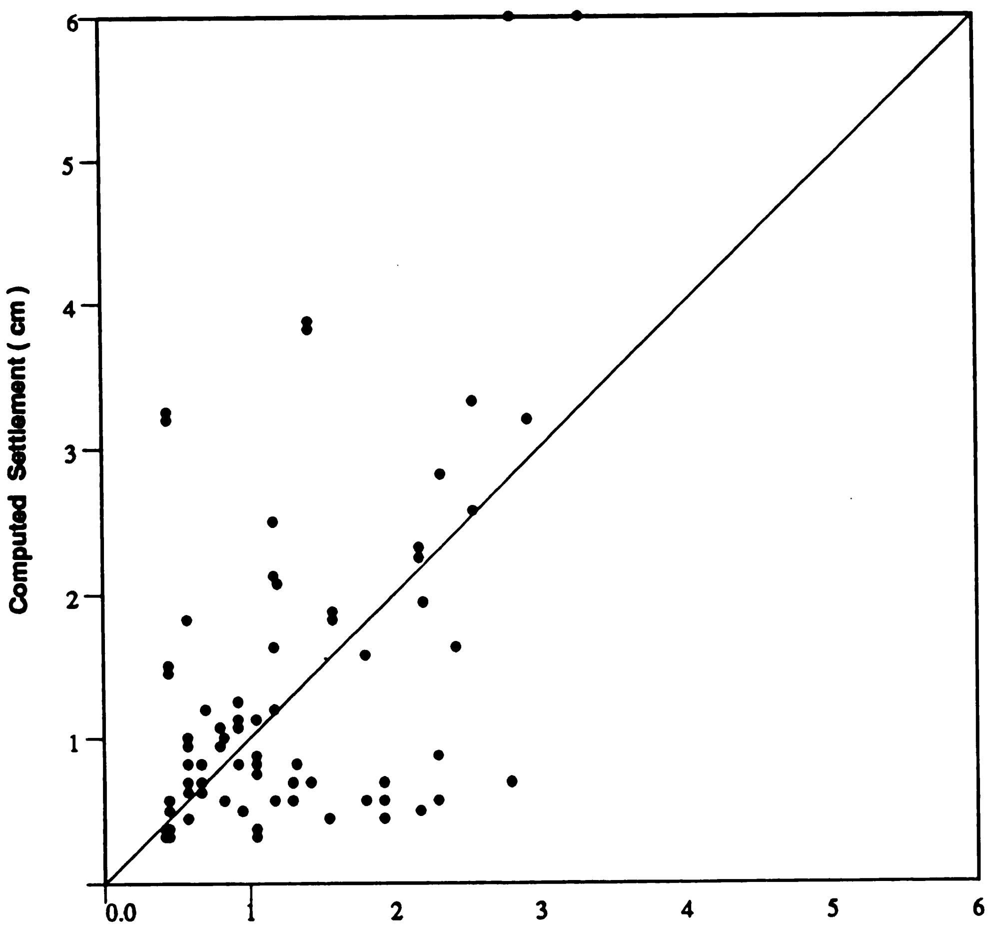


Figure 9. Comparison of Measured Settlements with Values Computed by Terzaghi Method (after Jayapalan and Boehm, 1986)

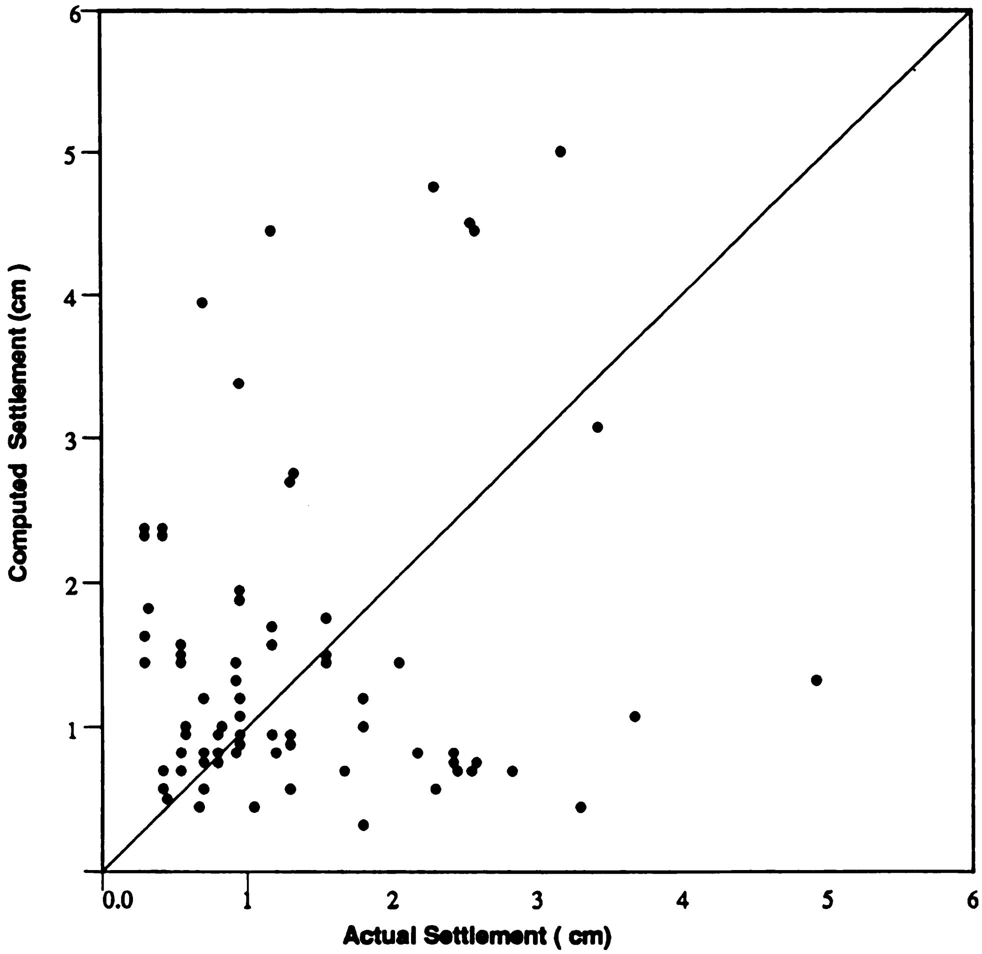


Figure 10 . Comparison of Measured Settlements with Values Computed by Schultze and Sherf's Method (after, Jayapalan and Boehm, 1986)

3.6 Importance of Stress History and Its Effect on Soil Settlement Prediction:

In recent years, engineers have become increasingly aware of the influence that horizontal ground stresses have on soil foundation behavior. Schmertmann (1985) listed a number of engineering problems wherein the lateral stress invokes a significant influence. Of those engineering problems, the soil settlement can be expected to change significantly with the coefficient of earth pressure (Ko). The latter is a function of preconsolidation pressure or overconsolidation ratio (OCR). Experiments in which sand was reloaded in the oedometer (Schmidt, 1966a) showed that the vertical strain upon reloading depends not only on the void ratio at the start of reloading and on the value of vertical pressure but also on the overconsolidation ratio (OCR). D'Applonia et al., (1970) attributed the potential reduction in soil compressibility of sand at the site of 300 footings on Lake Michigan to the high magnitude of horizontal ground stress as a result of a high past pressure or OCR. This means that confined sand may behave essentially as an elastic material after a large number of load applications. Janbu and Hjeldnes (1965) introduced a method for estimating sand compressibility in which the coefficient of earth pressure (Ko) was an important feature. Burland and Burbids (1987) introduced a settlement method where differentiation is proposed between overconsolidated soil and normally consolidated soils. However, based on the conclusion reached by Clayton et.al,(1988) their method is no better than the previous methods since it showed a poor performance in predicting actual settlement on sands.

This suggests that a better modeling of stress history is needed so that a better estimation of actual settlements for foundations on sand can be obtained.

3.7 New Approach for Improving the Predictions of Settlements of Shallow Foundations on sand:

It has been assumed that the standard penetration test results reflect the effect of soil stress history. However, many studies have indicated that the penetration tests of whatever nature are almost totally unaffected by stress history (Jamiolkowski et al., 1985; Clayton et al., 1985; Bellotti et al., 1986). Leonards and Frost (1987) noted that the ratio of SPT values for overconsolidated soil to those of normally consolidated soil was approaching unity while the corresponding soil modulus ratio was 6 to 10. This means that the previous SPT-based settlement methods may seriously overestimate the settlement of overconsolidated soils and ultimately lead to the rejection of a site or some unnecessary modifications where foundation settlement may actually be negligible. The question is; if the SPT cannot reflect the soil stress history to any significant extent, which is considered to be the main factor influencing soil compressibility, then how can foundation engineers have confidence in using any of SPT-based settlement methods for predicting foundations performances?. It must be obvious that any improvement in settlement prediction by the available methods cannot be expected unless the effect of soil stress history is properly accounted for, preferably in the form of coefficient of earth pressure (Ko).

Since the latter is a function of past soil history or over consolidation ratio (OCR), then it appears that the soil settlement bears a fundamental relation to this parameter. It has been known for a long time that OCR provides a forecast of how the soil will respond under applied load. For example, as OCR increases the soil settlement decreases and vice versa. Since Ko is directly related to OCR, then the same relation must hold for Ko. The relation between settlement and Ko is assumed to be exponential such that as Ko value increases the settlement decreases exponentially ($S \alpha e^{(-Ko)}$). This is analogous to the relationship between Ko and Skempton pore pressure (Au) (Figure 11). As may be seen at higher values of Ko corresponding to higher overconsolidation ratios, a lower pore pressure parameter is obtained. This relation is exponentially decreasing. Using the same line of reasoning, the soil settlement should decrease exponentially with increasing Ko value. As was shown earlier, all the SPT-based settlement methods are in the form of:

$$S = \frac{a * P}{N} \left(\frac{2 * B}{B + 1} \right)^2 \qquad \text{---------------} \qquad (3.15)$$

where **a** is a constant. In this study the original form of SPT-based settlement equation is maintained but with one modification that accounts explicitly for the effect of soil stress history in the form of coefficient of earth pressure (Ko):

$$S = \frac{2 * P}{N} \left(\frac{2 * B}{B + 1} \right)^2 \mathrm{Exp}(-Ko) \qquad \text{-----------} \qquad (3.16)$$

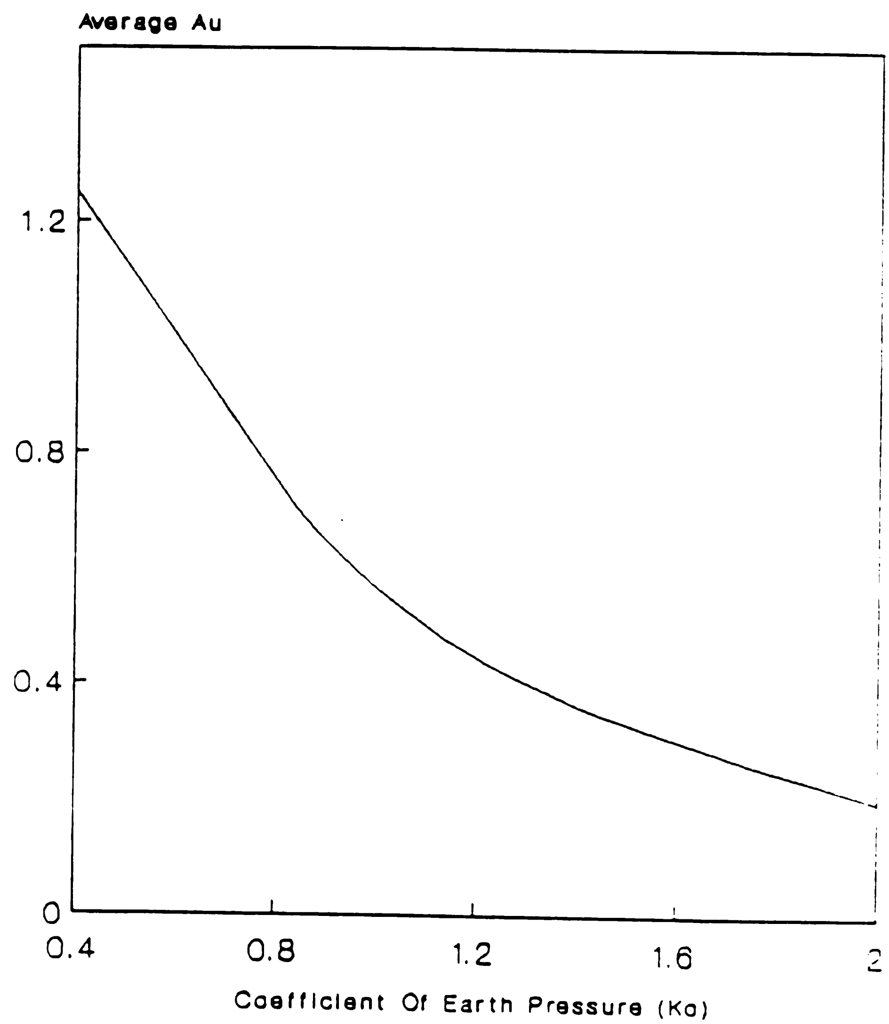


Figure 11. Relationship Between Skempton's Pore Pressure Parameter And Coefficient Of Earth Pressure(Holtz,1978)

a) Estimation of Coefficient of Lateral Earth Pressure:

Depending on the geological history of soil deposits at the site of a structure, it is possible to assess whether the soil is normally consolidated or overconsolidated. The value of coefficient of earth pressure (Ko) normally ranges from 0.35 to 1.0 . Typical Ko values for normally consolidated soil are 0.35 to 0.45 while values for overconsolidated soils range from 0.5 to 1.0. These limits are well established in geotechnical engineering. For example, in many studies a Ko value of 0.4 has been frequently assumed for normally consolidated soils (Christoffersen,1983). It is,therefore, suggested that the use of approximate value of Ko is superior to making no allowance whatsoever to account for the effect of soil history. In situations when evaluation of Ko becomes important then its value can be empirically obtained from results of Marchetti dilatometer (Marchetti, 1985) by:

$$Ko = 0.376 + 0.095\,KD - 0.00461qc/Pv \quad \text{--------------} \quad (3.17)$$

where KD is the horizontal stress index and qc is cone penetration resistance value, or from Schmertmann (1983) by:

$$Ko = \frac{40 + 23KD - 86KD(1-\sin d) + 152(1-\sin d) - 717(1-\sin d)^2}{192 - 717\,(1-\sin\phi)} \quad \text{---} \quad (3.18)$$

or based on the well known Jaky equation for normally consolidated soils:

$$K_{onc} = 1-\sin\phi \quad \text{--------------} \quad (3.19)$$

For overconsolidated soil the Ko can be approximated according to Mayne and Kulhawy, (1982) as:

$$Ko = K_{onc} * (OCR)^{sind} \qquad \text{-------------------} \qquad (3.20)$$

To demonstrate the effect of lateral coefficient of earth pressure (Ko) in the suggested functional form on settlements of foundations on sands, a comparison is made between a group of case histories of foundation design on normally consolidated soils and those case histories of foundations on overconsolidated soils. The results of the analysis are presented in Figures 12 and 13 and Table 4 to 5. It is readily seen that there is a significant improvement in the prediction of settlements as result of introducing the effect of soil history in the form of coefficient of earth pressure (Ko). It will be seen later that equation (3.16) is indeed a step in the right direction .

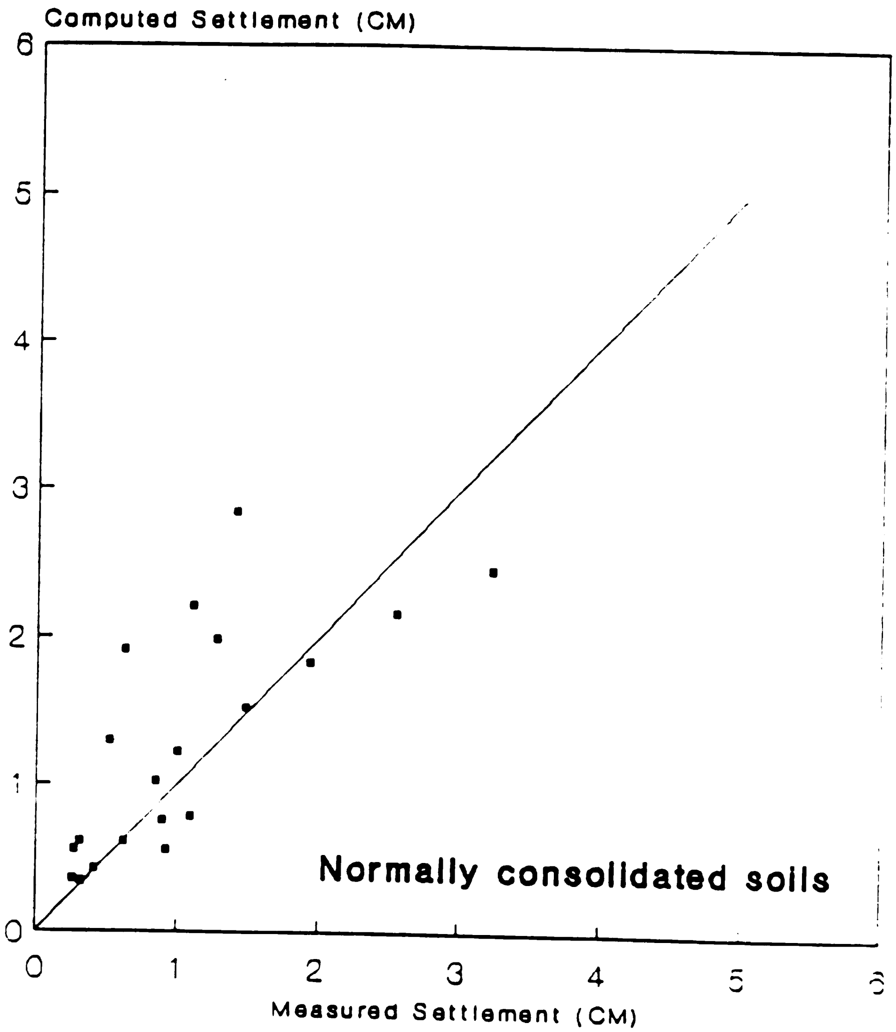


Figure 12. Comparison Of Measured Settlements With Values Computed By Proposed Method

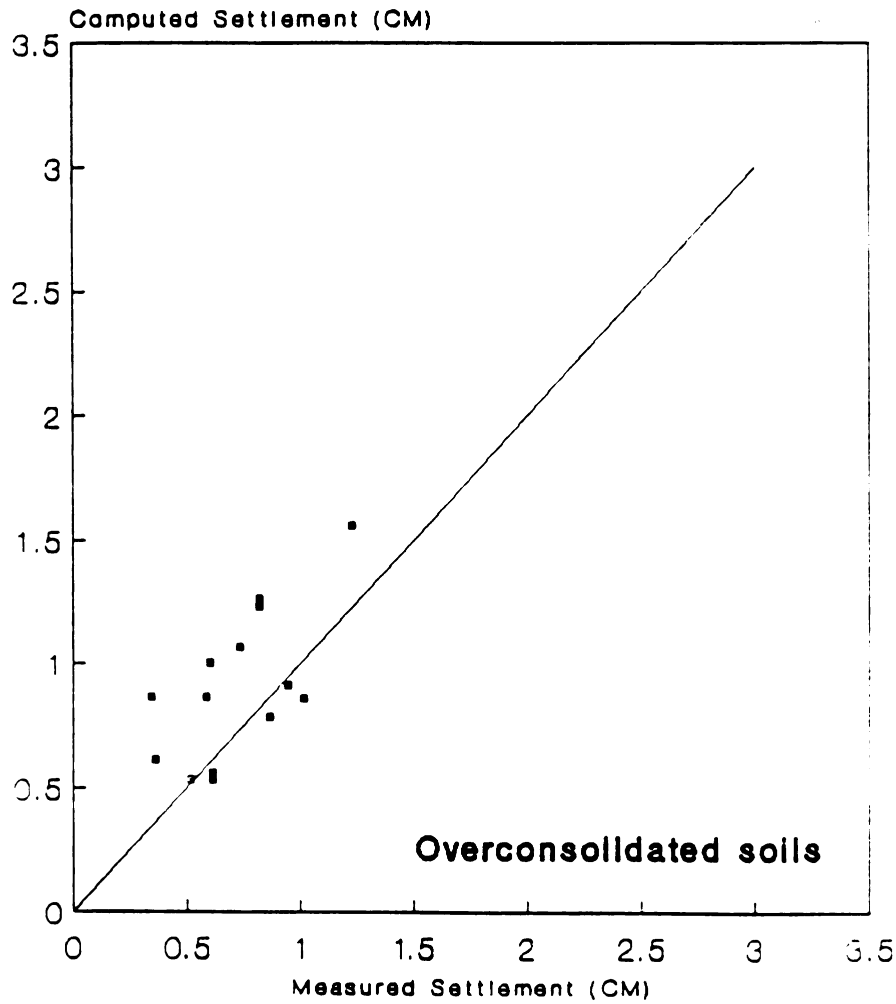


Figure 13. Comparison Of Measured Settlements With Values Computed By Proposed Method

Table 3.1: Data regarding settlements of case histories of shallow foundations on normally consolidated sand.

Case no.	Ref. no.	Type of Foundation	Dimention (ft)			Load	SPT (bl/ft) *	Ko	Exp(-Ko)	Settlement (in.)	
			B	L	D	(t/f)	ED	1-sin(phi)		Meas.	Pred.
49a	(25,63	strip	8.20	45.9	0.00	1.57	30	0.412	0.663	0.102	0.220
50b	=	footing	20.99	52.5	0.00	1.50	35	0.392	0.676	0.360	0.220
51c	=	footing	20.99	52.5	0.00	2.15	35	0.392	0.676	0.430	0.310
52d	(62,63)	strip	14.80	100.0	0.00	0.73	25	0.430	0.651	0.122	0.133
53e	=	strip	9.80	47.0	0.00	1.27	40	0.373	0.688	0.100	0.140
54f	(26,63)	strip	9.80	50.0	0.00	2.35	25	0.430	0.651	0.330	0.400
55g	=	strip	8.50	42.0	0.00	2.00	10	0.494	0.610	0.500	0.780
56h	=	footing	28.2	49.2	0.00	1.49	45	0.355	0.701	0.160	0.170
57i	(115,63)	mat	90.0	100.0	0.00	2.94	30	0.412	0.663	1.470	0.510
58j	(36,63	footing	43.0	90.0	0.00	1.76	30	0.412	0.663	0.350	0.300
59k	(43,63)	footing	20.0	20.0	0.00	0.44	8	0.501	0.606	0.120	0.240
60l	=	footing	20.0	20.0	0.00	0.66	8	0.501	0.606	0.240	0.240
61m	=	footing	20.0	20.0	0.00	0.87	8	0.501	0.606	0.390	0.480
62n	=	footing	20.0	20.0	0.00	1.10	8	0.501	0.606	0.580	0.600
63o	=	footing	20.0	20.0	0.00	1.31	8	0.501	0.606	0.760	0.720
64p	=	footing	20.0	20.0	0.00	1.53	8	0.501	0.606	1.000	0.850
65r	=	footing	20.0	20.0	0.00	1.75	8	0.501	0.606	1.270	0.970
66s	(85, 63)	mat	60.0	60.0	0.00	2.35	15	0.476	0.621	0.240	0.750
67t	(119,63)	footing	30.0	30.0	0.00	2.25	9	0.498	0.607	0.550	1.120
68u	(120)	footing	48.0	48.0	0.00	3.91	22	0.442	0.643	0.430	0.870
69v	(119,63)	footing	13.0	13.0	0.00	2.35	20	0.458	0.632	0.200	0.510

* DE refers to N-value selected by experienced engineers.

Table 3.2: Data regarding settlements of case histories of shallow foundations on overconsolidated sand.

Case no.	Ref. no.	Type of Foundation	Dimention (ft) B	L	D	ground water table (ft)	Load (t/f)	SPT (bl/ft) * DE	(Ko)	Exp(-Ko)	Settlement (in.) Meas.	Pred.
1a	(20,63,13)	footing	11.2	17.7	5.60	13.10	2.96	20	0.70	0.496	0.32	0.57
2b	=	footing	12.1	19.4	5.90	13.10	1.5-3.10	20	0.70	0.496	0.48	0.30-0.7
3c	=	footing	13.1	20.9	6.60	13.10	1.05-2.43	22	0.70	0.496	0.24-0.64	0.21-0.56
4d	=	footing	14.1	22.6	6.90	13.10	1.1-1.74	22	0.70	0.496	0.24-0.36	0.22-0.40
5e	=	footing	14.1	24.3	7.50	13.10	1.22-1.79	20	0.70	0.496	0.14-0.44	0.24-0.42
6f	=	footing	16.0	25.6	8.20	13.10	1.05-2.15	20	0.70	0.496	0.20-0.32	0.21-0.51
7g	=	footing	18.0	28.9	8.50	13.10	1.50	20	0.70	0.496	0.34	0.36
8h	=	footing	20.0	32.0	9.84	13.10	1.74	20	0.70	0.496	0.37	0.42
9i	=	footing	21.0	33.5	10.5	13.10	1.62	20	0.70	0.496	0.40	0.40
10j	=	footing	22.0	35.1	11.15	13.10	1.22	21	0.70	0.496	0.57	0.28
11k	=	footing	23.0	36.7	11.5	13.10	1.27	22	0.70	0.496	0.23	0.29
12l	=	footing	9.84	15.74	9.84	13.10	1.32	22	0.70	0.496	0.13	0.27
13m	=	footing	9.84	15.74	4.92	13.10	1.32	20	0.70	0.496	0.23	0.27
14n	=	footing	13.1	20.99	13.1	13.10	1.57	20	0.70	0.496	0.26	0.34
15o	=	footing	13.1	20.99	6.56	13.10	1.57	20	0.70	0.496	0.29	0.34
16p	=	footing	16.1	25.72	16.1	13.10	1.82	20	0.70	0.496	0.32	0.41
17r	=	footing	16.1	25.72	8.0	13.10	1.82	20	0.70	0.496	0.38	0.41
18s	=	footing	19.0	30.40	19.0	13.10	2.06	20	0.70	0.496	0.37	0.47
19t	=	footing	19.00	30.40	9.50	13.10	2.06	20	0.70	0.496	0.40	0.47
20u	=	footing	21.98	35.17	21.9	13.10	2.31	20	0.70	0.496	0.41	0.53
21v	=	footing	10.99	35.17	10.9	13.10	2.31	20	0.70	0.496	0.49	0.53

* DE refers to N-value used by designer or other experienced foundations engineers.

3.8 Available Procedures For Selecting Design N-Value

A) Methods recommended in Literature:

Several procedures may be followed in the selection of a design N-value. Terzaghi and Peck (1948) presented a chart for estimating allowable soil pressures for shallow foundations on dry sand based on the results of Standard Penetration Tests (N-values). This chart gives a relationship of the Standard Penetration Test (N-values) of sand, the width of footing (in feet) and the allowable soil pressure (in tons per square foot) for 1 inch of settlement. In order to obtain the N-value, which is to be entered in the chart, the sand penetration resistance at the site of a structure should be determined by making standard penetration tests in a number of borings, preferably at least one for every 4 to 6 footings at the site. For each boring, the Standard penetration value (N) should be determined at intervals of 2 1/2 ft or 5 ft in the vertical direction, and the average N-value should be determined for each boring for the sand between the footing base level and a depth B-2B (footing width) below this level. The smallest average N-value obtained from borings should be used for design of all borings, Bazaraa (1967). Wu and kraft (1967) investigated the consequences of adopting the minimum criterion on the foundation performance by means of probability and statistical analysis. In their study a series of standard penetration tests was performed in an out-wash soil deposit. The average N-value of each boring was determined and the minimum one was selected for design. They concluded that majority of footings will be designed very conservatively if the minimum average of N-value is adopted in design. They suggested that the theory of probability should be used an alternative for making a decision concerning the most probable representative soil strength parameters that should be used in design of soil foundations.

From results of standard penetration tests on a number of sites (Meyerhof, 1956) proposed a relationship between N-values and bearing capacity of shallow foundations. He recommended that the average of SPT (N-values) within a depth B below the base of footings be used in design of foundations. He further suggested that all penetration tests tend to become unreliable as the maximum particle size approaches the diameter of penetrometer or sampling spoon, and that the minimum single N-value of the penetration resistance be used if the maximum size of soil particles exceeds about 1/2 in. De Mello (1971), however,considered such a recommendation to be unacceptable; first because it fails to place the SPT in the conceptually correct position of any test available to the engineer, i.e., as a tool to be used with due specific interpretations; second because it neglects the fact that one of the greatest advantages of the SPT in comparison with non-sampling penetrometers is that specific information on the sampling soil could be obtained, i.e., if the gravel did block the penetrometer it would be immediately detected on the subsequent resumption of penetration. In addition, the engineer could easily inspect the sampling tube for gravels. The interpretation of N-value should, therefore, be the responsibility of the designer who has access to the information gathered from field tests,and the selection of minimum single N-value thus appears to be too conservative.

Meyerhof (1965) reviewed the design and performance of spread footings and rafts in relations to the predictions of settlement based on the standard penetration values. He indicated that the interpretation of the results of the standard penetration test, as done by Terzaghi and Peck (1948), for a given settlement of shallow foundations of sand and gravel are rather conservative. He recommended that the allowable bearing pressure should be increased by 50%. This recommendation clearly indicates that neither the minimum average N-value of a series of borings nor a minimum single N-value is a proper criterion for selecting a design N-value for soil foundation. Moreover, his recommendation shows implicitly that the average of N-values also is not a proper criterion since the increase of soil bearing pressures by 50% implies an increase of N-value by some factor.

Design charts for spread footings on sand were proposed by Peck et al.,(1953) where the allowable bearing pressure was related to the penetration resistance at a total settlement of 25mm (1.0 in). These design charts were modified latter (1974) by the same authors where the penetration resistance values were corrected with respect to the effective overburden pressure as proposed by Bazaraa (1967). The revised design charts were somewhat less conservative than ones recommended earlier ln 1953 by Peck et.al, but the criterion for adopting N-value in the design remains the minimum average N-value when design is based on a several soil borings. In their discussion about the standard penetration test values and the allowable soil pressure that could be obtained from the newly developed curves for foundation design, they recommended the design engineer be aware of the variability of the soil as reflected in the variation in the N-values from boring to boring at the site of a structure. They further recommended that such a variation be taken into account before using the developed design curves.

It might be reasonable to raise the following question: How does the design engineer take into account the variations in the SPT results when using their design charts? Should a designer assume the worst soil condition? Or should he use probability theory? Their recommendation clearly demonstrates the need for developing a method that takes in to account the variation of the in-situ test results. The conservatism resulting from the adoption of lowest values of soil parameters for design of foundations leads to more waste than is realized (Peck, 1977). The minimum criterion leads to a less satisfactory solution than would be achieved by developing other criteria or by accepting more reasonable risks. A similar conclusion was reached by Grivas and Harr (1979). Dunn et al., (1980) stated that the general practice in footing design set by Terzaghi and Peck (1948) remains the common practice among soil engineers despite the conservatism that is associated with such a criterion. Linderburg (1981) recommended that the minimum single N-value should be adopted for design soil foundations and this was strictly recommended for practicing engineers. However, Das (1984) pointed out that the design N-value for shallow foundation engineering should be determined by taking into account the Standard Penetration Test data that are within a depth of 2B to 3B below the base of footings and that the average of these values should be considered in the design. But, the variations in the results of in-situ test should not be neglected in the interpretation of test results, however, he did not show how the soil variation could be taken into account.

On the other hand, Bowles (1988) stated that the early recommendations were to adopt the minimum N-value in the borings or an average of all values. Both methods were used in foundation design but currently the common practice is to use an average N but in the zone of interest. His interpretation for the zone of interest is from about one half the footing width B above the estimated base location to a depth of 2B below that base. His statement is really ambiguous: first, neither previously nor currently a study has been done regarding the method that should be used for selecting a design N-value. Second, his interpretation for zone of interest as he called it is not clearly defined and it is contrary to what has been suggested by majority of engineers. This clearly shows that there is no acceptable form of selecting N-value for design foundation and the existing recommendations are not more than arbitrary ones which need to be investigated and more rational criterion be developed.

A) <u>Methods As Recommended In Actual Foundations Design</u>:

Two studies were conducted to show how the foundation engineers make their engineering judgement during the processes of design (Lambe, 1971; Wolff,1988,1989). The first author presented two case histories to eleven students at M.I.T. University of which ten were advanced graduate students and were highly trained academically but were inexperienced engineers. In both case histories they were asked to predict the field performance of Las Tortolas Dam and Oil Storage Reservoir. However, the author did not show what soil parameters were given. Two important and interesting facts came out of the case studies of the first author, namely:

1- Poor agreement between predicted and measurement performance;

2- Poor agreement among the predictions by the various engineers.

One of the most significant differences among students prediction of field performance was the selection of soil design parameter values such as different value of ϕ and E that they were given. He indicated that all students agreed that the selection of appropriate soil design value was the most difficult task and it was the primary reason for not predicting the field performance of both case histories. He concluded that inability of engineers to predict the performance of structures that are founded on soils arises from their inability to determine the appropriate soil design parameters. This again shows that when the designers are given different values of N for the purpose of determining the soil design parameters ,i.e., ϕ, and E, the designers need a lot judgement, since there is no available criterion for selecting such values.

Using different methods or approaches for estimating the sand strength parameter-value for the design always result in different values of ϕ. The scatter is significant and the ϕ-value can range from 36-50 for different methods (Bauer 1979). If the ϕ-value of 50 is used in the design the theoretical bearing capacity of footing will increase by several hundred percent. On the other hand if 36 is used the theoretical bearing capacity will decrease by a significant percentage. He stated that the variation of this parameter should be taken in consideration and this will definitly assist the practicing engineer in selecting a "design" strength parameter ϕ-value for the design purpose.

Wolff (1987) presented a civil engineering problem which was the design of a shallow foundation (spread footing) on sand to a diverse group of 39 experienced engineers. The total work experience of the respondents averaged about 15 years. The designers were given two borings with SPT data at the site of footing with a few load values that would be the typical foundation information available to a design engineer in majority of cases. All 39 designer engineers worked from the same site and from the same available information that were given, yet there were different approaches for selecting a design N-value. This led to substantial differences in outcomes of design calculations. The first step in the design involved the selection of an appropriate N-value. The design N-values that were selected by engineers ranged from 15 to 26 with a mean value of about 19. This variation in the magnitude of selected N-value led to different values of recommended footing sizes. The latter in term of area varied from 24.0 ft^2 to 94 ft^2. The results of this investigation showed that 46% of the design engineers used minimum uncorrected N-value, 15% used minimum average of uncorrected N-value, 10% used average of averages of uncorrected N-value, 5% used corrected average of all N-values, 18% used lowest corrected cumulative average of N-value and 5%, the minimum cumulative average.

The results of this study contradict Bowles' statement that the current practice is to select the corrected average N-value. Moreover, it supports Lambe's (1971) conclusion that the selection of soil design parameters is the most difficult and important step in the design of foundation engineering. However, it is often a common practice to select the minimum SPT (N) (Liao and Whitman; 1988). Again, the recommendations of Liao and Whitman (1988) contradict the statement made by Bowles (1988) and also ignores to admit that the minimum N-value is overly conservative criterion as shown in the arguments of many researchers in this chapter. Moreover, it emphasizes the opinion of Lambe (1971) in which he stated that the inability of an engineer to select a proper N-value for design leads always to poor prediction of civil engineering foundation field performance. It also supports the idea of both Peck (1977) and Grivas and Harr (1979) that a majority of geotechnical engineers select the minimum design N-value which leads to more waste than is realized.

DeMello (1970) stated that the minimum criterion that was suggested by Terzaghi and Peck (1948, 1967) to be adopted in design was indeed a recommendation and it should be altered since it is too conservative, as shown in most cases of foundation design. The selection of design N-value is one of the problems facing soil engineers since all correlations in the design depend on N-values of the soil foundation; consequently the misinterpretation of these values may lead to unfavorable circumistances.

D'Appolonia et al. (1968) used another approach for selecting an N-value for design of more than 340 footings on the shore of Lake Michigan in Nothern Indiana. Their procedure is as follows:

The variation in average blow-count was plotted as a function of elevation for 96 borings in the vicinity of column footings as shown in Figure (14). The curve was obtained by taking the blow-counts from each boring at a given elevation and then taking the average of all borings at that elevation. The SPT resistance used in the design is the average of averages of all SPT values within a depth B below the base of footings. It is interesting to see that the same case history was analyzed by many other experienced designer engineers (Schmertmann 1970; Oweises 1979; Jeyaplan and Boehm 1986; and Byrne and Russell, 1987) and yet each one selected a different design N-value to support the validity of their studies. The selected N-values for the same case history were 34, 15, 20 and 15 respectively.

DeBeer (1948) reported on the design of a shallow foundation for a Belgian bridge pier. The soil of the bridge foundation consists mainly of sand material. Schmertmann (1970), and Jeyapalan and Boehm, (1986)

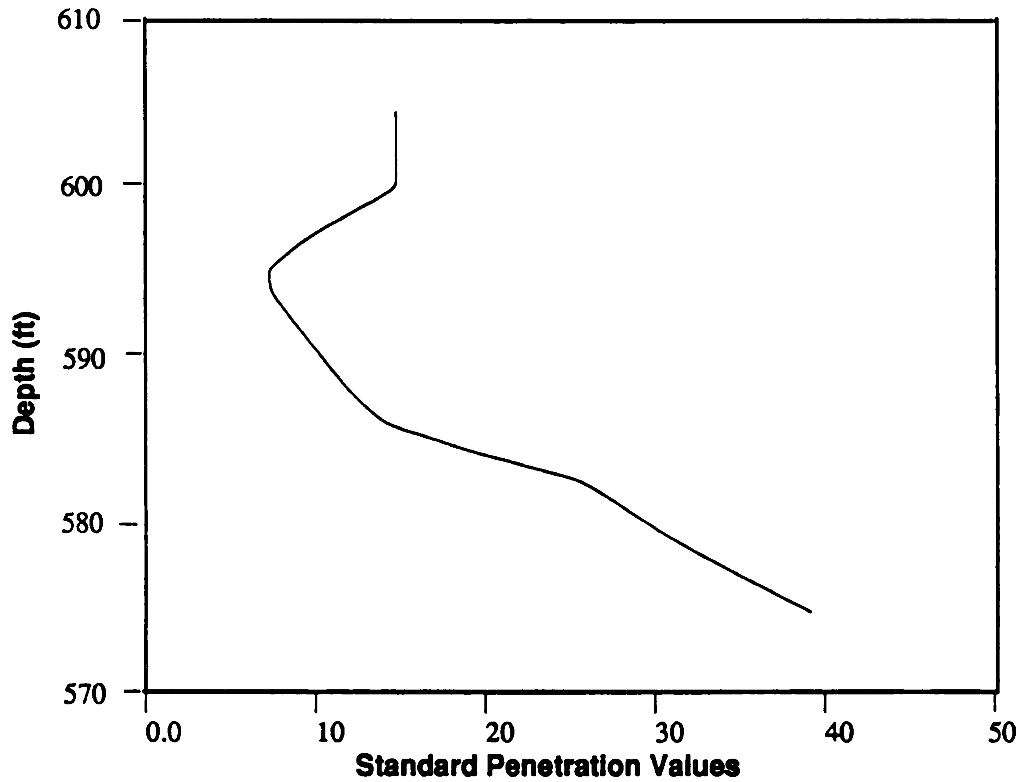


Figure 14 . The Results of Standard Penetration Tests at the Site of MILL Building at Lake Michigan (after D'Applonia ,1968)

analyzed this case history for expected settlement of the bridge foundation. Both used the same settlement equation but two different values of penetration test which resulted in substantialy different predicted settlement outcomes. In another case of a bridge foundation, DeBeer and Martens (1956), Schmertmann (1970) and Jeyapalan and Boehm (1986) worked from the same soil and structural data to predict the settlement of the foundation; however, the difference in prediction of settlement was substantial as a result of assuming different N-values, 18, 25 and 34 respectively, in their calculations.

Grimes and Cantlay (1965) reported on a case history that involved a twenty-story office block at Lagos, Nigeria. This case history was analyzed by Shmertmann (1970), Jeypalan and Rolland (1986), and Bazaraa (1968). The first author used results of cone penetration tests in his analysis in which he adopted a value of 120 t/ft^2(qc) that is equivalent to N-value of 34 as he indicated in his analysis. The second author considered N-value of 30 as the most representative value for design of foundation while the third author assumed N-value of 17 to be used in the analysis of soil foundation. These are knowledgeable and experienced engineers and yet there was a significant difference in their decision about the status of soil condition at the site of a structure. Clearly such disagreement among design engineers about the selection of design N-value will result in different predictions of foundation performance.

Casagrande (1966) stated that the designer's misinterpretation of N-values have resulted in many cases to very costly buildings,i.e, changing a shallow foundation to pile foundation due to wrong interpretation of N-values of the soil at the site of a structure. It is the opinion of this writer that if there was a reasonable quantative method for selecting N-value for foundation design, there would have not been so many empirical equations for settlement prediction or at least the existing SPT-based equations would have been in some other forms.

Baker (1965) reported on the selection of SPT value for the design of foundation for two chemical storage tanks. For the foundation of the first structure the minimum single N-value was used, while for the second foundation the average of SPT values was adopted for the design. Bazaraa (1967) reported some details about soil conditions at the two sites as well as dimensions and applied pressures of the same structures. In his analysis of soil strength of both sites the minimum average of N-values was selected for estimation of settlements of foundations.

Three case histories of shallow foundations in Germany, Thyssen building and Ministry building in Dusseldorf and a reactor building in Stetternich, were analyzed by Schultze (1962, 1963), Bazaraa (1967) and Burland and Burbidge (1986). There was a significant difference among the three experienced engineers concerning the selected N-value for design of each case history. For the Thyssen building, the value of selectd N was 18, 23, and 20 respectively. For second case history,the selected N-value was 12, 16 and 14 respectively while for the third case, the selected N-value was 30, 29, and 25 respectively.

Similar difference in the selected N-value was noted among the same foundation engineers in the analysis of Student building in Aachen, Germany. Again, this shows that available procedures for selecting design N-value are not always consistent and the selection of N-value depends to a great extent on the experience of foundation engineers rather than on any other factor.

For simplification, the different procedures for selection of design N-value as illustrated in this chapter are summarized and presented in Table (3.3).

Table 3.3. Recommended methods of selecting SPT value for design of shallow foundations

References	* Method
Terzaghi and Peck (1948)	minimum of averages
Meyerhof (1956)	average
DeMello (1974)	average
Bazaraa (1967)	minimum of averages or average of averages
Wu and Kraft (1968)	minimum of averages or theory of probability
Peck et al., (1974)	minimum of averages or average
Dunn et al., (1980)	minimum of averages or average
Linderburg (1981)	minimum N-value
Das (1984)	average
Bowles (1988)	average

*** Minimum average (Nmm): N values in each boring are averaged and the minimum of these averages is taken.**
Total average(Ntavg) : N values in each boring(s) are averaged.
Minimum N value(Nmin): The Minimum N value found in Boring(s).

CHAPTER 4

PROPOSED MATHEMATICAL ALGORITHM FOR SELECTION OF DESIGN STANDARD PENETRATION VALUE

4.1 Introduction:

In chapter two, it was pointed out that Standard Penetration Test values are a function of many variables which leads to significant scatter in the test data. Even though some of important variables are recognized by geotechnical engineers, differences exist among them in the selecting a "design" N-value for a given set of conditions. A great deal of variability in recommended designs can be attributed to the lack of a unified method for selection of the N-value. The need for an engineer to select a design N-value implies a unique value exists for the entire soil at the site of a structure. This is comparable to stating that the N-values at the site of a structure have no inherent variability. The variation in the results of standard penetration tests is significant and as a result design engineers tend to design a foundation on the basis of some form of conservative criterion. For instance, the engineer might consider the minimum single N-value or the minimum average N-value to adopt in the design of foundation. Such approach to design assumes the worst soil condition. It can be criticized on the grounds that the engineer ignores other soil N-values except the minimum one. Soil conditions to him are expected to be always worse than what the SPT reveals. Although under certain conditions a designer may follow the minimum criterion in order to minimize losses rather than maximize gains, most engineers are not likely to be guided by such extreme pessimism.

At the other extreme, the designer may apply the maximum criterion particularly when the N-values in boring(s) within the vicinity of a site are increasing regularly with depth. Following this criterion, the engineer would consider the maximum N-value or maximum average N-value which represents the most optimistic decision about the soil condition at the site of a structure. This criterion reflects the attitude of a foundation engineer who believes the soil at the area of structure has the best characteristics as the one shown in soil boring log. He ignores all other possibilities that might arise. Most engineers are not likely to be guided by such extreme optimism. Thus, the maximum criterion does not appear to be a satisfactory guide for adopting N-value in foundation design.

Another criterion that might be used for selecting design N-value is the arithmetic mean of N-values in the boring(s). Bowles (1988) stated that such approach is recently being adopted in design, however, a recent study by Wolff (1988, 1989) showed that most of practicing engineers adopted the minimum criterion in the selection of N-value for foundation design. This was later shown (liao et al., 1988) to be the only criterion that enjoys the wide application in the geotechnical engineering design. The arithmetic mean of N-values is affected by the extreme values and this approach does not show the design engineer the magnitude of uncertainty associated with this approach of selection. This criterion might be of academic interest more than in practice.

The theory of probability and statistical analysis as alternative approach has been gaining ground in recent years for application in geotechnical engineering. However,this approach is mathmetically oriented and this condition has acted to restrain the application of this method because of a general unfamiliarty of the designers with this method (Whitman, 1984). Moreover, the choice of the probability distribution may be dictated by mathematical convenience. In many cases, the functional form of the required probability distribution may not be easy to determine, or more than one distribution may fit the available data. The mathematical relations serve only to carry the intuition of engineers to its logical conclusion. It, therefore, is not a popular tool for design engineers.

4.2 Hypothesis of The Proposed Algorithm for Selection Design N-value:

a) Definition of Design N-value

The design N-value is defined as the value selected by a design engineer from a series of SPT measurements to characterize the foundation soils. This value will be used either directly to check the bearing capacity and foundation settlement or indirectly by using it to estimate design values for angle of internal friction (ϕ) and modulus of elasticity (E) for evaluation the foundation performance.

b) Hypothesis

In mathematical form, the design N-value (N_d) should be expressed as a function of the effective overburden pressure (Pv), a parameter accounting for variation of soil properties (grain size distribution, soil aging ,relative density....etc.) (C), the amount of energy that is delivered to the split spoon sampler (ERr), the set of measured N-values [N1,N2.......Nn], and the bore hole diameter (Dc),. In its simplest form this function could be set up as follows:

$$N_d = f\ (\ Pv,\ C,\ ERr,\ N',\ Dc,\ ) \quad \text{------------------} \quad (4.1)$$

where ;

$$P_v = f\ (\ P_{v1}\ ,P_{v2}\ ,P_{v3}\ \ldots\ldots.\ Pvn\ ) \quad \text{----------------} \quad (4.2)$$

and

$$\mathbf{N' = f\ (N_1\ ,\ N_2\ ,\ N_3\ .... Nn\ )} \quad \text{--------------} \quad (4.3)$$

In view of the forementioned information this functional relationship may be formulated into following form of equation that shows explicity the importance of SPT scatter in the decision making of a design engineer about N-design value.

$$\mathbf{N_d = \frac{Er\%}{60} * Dc * N''} \quad \text{--------------} \quad (4.4)$$

where ;

$$\mathbf{N'' = [\ A * C + (1 - C\ ) * B\ ]} \quad \text{--------------} \quad (4.5)$$

$\frac{Er\%}{60}$	is the energy correction factor;
Dc	is the borehole size correction factor;
A	is some low N-value to be determined (may be Nmin.,Nmm,..etc.);
B	is some high N-value to be determined (may be Ntavg, Nxavg.,..etc.);
C	is a parameter that accounts for data scatter;

The scatter of any data, however, can be quantified by its coefficient of variation (C), the ratio of standard deviation to mean value. It is significant to note that if the coefficient of variation (C) is 1 such that there is 100% variation in N measurements, the suggested equation should result in the minimum criteria that give value for N_d close to that currently selected by design engineers. On the other hand, if the coefficient of variation (C) is zero such that there is zero variation in measurements, the same equation should lead to less conservative criteria that give value for N close to one most engineer will select for design. It, however, provides a basis for more rational selection of the design N-value when the scatter in the test results is not in the extremes.

The selected design N-value that is suggested by the proposed equation will be compared with one that was used in actual foundation design by practicing engineers. The following chapter of case histories for shallow foundations design on sand will be used for comparison and verification of the proposed equation.

4.3 Factors to be Considered in Developing the Proposed Algorithm:

a) Scatter of Standard Penetration Test Results

Variability of soil test results is a key issue in almost every aspect of geotechnical engineering. In foundation engineering design, the scatter of Standard Penetration Test (SPT) values plays an important role in the decision of designers. Indeed, the scatter of test data is one of the prime reasons for disagreement among designer engineers about which N-value be chosen, from a range of values obtained by multiple soil boring(s) within the site of a structure, and tendency toward conservatism and inconsistency. For instance, a series of measurements of N-value at different depths may be available at the site of a structure. Typically these measurements will take the following form, which is a compilation of Brazilian penetration tests reported by Rios and Silva (1948).

Boring 1		Boring 2	
Depth Below Ground Surface (ft)	Brazilian Penetration Value	Depth Below Ground Surface (ft)	Brazilian Penetration Value
22.9	10 (14)	22.9	5 (7)
26.2	11 (15)	26.2	3 (4)
32.8	5 (7)	32.8	5 (7)
36.1	25 (32)	36.1	5 (7)
49.2	15 (18)	49.2	5 (7)
52.5	17 (20)	52.5	6 (9)
55.8	10 (11)	55.8	8 (8)
59.1	7 (8)	59.1	15 (17)
65.6	20 (22)	65.6	9 (10)
68.9	15 (16)	68.9	10 (10)
75.5	13 (14)	75.5	2 (2)
82.0	15 (15)	82.0	23 (23)
88.8	26 (26)	88.8	22 (21)
Meam	17		10

Scatter of Test Results is 55 %

Number between bracket refers to corrected value for overburden pressure.

To eliminate the effect of depth, the design engineer might apply the correction factor for overburden pressure, the most widely used is cn = 0.77 * log (Pv/20) Peck et al., (1974), then the designer is left with the decision to make about which N value might be the representative of soil condition. Each criterion will result in a different N-value and consequently in a different outcome of anticipated foundation settlement and bearing capacity.

For purpose of comparison the results are illustrated in Table (4.1). Definitions of these criteria are given in Table (3.3) in chapter 3.

Table 4.1. Comparison between common criteria for selecting design N-value

Methods	Selected N-value
Minimum N-value (Nmin)	2
Minimum average (Nmm)	10
Total average (Ntavg)	14
Maximum average (Nxavg)	17

An appropriate way to interpret the results of N is to recognize that the design engineer would generally tend to be conservative in selection of N-value when there is a considerable scatter in N-values. On the other hand, the same designer would tend to be less conservative where there is a slight scatter in the test values. Thus, the scattter of test results needs to be accounted for, preferably in the form of a parameter that accounts for scatter. Indeed the variation in the test results is a fundamental term in the mathmetical relations for evaluating the reliability and safety of any engineering structures. The greater the scatter of test results, the greater must be the factor of safety (Osgood, 1982), which implies a lower value for strength of the material in question. This supports the aforementioned view that it is advantageous to account for the effect of scatter in decision making of selecting N-value or any other soil parameters for purpose of foundation engineering design.

a.1) Relationship Between Variation of Test Data and Reliability of a Structure:

The purpose of analyzing the variation of soil materials as is indicated by wide scatter of test results is to allow a better characterization of the soil parameters. This characterization includes the most likely value and possible error and can be represented mathematically by mean and standard deviation. If the results of in-situ tests (SPT or CPT) should be correlated with the strength or deformation of soil which is the case in geotechnical engineering, then the variations of test results should be considered. The degree of conservatism then may be chosen with respect to the scatter of data. There are many methods to estimate spatial variation of soil materials.

In these methods the mean value of the soil parameters is usually reduced by some correction factors(Wu et al., 1989) due to variation of test results at the site of a structure or each observed data is given a weight denoted by some factors according to their location with respect to critical area (the area that is near to the location of a structure). The data scatter has been used to evaluate the safety factor and reliability of structures and machines (Figure (15)). The later shows that the higher the scatter in test results, the higher the required factor of safety. This, again, shows the importance of accounting for variation of design parameters in the decision of a design engineer.

b) Actual Energy Delivered To The SPT Rod:

Recently the concept of energy and its importance in the interpretation of SPT data has drawn a lot attention in different parts of the world, among soil foundation investigators. Field data show that the SPT values for a given soil are inversely proportional to the actual amount of energy ($N \alpha 1/E$) in the drilling rod (Schmertmann, 1975 and Schmertmann et al., 1979). Similar findings were reported by Kovacs (1978, 1981, 1982). The variability of SPT values obtained by different drillers and drill rigs testing the same soil can be substantial due to the differences in the amount of energy that actually reaches the drill rods. It is misleading, therefore, to select a unique value for N in foundation design from SPT data without modification it for the corresponding energy or normalizing to a common energy. The effect of this factor on SPT results is detailed in chapter 2. In this study the energy ratio of 60% (actual amount of energy in rod with respect to the

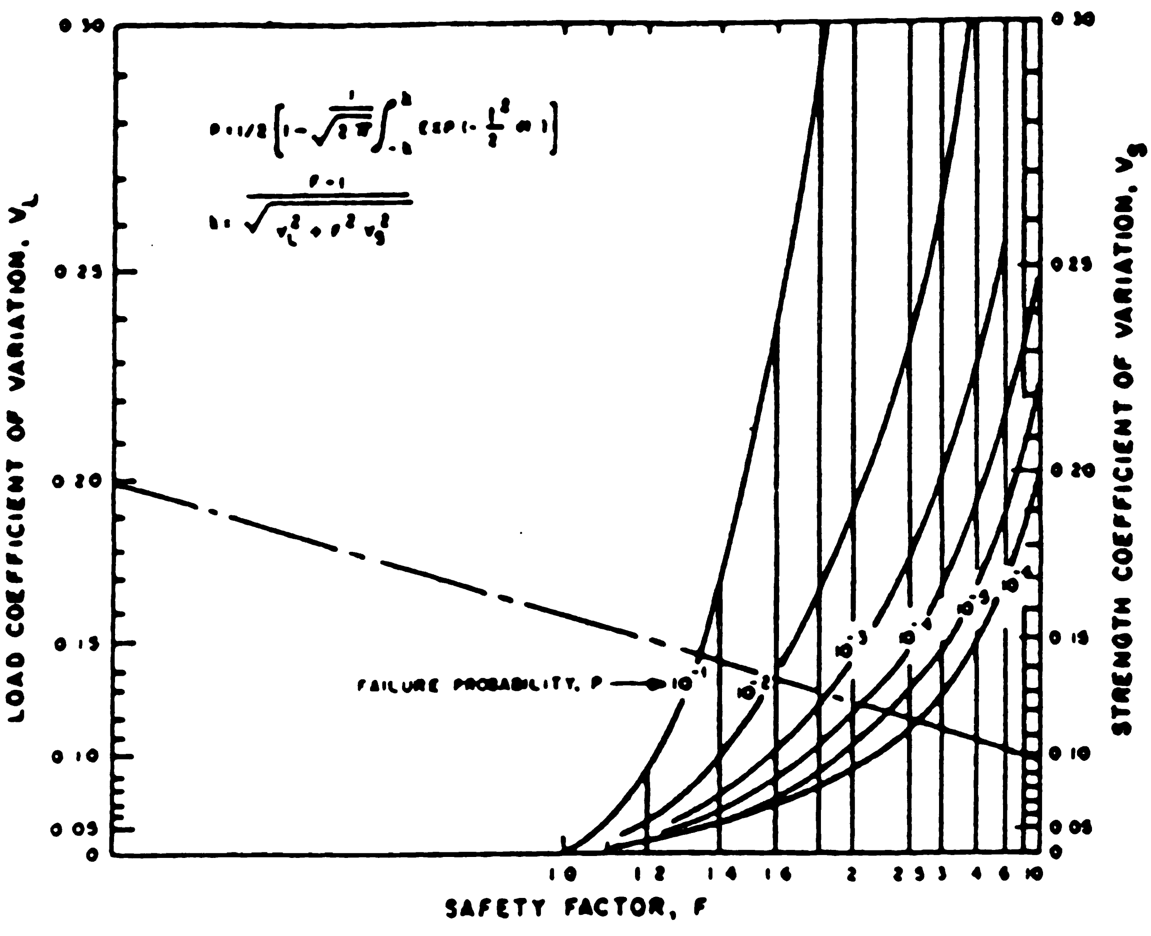


Figure 15. Relationship Between Safety Factor and Strength Coefficient of Variation (after, Osgood, 1982)

theoretical value) is considered to be reasonable for standardization of SPT value as a common energy base for the reason previously given.

c) <u>Bore Hole Size Effect:</u>

Bore hole diameter has an effect on the stress relief at a depth that the SPT performed. The effect of such a factor on the results of SPT is detailed in chapter 2 of this study.

CHAPTER 5

Testing Of Algorithm Using Actual Case Histories

5.1 General Procedure:

The purpose of this chapter is to test the proposed algorithm presented in Chapter 4 and to make a direct check of the reliability of the procedure suggested in Chapter 2 for improving the estimation of shallow foundations settlements on sand. To do so, it is essential to have case records for the settlements of structures on such foundations. These records must contain adequate data regarding the dimensions of the foundations, the applied pressures, the measured settlements and the soil conditions. The standard penetration N-values must also be known. The case records which will be included in this study include not only all the previously mentioned data but contain sufficient information regarding soil stress history as well.

Eighty six case histories for the settlement behavior of shallow foundations were collected for this study. These case histories cover a wide range of structures; from a simple one story building to towers and nuclear power plants. The data were analyzed in two stages: In the first stage the soil boring(s) data (SPT values) for a total of 21 case records along with description of a necessary design information were analyzed in detail. The analysis of the data from each case history was made as detailed below:

1. Standard penetration values were tabulated for each soil boring(s). The N-values of all soil boring(s) from soil surface to a depth of 2B were added and averaged. The standard deviation and coefficient of variation were then calculated.

2. For each individual soil boring, the N-values were averaged within the significant depth between the base of a footing and a depth below the base equal to the 2B of footing width. The lowest average N-value obtained in this manner, from a group of borings at a site, was called (**Nmm**) and the maximum average was called (**Nxavg**). The average of all N values from all boring(s) obtained from base of footing to 2B was called (**Ntavg**) and the lowest single N-value in the boring(s) is called (**Nmin**). As settlement prediction was of interest, no correction for overburden pressure was made for the reason explained in Chapter 2. It is necessary to note that in most cases the soil borings were terminated at a depth less than 2B, especially for large footings or mats. In such cases, there was no other alternative but to accept the N-values that were presented in the case history.

3. The parameters, A and B, of the proposed algorithm for selecting N-value were then investigated before a final form of an algorithm was adopted. The following five hypotheses were used to provide enough information for both parameters in the proposed algorithm to be determined:

First hypothesis

A = Nmin

B = Nxavg

Second hypothesis

A = Nmin

B = Nmm

Third hypothesis

A = Nmin

B = Ntavg

Fourth hypothesis

A = Nmm

B = Nxavg

Fifth hypothesis

A = Nmm

B = Ntavg

The values of A and B for each hypothesis were then inserted in the proposed algorithm to get the N-design value. The results of each hypothesis were then compared with N-value that was suggested by a designer in that particular case history as well as the N-value that obtained by the most common recommended methods of selecting N-value in geotechnical engineering. The best matched hypothesis with a designer's value was then adopted for the final form of proposed algorithm.

Second Stage of the Data Analysis

The adopted form of the algorithm (to be defined later) from stage one was then used to analyze the SPT data for a total number of sixty five more case histories of actual shallow foundations on sand. However, in this stage the details of analysis were omitted and only the necessary information were retained for analyses and verification of the proposed algorithm.

The settlements of soil foundations for all case histories were then estimated by means of the following five different procedures:

1. $$S = X_B * \frac{2 * P}{N} \left(\frac{2 * B}{B + 1} \right)^2 \quad \text{---------------} \quad (5.1)$$

This represented the settlement estimated according to Bazzara (1967).

2. $$S = \frac{P}{0.22 * N_1 * Cw} \quad \text{-----------------------} \quad (5.2)$$

This represented the settlement estimated according to Peck et al. (1974).

3. $$S = \frac{2 * P}{N} \left(\frac{2 * B}{B + 1} \right)^2 \quad \text{------------------} \quad (5.3)$$

This represented the settlement estimated according Meyerhof (1965).

4. $$S = \frac{0.25 * P}{N_1} \left(\frac{2 * B}{B + 0.3} \right)^2 [\, 1 - 0.25\ (\, D / B\,)\,] \quad \text{---} \quad (5.4)$$

This represented the settlement estimated according to D'Applonia (1970)

5. $$S = \frac{2 * P}{N_d} \left(\frac{2 * B}{B + 1} \right)^2 * \text{Exp}\,(-K_o) \qquad \text{------------} \qquad (5.5)$$

This is the suggested method for estimating settlements of shallow foundations on sand proposed in chapter 3 of this study. The settlement is estimated by introducing the N value of the proposed algorithm together with the effect of soil stress history. The latter was introduced in the form of coefficient of earth pressure (K_o). To obtain K_o, the case histories were divided into two categories: Over consolidated soil foundations and normally consolidated soil foundations. For the first category a K_o value was known while for second category a value of 0.4 was assumed. This value was considered to be within a reasonable accuracy based on the information given in the case histories. However, in cases where there was no enough information available about the soil foundations' status, the well known Jaky equation, $Ko = 1 - \sin\phi$, was used.

To estimate the effect of energy on predictions of actual settlements of shallow foundations presented in this study, the N-values of each case history were converted to equivalent N_{60}, based on the assumption that the SPT results of each case history were obtained by American original Donut hammer (Skempton (1986) used same assumption). The final settlements were then estimated by means of the forementioned methods.

In the majority of case histories, the standard procedure was used to obtain SPT values such that there was no deviation from original bore hole diameter (4 inches), therefore its value was taken 1.0 in the proposed algorithm. In case there was no enough information to calculate coefficient of variation (C) a value of 30% was assumed (Wu, 1974 and Yoder and Witczak, 1975 used same assumption).

The comparison of the results obtained from these different procedures for estimating settlements of footings on sand and the adoption of any one or combination of them as the final suggested method is reserved for the final section of this chapter.

5.2 Analysis of Case Histories and Testing Hypothesis:

A. Detailed Analysis:

Case History No.1: Ipiranga Building

Vargas (1948), Meryerhof (1965), Bazaraa (1967) and Jeyapalan and Boehm (1986) reported on design of the Ipiranga building, Sao Paulo,Brazil.

Soil Conditions

The soil consisted of 6.5 to 10 ft of fill and stiff clay underlain by about 13 feet of medium sand with traces of clay, then by 3 feet of compact fine clayey sand where a stiff plastic clay was encountered and continued to a depth of 36 feet. The stiff clay was then underlain by compact clayey sand which extended to the end of the soil borings. Table (5.1) presents the results of Brazilian penetration tests at the site of a structure and equivalent SPT values (Bazaraa , 1967).

The ground water table was observed at a depth of 14.7 feet below ground surface.

Structure and Foundation

This structure is an 18-story reinforced concrete building. The area of the floor is 8288 ft^2. The dead load of the building is about 14,800 tons and the live load is about 1,000 tons. The building is founded on spread footings covering 85 percent of the entire area. The base of the foundation is 14.7 ft. below ground surface.

Analysis: Case history 1

Table (5.1)

Results of Brazilian Penetration tests
at the site of Ipirange Building, Brazil

Boring 1		Boring 2	
Depth Below Ground Surface (ft)	Brazilian Penetration Value *	Depth Below Ground Surface (ft)	Brazilian Penetration Value
18	7 (11)	14.7	10 (16)
22.9	10 (16)	19.7	5 (8)
31.2	5 (8)	32.8	5 (8)
35.1	9 (11)	36.0	7 (11)
37.7	9 (11)	38.4	4 (6)
41.0	6 (10)	43.6	3 (5)
44.9	8 (13)	47.6	6 (10)
47.6	15 (24)	59.1	7 (11)
$Mean_1$	13	$Mean_2$	9

Total mean of both borings = 11

Standard deviation = 4.56

Coefficient of variation = 0.415

* Number between bracket refers to equivalent SPT value.

Loads and Settlements

The total load applied to the base of the footings was 2.25 tsf. The settlements were recorded for 30 reference points embedded in the columns. The final measured average settlement was 0.56 in.

Designer Approach:

Selected N value for the foundation design = 10

The same case history was analyzed by Jeyapalan and Boehm (1986). The N-value of 9 was assumed to be the most representative value for the foundation.

Proposed Algorithm:

1. SPT values within a depth of 2B from ground surface:

   The mean N values of all soil borings from ground surface to a depth of 2B = 11

   The Standard deviation = 4.56

   The Coefficient of variation (C) = 0.415

2. SPT values within a depth of 2B from the base of foundation:

   The mean N values of all soil borings (Ntavg) = 11

The minimum N-value of all soil borings (Nmin) = 5

The maximum mean N values of all soil borings (Nxavg) = 13

The minimum mean N values of all soil borings (Nmm) = 9

The design N-value is then determined as:

$$N_d = \frac{ERr}{60} * D_c * [\ A * C + (1 - C) * B\ ]$$

The parameter C is Coefficient of variation (Table 5.1).

The parameters A and B are assuming one of the following combinations.

First combination of parameters A and B

$A = N_{min} = 5$

$B = N_{xavg} = 13$

$N_d = [\ 5 * 0.415 + (\ 1\text{-}0.415\ ) * 13\ ] = 9.68$

Second combination of parameters A and B

$A = N_{min} = 5$

$B = N_{mm} = 9$

$N_d = [\ 5 * 0.415 + (\ 1\text{-}0.415\ ) * 9\ ] = 7.3$

Third combination of parameters A and B

$A = N_{min} = 5$

$B = N_{tavg} = 11$

$N_d = [\ 5 * 0.415 + (\ 1\text{-}0.415\ ) * 11\ ] = 8.51$

Fourth combination of parameters A and B

$$A = N_{mm} = 9$$

$$B = N_{xavg} = 13$$

$$\mathbf{N_d} = [\ 9 * 0.415 + (\ 1\text{-}0.415\ ) * 13\ ] = 11.34$$

Fifth combination of parameters A and B

$$A = N_{mm} = 9$$

$$B = N_{tavg} = 11$$

$$\mathbf{N_d} = [\ 9 * 0.415 + (\ 1\text{-}0.415\ ) * 11\ ] = 10.17$$

Table 5.2. Comparison between results of algorithm, common criteria and a designer N value for case history no.1

Designer approach	* proposed equation					Common criteria			
Selected N-value	(1)	(2)	(3)	(4)	(5)	Nmin	Nmm	Nxavg	Ntavg
10	10	7	9	11	10	5	9	13	11

* Number between brackets refers to the different combinations of parameters A and B in the proposed equation.

The detailed analysis of the case histories No. 2 to No. 21 are presented in the Appendix.

The detailed analysis of twenty one case histories of actual foundations on sand was used to examine five hypotheses in order to determine the two main parameters (A and B) of the proposed algorithm. Figures 16,17,18,19 and 20 show the comparison between selected N-design value in actual foundation design and values of each hypothesis using the proposed algorithm. The results of comparison suggest that all five hypotheses are reasonably accurate. However, the fifth hypothesis will be adopted in the proposed algorithm not only because it bears encouraging similarity to a designer approach in selecting N-value but also it combines the advantages of taking the effect of all N-values in consideration. It is interesting to note that the parameter A in the fifth hypothesis represents minimum criterion while the B parameter represents mean criterion. Both of these criteria were found to be, as presented in chapter 3, the most common used in selection of design N value. Moreover, it may be seen from analysis of these twenty one case histories that if the parameter A and B are defined to be minimum and mean N values respectively, (fifth hypothesis), then the algorithm will be conservative when scatter of SPT values are high, however the same algorithm will be less conservative when scatter of SPT values (N) is low. This is consistent with what majority of practicing and knowledgeable engineers will do.

To show that the adopted algorithm is a reasonable choice, more than 65 other case records of shallow foundations on sand were examined for checking its reliability. In this stage, however, the detailed analyses were omitted and only the necessary data were taken for analysis and verification of the fifth algorithm.

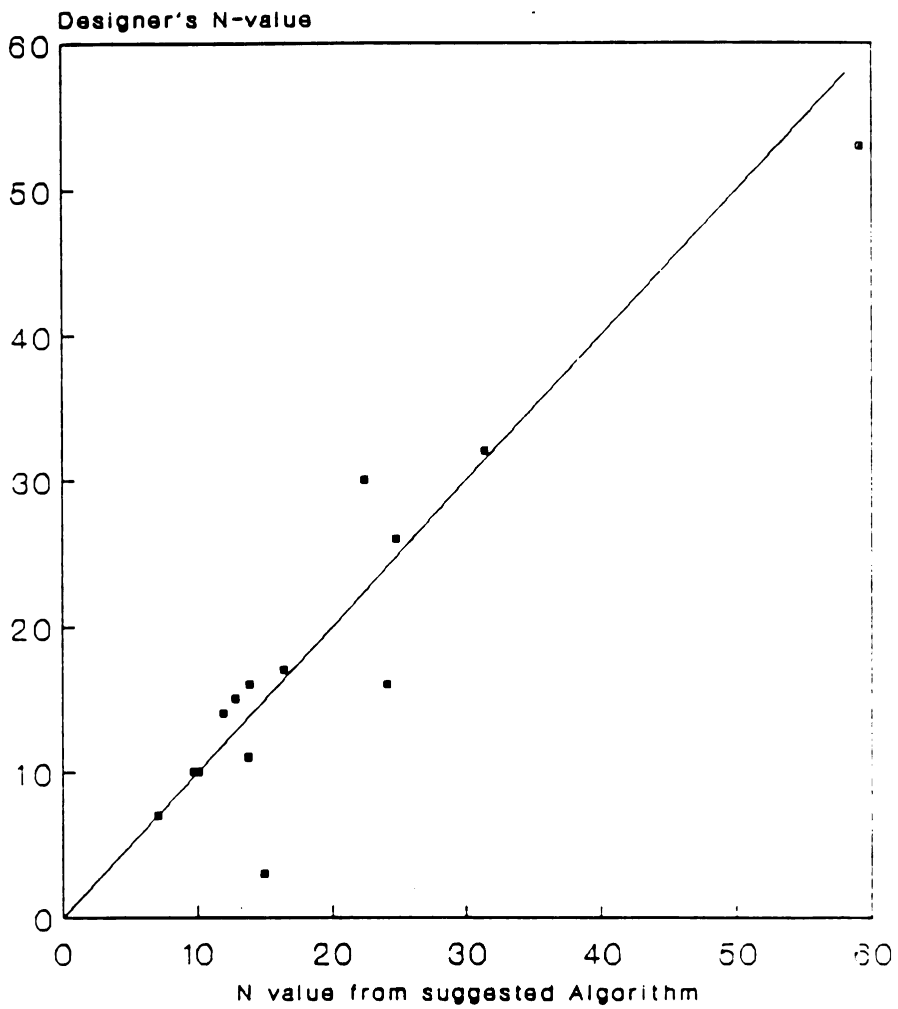


Figure 16. Results Of First Hypothesis Of The Proposed Algorithm

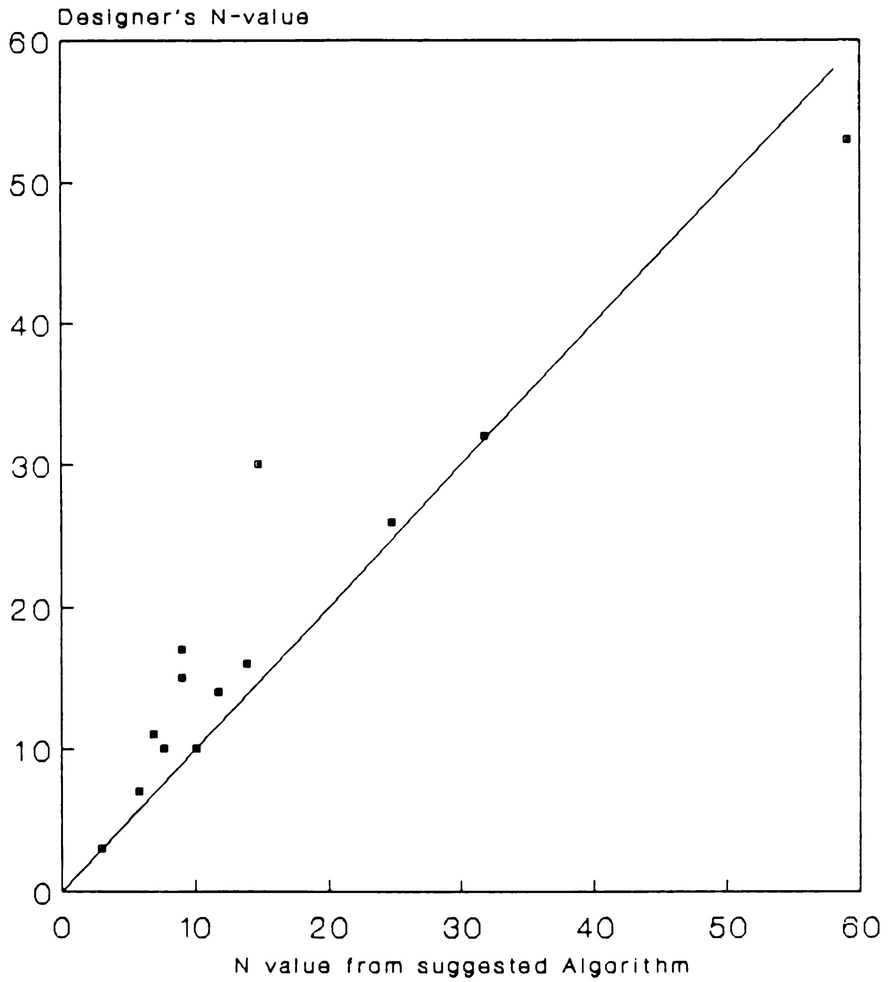


Figure 17. Results Of Second Hypothesis Of The Proposed Algorithm

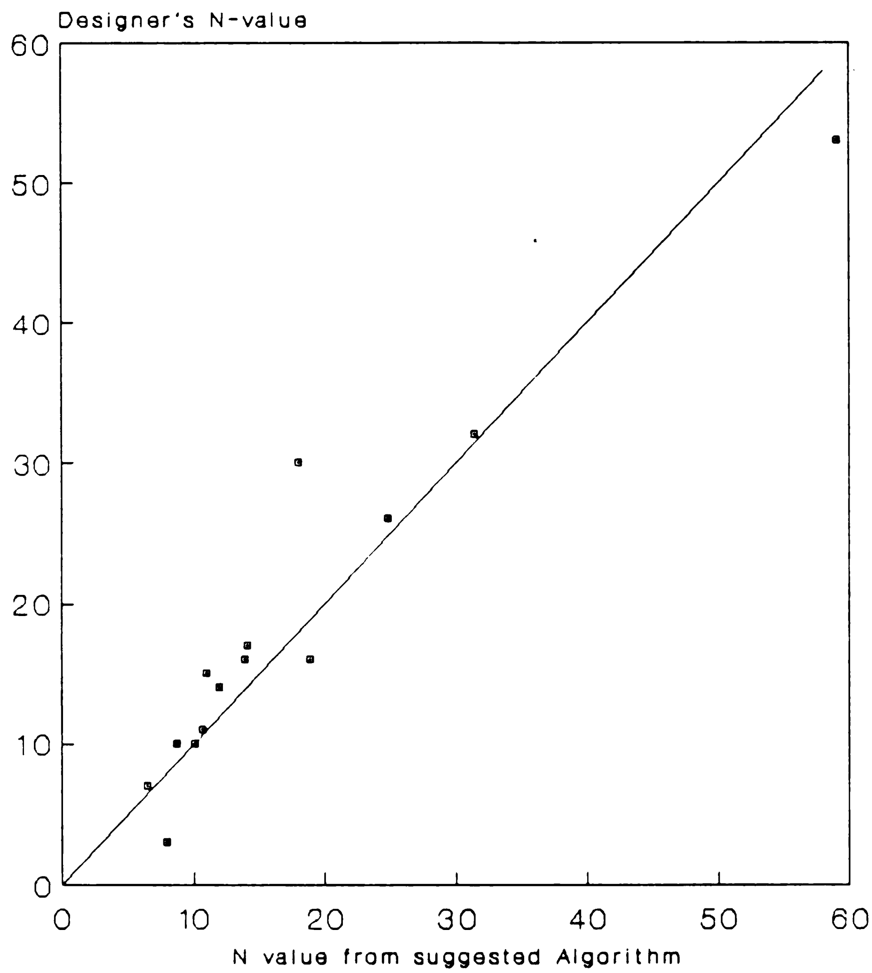


Figure 18. Results Of Third Hypothesis Of The Proposed Algorithm

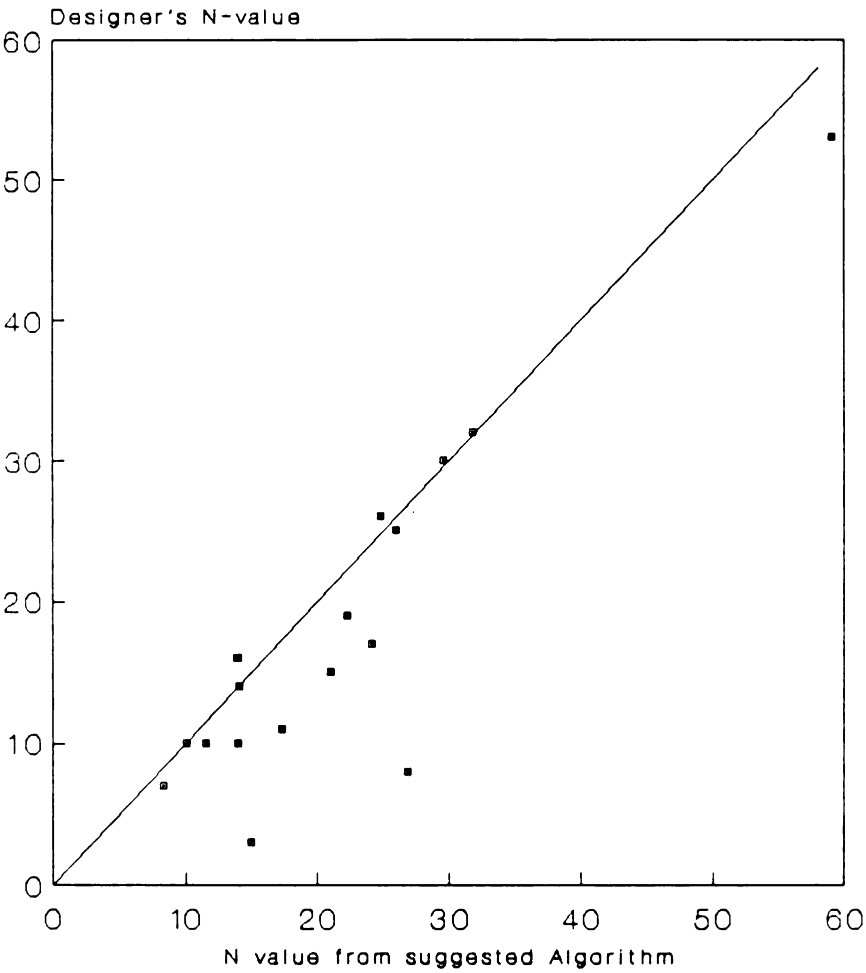


Figure 19. Results Of Fourth Hypothesis Of The Proposed Algorithm

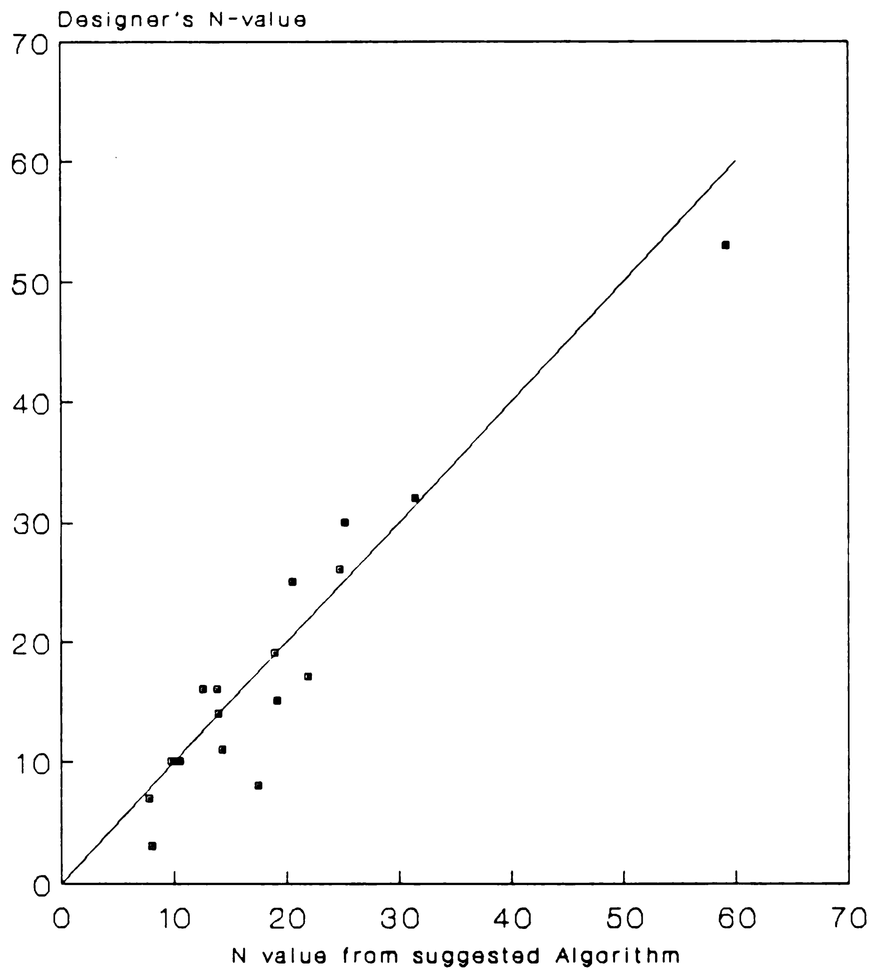


Figure 20. Results Of Fifth Hypothesis Of The Proposed Algorithm

B. Overview analysis:

To simplify the study of the data for the additional 65 case histories of shallow foundations on sand investigated in this study, summary data are presented in Table (5.3). These include data regarding the type and dimensions of foundation, depth of base below ground surface, depth of water table, computed N-design value by the proposed algorithm, and the N value chosen by a designer or other experienced and knowledgeable foundation engineers. The results of this investigation were plotted and presented in Figure (21). As may be seen from this figure, there is a remarkable agreement between results of proposed algorithm and those suggested by design engineer. The analysis of these case histories lends further support and confidence to the proposed method and to the use of the fifth hypothesis as a reasonable choice. It may be seen that in some cases there is a slight difference between the N-design value predicted by proposed algorithm and those recommended by a design engineer. It will be shown latter in this chapter that these differences might be a positive aspect of the proposed algorithm.

It is intended to deal with two main problems in the remaining part of this chapter. The first is to investigate the reliability of the proposed methods. The second is to investigate the effect of energy concept on the validity of the proposed settlement equation and those that are most widely used in predictions of settlements of shallow foundations.

Table 5.3: Data regarding case histories for settlements of shallow foundations on sand.

Case no.	Ref. no.	Type of Foundation	Dimention (ft) B	L	D	ground water table (ft)	Load (t/f)	SPT (bl/ft) * DE	** EQ	(Ko)	Exp(-Ko)	Settlement (in.) Meas.	Pred.
22	(102,13)	footing	8.5	35.0	3.28	-	2.99	37	34	0.40	0.670	0.43	0.38
23	(3, 6)	mat	120	-	0.00	4.92	2.10	26	24	0.40	0.670	0.71	0.47
24	(3, 6)	mat	40	-	3.90	23.94	1.62	14	13	0.40	0.670	0.86	0.63
25	(20,63,13)	footing	11.2	17.7	5.60	13.10	2.96	20	19	0.70	0.496	0.32	0.57
26	(20,63,13)	footing	12.1	19.4	5.90	13.10	1.5-3.10	20	19	0.70	0.496	0.48	0.30-0.7
27	=	footing	13.1	20.9	6.60	13.10	1.05-2.43	22	19	0.70	0.496	0.24-0.64	0.21-0.56
28	=	footing	14.1	22.6	6.90	13.10	1.1-1.74	22	19	0.70	0.496	0.24-0.36	0.22-0.40
29	=	footing	14.1	24.3	7.50	13.10	1.22-1.79	20	19	0.70	0.496	0.14-0.44	0.24-0.42
30	=	footing	16.0	25.6	8.20	13.10	1.05-2.15	20	19	0.70	0.496	0.20-0.32	0.21-0.51
31	=	footing	18.0	28.9	8.50	13.10	1.50	20	19	0.70	0.496	0.34	0.31
32	=	footing	20.0	32.0	9.84	13.10	1.74	20	19	0.70	0.496	0.37	0.36
33	=	footing	21.0	33.5	10.5	13.10	1.62	20	19	0.70	0.496	0.40	0.34
34	=	footing	22.0	35.1	11.15	13.10	1.22	21	20	0.70	0.496	0.57	0.26
35	=	footing	23.0	36.7	11.5	13.10	1.27	22	20	0.70	0.496	0.23	0.27
36	(13)	footing	11.5	11.5	4.92	0.00	0.27	12	11	0.40	0.670	0.11	0.11
37	(13)	mat	72.2	246	16.4	32.80	0.89	21	20	0.40	0.670	0.30	0.23
38	(13)	mat	72.2	-	16.4	32.80	0.86	21	20	0.40	0.670	0.41	0.22
39	(13)	footing	8.2	-	0.00	32.80	2.74	16	16	0.40	0.670	0.43	0.58
40	(13)	footing	3.28	-	0.00	32.80	2.65	16	16	0.40	0.670	0.39	0.42
41	(74)	footing	3.94	3.94	0.00	-	3.46	33	31	0.40	0.670	0.11	0.38
42	(71)	mat	36.0	73.5	56.0	0.00	3.67	23	21	0.40	0.670	0.47	0.89
43	(6,13)	strip	7.87	399.9	6.60	17.38	1.82	12	11	0.40	0.670	0.62	0.70
44	(6,13)	strip	15.1	399.9	6.60	17.38	2.03	12	11	0.40	0.670	0.75	0.87
45	(54,13)	footing	3.93	3.93	1.60	non	3.24	50	47	0.40	0.670	0.18	0.23

DE refers to N-value used by designer or other experienced foundations engineers.
EQ refers to N-value obtained by proposed algorithm.

Table 5.3 cont'.: Data regarding case histories for settlements of shallow foundations on sand.

Case no.	Ref. no.	Type of Foundation	Dimention (ft)			Ground Water Table	Load	SPT (bl/ft)		(Ko)	Exp(-Ko)	Settlement (in.)	
			B	L	D	(ft)	(t/f)	* DE	** EQ			Meas.	Pred.
46	(54,13)	footing	4.60	4.60	12.10	4.92	3.24	50	47	0.40	0.67	0.06	0.26
47	(54,13)	footing	2.95	2.95	3.90	12.10	3.24	30	29	0.40	0.67	0.16	0.33
48	=	footing	2.95	2.95	10.0	2.95	3.24	20	19	0.40	0.67	0.26	0.48
49	=	footing	2.95	2.95	3.93	5.90	3.24	20	19	0.40	0.67	0.11	0.48
50	(13)	mat	442.9	587.2	68.6	0.00	5.41	60	60	0.40	0.67	0.74	0.53
51	(100,99)	mat	57.7	275.2	35.0	0.00	2.60	20	19	0.40	0.67	0.83	0.67
52	(100,99)	mat	52.5	141.1	23.6	8.20	2.47	14	13	0.40	0.67	0.71	0.91
53	(100,99)	-	67.3	-	11.5	24.6	1.87	10	10	0.40	0.67	0.38	0.88
54	(100,13)	footing	47.6	47.6	11.5	0.00	2.76	26	22	0.40	0.67	0.61	0.54
55	(102,13)	strip	8.5	35.1	3.28	0.00	3.17	37	34	0.40	0.67	0.43	0.40
56	(112,13)	-	80	-	0.00	0.00	1.30	27	27	0.40	0.67	0.56	0.25
57	(112,13)	footing	6.89	7.87	7,87	0.00	6.32	50	47	0.40	0.67	0.17	0.58
58	(112,13)	footing	6.89	6.89	4.92	0.00	7.53	50	47	0.40	0.67	0.10	0.69
59	(42)	mat	111.5	187	25.9	0.00	2.92	30	29	0.40	0.67	0.86	0.53
60	(70,63)	mat	77.4	88.2	9.80	0.00	1.81	35	32	0.40	0.67	0.61	0.30
61	(70,63)	footing	5.90	5.90	9.80	0.00	2.48	25	22	0.40	0.67	0.14	0.42
62	(70,63)	footing	4.59	4.59	9.80	0.00	2.48	25	22	0.40	0.67	0.16	0.39
63	(70,63)	footing	7.20	-	9.80	0.00	3.07	25	22	0.40	0.67	0.41	0.54
64	(70,63)	footing	14.76	18.7	9.80	0.00	2.10	35	32	0.40	0.67	0.16	0.31
65	(70,63)	strip	49.20	239	9.80	8.52	0.88	35	32	0.40	0.67	0.21	0.14
66	(70,63)	strip	4.59	-	1.31	8.52	2.44	25	22	0.40	0.67	0.30	0.38
67	(70,63)	strip	5.25	41.3	1.31	8.85	2.70	25	22	0.40	0.67	0.37	0.45
68	(70,63)	strip	3.93	41.66	0.98	8.85	2.70	25	22	0.40	0.67	0.39	0.40
69	(70,63)	strip	2.62	60.36	0.98	8.85	3.18	25	22	0.40	0.67	0.23	0.39
	(70,63)	strip	5.91	79.1	0.98	0.00	2.23	25	22	0.40	0.67	0.66	0.40

* refers to N-value used by designer or other experienced foundations engineers.
EQ refers to N-value obtained by proposed algorithm.

Table 5.3 cont'.: Data regarding case histories for settlements of shallow foundations on sand.

Case no.	Ref. no.	Type of Foundation	Dimention (ft) B	L	D	Ground Water Table (ft)	Load (t/f)	SPT (bl/ft) DE*	EQ**	(Ko)	Exp(-Ko)	Settlement (in.) Meas.	Pred.
71	(114,13)	strip	26.9	200.0	-	21.30	0.378	5	5	0.40	0.67	0.75	0.41
72	(114,13)	mat	99.0	101.0	8.90	0.00	4.175	18	17	0.40	0.67	0.98	1.20
73	(6,86,13)	mat	73.8	213.0	32.8	32.80	2.65-3.19	20	19	0.40	0.67	0.59-1.1	0.68-0.83
74	(88,13)	-	32.8	-	4.92	4.90	2.60	60	54	0.40	0.67	0.275	0.262
75	(66,63,6)	footing	3.60	3.60	3.90	4.90	0.85	13	12	0.40	0.67	0.08	0.23
76	(66,63,6)	footing	4.92	4.92	3.90	4.90	0.83	13	12	0.40	0.67	0.08	0.23
77	(66,63,6)	footing	4.92	4.92	3.90	0.00	0.83	13	12	0.40	0.67	0.05	0.23
78	(36,6,63,99)	mat	42.6	103	6.8	0.00	2.10	18	17	0.40	0.67	0.82	0.60
79	(36,6,63,99)	mat	42.6	89.9	6.8	5.90	2.10	18	17	0.40	0.67	0.70	0.60
80	=	footing	42.6	73.8	6.8	5.90	2.10	18	17	0.40	0.67	0.55	0.60
81	(13,52)	-	60.0	-	0.98	18.4	0.44	20	19	0.40	0.67	0.19	0.13
82	=	-	49.8	-	0.98	5.90	0.36	20	19	0.40	0.67	0.11	0.10
83	(54,13)	footing	13.1	22.9	16.4	18.3	5.60	37	34	0.40	0.67	0.47	0.76
84	(6,13)	footing	3.93	3.93	8.50	8.20	2.33	29	28	0.40	0.67	0.10	0.27
85	=	footing	3.93	3.93	8.50	8.20	2.33	26	26	0.40	0.67	0.06	0.26
86	=	footing	3.93	3.93	8.50	8.20	2.33	18	17	0.40	0.67	0.34	0.44

* DE refers to N-value selected by designer or other experienced engineers.

** EQ refers to N-value obtained by proposed algorithm.

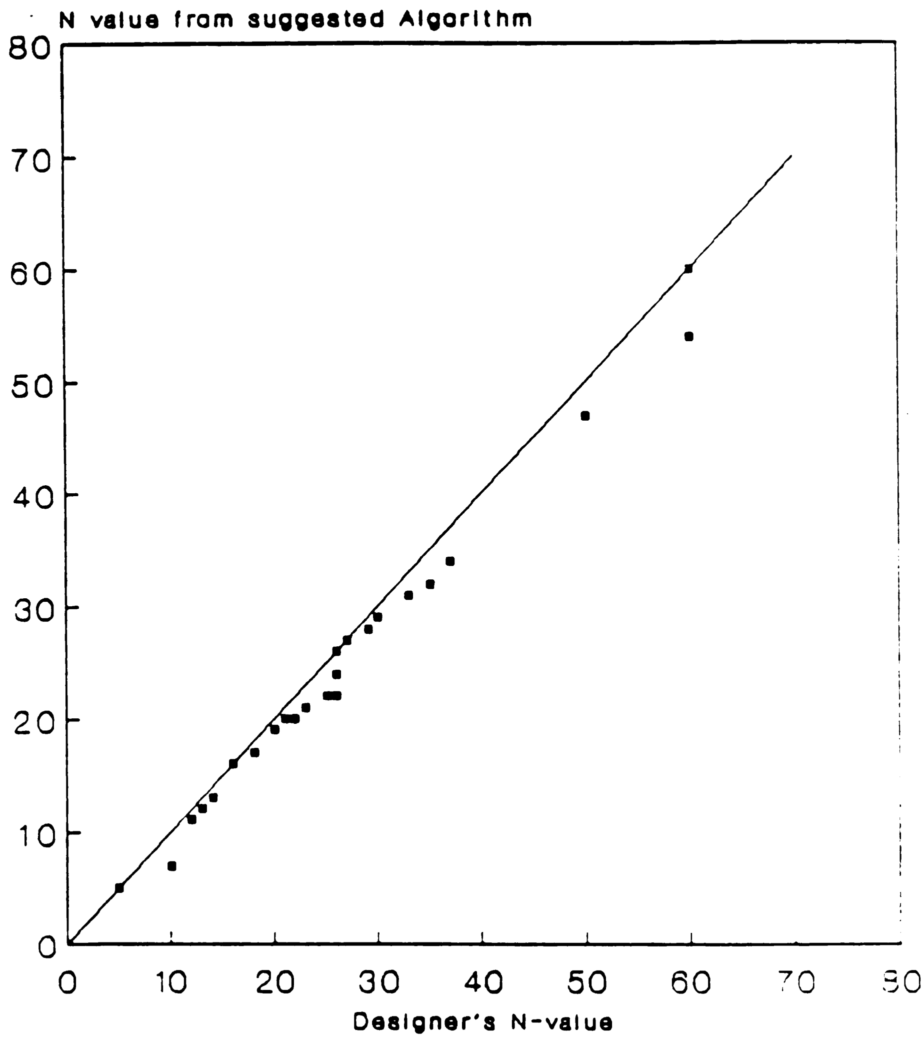


Figure 21. Final Results Of The Adopted Hypothesis Of The Proposed Algorithm

5.3 Reliability of the Proposed Methods:

Chapter 3 includes a study of soil stress history and presents a new approach for estimating settlements of foundations on sand. This method assumes that the N-design value is selected by the proposed algorithm and that the coefficient of earth pressure (Ko) is known with reasonable accuracy. These case histories, therefore, constitute a means for investigating the reliability of the proposed method and whether or not the adopted algorithm has made any significant improvement in settlement predictions of foundations on sand. Four different settlement methods, which are frequently used in foundations design, together with the proposed method are used in this investigation. Equation (1) represents Bazaraa and Peck's method, equation (2) represents Meyerhof's method, equation (3) represents D'Applonia et al. method, equation (4) represents Peck et al. method whereas equation (5) represents the suggested method. Figures 22,23,24,25, and 26 present a comparison between measured and computed settlements by means of these five methods for all 87 case histories presented in this study. These figures show clearly that Peck et al. method for estimating settlements of shallow foundations on sand is very conservative. If stress history represented by coefficient of earth pressure (Ko) is used in conjunction with N-value selected by proposed algorithm, the settlement predictions become much more reasonable. Both Meyerhof equation and Bazaraa's equation tend to overestimate the settlements whereas D'Applonia's method seems to seriously underestimate the settlements in some cases. All the available

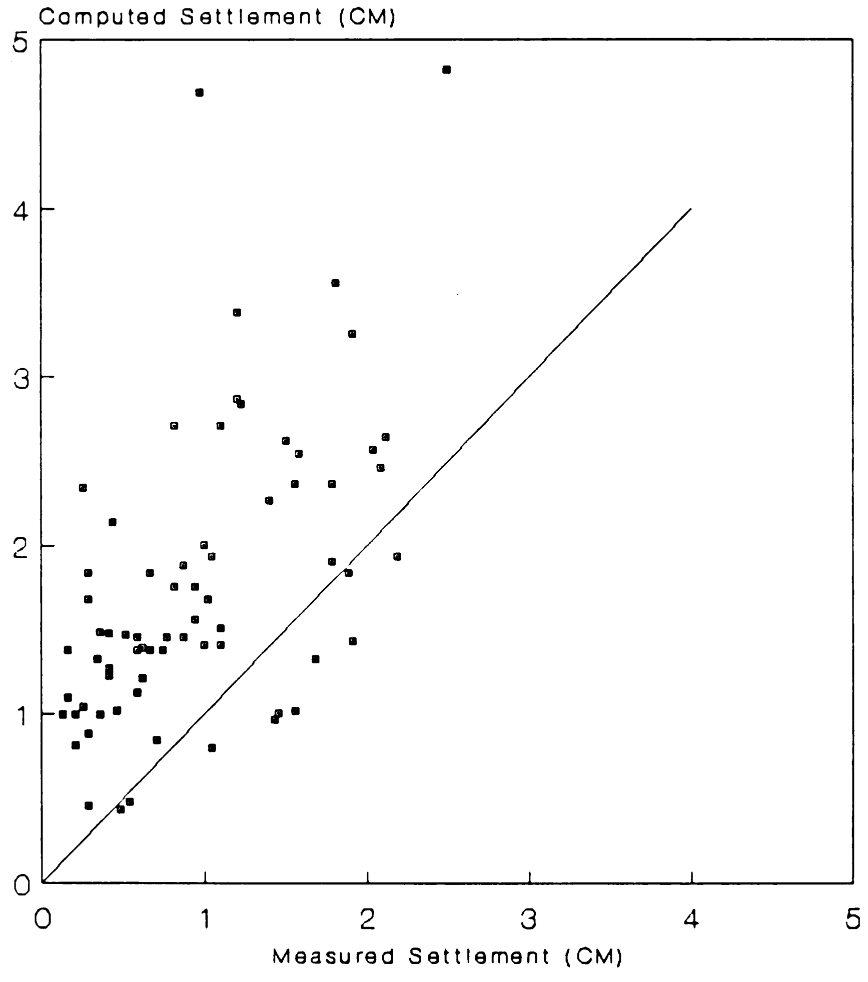


Figure 23. Comparison Of Measured Settlements With Values Computed By Meyerhof Method

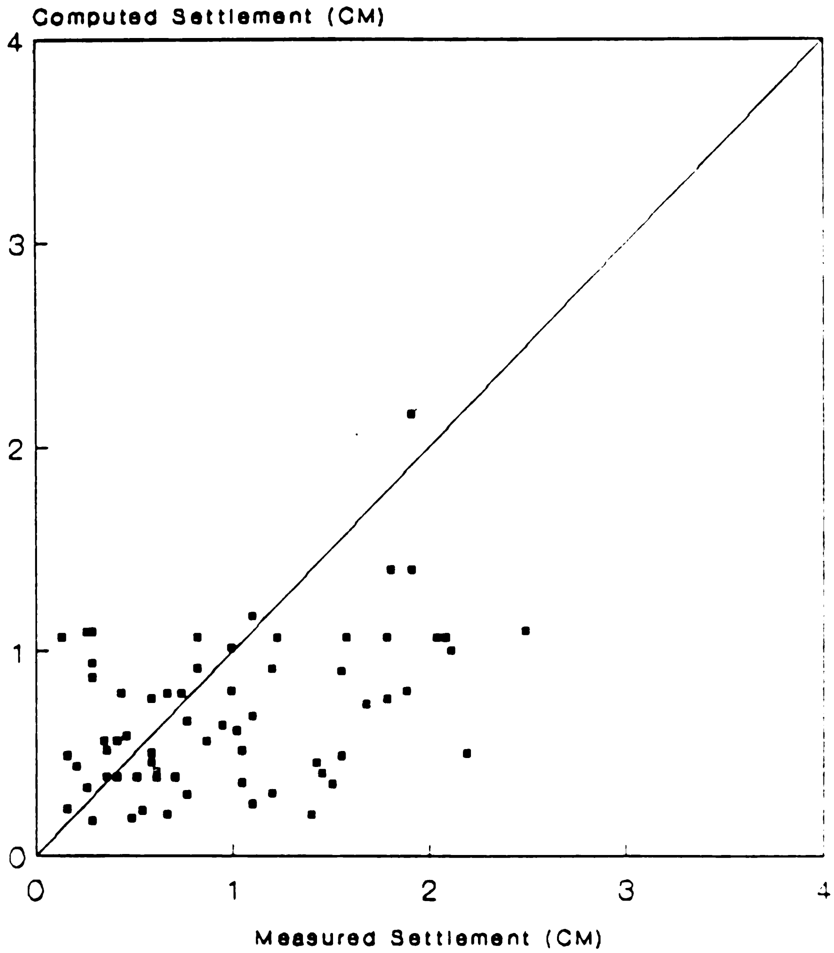


Figure 24. Comparison Of Measured Settlements With Values Computed By D'Applonia Method

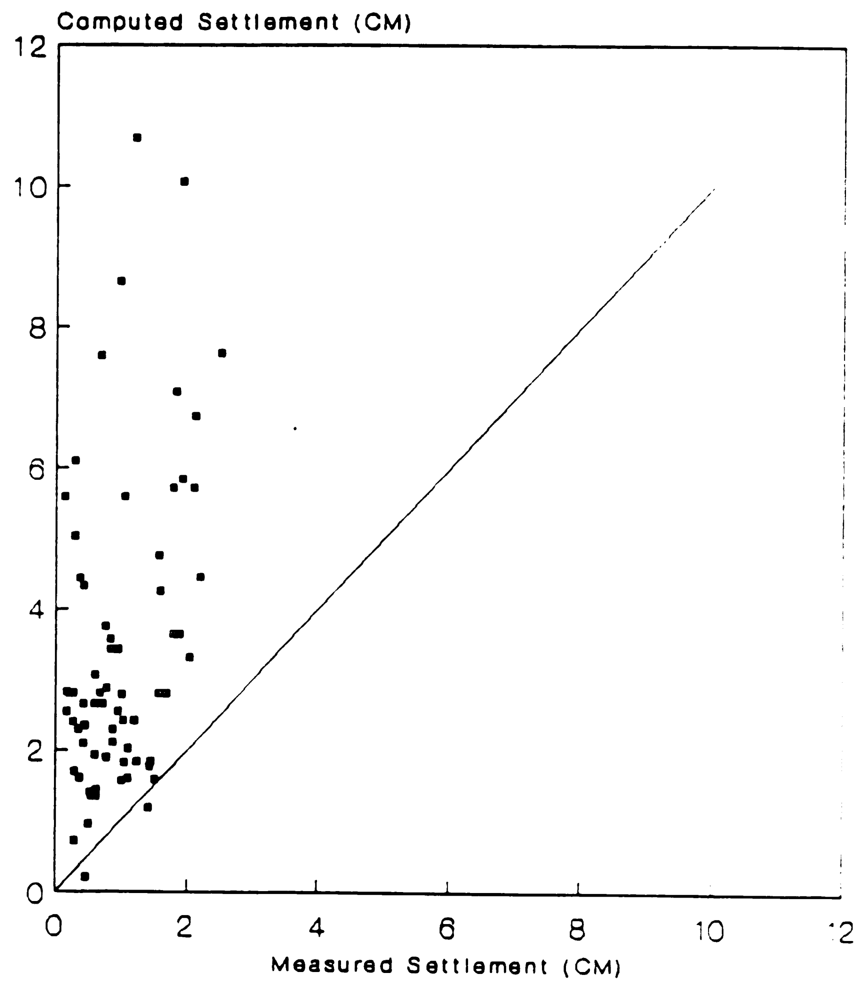


Figure 25. Comparison Of Measured Settlements With Values Computed By Peck et al. Method

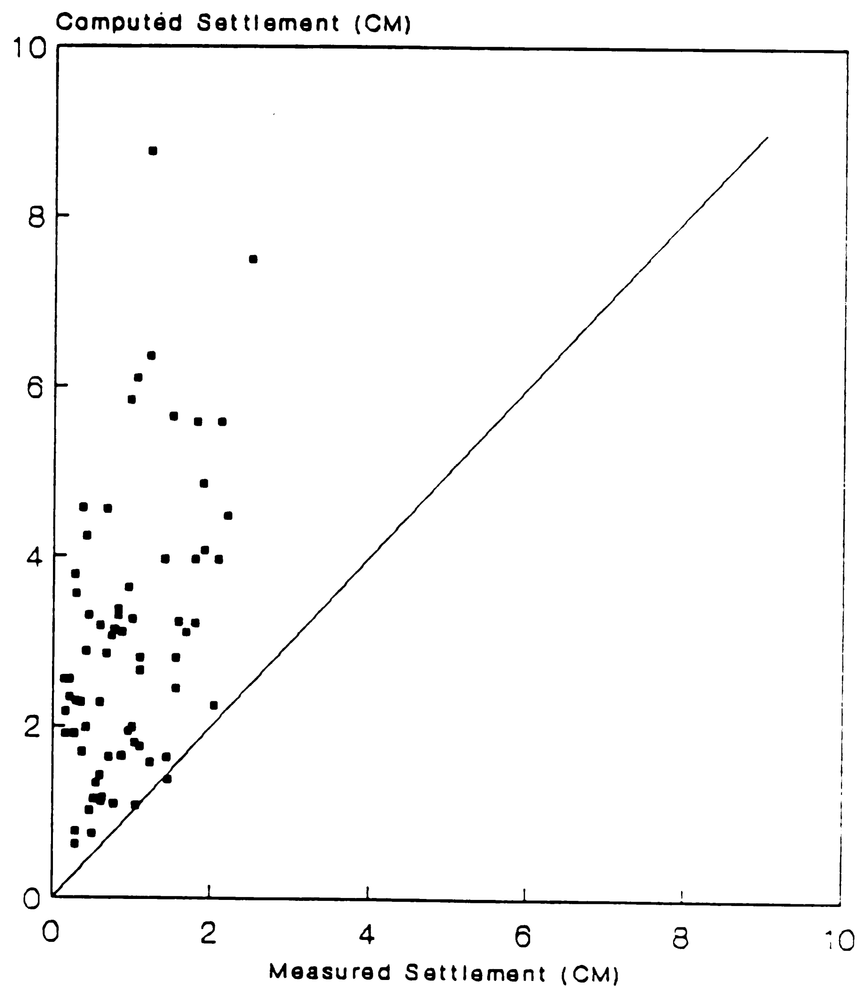


Figure 26. Comparison Of Measured Settlements With Values Computed By Bazaraa Method

results for the ratios of computed to measured settlements (SP/SM) for all case histories are presented in Table (5.4). These results indicate clearly that the Peck et al., (1974) method is very conservative. This method predicts 1 to 5.6 times the measured settlement values. The Bazaraa and Peck method given by equation (3.8) predicts, on the average, a settlement 1.1 to 4.6 times the measured values. The Meyerhof's method given by equation (3.11) predicted on the average, a settlement 1 to 2.73 times the measured value. The D'Applonia method given by equation (3.12) predicts, on the average, a settlement 0.82 times the measured values. However, this method has two disadvantages. The first is that it tends to underestimate the settlement. The second disadvantage is that in many cases this tendency is so significant that the computed settlement is only 0.22 to 0.64 times the measured value.

The suggested method given by equation (3.16) shows that it is not only a reliable method but it is significantly better than the other four settlement methods used for comparison. On the average, the settlements predicted by means of this method were 1.65 times the measured values. The suggested procedure underestimated the settlements in 23 out 87 cases. However, the ratio of predicted to measured settlements for these 23 cases, on the average, was about 0.89. This is considered to be acceptable for most practical foundation engineering problems. It is important to note that the soil foundation for those cases with unknown past stress history were assumed to be normally consolidated and the Ko value of 0.4 was used in settlement calculations by the suggested method.

Table 5.4. Comparison Of Settlement Predictions By Different Methods

Prediction Method	Mean SP/SM	Standard Deviation	$(SP/SM)_{10}$*	$(SP/SM)_{90}$**
Suggested	1.65	1.24	0.85	1.7
Meyerhof	2.73	2.10	1.52	2.7
Bazaraa	4.59	3.96	1.80	6.9
D'Applonia	0.84	1.49	0.35	0.7
Peck et al.	5.56	6.52	2.00	8.0

* for which 10% of observations are smaller.

** for which 90% of observations are smaller.

Such assumptions might be the cause of some discrepancies between predicted and measured values of settlements. Nevertheless, the results of this investigation proved that the suggested method is consistent and reasonably accurate. The method is simple. It takes into account the effect of soil stress history and it utilizes the most representative N-value at the site of a structure by using the proposed algorithm. Considering the complexity of soil and the empirical nature of the equation, no doubt this is a step in right direction and an improvement in foundation design.

5.4 Effect of Energy on the Reliability of the Proposed Methods:

In chapter two, the effect of energy on standard penetration test results was studied and it is included in the proposed algorithm. It is therefore, deemed necessary to investigate such an effect on settlement predictions. For this investigation, the previous four methods together with a suggested method were used. The standard penetration results for soil foundations of the case histories presented in this study were assumed to have been obtained by the original American donut hammer. This assumption is reasonable because this type of hammer was common and the most used in soil investigation during the time when the foundations of the these case histories were explored. The results of this investigation are presented in Figures 27,28,29,30 and 31. These figures show that all methods tend to overestimate the measured settlements. However, the proposed method remained to be consistent and more reasonable than the other four methods used for comparisons. Based on these results, it appears that the existing settlement methods can seriously overestimate the measured value if the SPT results are

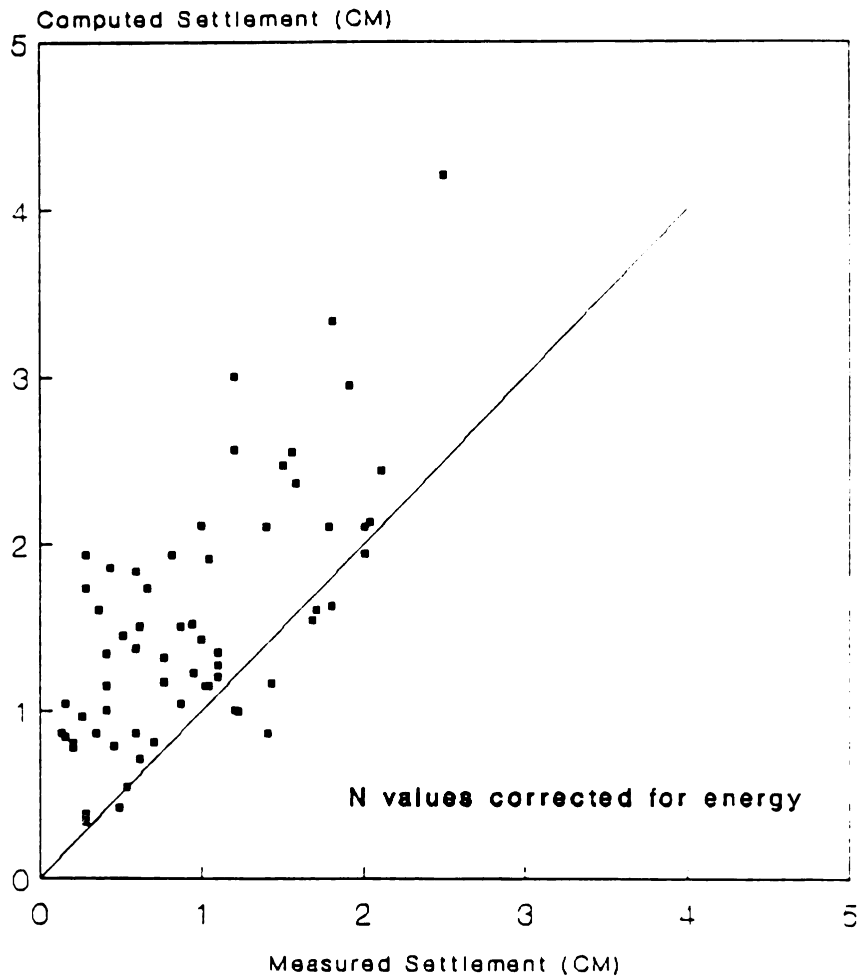


Figure 27. Comparison Of Measured Settlements With Values Computed By Suggested Method

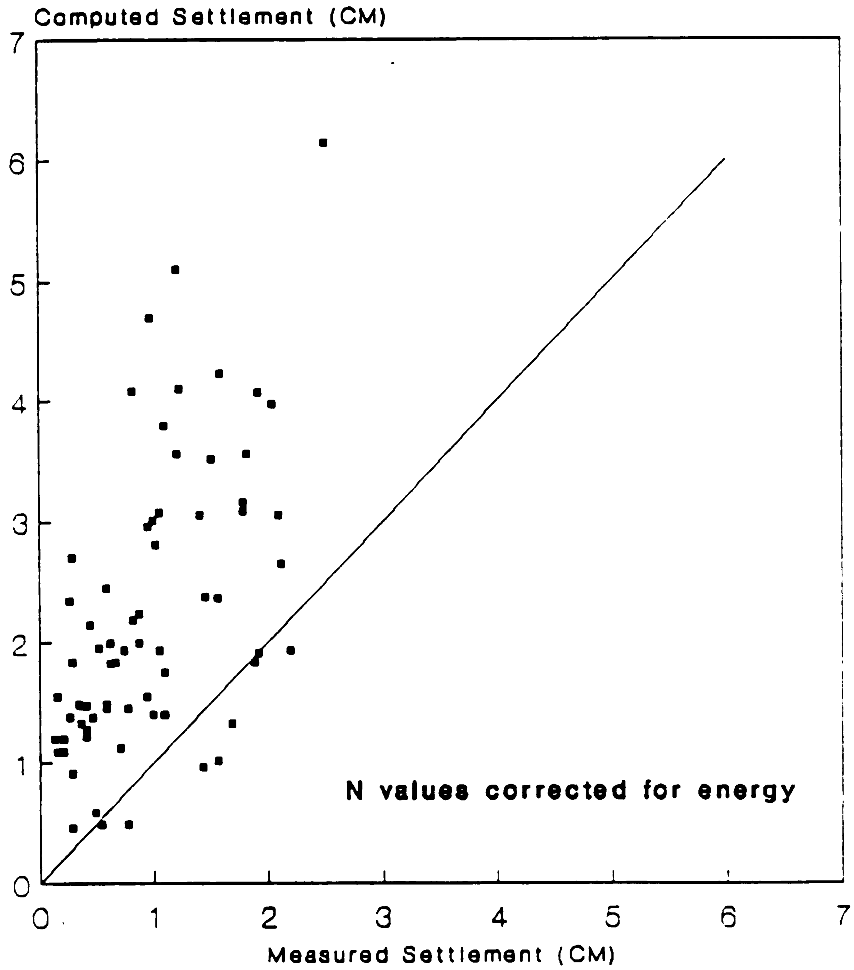


Figure 28. Comparison Of Measured Settlements With Values Computed By Meyerhof Method

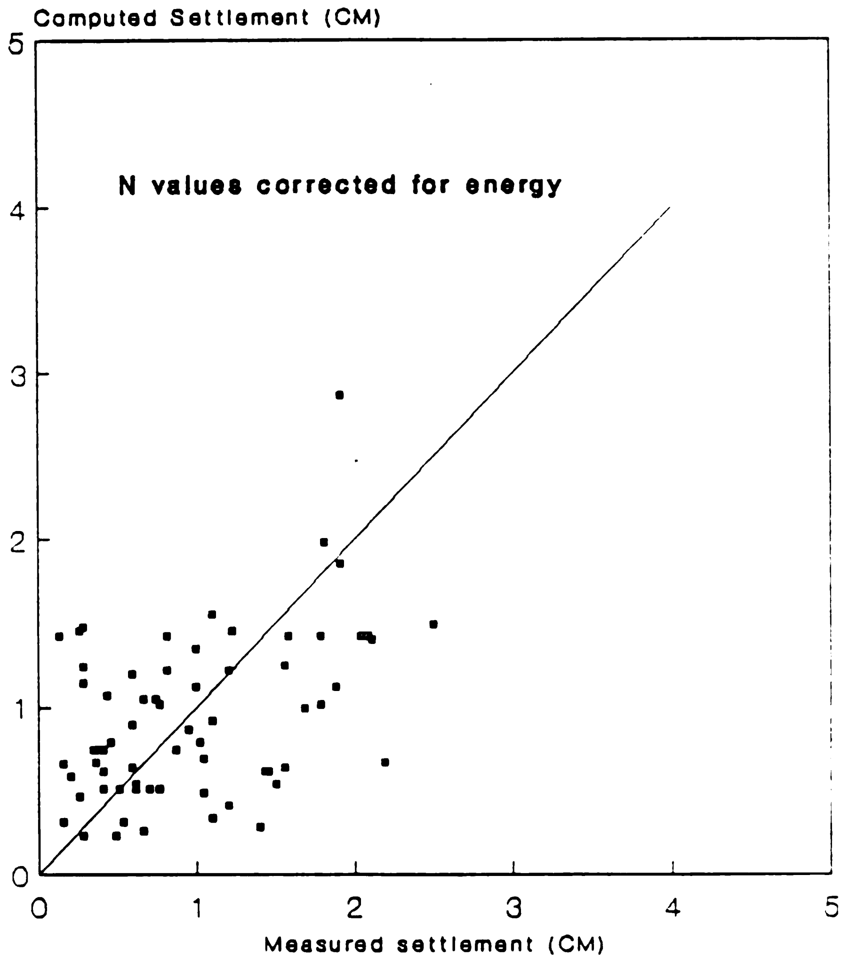


Figure 29. Comparison Of Measured Settlements With

Values Computed By D'Applonia Method

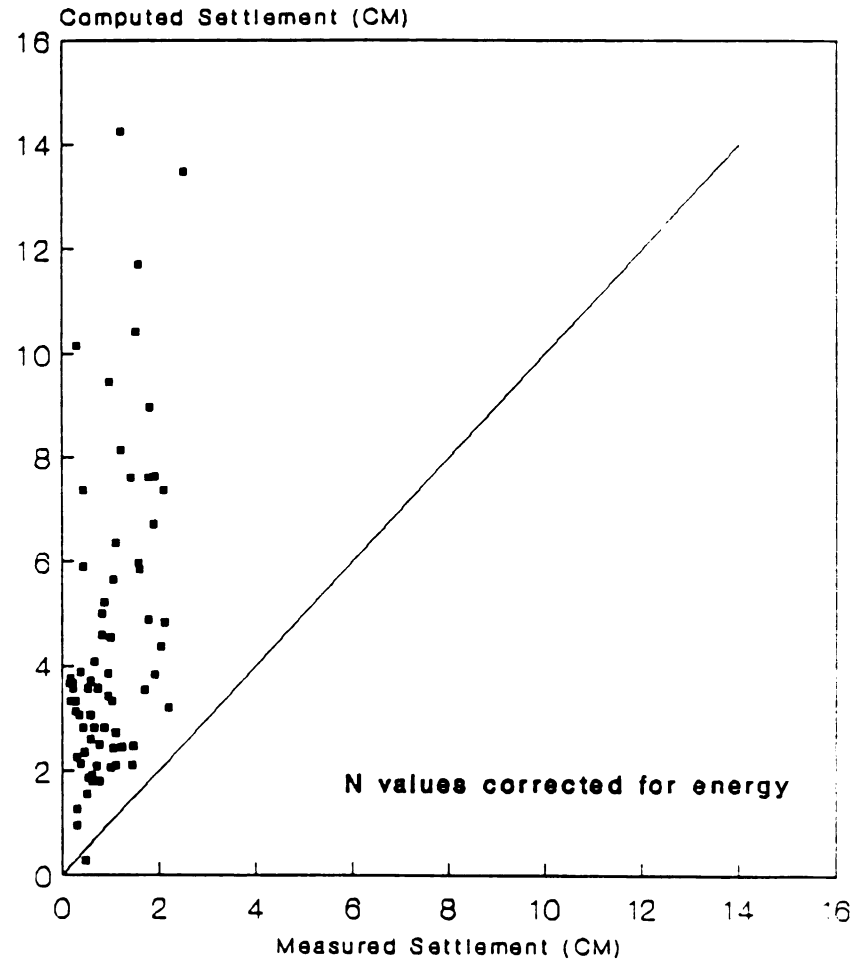


Figure 30. Comparison Of Measured Settlements With Values Computed By Peck et al. Method

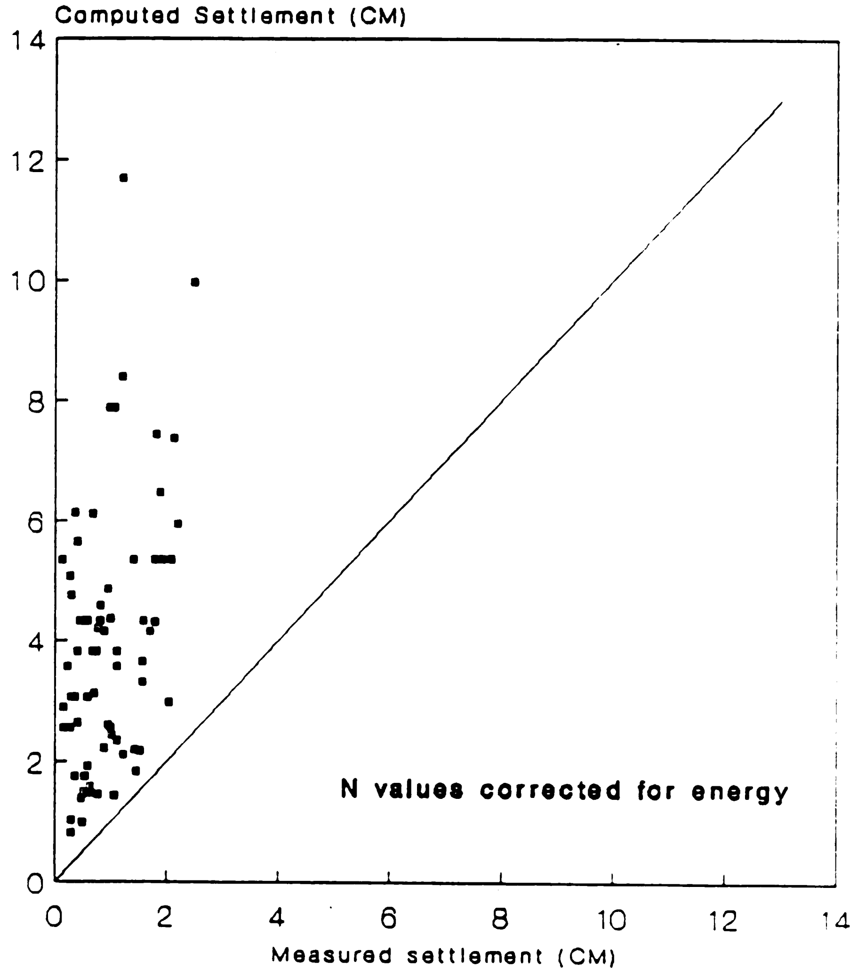


Figure 31. Comparison Of Measured Settlements With Values Computed By Bazaraa Method

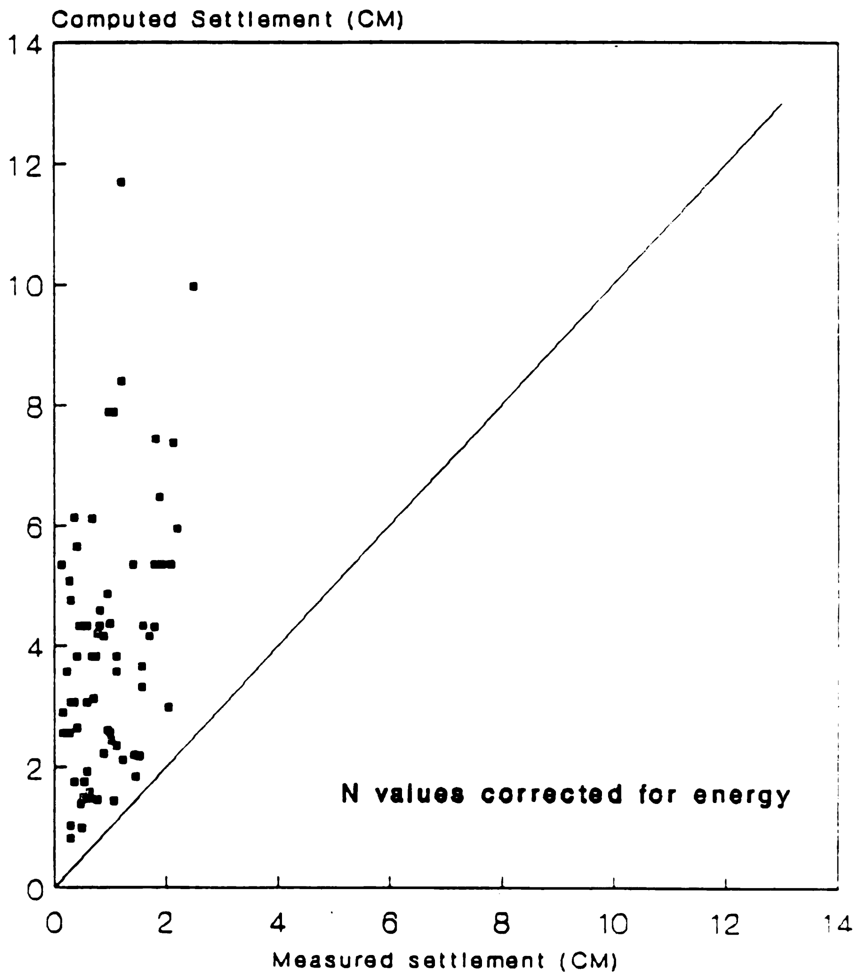


Figure 31. Comparison Of Measured Settlements With Values Computed By Bazaraa Method

SPT results are corrected for energy. Although no conclusive statement can be made due to lack of enough information about energy, the results of this investigation did suggest that the proposed settlement method is consistent and reliable. A need for future study regarding the effect of energy correction on the SPT-based settlement procedures seems to be warranted.

CHAPTER 6

SUMMARY AND CONCLUSIONS

6.1 Summary

In-situ tests form an essential part of foundation exploration at the onset of a project. Of those in-situ tests, the standard penetration test (SPT) is still the most common method at the present time. This method is of practical interest because it is simple, cost effective and allows the material penetrated to be visually identified. It is the results of SPT that are correlated with virtually every soil parameter used for evaluating either settlement or bearing capacity of foundations. Due to a wide variation of SPT results, a higher degree of engineering judgement is required to select an appropriate design N-value. Any misinterpretation of SPT data may lead to unfavorable circumstances. It is, therefore, important to be able to select a truly representative N-value that can be used in predicting soil foundation performance.

The factors that influence SPT were discussed and the different methods for selecting design N-value for foundations on sand were compared. It was noted that interpretation of soil boring data and selection of a representative N-value depends to a great extent on the experience and the knowledge of the foundation engineer rather than on a specific procedure or method.

To formulate an algorithm for consistent selection of a design N-value, the design N-value was taken to be a function of the pertinent variables. It was found that it is the scatter of SPT results that is the essential part in the interpretation and selection of the design N-value.

A mathematical algorithm that includes the effect of SPT variation was then developed. Five hypotheses were presented to determine the main variables in the proposed algorithm. Numerous case histories for actual foundations design on sand were analyzed by the proposed algorithm. In all cases the proposed algorithm was found to be superior with respect to current recommended procedures for selecting a design N-value in foundation engineering.

Foundations have to meet certain criteria that ensure safety against bearing failure and minimal settlement. Foundations on sand are usually controlled by settlement considerations rather than bearing capacity. It is therefore, necessary to be able to estimate foundation settlements with reasonable accuracy. The factors that influence the compressibility of sand were discussed and the different available correlations between soil compressibility and standard penetration test results were compared. It was found that no unique correlation exists and the soil modulus can assume significantly different values for the same SPT resistance value depending on which correlation is used. The importance of soil stress history with respect to soil compressibility was discussed. The relationship between foundation settlements and coefficient of earth pressure (Ko) was shown to fit an exponential equation and a new procedure for estimating foundation settlements on sand was proposed. The new proposed procedure includes the effect of soil stress history in the form of coefficient of earth pressure (Ko) and SPT resistance value that is selected by the proposed algorithm.

A variety of case histories of actual settlements for foundations on sand were presented and analyzed for checking the reliability of the proposed settlement procedure. The results of this investigation suggest that the proposed method is reliable and can be used in actual foundation design with confidence. The effect of energy on SPT results and ultimately on settlement predictions of foundations on sand was then investigated.

6.2 Conclusions:

Based on the results of this study and from field data and laboratory tests by other researchers, the following conclusions are made:

1. Coefficient of earth pressure (Ko) is an important soil parameter that should not be neglected in evaluation of foundation settlements on sand. This parameter is a function of soil stress history and it can be assumed to have an exponential relationship with soil settlements.

2. Evaluation of soil modulus (E) from results of the standard penetration test can be misleading unless soil stress history is known. Any correlation between E and SPT results should be treated with caution.

3. Correction of SPT results for effects of energy does not seem to significantly improve settlement predictions if the existing SPT-based settlement procedures are used.

4. A mathematical algorithm for selecting the design N-value from results of standard penetration test was proposed. The proposed algorithm can be summarized as follows;

a: Perform standard penetration tests, preferably at a number of locations within the site of a structure. Present the SPT results as a function of a depth for each soil boring(s).

b: Obtain the average of all SPT results from all soil borings that are within a distance B-2B from bottom of footing(s) and call it Ntavg. In case there is only one soil boring, then Ntavg is replaced by the average of N-values within above mentioned distance, in the main equation.

c: Calculate coefficient of variation for entire SPT results that are within a significant depth (within a distance B-2B). If sufficient SPT measurements (N-values) are not available, then assume coefficient of variation of 30% for N-values. Assign the result of this step to variable C.

d: Determine mean value of SPT results in each soil boring. The SPT results are those that are within a distance of B-2B from the bottom of footing(s).

e: Locate the minimum mean value among those calculated in (d) and call it Nmm. In case there is only one soil boring, which is not a common engineering practice, select the the minimum N value in the soil boring that is within the same distance as in (d) and call it Nmin.

f: Select the N value for footing design according to the following equation:

$$N_d = \frac{ERr\%}{60} * [\ Nmm * C + (1 - C) * Ntavg\ ]$$

g: Compute the foundation's settlement according to the following equation:

$$S = \frac{2 * P}{N_d} \left(\frac{2 * B}{B + 1} \right) * Exp\,(- K_o)$$

Both equations appeared to be reliable and consistent, for all case histories of foundations analyzed in this study, as shown in Figure (21) and (22) respectively.

Suggestions for Future Work:

For future research, investigation of the following topics appears to be warranted:

1. Since the standard penetration test does not reflect the soil stress history to any significant degree, it is of great interest to investigate in more detail the effect of changing the coefficient of earth pressure on settlements of footings on sand. This probably can be done in the laboratory by constructing small models of footings.

2. Investigate the effect of anvil size on the efficiency of hammer in SPT. This probably can be investigated theoretically. The results of such investigation may lead to design a more efficient hammer and eliminate the need for SPT results to be corrected for energy effect.

APPENDIX

Case History No.2: Banco do Brasil Building

Rios and Silvas (1948),Vergas(1961),Meryerhof(1965),Bazaraa(1967),and Jeyapalan and Boehm(1986) reported on design of the Banco do Brasil,Sao Paulo,Brazil.

Soil Conditions

The soil consisted of 30 to 60 ft of alternating layers of sand, clayey sand and silty clay underlain by sand which continued to a depth of more than 90 ft below ground surface. Table 5.5 presents the results of Barazilian penetration tests for both soil borings at the site of a structure and equivalent SPT values (Bazzaraa, 1967).

The ground water table was observed at a depth of 24.60 feet below ground surface.

Structure and Foundation

The Banco do Brasil is 25-story bank building occupying an area of about 16146 ft^2. The building is founded on a reinforced concrete mat. The mat is 73.8 ft wide and 213 ft long, and founded at a depth of about 19.68 ft below ground surface.

Analysis: Case history 2

Table (5.5)

Results of Brazilian Penetration tests at the site of Banco do Brasil Building, Brazil *

Boring 1		Boring 2	
Depth Below Ground Surface (ft)	Brazilian Penetration Value	Depth Below Ground Surface (ft)	Brazilian Penetration Value
22.9	10 (16)	22.9	5 (8)
26.2	11 (17.6)	26.2	3 (4.8)
32.8	5 (8)	32.8	5 (8)
36.1	25 (40)	36.1	5 (8)
49.2	15 (24)	49.2	5 (9.6)
52.5	17 (27.2)	52.5	6 (12.8)
55.8	10 (16)	55.8	8 (12.8)
59.1	7 (11.2)	59.1	15 (24)
65.6	20 (32)	65.6	9 (14.4)
68.9	15 (24)	68.9	10 (16)
75.5	13 (20.8)	75.5	2 (3.2)
82.0	15 (24)	82.0	23 (36.8)
88.8	26 (41.6)	88.8	22 (35.2)
Meam	26.08		17.28

* The numbers between parentheses are equivalent to SPT values.

Loads and Settlements

The total load applied to the base of the mat of this building was estimated to be between 2.65 tsf to 3.19 tsf with an average of a bout 2.96 tsf. The minimum and maximum measured settlements were 0.53 in. and 1.1 in. respectively.

Designer Approach:

Selected N value for the foundation design = 15

The same case history was analyzed by Jeyapalan and Boehm(1986). The N-value of 15 was assumed to be the most representative value for the foundation.

Proposed Algorithm:

1. SPT values within a depth of 2B from ground surface;

   The mean N values of all soil borings from ground surface to a depth of 2B = 19

   The Standard deviation = 11.103

   The Coefficient of variation (C) = 0.582

2. SPT values within a depth of 2B from the base of foundation;

   The mean N values of all soil borings (Ntavg) = 21.68

The minimum N-value of all soil borings (Nmin) = 3

The maximum mean N values of all soil borings (Nxavg) = 26

The minimum mean N values of all soil borings (Nmm) = 17

The design N-value is then determined as ;

$$N_d = \frac{ERr}{60} * D_c * [\ A * C + (1 - C) * B\ ]$$

The parameter C is Coefficient of variation (Table 5.5).

The parameters A and B are assuming one of the following combinations.

First combination of parameters A and B

$A = N_{min} = 3.2$

$B = N_{xavg} = 26.08$

$N_d = [\ 3.28 * 0.582 + (1\text{-}0.582) * 26\ ] = 12.76$

Second combination of parameters A and B

$A = N_{min} = 3.2$

$B = N_{mm} = 17.28$

$N_d = [\ 3.2 * 0.582 + (1\text{-}0.582) * 17.28\ ] = 9.08$

Third combination of parameters A and B

$A = N_{min} = 3.2$

$B = N_{tavg} = 21.68$

$N_d = [\ 3.2 * 0.582 + (1\text{-}0.582) * 21.68\ ] = 10.92$

Fourth combination of parameters A and B

$A = N_{mm} = 17.28$

$B = N_{xavg} = 26.08$

$$N_d = [\ 17.28 \ * 0.582 + (\ 1\text{-}0.582\ ) * 26.08\ ] = 20.96$$

Fifth combination of parameters A and B

$A = N_{mm} = 17.28$

$B = N_{tavg} = 21.68$

$$N_d = [\ 17.28 \ * 0.582 + (\ 1\text{-}0.582\ ) * 21.68\ ] = 19.11$$

Table 5.6. Comparison between results of algorithm, common criteria and a designer N value for case history No.2

Designer approach	* proposed equation					Common criteria			
Selected N-value	(1)	(2)	(3)	(4)	(5)	Nmin	Nmm	Nxavg	Ntavg
15	12.76	9.08	10.92	20.96	19.11	3.2	17.28	26.08	21.68

* Number between brackets refers to the different combinations of parameters A and B in the proposed equation.

Case History No.3 : Machine Shop of the Republic Steel Plant, South Chicago, Illinois, U.S.A

Peck's files (1941-1943), Bazaraa (1967)

Soil Conditions

The subsurface soil consisted of 1 feet to 3 feet thick of peat underlain by a bout 11 feet of fine silty sand. A considerable thick layer of plastic medium hard clay was then encountered and continued to the end of a deepest boring some 65 feet below ground surface. Table (5.7) includes standard penetration test results of 15 soil borings for the sand layer at the site of a structure.

The ground water table was observed at a depth of 2 feet below the original ground surface.

Structure and Foundation

The Machine shop is a two-story reinforced concrete building with dimensions of a bout 270 ft x 400 ft in plan. The Shop is founded on 7 strip footings which support columns at 20 ft center. The footings are 6 ft to 15 ft wide. The top 2 ft of soil was excavated and the grade was raised 11 ft using a hydraulic sand fill and then all footings were constructed with their bases at 6.5 ft below the final ground surface.

Analysis: Case history 3

Table (5.7)

Results of Standard Penetration Tests
at the Site of Machine Shop of the Republic Steel
Plant, South Chicaco, Illinois, U.S.A.

Elevation	Soil Borings No.														
(ft)	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
2.5	14			9	9	6	3	8							
5.0		3	12		17	14	3		25	11	27				
7.5	18	13	20	10	16	15		8	14	13				4	27
10	14		16	10	16							12	9	4	29
12.5										13	26	12			
Meam	15.3	8	16	9.6	14.5	11.6	3	8	19.5	12.3	26.5	12	9	4	28

Total mean = 13.33
Standard deviation = 7.0
Coefficient of variation = 0.525

Loads and Settlements

Dead load on the bases of footings that support the interior columns was 0.7 tsf however, The maximum load including the live load and wind load was not known but the design recommended load on spread footing of similar structure and soil foundation at the nearby location was 1.75 tsf and it was assumed to be the most reasonable maximum load for design the Machine Shop. The average and maximum measured settlements of the 15 feet wide footings were 0.75 in. and 1.32 in. respectively.

Designer Approach:

Selected N value for the foundation design = 3
this was assumed to be the most representative
value for this foundation.

Proposed Algorithm:

1. SPT values within a depth of 2B from ground surface;

   The mean N values of all soil borings from ground
   surface to a depth of 2B = 13.33

   The Standard deviation = 7.0

   The Coefficient of variation (C) = 0.525

2. SPT values within a depth of 2B from the base of foundation;

   The mean N values of all soil borings (Ntavg) = 13.33

The minimum N-value of all soil borings (Nmin) = 3

The maximum mean N values of all soil borings (Nxavg) = 28

The minimum mean N values of all soil borings (Nmm) = 3

The design N-value is then determined as ;

$$N_d = \frac{ERr}{60} * D_c * [\ A * C + (1 - C) * B\ ]$$

The parameter C is Coefficient of variation (Table 5.7).

The parameters A and B are assuming one of the following combinations.

First combination of parameters A and B

$A = N_{min} = 3$

$B = N_{xavg} = 28$

$N_d = [\ 3 * 0.525 + (1-0.525) * 28.0\ ] = 14.87$

Second combination of parameters A and B

$A = N_{min} = 3$

$B = N_{mm} = 3$

$N_d = [\ 3 * 0.525 + (1-0.525) * 3\ ] = 3$

Third combination of parameters A and B

$A = N_{min} = 3$

$B = N_{tavg} = 13.33$

$N_d = [\ 3 * 0.525 + (1-0.525) * 13.33\ ] = 7.91$

Fourth combination of parameters A and B

$A = N_{mm} = 3$

$B = N_{xavg} = 28$

$N_d = [\ 3 * 0.525 + (\ 1-0.525\ ) * 28.0\ ] = 14.87$

Fifth combination of parameters A and B

$A = N_{mm} = 3$

$B = N_{tavg} = 13.33$

$N_d = [\ 3 * 0.525 + (\ 1-0.525\ ) * 13.33\ ] = 7.91$

Table 5.8. Comparison between results of algorithm, common criteria and a designer N value for case history No.3

Designer approach	* proposed equation					Common criteria			
Selected N-value	(1)	(2)	(3)	(4)	(5)	Nmin	Nmm	Nxavg	Ntavg
3.0	14.87	3.0	7.91	14.87	7.91	3.0	3.0	28.0	13.33

* Number between brackets refers to the different combinations of parameters A and B in the proposed equation.

Case History No.4 : Boiler and Erection Shop at Silvis , Illinois ,U.S.A

Peck's files , Bazaraa (1967)

Soil Conditions

Five soil borings were drilled at the proposed Machine Shop site. Borings 1 ,3 ,and 4 were drilled to a depth of a bout 20 feet and boring 2 was drilled to a depth of about 30 feet. Boring 5 was extended to a depth of 70 feet below ground surface. The overburden soils at the site consisted of 4 feet fill underlain by 25 feet of a stratified sand and then hard clay of 6 feet thick. Shale was located immediately after clay and it extended to the end of the deepest boring. Table (5.9) presents the results of Standard Penetration Test for the five soil borings at the location of the structure.

The ground water table was observed at a depth of 16 feet below ground surface.

Structure and Foundation

The Shop is a steel truss building which covers an area of 276 ft x 860 ft . The building is founded on spread concrete footings with dimensions of 7.5 ft x 8.0 ft in plan. The final grade of foundation bases is 7 ft-8 in. below the concrete floor of the structure.

Analysis: Case history 4

Table (5.9)

Results of Standard Penetration tests at the Site of Boiler and Shop at Silvis, Illinois, U.S.A.

Elevation (ft)	Soil Borings No. (1)	(2)	(3)	(4)	(5)
7.5	11	17	13	14	10
12.5	9	8	10	10	10
15.0	4			3	6
17.5	10			9	12
20.0		9			7
22.5		8			4
25.0		9			4
27.5		4			3
30.0		9			12
32.5		18			27
Mean	7.75	8.83	8.66	8.0	7.14

Loads and Settlements

The total load applied to the base of each footings of this building was estimated to be 5.25 tsf . This corresponds to a dead load and live load together with the effect of eccentricity . The effect of snow and wind was included in live load. The total settlements of the footings were not known because no reliable measurement was made at the time of construction. However, based on visual inspection the maximum settlement was estimated to be 3.67 in.

Designer Approach:

Selected N value for the foundation design = 7
this was assumed to be the most representative
value for this foundation.

Proposed Algorithm:

1. SPT values within a depth of 2B from ground surface;

   The mean N values of all soil borings from ground
   surface to a depth of 2B = 7.96

   The Standard deviation = 6.123

   The Coefficient of variation (C) = 0.311

2. SPT values within a depth of 2B from the base of foundation;

   The mean N values of all soil borings (Ntavg) = 7.96

The minimum N-value of all soil borings (Nmin) = 3

The maximum mean N values of all soil borings (Nxavg) = 8.86

The minimum mean N values of all soil borings (Nmm) = 7.14

The design N-value is then determined as ;

$$N_d = \frac{ERr}{60} * D_c * [\ A * C + (1 - C) * B \ ]$$

The parameter C is Coefficient of variation (Table 5.9).

The parameters A and B are assuming one of the following combinations.

First combination of parameters A and B

$A = N_{min} = 3$

$B = N_{xavg} = 8.86$

$N_d = [\ 3 * 0.311 + (1-0.311) * 8.86\ ] = 7.0$

Second combination of parameters A and B

$A = N_{min} = 3$

$B = N_{mm} = 7.14$

$N_d = [\ 3 * 0.311 + (1-0.311) * 7.14\ ] = 5.85$

Third combination of parameters A and B

$A = N_{min} = 3$

$B = N_{tavg} = 7.96$

$N_d = [\ 3 * 0.311 + (1-0.311) * 7.96\ ] = 6.42$

Fourth combination of parameters A and B

$A = N_{mm} = 7.14$

$B = N_{xavg} = 8.86$

$$\mathbf{N_d} = [\ 7.14\ *\ 0.311 + (\ 1-0.311\ ) * 8.86\ ] = 8.33$$

Fifth combination of parameters A and B

$A = N_{mm} = 7.14$

$B = N_{tavg} = 7.96$

$$\mathbf{N_d} = [\ 7.14\ *\ 0.311 + (\ 1-0.311\ ) * 7.96\ ] = 7.70$$

Table 5.10. Comparison between results of algorithm, common criteria and a designer N value for case history No.4

Designer approach	* proposed equation					Common criteria			
Selected N-value	(1)	(2)	(3)	(4)	(5)	Nmin	Nmm	Nxavg	Ntavg
7	7.0	5.85	6.42	8.33	7.70	3	7.14	8.86	7.96

* Numbers between brackets refer to the different combinations of parameters A and B in the proposed equation.

Case History No.5 : Catalytic Cracker at Whiting, Indiana, U.S.A

Peck's files, Bazaraa (1967)

Soil Conditions

Detailed soil exploration at this site revealed existence of 2 ft to 10 ft of miscellaneous fill followed by about 30 ft of fine gray sand. Beneath fine sand, there was a layer of blue clay of approximately 40 ft thick. The clay layer underlain by very hard silt continuing to bedrock at a depth of a bout 87 ft below original ground surface. Table (5.11) presents standard penetration test values for 8 soil borings at the site of a structure.

The ground water table was at a depth of about 2 ft below the original ground surface.

Structure and Foundation

The Catalytic cracker is composed of two units; the regenerator and the fractionator. Both units were founded on reinforced concrete slabs that are 86 ft x 87 and 83 ft x 84 ft in plan respectively. The upper portion 7.5 ft of fill was removed and the 11 ft of clean sand was added and compacted. The first 6 ft of this compacted sand was then excavated and the concrete slabs were poured. The ground surface was then brought to its final grade by adding three more feet of compacted sand ,such that the base of slabs were founded at 9 ft below final grade.

Analysis: Case history 5

Table (5.11)

Results of Standard Penetration Tests
at the Site of Catalytic Cracker at Whiting, Indiana,
U.S.A.

Elevation	Soil Borings No.							
(ft)	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
5.00							8	
7.50	18						20	10
10.0				16		4	21	
12.5	22	8	8	24	13	2	29	16
15.0	36	25	30	37	35	23	42	51
20.0	45	20		34	42	33	59	42
22.5	36	39	52					
25.0	42		36	40	41	24	52	18
27.5		34						
30.0				18	46	22	22	24
32.5	18		23					
35.0		18		12		12	10	
37.5					14			
Meam	31.0	24.0	29.8	25.86	31.83	17.14	29.22	26.83

Total mean = 26.906
Standard deviation = 13.90.
Coefficient of variation = 0.517

Loads and Settlements

The recommended design load at the base of one unit was 1.75 tsf. This was the maximum applied load that includes the effect of wind pressure as well. Under the full load the maximum measured settlement was 1.4 in. This includes 0.3 in. elastic settlement from clay layer. The settlement due to sand,therefore, is 1.1 in.

Designer Approach:

Selected N value for the foundation design = 17
this was assumed to be the most representative
value for foundation of both units.

Proposed Algorithm:

1. SPT values within a depth of 2B from ground surface;

   The mean N values of all soil borings from ground surface to a depth of 2B = 26.90

   The Standard deviation = 13.90

   The Coefficient of variation (C) = 0.517

2. SPT values within a depth of 2B from the base of foundation;

   The mean N values of all soil borings (Ntavg) = 26.90

The minimum N-value of all soil borings (Nmin) = 2

The maximum mean N values of all soil borings (Nxavg) = 31.83

The minimum mean N values of all soil borings (Nmm) = 17.14

The design N-value is then determined as ;

$$\mathbf{N_d = \frac{\underline{ERr}}{60} * D_c * [\ A * C + (1 - C) * B\ ]}$$

The parameter C is Coefficient of variation Table (5.11).

The parameters A and B are assuming one of the following combinations.

First combination of parameters A and B

$A = N_{min} = 2$

$B = N_{xavg} = 31.83$

$\mathbf{N_d} = [\ 2 * 0.517 + (1-0.517) * 31.83\ ] = 16.41$

Second combination of parameters A and B

$A = N_{min} = 2$

$B = N_{mm} = 17.14$

$\mathbf{N_d} = [\ 2 * 0.517 + (1-0.517) * 17.14\ ] = 9.31$

Third combination of parameters A and B

$A = N_{min} = 2$

$B = N_{tavg} = 26.90$

$\mathbf{N_d} = [\ 2 * 0.517 + (1-0.517) * 26.90\ ] = 14.03$

Fourth combination of parameters A and B

$A = N_{mm} = 17.14$

$B = N_{xavg} = 31.83$

$$N_d = [\ 17.14 * 0.517 + (\ 1\text{-}0.517\ ) * 31.83\ ] = 24.24$$

Fifth combination of parameters A and B

$A = N_{mm} = 17.14$

$B = N_{tavg} = 26.90$

$$N_d = [\ 17.14 * 0.517 + (\ 1\text{-}0.517\ ) * 26.90\ ] = 21.86$$

Table 5.12. Comparison between results of algorithm, common criteria and a designer N value for case history No.5

Designer approach	* proposed equation					Common criteria			
Selected N-value	(1)	(2)	(3)	(4)	(5)	Nmin	Nmm	Nxavg	Ntavg
17	16.41	9.31	14.03	24.24	21.86	2.0	17.14	31.83	26.90

* Number between brackets refers to the different combinations of parameters A and B in the proposed equation.

Case History No.6: Pier 3 of Bridge 50 at Noxon, Montana, U.S.A

Peck's files, Bazaraa (1967)

Soil Conditions

Subsurface soil was primarily coarse sand and gravel that extended to a depth of more than 100 ft below the base of footing. Seven soil borings were made at the site of the structure. However, the penetration test was performed for one of soil borings at the site of Pier 3 and its results were used as a basis for the design of pier footing. It is important to indicate that the test was not standard since a havier hammer of 250 lb was used. Nevertheless its results were treated as standard values. Table (5.13) presents penetration test values for soil boring at the site of the structure.

The ground water table was at a depth of about 6 ft below the original ground surface .

Structure and Foundation

The bridge rests on two abutments and two piers. These four supports were positioned at about 120 ft centers. The pier covers a plan area of about 9 by 19 feet and its height is 107 ft from the bottom of footing to the top of the cap. The pier is founded on footing 40 ft wide and 40 ft long. The base of footing was at 8 ft below ground surface.

Analysis: Case history 6

Table (5.13)

Results of Penetration Test at the Site of Pier 3 of Bridge 50 at Noxon Rapids, Montana,U.S.A

Soil Boring	
Depth Below Ground Surface (ft)	Standard Penetration Value
20	60
25	50
30	45
35	50
40	55
45	50
50	45
55	65
60	90
65	80
70	75
75	76
80	80
85	65
Mean	63.28

Total mean = 63.28

Standard deviation = 14.819

Coefficient of variation = 0.234

Loads and Settlements

The pressure due the footing own weight was about 1.35 tsf. This pressure was increased to 2.7 tsf as a result of placing a rock fill on the top of footing . Maximum measured settlement under the last pressure was about 0.216 inch.

Designer Approach:

Selected N value for the foundation design = 53
this was assumed to be the most representative
value for foundation of pier 3 .

Proposed Algorithm:

1. SPT values within a depth of 2B from ground surface;

   The mean N values of soil boring from ground
   surface to a depth of 2B = 63.28
   The Standard deviation = 14.82
   The Coefficient of variation (C) = 0.234

2. SPT values within a depth of 2B from the base of foundation;

   The mean N values of soil boring (Ntavg) = 63.28

The minimum N-value of soil boring (Nmin) = 45

The maximum N value of soil boring (Nxavg) = 90

The minimum mean N values of boring (Nmm) = 63.28

The design N-value is then determined as ;

$$N_d = \frac{ERr}{60} * D_c * [\ A * C + (1 - C) * B\ ]$$

The parameter C is Coefficient of variation (Table 5.13).

The parameters A and B are assuming one of the following combinations.

First combination of parameters A and B

$A = N_{min} = 45$

$B = N_{max} = 90$

$N_d = [\ 45 * 0.234 + (1-0.234) * 90\ ] = 79.47$

Second combination of parameters A and B

$A = N_{min} = 45$

$B = N_{mm} = 63.28$

$N_d = [\ 45 * 0.234 + (1-0.234) * 63.28\ ] = 59.0$

Third combination of parameters A and B

$A = N_{min} = 45$

$B = N_{tavg} = 63.28$

$N_d = [\ 45 * 0.234 + (1-0.234) * 63.28\ ] = 59.0$

Fourth combination of parameters A and B

$A = N_{mm} = 63.28$

$B = N_{max} = 90$

$$\mathbf{N_d} = [\ 62.28\ *\ 0.234 + (\ 1\text{-}0.234\ )\ *\ 90\ ] = 83.51$$

Fifth combination of parameters **A** and **B**

$A = N_{mm} = 63.28$

$B = N_{tavg} = 63.28$

$$\mathbf{N_d} = [\ 63.28\ *\ 0.234 + (\ 1\text{-}0.234\ )\ *\ 63.28\ ] = 63.28$$

Table 5.14. Comparison between results of algorithm, common criteria and a designer N value for case history No.6

Designer approach	* proposed equation					Common criteria		
Selected N-value	(1)	(2)	(3)	(4)	(5)	Nmin	Nmm	Ntavg
53	79.47	59	59	83.51	63.28	45	63.28	63.28

* Numbers between brackets refer to the different combinations of parameters A and B in the proposed equation.

Case History No.7: M.I.T Student Center at Boston, Massachusetts, U.S.A

Horn's files, Gass (1964), Bazaraa (1967)

Soil Conditions

The soil exploration at the area of the structure was performed by standard penetration tests. The soil borings revealed existence of about 10 ft of fill followed by about 4 ft of Peat and organic silt. Underneath this organic material, there was about 25 ft of outwash sand and gravel and then a thick layer of inorganic Boston blue clay extended to a great depth of some 70 ft below the ground surface. This clay underlain by a thin layer of glacial till followed by Shale. Table (5.15) presents the results of standard penetration tests for 15 soil borings at the site of the structure.

The ground water table was observed at a depth of 7 feet below the original ground surface.

Structure and Foundation

The Structure is a 4-story reinforced concrete frame building. It was founded on a heavily-reinforced mat foundation that covers a plan area of 142 by 236 feet. The mat consisted of four slabs that range from 3 ft to 10 ft in thickness. The base of the mat was located at about 12 ft below the original ground surface.

Analysis: Case history 7

Table (5.15)

Results of Standard Penetration Tests
at the Site of the M.I.T Student Center ,
Boston, Massachusetts, U.S.A.

Elevation	Soil Borings No.														
(ft)	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
10					10				11						
15		12		28	7		26	2	16	23			14	18	11
20	34	9	10	12	32	13	25	24	14	20	19	36	34	19	22
25	17	16	23	13	16	16	24	22	17	19	10	16	15	15	7
30	16	23		12	12	17	15	22	21	19	13	15	19	12	5
35	12	28	7	17	33			24		22	12		15	13	7
40					9								13		
Meam	19.7	18	13	16	17	15	21	19	16	20.6	13.5	22.3	18.3	15	10.4

Total mean = 17.086

Standard deviation = 7.30

Coefficient of variation = 0.427

Loads and Settlements

The mat was poured in 4 sections. The thickness of four sections were 3 in., 5 in.,7 in. and 10 in. respectively. The average applied pressure due to the dead load of the mat own weight was 0.432 tsf. The corresponding measured settlement was 0.216 inch.

Designer Approach:

Selected N value for the foundation design = 11
this was assumed to be the most representative
value for this foundation.

Proposed Algorithm:

1. SPT values within a depth of 2B from ground surface;

   The mean N values of all soil borings from ground surface to a depth of 2B = 17.086

   The Standard deviation = 7.30

   The Coefficient of variation (C) = 0.427

2. SPT values within a depth of 2B from the base of foundation;

   The mean N values of all soil borings (Ntavg) = 17.086

The minimum N-value of all soil borings (Nmin) = 10.4

The maximum mean N values of all soil borings (Nxavg)= 22.3

The minimum mean N values of all soil borings (Nmm) = 2

The design N-value is then determined as ;

$$N_d = \frac{ERr}{60} * D_c * [\ A * C + (1 - C) * B\ ]$$

The parameter C is Coefficient of variation Table (5.15).

The parameters A and B are assuming one of the following combinations.

First combination of parameters A and B

$A = N_{min} = 2$

$B = N_{xavg} = 22.3$

$N_d = [\ 2 * 0.427 + (1-0.427) * 22.3\ ] = 13.65$

Second combination of parameters A and B

$A = N_{min} = 2$

$B = N_{mm} = 10.4$

$N_d = [\ 2 * 0.427 + (1-0.427) * 10.4\ ] = 6.8$

Third combination of parameters A and B

$A = N_{min} = 2$

$B = N_{tavg} = 17.086$

$N_d = [\ 2 * 0.427 + (1-0.427) * 17.086\ ] = 10.6$

Fourth combination of parameters A and B

$A = N_{mm} = 10.4$

$B = N_{xavg} = 22.3$

$$\mathbf{N_d} = [\ 10.4 * 0.427 + (\ 1\text{-}0.427\ ) * 22.3\ ] = 17.2$$

Fifth combination of parameters A and B

$A = N_{mm} = 10.4$

$B = N_{tavg} = 17.086$

$$\mathbf{N_d} = [\ 10.4 * 0.427 + (\ 1\text{-}0.427\ ) * 17.086\ ] = 14.2$$

Table 5.16. Comparison between results of algorithm, common criteria and a designer N value for case history No.7

Designer approach	* proposed equation					Common criteria			
Selected N-value	(1)	(2)	(3)	(4)	(5)	Nmin	Nmm	Nxavg	Ntavg
11	13.65	6.8	10.6	17.2	14.2	2.0	10.4	22.3	17.086

* Number between brackets refers to the different combinations of parameters A and B in the proposed equation.

Case History No.8: Proposed Plant Site at Hennepin, Illinois, U.S.A

Ireland's files, Bazaraa (1967)

Soil Conditions

The field testing program consisted of fifty soil borings and a number of plate load tests. The subsurface soil characterized by glacial deposits that consisted of 6 ft of silty and clayey material overlying a thick layer of fine to coarse sand with gravel and occasionally traces of silt and clay. Three soil borings together with three plate load tests were used as a basis for primary site investigation and prediction of likelihood settlement. Table (5.17) presents results of standard penetration tests at the site of the structure. No water table was encountered in any soil borings during site exploration.

Structure and Foundation

This is a structure steel plant that covers an area of about 9000 by 9000 feet in plan. The plant consisted of different structures being supported by a group of spread footings founded at about 10 ft below ground surface. This case history is mainly deals with results of three plate load tests with three soil borings. Each soil boring accompanying by results of one plate load test was analyzed separately. In this study ,therefore, similar approach was used so that a consistent comparison can be made between results of this study and a designer.

Analysis: Case history 8

Table (5.17)

Results of Standard Penetration Tests
at the Site of Proposed Plant Steel, at Hennepin,
Illinois, U.S.A.

Elevation	Soil Borings No.		
(ft)	(1)	(2)	(3)
545.0		16	
540.0		19	8
537.5	34	25	15
535.0	32	27	11
530.0	30	25	14
527.5		23	22
525.0	21	25	24
522.5			25
520.0	25	21	27
515.0	28	23	21
510.0	23	20	
505.0	21	41	22
502.5	23	39	24
500.0	26	35	
495.0	30	34	32

Total mean = 21.50
Standard deviation = 7.959
Coefficient of variation = 0.370

Loads and Settlements

Three load tests were conducted in pits about 16 ft by 22 ft in plan. The depth of pits were between 7 ft and 9.7 ft. Three standard 4 ft square plates were placed at the bottom of each pit and applied load was increased in increments of 2 tsf up to 10 tsf. The recommended design pressure for the spread footings of the actual structure is 4 tsf. Under this pressure the actual settlements of the three standard square plates were 0.18 in., 0.19 in. and 0.63 in.

A. Plate load test and soil boring No. 1

Designer Approach:

Selected N value from first boring for load
test No.1 = 32
this was assumed to be the most representative
value for estimating foundation settlement.

Proposed Algorithm:

1. SPT values within a depth of 2B from ground surface;

    The mean N values of all soil borings from ground surface to a depth of 2B = 21.50

    The Standard deviation = 7.959

    The Coefficient of variation (C) = 0.370

2. SPT values within a depth of 2B from the base of foundation;

    The mean N values of soil boring No.1 (Ntavg) = 32.0

The minimum N-value of soil boring No.1 (Nmin) = 30

The maximum N value of soil borings No.1 (Nmax) = 34

The minimum mean N value of soil boring No.1 (Nmm) = 32

The design N-value is then determined as ;

$$N_d = \frac{ERr}{60} * D_c * [\ A * C + (1 - C) * B\ ]$$

The parameter C is Coefficient of variation Table (5.17).

The parameters A and B are assuming one of the following combinations.

First combination of parameters A and B

$A = N_{min} = 30$

$B = N_{max} = 34$

$N_d = [\ 30 * 0.37 + (\ 1\text{-}0.37\ ) * 34\ ] = 32.52$

Second combination of parameters A and B

$A = N_{min} = 30$

$B = N_{mm} = 32$

$N_d = [\ 30 * 0.37 + (\ 1\text{-}0.37\ ) * 32\ ] = 31.26$

Third combination of parameters A and B

$A = N_{min} = 30$

$B = N_{tavg} = 32$

$N_d = [\ 30 * 0.370 + (\ 1\text{-}0.370\ ) * 32\ ] = 31.26$

Fourth combination of parameters A and B

$A = N_{mm} = 32$

$B = N_{max} = 34$

$N_d = [\ 32 * 0.37 + (\ 1-0.37\ ) * 34\ ] = 33.26$

Fifth combination of parameters A and B

$A = N_{mm} = 32$

$B = N_{tavg} = 32$

$N_d = [\ 32 * 0.37 + (\ 1-0.37\ ) * 32\ ] = 32$

Table 5.18. Comparison between results of algorithm, common criteria and a designer N value for case history No.8

Designer approach	* proposed equation					Common criteria			
Selected N-value	(1)	(2)	(3)	(4)	(5)	Nmin	Nmm	Nmax	Ntavg
32	32.52	31.26	31.26	33.26	32	30	32	43	32

* Numbers between parentheses refer to the different combinations of parameters A and B in the proposed algorithm.

Case History No.9: Proposed Plant Site at Hennepin, Illinois, U.S.A

Ireland's files, Bazaraa (1967)

Soil Conditions

The field testing program consisted of fifty soil borings and a number of plate load tests. The subsurface soil characterized by glacial deposits that consisted of 6 ft of silty and clayey material overlying a thick layer of fine to coarse sand with gravel and occasionally traces of silt and clay. Three soil borings together with three plate load tests were used as a basis for primary site investigation and prediction of likelihood settlement. Table (5.17) presents results of standard penetration tests at the site of the structure. No water table was encountered in any soil borings during site exploration.

Structure and Foundation

This is a structure steel plant that covers an area of about 9000 by 9000 feet in plan. The plant consisted of different structures being supported by a group of spread footings founded at about 10 ft below ground surface. This case history is mainly deals with results of three plate load tests with three soil borings. Each soil boring accompanying by results of one plate load test was analyzed separately. In this study ,therefore, similar approach was used so that a consistent comparison can be made between results of this study and a designer.

Analysis: Case history 9

Table (5.17)

Results of Standard Penetration Tests at the Site of Proposed Plant Steel, at Hennepin, Illinois, U.S.A.

Elevation	Soil Borings No.		
(ft)	(1)	(2)	(3)
545.0		16	
540.0		19	8
537.5	34	25	15
535.0	32	27	11
530.0	30	25	14
527.5		23	22
525.0	21	25	24
522.5			25
520.0	25	21	27
515.0	28	23	21
510.0	23	20	
505.0	21	41	22
502.5	23	39	24
500.0	26	35	
495.0	30	34	32

Total mean = 21.50
Standard deviation = 7.959
Coefficient of variation = 0.370

Loads and Settlements

Three load tests were conducted in pits about 16 ft by 22 ft in plan. The depth of pits were between 7 ft and 9.7 ft. Three standard 4 ft square plates were placed at the bottom of each pit and applied load was increased in increments of 2 tsf up to 10 tsf. The recommended design pressure for the spread footings of the actual structure is 4 tsf. Under this pressure the actual settlements of the three standard square plates were 0.18 in., 0.19 in. and 0.63 in.

B. Plate load test and soil boring No. 2

Designer Approach:

Selected N value from first boring for load test No.2 = 26

this was assumed to be the most representative value for estimating foundation settlement.

Proposed Algorithm:

1. SPT values within a depth of 2B from ground surface;

   The mean N values of all soil borings from ground surface to a depth of 2B = 21.50

   The Standard deviation = 7.959

   The Coefficient of variation (C) = 0.370

2. SPT values within a depth of 2B from the base of foundation;

   The mean N values of soil boring No.2 (Ntavg) = 25.0

The minimum N-value of soil boring No.2 (Nmin) = 23

The maximum N value of soil borings No.2 (Nmax) = 27

The minimum mean N value of soil boring No.2 (Nmm) = 25

The design N-value is then determined as ;

$$N_d = \frac{ERr}{60} * D_c * [\ A * C + (1 - C) * B\ ]$$

The parameter C is Coefficient of variation Table (5.17).

The parameters A and B are assuming one of the following combinations.

First combination of parameters A and B

$A = N_{min} = 23$

$B = N_{max} = 27$

$N_d = [\ 23 * 0.37 + (1 - 0.37) * 27\ ] = 25.52$

Second combination of parameters A and B

$A = N_{min} = 23$

$B = N_{mm} = 25$

$N_d = [\ 23 * 0.37 + (1 - 0.37) * 25\ ] = 24.26$

Third combination of parameters A and B

$A = N_{min} = 23$

$B = N_{tavg} = 25$

$N_d = [\ 23 * 0.370 + (1 - 0.370) * 25\ ] = 24.26$

Fourth combination of parameters A and B

$A = N_{mm} = 25$

$B = N_{max} = 27$

$$\mathbf{N_d} = [\ 25 * 0.37 + (\ 1\text{-}0.37\ ) * 27\ ] = 26.26$$

Fifth combination of parameters A and B

$A = N_{mm} = 25$

$B = N_{tavg} = 25$

$$\mathbf{N_d} = [\ 25 * 0.37 + (\ 1\text{-}0.37\ ) * 25\ ] = 25$$

Table 5.19. Comparison between results of algorithm, common criteria and a designer N value for case history No.9

Designer approach	* proposed equation					Common criteria			
Selected N-value	(1)	(2)	(3)	(4)	(5)	Nmin	Nmm	Nmax	Ntavg
26	25.52	24.26	24.26	26.26	25	23	25	27	25

* Numbers between parentheses refer to the different combinations of parameters A and B in the proposed algorithm.

Case History No.10: Proposed Plant Site at Hennepin, Illinois, U.S.A

Ireland's files, Bazaraa (1967)

Soil Conditions

The field testing program consisted of fifty soil borings and a number of plate load tests. The subsurface soil characterized by glacial deposits that consisted of 6 ft of silty and clayey material overlying a thick layer of fine to coarse sand with gravel and occasionally traces of silt and clay. Three soil borings together with three plate load tests were used as a basis for primary site investigation and prediction of likelihood settlement. Table (5.15) presents results of standard penetration tests at the site of the structure. No water table was encountered in any soil borings during site exploration.

Structure and Foundation

This is a structure steel plant that covers an area of about 9000 by 9000 feet in plan. The plant consisted of different structures being supported by a group of spread footings founded at about 10 ft below ground surface. This case history is mainly deals with results of three plate load tests with three soil borings. Each soil boring accompanying by results of one plate load test was analyzed separately. In this study ,therefore, similar approach was used so that a consistent comparison can be made between results of this study and a designer.

Analysis: Case history 10

Table (5.17)

Results of Standard Penetration Tests
at the site of Proposed Plant Steel, at Hennepin,
Illinois, U.S.A.

Elevation	Soil Borings No.		
(ft)	(1)	(2)	(3)
545.0		16	
540.0		19	8
537.5	34	25	15
535.0	32	27	11
530.0	30	25	14
527.5		23	22
525.0	21	25	24
522.5			25
520.0	25	21	27
515.0	28	23	21
510.0	23	20	
505.0	21	41	22
502.5	23	39	24
500.0	26	35	
495.0	30	34	32

Total mean = 21.50
Standard deviation = 7.959
Coefficient of variation = 0.370

Loads and Settlements

Three load tests were conducted in pits about 16 ft by 22 ft in plan. The depth of pits were between 7 ft and 9.7 ft. Three standard 4 ft square plates were placed at the bottom of each pit and applied load was increased in increments of 2 tsf up to 10 tsf. The recommended design pressure for the spread footings of the actual structure is 4 tsf. Under this pressure the actual settlements of the three standard square plates were 0.18 in., 0.19 in. and 0.63 in.

C. Plate load test and soil boring No. 3

Designer Approach:

Selected N value from first boring for load test No.3 = 16

this was assumed to be the most representative value for estimating foundation settlement.

Proposed Algorithm:

1. SPT values within a depth of 2B from ground surface;

   The mean N values of all soil borings from ground surface to a depth of 2B = 21.50

   The Standard deviation = 7.959

   The Coefficient of variation (C) = 0.370

2. SPT values within a depth of 2B from the base of foundation;

   The mean N values of soil boring No.3 (Ntavg) = 15.5

The minimum N-value of soil boring No.3 (Nmin) = 11

The maximum N value of soil borings No.2 (Nmax) = 22

The minimum mean N value of soil boring No.2 (Nmm) = 15.5

The design N-value is then determined as ;

$$N_d = \frac{ERr}{60} * D_c * [\ A * C + (1 - C) * B\ ]$$

The parameter C is Coefficient of variation Table (5.17).

The parameters A and B are assuming one of the following combinations.

First combination of parameters A and B

$A = N_{min} = 11$

$B = N_{max} = 22$

$N_d = [\ 11 * 0.37 + (1\text{-}0.37) * 22\ ] = 17.93$

Second combination of parameters A and B

$A = N_{min} = 11$

$B = N_{mm} = 15.5$

$N_d = [\ 11 * 0.37 + (1\text{-}0.37) * 15.5\ ] = 13.84$

Third combination of parameters A and B

$A = N_{min} = 11$

$B = N_{tavg} = 15.5$

$N_d = [\ 11 * 0.370 + (1\text{-}0.370) * 15.5\ ] = 13.84$

Fourth combination of parameters A and B

$A = N_{mm} = 15.5$

$B = N_{max} = 22$

$$N_d = [\ 15.5 * 0.37 + (\ 1-0.37\ ) * 22\ ] = 19.59$$

Fifth combination of parameters A and B

$A = N_{mm} = 15.5$

$B = N_{tavg} = 15.5$

$$N_d = [\ 15.5 * 0.37 + (\ 1-0.37\ ) * 15.5\ ] = 15.5$$

Table 5.20. Comparison between results of algorithm, common criteria and a designer N value for case history No.10

Designer approach	* proposed equation					Common criteria			
Selected N-value	(1)	(2)	(3)	(4)	(5)	Nmin	Nmm	Nmax	Ntavg
16	17.93	13.84	13.84	19.59	15.5	11	15.5	22	15.5

* Numbers between parentheses refer to the different combinations of parameters A and B in the proposed algorithm.

Case History No.11: Proposed Plant Site at Hennepin, Illinois, U.S.A

Ireland's files, Bazaraa (1967)

Soil Conditions

The field testing program consisted of fifty soil borings and a number of plate load tests. The subsurface soil characterized by glacial deposits that consisted of 6 ft of silty and clayey material overlying a thick layer of fine to coarse sand with gravel and occasionally traces of silt and clay. Three soil borings together with three plate load tests were used as a basis for primary site investigation and prediction of likelihood settlement. Table (5.17) presents results of standard penetration tests at the site of the structure. No water table was encountered in any soil borings during site exploration.

Structure and Foundation

This is a structure steel plant that covers an area of about 9000 by 9000 feet in plan. The plant consisted of different structures being supported by a group of spread footings founded at about 10 ft below ground surface. This case history is mainly deals with results of three plate load tests with three soil borings. Each soil boring accompanying by results of one plate load test was analyzed separately. In this study ,therefore, similar approach was used so that a consistent comparison can be made between results of this study and a designer.

Analysis: Case history 11

Table (5.17)

Results of Standard Penetration Tests
at the Site of Proposed Plant Steel, at Hennepin,
Illinois, U.S.A.

Elevation	Soil Borings No.		
(ft)	(1)	(2)	(3)
545.0		16	
540.0		19	8
537.5	34	25	15
535.0	32	27	11
530.0	30	25	14
527.5		23	22
525.0	21	25	24
522.5			25
520.0	25	21	27
515.0	28	23	21
510.0	23	20	
505.0	21	41	22
502.5	23	39	24
500.0	26	35	
495.0	30	34	32

Total mean = 21.50
Standard deviation = 7.959
Coefficient of variation = 0.370

Loads and Settlements

Three load tests were conducted in pits about 16 ft by 22 ft in plan. The depth of pits were between 7 ft and 9.7 ft. Three standard 4 ft square plates were placed at the bottom of each pit and applied load was increased in increments of 2 tsf up to 10 tsf. The recommended design pressure for the spread footings of the actual structure is 4 tsf. Under this pressure the actual settlements of the three standard square plates were 0.18 in., 0.19 in. and 0.63 in.

D. Plate load tests and soil borings for three locations.

Designer Approach:

Selected N value from all borings = 16
this was assumed to be the most representative value for estimating foundation settlement of actual structure.

Proposed Algorithm:

1. SPT values within a depth of 2B from ground surface;

   The mean N values of all soil borings from ground surface to a depth of 2B = 21.50

   The Standard deviation = 7.959

   The Coefficient of variation (C) = 0.370

2. SPT values within a depth of 2B from the base of foundation;

   The mean N values of all soil borings (Ntavg) = 23.45

The minimum N-value of all soil borings (Nmin) = 11

The maximum mean N values of all soil borings (Nxavg) = 32

The minimum mean N values of all soil borings (Nmm) = 15.5

The design N-value is then determined as ;

$$\mathbf{N_d = \frac{ERr}{60} * D_c * [\ A * C + (1 - C) * B\ ]}$$

The parameter C is Coefficient of variation Table (5.17).

The parameters **A** and **B** are assuming one of the following combinations.

First combination of parameters A and B

$A = N_{min} = 11$

$B = N_{xavg} = 32$

$\mathbf{N_d}$ = [11 * 0.370 + (1-0.370) * 32] = 24.23

Second combination of parameters A and B

$A = N_{min} = 11$

$B = N_{mm} = 15.5$

$\mathbf{N_d}$ = [11 * 0.370 + (1-0.370) * 15.5] = 13.83

Third combination of parameters A and B

$A = N_{min} = 11$

$B = N_{tavg} = 23.45$

$\mathbf{N_d}$ = [11 * 0.370 + (1-0.370) * 23.45] = 18.84

Fourth combination of parameters A and B

$A = N_{mm} = 15.5$

$B = N_{xavg} = 32$

$$N_d = [\ 15.5 * 0.370 + (\ 1\text{-}0.370\ ) * 32\ ] = 25.89$$

Fifth combination of parameters A and B

$A = N_{mm} = 15.5$

$B = N_{tavg} = 23.45$

$$N_d = [\ 15.5 * 0.370 + (\ 1\text{-}0.370\ ) * 23.45\ ] = 20.51$$

Table 5.21. Comparison between results of algorithm, common criteria and a designer N value for case history No.11

Designer approach	* proposed equation					Common criteria			
Selected N-value	(1)	(2)	(3)	(4)	(5)	Nmin	Nmm	Nxavg	Ntavg
25	24.23	13.83	18.84	25.89	20.51	11	15.5	32	23.45

* Number between parentheses refer to the different combinations of parameters A and B in the proposed equation.

Case History No.12: H-Frame Transmission Tower Structure, U.S.A

LaGatta and Keller (1984)

Soil Conditions

Two soil borings were made at the site of the structure. The soil borings consists of about 20.5 ft of a predominantly non plastic silt. The silt contained lenses of fine sand up to 2 feet thick. Table 5.22 presented the results of standard penetration tests at the location of the structure. Under silt layer there was a thick layer of glacial till that consists mainly of a dense gravel sand with boulders. The results of SPT in glacial till were found to be greater than 50 blows/ft. Dry unit weight of undisturbed soil were between 87.7 to 109.5 pcf while the specific gravity was 2.71.

The ground water level was observed at the base of footings during the load testing program.

Structure and Foundation

The Structure is a 61-ft-high H-fame transmission tower. It was founded on two reinforced concrete shallow spread footings. The size of left footing was 26 ft x 13 ft x 4 ft while size of right footing was 24 ft x 12 ft x 4 ft. The distance between footings was 11 ft. The top soil layer was excavated to a depth of 4.5 ft and then a thin layer of 6 in widely graded sand was placed at the bottom of excavation prior to the footing construction. The base of the footings was located at about 4 ft below the original ground surface.

Analysis: Case history 12

Table (5.22)

Results of Standard Penetration Tests
at the Site of a H-frame Transmission Tower,
U.S.A.

Elevation	Soil Borings No.	
(ft)	(1)	(2)
7.50	15	28
12.50	19	11
17.50	11	10
20.00		11
22.50	12	9
Mean	14.25	13.40

Total mean = 14.0
Standard deviation = 6.062
Coefficient of variation = 0.433

Loads and Settlements

This case history represents a load testing program of a H-frame transmission tower foundation. Vertical load tests were performed on two footings that were used as actual foundation for the structure. Nominal vertical loads applied in increment of 100 kips up to maximum load of 500 kips. The actual measured settlements corresponding to a maximum load of 500 kips were extrapolated from curve of settlements and were found to be 0.24 in. and 0.36 in. for left and right footing respectively.

Designer Approach:

Selected N value for the foundation design = 14
this was assumed to be the most representative
value for this foundation.

Proposed Algorithm:

1. SPT values within a depth of 2B from ground surface;

   The mean N values of all soil borings from ground surface to a depth of 2B = 14.0

   The Standard deviation = 6.062

   The Coefficient of variation (C) = 0.433

2. SPT values within a depth of 2B from the base of foundation;

   The mean N values of all soil borings (Ntavg) = 14.0

The minimum N-value of all soil borings (Nmin) = 9

The maximum mean N values of all soil borings (Nxavg)= 14.25

The minimum mean N values of all soil borings (Nmm) = 13.80

The design N-value is then determined as ;

$$N_d = \frac{ERr}{60} * D_c * [\ A * C + (1 - C) * B\ ]$$

The parameter C is Coefficient of variation Table (5.22).

The parameters A and B are assuming one of the following combinations.

First combination of parameters A and B

$A = N_{min} = 9$

$B = N_{xavg} = 14.25$

$N_d = [\ 9 \ * 0.433 + (1-0.433) * 14.25\ ] = 11.97$

Second combination of parameters A and B

$A = N_{min} = 9$

$B = N_{mm} = 13.8$

$N_d = [\ 9 \ * 0.433 + (1-0.433) * 13.8\ ] = 11.72$

Third combination of parameters A and B

$A = N_{min} = 9$

$B = N_{tavg} = 14$

$N_d = [\ 9 \ * 0.433 + (1-0.433) * 14\ ] = 11.83$

Fourth combination of parameters **A** and **B**

$A = N_{mm} = 13.8$

$B = N_{xavg} = 14.25$

$$\mathbf{N_d} = [\ 13.8 * 0.433 + (\ 1\text{-}0.433\ ) * 14.25\ ] = 14.05$$

Fifth combination of parameters **A** and **B**

$A = N_{mm} = 13.8$

$B = N_{tavg} = 14$

$$\mathbf{N_d} = [\ 13.8 * 0.433 + (\ 1\text{-}0.433\ ) * 14\ ] = 13.91$$

Table 5.23. Comparison between results of algorithm, common criteria and a designer N value for case history No.12

Designer approach	* proposed equation					Common criteria			
Selected N-value	(1)	(2)	(3)	(4)	(5)	Nmin	Nmm	Nxavg	Ntavg
14	11.97	11.72	11.83	14.05	13.9	9.0	13.8	14.25	14.25

* Number between brackets refers to the different combinations of parameters A and B in the proposed equation.

Case History No.13: A Tall Chimney Foundation, Maryland, U.S.A

Davie and Lewis (1988)

Soil Conditions

Field investigation revealed that the soil consisted of 13 ft. of light to dark brown fine sand with terraces of silt underlain by 15 ft. of brown and gray fine to coarse sand with some cobbles and boulders. This layer of fine to coarse sand was followed by bedrock that extended to a great depth. The results of standard penetration tests in two soil borings are presented in Table (5.24).

The ground water level was encountered at a depth of a bout 2 ft below original ground surface.

Structure and Foundation

This is a 350-foot-high concrete chimney that has three steel flues. The outside diameters for the base and top shell concrete were 29.6 ft and 25.6 ft respectively. The chimney is supported on a large reinforced concrete square mat that covers an area of 54 ft by 54 ft in plan. The final base of mat foundation was at about 7 feet below the original ground surface.

Analysis: Case history 13

Table (5.24)

Results of Standard Penetration Tests
at the Site of a Tall Chimney Foundation,
Maryland, U.S.A.

Elevation	Soil Borings No.	
(ft)	(1)	(2)
70.0	8	8
67.0	17	19
66.0	12	33
64.0	19	25
62.0	22	37
60.0	20	34
57.0	23	54
55.0	8	
52.0	38	39
45.0	23	
Mean	21.86	37.80

Total mean = 24.38

Standard deviation = 12.61

Coefficient of variation = 0.517

Load And Settlement:

The applied dead load at the base of mat foundation was about 7000 kips resulting in a maximum bearing pressure of 2.4 ksf. This load does not include the dynamic load from wind pressure. The maximum actual measured settlement was 0.25 in.

Designer Approach:

Selected N value for the foundation design = 30
this was assumed to be the most representative
value for this foundation.

Proposed Algorithm:

1. SPT values within a depth of 2B from ground surface;

    The mean N values of all soil borings from ground surface to a depth of 2B = 24.38

    The Standard deviation = 12.61

    The Coefficient of variation (C) = 0.517

2. SPT values within a depth of 2B from the base of foundation;

    The mean N values of all soil borings (Ntavg) = 28.5

The minimum N-value of all soil borings (Nmin) = 8

The maximum mean N values of all soil borings (Nxavg)= 37.8

The minimum mean N values of all soil borings (Nmm) = 21.86

The design N-value is then determined as ;

$$N_d = \frac{ERr}{60} * D_c * [\ A * C + (1 - C) * B\ ]$$

The parameter C is Coefficient of variation Table (5.24).

The parameters A and B are assuming one of the following combinations.

First combination of parameters A and B

$A = N_{min} = 8$

$B = N_{xavg} = 37.8$

$N_d = [\ 8 * 0.517 + (\ 1-0.517\ ) * 37.8\ ] = 22.39$

Second combination of parameters A and B

$A = N_{min} = 8$

$B = N_{mm} = 21.86$

$N_d = [\ 8 * 0.517 + (\ 1-0.517\ ) * 21.86\ ] = 14.69$

Third combination of parameters A and B

$A = N_{min} = 8$

$B = N_{tavg} = 28.5$

$N_d = [\ 8 * 0.517 + (\ 1-0.517\ ) * 28.5\ ] = 17.90$

Fourth combination of parameters **A** and **B**

$A = N_{mm} = 21.86$

$B = N_{xavg} = 37.8$

$\mathbf{N_d} = [\ 21.86 * 0.517 + (\ 1\text{-}0.517\ ) * 37.8\ ] = 29.56$

Fifth combination of parameters **A** and **B**

$A = N_{mm} = 21.86$

$B = N_{tavg} = 28.5$

$\mathbf{N_d} = [\ 21.86 * 0.517 + (\ 1\text{-}0.517\ ) * 28.5\ ] = 25.1$

Table 5.25. Comparison between results of algorithm, common criteria and a designer N value for case history No.13

Designer approach	* proposed equation					Common criteria			
Selected N-value	(1)	(2)	(3)	(4)	(5)	Nmin	Nmm	Nxavg	Ntavg
30	22.39	14.69	17.90	29.56	25.1	8.0	21.86	37.80	28.50

*** Number between brackets refers to the different combinations of parameters A and B in the proposed equation.**

Case History No.14: Steel Mill Factory, Lesaka, Spain

Picornell and del Monte (1988)

Soil Conditions

The subsurface soils consisted of two distinctive strata; the upper strata is about 23 ft thick and consisted mainly of greenish gray silty gravel with sand and frequent boulders. However, in some parts of the site of the structure ,there were irregular lenses of medium stiff to stiff silty clay embedded in this strata. The second strata consisted of alternating layers of loose to medium dense silty sand and sandy silt with varying amount of gravel. The thickness of this strata is variable but generally it was a thick layer that exceeds 100 ft in some parts of the investigated site. This strata underlain by a thick layer of limestone bed rock. The penetration tests were performed in two soil borings, however, the results of one boring were considered dependable since the readings of the other boring were abnormal due to frequent presence of boulders. Table (5.26) presents results of standard penetration tests for one boring at the site of the structure. The ground water level was at a depth of a bout 2 ft below original ground surface.

Structure and Foundation

The actual structure is a steel mill factory in Northern Spain. However this study deals only with site investigation and specially with result of field load test performed at the location of the structure. The load test was made in the area where silty sand layer was closer to the ground surface. The loaded area was about 65 ft by 50 ft in plan.

Analysis: Case history 14

Table (5.26)
Results of Standard Penetration Tests
at the Site of a Steel Mill Factory,
Lesaka, Spain

Elevation (ft)	Soil Borings No. (1)
5.0	36
10.0	24
15.0	14
20.0	17
23.8	10
26.2	9
31.2	9
36.1	14
41.3	7
45.9	17
50.8	10
55.8	23
70.0	18
80.0	34
85.0	53
Mean	19.66

Total mean = 19.66

Standard deviation = 12.73

Coefficient of variation = 0.647

Load And Settlement:

The load testing program consisted of building a stock piles of steel sheet coils on the surface of the silty sand layer that covers an area of 65 ft by 50 ft in plan. The applied load resulted in average static bearing pressure of 3.1 ksf or 1.55 tsf. The actual measured average settlement was 0.9 in.

Designer Approach:

Selected N value for the foundation design = 10 this was assumed to be the most representative value for this foundation.

Proposed Algorithm:

1. SPT values within a depth of 2B from ground surface;

   The mean N values of soil boring from ground surface to a depth of 2B = 19.66

   The Standard deviation = 12.73

   The Coefficient of variation (C) = 0.647

2. SPT values within a depth of 2B from the base of foundation;

   The mean N values of soil boring (Ntavg) = 12.37

The minimum N-value of soil boring (Nmin) = 7

The maximum N values of soil boring (Nmax) = 53

The minimum mean N values of soil boring (Nmm) = 12.37

The design N-value is then determined as ;

$$N_d = \frac{ERr}{60} * D_c * [\ A * C + (1 - C) * B\ ]$$

The parameter C is Coefficient of variation Table (5.26).

The parameters A and B are assuming one of the following combinations.

First combination of parameters A and B

$A = N_{min} = 7$

$B = N_{max} = 53$

$N_d = [\ 7 * 0.647 + (1-0.647) * 53\ ] = 23.24$

Second combination of parameters A and B

$A = N_{min} = 7$

$B = N_{mm} = 12.37$

$N_d = [\ 7 * 0.647 + (1-0.647) * 12.37\ ] = 8.89$

Third combination of parameters A and B

$A = N_{min} = 7$

$B = N_{tavg} = 12.37$

$N_d = [\ 7 * 0.647 + (1-0.647) * 12.37\ ] = 8.89$

Fourth combination of parameters A and B

$A = N_{mm} = 12.37$

$B = N_{max} = 53$

$$\mathbf{N_d} = [\ 12.37 * 0.647 + (\ 1\text{-}0.647\ ) * 53\ ] = 26.71$$

Fifth combination of parameters A and B

$A = N_{mm} = 12.37$

$B = N_{tavg} = 12.37$

$$\mathbf{N_d} = [\ 12.37 * 0.647 + (\ 1\text{-}0.647\ ) * 12.37\ ] = 12.37$$

Table 5.27. Comparison between results of algorithm, common criteria and a designer N value for case history No.14

Designer approach	* proposed equation					Common criteria			
Selected N-value	(1)	(2)	(3)	(4)	(5)	Nmin	Nmm	Nmax	Ntavg
10	23.24	8.89	8.89	26.71	12.37	7.0	12.37	53.0	12.37

* Number between brackets refers to the different combinations of parameters A and B in the proposed equation.

Case History No.15: Thyssen Building
Dusseldor, Germany
Schultze (1962,1963),

Soil Conditions

Six soil borings were made at this site. The subsurface soil consisted of about 85 ft. of gravelly sand underlain by a layer of very fine dense sand that continued to the end of borings some 121 ft below original ground surface. The ground water table was encountered at a depth of approximately 28 ft below the ground surface.

Structure and Foundation

This is a building occupying an area of 57.7 ft by 257.6 ft. It is a reinforced concrete structure with steel skeleton. It consist of 3 basements and 30 stories and it is a bout 344 ft high. It founded on a raft with its base at 35 ft below ground surface.

Load and Settlement:

The total maximum applied pressure on the base of a raft was 2.5 tsf. The corresponding maximum measured settlement was 0.866 in.

Load And Settlement:

The total applied Dead load was about 2.45 tsf and the corresponding measured settlement was about 0.83 in. testing program consisted of building a stock piles of steel sheet coils on the surface of the silty sand layer that covers an area of 65 ft by 50 ft in plan. The applied load resulted in average static bearing pressure of 3.1 ksf or 1.55 tsf. The actual measured average settlement was 0.9 in.

Designer Approach:

Selected N value for the foundation design = 18
this was assumed to be the most representative
value for this foundation.

Proposed Algorithm:

1. SPT values within a depth of 2B from ground surface;

   The mean N values of all soil borings from ground surface to a depth of 2B = 20.0

   The Standard deviation = 6.0

   The Coefficient of variation (C) = 0.30

2. SPT values within a depth of 2B from the base of foundation;

   The mean N values of soil boring (Ntavg) = 20

The minimum N-value of soil borings (Nmin) = 12

The maximum mean N values of soil borings (Nxavg) = 25

The minimum mean N values of soil borings (Nmm) = 16

The design N-value is then determined as ;

$$\mathbf{N_d = \frac{ERr}{60} * D_c * [\ A * C + (1 - C) * B\ ]}$$

The parameters A and B are assuming one of the following combinations.

First combination of parameters A and B

$A = N_{min} = 12$

$B = N_{xavg} = 25$

$\mathbf{N_d}$ = [12 * 0.30 + (1-0.30) * 25] = 21.1

Second combination of parameters A and B

$A = N_{min} = 12$

$B = N_{mm} = 16$

$\mathbf{N_d}$ = [12 * 0.30 + (1-0.30) * 16] = 14.8

Third combination of parameters A and B

$A = N_{min} = 12$

$B = N_{tavg} = 20$

$\mathbf{N_d}$ = [12 * 0.30 + (1-0.30) * 20] = 17.6

Fourth combination of parameters **A** and **B**

$A = N_{mm} = 16$

$B = N_{xavg} = 25$

$\mathbf{N_d} = [\ 16 * 0.30 + (\ 1-0.30\ ) * 25\ ] = 22.3$

Fifth combination of parameters **A** and **B**

$A = N_{mm} = 16$

$B = N_{tavg} = 20$

$\mathbf{N_d} = [\ 16 * 0.30 + (\ 1-0.30\ ) * 20\ ] = 18.8$

Table 5.28. Comparison between results of algorithm, common criteria and a designer N value for case history No.15

Designer approach	* proposed equation					Common criteria			
Selected N-value	(1)	(2)	(3)	(4)	(5)	Nmin	Nmm	Nxavg	Ntavg
18	21.1	14.8	17.6	22.30	18.80	12	16	25	20

* Number between brackets refers to the different combinations of parameters A and B in the proposed equation.

Case History No.16: Ministry Building
Dusseldorf , Germany
Schultze (1962,1963),

Soil Conditions

Based on the results of three soil borings, The subsurface soils were classified as sandy gravel. The first 36 ft of soil was predominantly coarse gravel underlain by about 5 ft of sand and then by coarse gravel followed by medium-dense to dense sandy gravel that extended to a great depth. Schultze (1962) showed the results of standard penetration tests of one soil boring at the site of the structure. The ground water level was at a depth of about 18 ft below the original ground surface.

Structure and Foundation

This structure is a 20-story reinforced concrete building with steel skeleton. It covers an area of about 52 ft by 141 ft in plan. It consist of 2 basements and it is about 213 ft high. The building is built on a raft foundation with its base at about 24 ft below ground surface.

Load And Settlement:

The total applied Dead load was about 2.32 tsf and the corresponding measured settlement was about 0.84 in.

Load and Settlement:

The total applied Dead load was about 2.45 tsf and the corresponding measured settlement was about 0.83 in. testing program consisted of building a stock piles of steel sheet coils on the surface of the silty sand layer that covers an area of 65 ft by 50 ft in plan. The applied load resulted in average static bearing pressure of 3.1 ksf or 1.55 tsf. The actual measured average settlement was 0.9 in.

Designer Approach:

Selected N value for the foundation design = 16
this was assumed to be the most representative
value for this foundation.

Proposed Algorithm:

1. SPT values within a depth of 2B from ground surface;

   The mean N values of all soil borings from ground surface to a depth of 2B = 16.0

   The Standard deviation = 5.0

   The Coefficient of variation (C) = 0.312

2. SPT values within a depth of 2B from the base of foundation;

   The mean N values of soil boring (Ntavg) = 14

The minimum N-value of soil borings (Nmin) = 9

The maximum mean N values of soil borings(Nxavg) = 18

The minimum mean N values of soil boring (Nmm) = 14

The design N-value is then determined as ;

$$N_d = \frac{ERr}{60} * D_c * [\ A * C + (1 - C) * B\ ]$$

The parameters A and B are assuming one of the following combinations.

First combination of parameters A and B

$A = N_{min} = 9$

$B = N_{xavg} = 18$

$N_d = [\ 9 * 0.321 + (1-0.321) * 18\ ] = 15.1$

Second combination of parameters A and B

$A = N_{min} = 9$

$B = N_{mm} = 14$

$N_d = [\ 9 * 0.321 + (1-0.321) * 14\ ] = 12.4$

Third combination of parameters A and B

$A = N_{min} = 9$

$B = N_{tavg} = 14$

$N_d = [\ 9 * 0.321 + (1-0.321) * 14\ ] = 12.4$

Fourth combination of parameters A and B

$A = N_{mm} = 14$

$B = N_{max} = 18$

$$N_d = [\ 14 * 0.321 + (\ 1-0.321\ ) * 18\ ] = 16.72$$

Fifth combination of parameters A and B

$A = N_{mm} = 14$

$B = N_{tavg} = 14$

$$N_d = [\ 14 * 0.321 + (\ 1-0.321\ ) * 14\ ] = 14$$

Table 5.29. Comparison between results of algorithm, common criteria and a designer N value for case history No.16

Designer approach	* proposed equation					Common criteria			
Selected N-value	(1)	(2)	(3)	(4)	(5)	Nmin	Nmm	Nxavg	Ntavg
16	15.11	12.4	12.4	16.72	14.0	9.0	14.0	18.0	14.0

* Number between brackets refers to the different combinations of parameters A and B in the proposed equation.

Case History No.17: A chimney

Coloqne, Germany

Schultze (1962,1963),

Soil Conditions

The Results of one soil boring indicated that the subsurface soil consisted of about 16 ft of gravelly sand followed by a thick layer of sand with varying amount of gravels that extended to a great depth below ground surface. The ground water table was encountered at a depth of approximately 18 ft below the ground surface.

Structure and Foundation

The structure is a 394 ft high reinforced concrete chimney with base diameter of about 23 ft. It is founded on a circular footing that has a diameter of about 67 ft. The base of the footing was located at 11.5 ft below ground surface.

Load And Settlement:

The total maximum applied load was about 1.80 tsf and the corresponding measured settlement was about 0.315 in.

Load And Settlement:

The total applied Dead load was about 2.45 tsf and the corresponding measured settlement was about 0.83 in. testing program consisted of building a stock piles of steel sheet coils on the surface of the silty sand layer that covers an area of 65 ft by 50 ft in plan. The applied load resulted in average static bearing pressure of 3.1 ksf or 1.55 tsf. The actual measured average settlement was 0.9 in.

Designer Approach:

Selected N value for the foundation design = 8
this was assumed to be the most representative
value for this foundation.

Proposed Algorithm:

1. SPT values within a depth of 2B from ground surface;

   The mean N values of soil boring from ground
   surface to a depth of 2B = 10
   The Standard deviation = 3.92
   The Coefficient of variation (C) = 0.428

2. SPT values within a depth of 2B from the base of foundation;

   The mean N values of soil boring (Ntavg) = 9

The minimum N-value of soil boring (Nmin) = 4

The maximum N values of soil boring (Nmax) = 16

The minimum mean N values of soil boring (Nmm) = 9

The design N-value is then determined as ;

$$N_d = \frac{ERr}{60} * D_c * [\ A * C + (1 - C) * B\ ]$$

The parameters A and B are assuming one of the following combinations.

First combination of parameters A and B

$A = N_{min} = 4$

$B = N_{max} = 16$

$N_d = [\ 4 * 0.428 + (1-0.428) * 16\ ] = 10.86$

Second combination of parameters A and B

$A = N_{min} = 4$

$B = N_{mm} = 9$

$N_d = [\ 4 * 0.428 + (1-0.428) * 9\ ] = 6.86$

Third combination of parameters A and B

$A = N_{min} = 4$

$B = N_{tavg} = 9$

$N_d = [\ 4 * 0.428 + (1-0.428) * 9\ ] = 6.86$

Fourth combination of parameters A and B

$A = N_{mm} = 9$

$B = N_{max} = 16$

$$N_d = [\ 9 * 0.428 + (\ 1-0.428\ ) * 16\ ] = 13$$

Fifth combination of parameters A and B

$A = N_{mm} = 9$

$B = N_{tavg} = 9$

$$N_d = [\ 9 * 0.428 + (\ 1-0.428\ ) * 9\ ] = 9$$

Table 5.30. Comparison between results of algorithm, common criteria and a designer N value for case history No.17

Designer approach	* proposed equation					Common criteria			
Selected N-value	(1)	(2)	(3)	(4)	(5)	Nmin	Nmm	Nmax	Ntavg
8	10.86	6.86	6.86	13	9.0	4.0	9.0	16.0	9.0

* Number between brackets refers to the different combinations of parameters A and B in the proposed equation.

Case History No.18: Chimney of a Copper Factory, Duisburg, Germany

Schultze (1963)

Soil Conditions

Subsurface soil consisted of about 79 ft of medium dense sand and gravel followed by dense sandy clay that extended to the end of soil boring. Standard penetration tests were made at small interval 1-2 ft along one soil boring which was made immediately beneath the foundation. Figure (32), presented the SPT resistance values at the site of the structure. The ground water table was observed at a depth of approximately 36 ft below the ground surface.

Structure and Foundation

The structure consists of a reinforced concrete boiler building 219.8 ft high with a chimney extending 393.7 ft above ground surface. The structure is constructed on a square footing that is 47.57 ft by 47.57 ft in plan. The footing is founded about 11.48 ft below original ground surface.

Load and Settlement:

The total maximum applied pressure on the base of a footing was 2.60 tsf. The corresponding maximum measured settlement was 0.708 in.

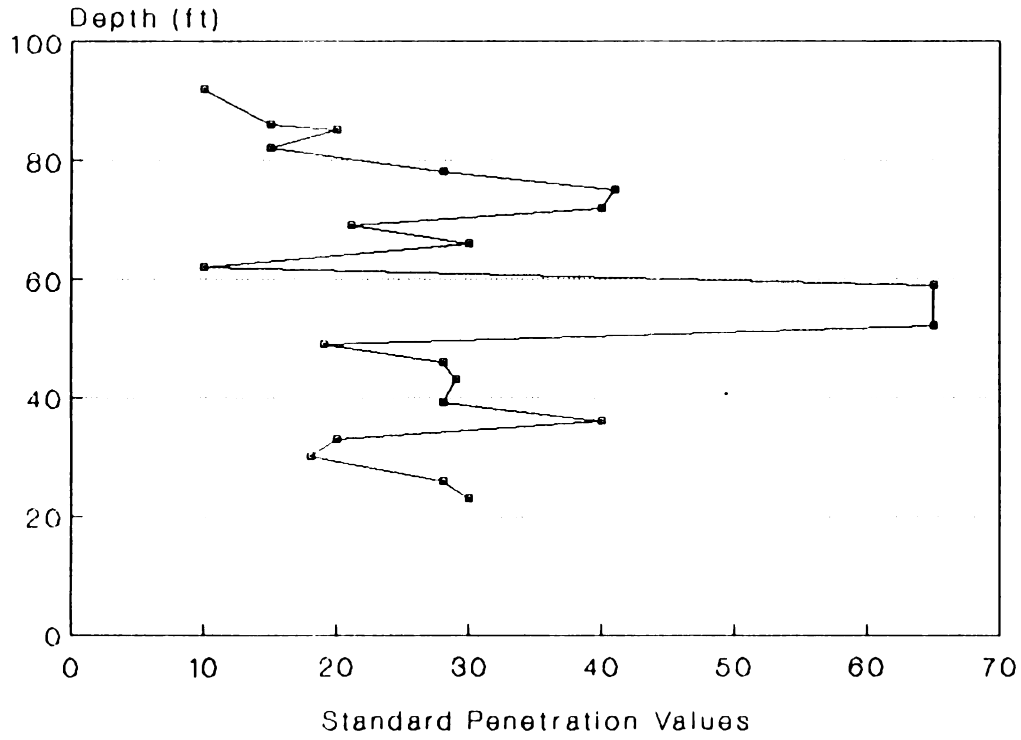


Figure 32. Results of Standard Penetration Test for a Chimney at Duisburg, Germany

Load and Settlement:

The total applied Dead load was about 2.45 tsf and the corresponding measured settlement was about 0.83 in. testing program consisted of building a stock piles of steel sheet coils on the surface of the silty sand layer that covers an area of 65 ft by 50 ft in plan. The applied load resulted in average static bearing pressure of 3.1 ksf or 1.55 tsf. The actual measured average settlement was 0.9 in.

Designer Approach:

Selected N value for the foundation design = 20
this was assumed to be the most representative
value for this foundation.

Proposed Algorithm:

1. SPT values within a depth of 2B from ground surface;

   The mean N values of all soil borings from ground
   surface to a depth of 2B = 28.57

   The Standard deviation = 15.068

   The Coefficient of variation (C) = 0.527

2. SPT values within a depth of 2B from the base of foundation;

   The mean N values of soil boring (Ntavg) = 28.57

The minimum N-value of soil boring (Nmin) = 10

The maximum N value of soil boring (Nmax) = 65

The minimum mean N values of soil boring (Nmm) = 28.57

The design N-value is then determined as ;

$$\mathbf{N_d = \frac{ERr}{60} * D_c * [\ A * C + (1 - C) * B\ ]}$$

The parameter C is Coefficient of variation Figure (32).

The parameters A and B are assuming one of the following combinations.

First combination of parameters A and B

$A = N_{min} = 10$

$B = N_{max} = 65$

$\mathbf{N_d} = [\ 10 * 0.527 + (\ 1\text{-}0.527\ ) * 65\ ] = 36$

Second combination of parameters A and B

$A = N_{min} = 10$

$B = N_{mm} = 28.57$

$\mathbf{N_d} = [\ 10 * 0.527 + (\ 1\text{-}0.527\ ) * 28.57\ ] = 18.78$

Third combination of parameters A and B

$A = N_{min} = 10$

$B = N_{tavg} = 28.57$

$\mathbf{N_d} = [\ 10 * 0.527 + (\ 1\text{-}0.527\ ) * 28.57\ ] = 18.78$

Fourth combination of parameters A and B

$A = N_{mm} = 28.57$

$B = N_{max} = 65$

$$N_d = [\ 28.57 * 0.527 + (\ 1-0.527\ ) * 65\ ] = 45.8$$

Fifth combination of parameters A and B

$A = N_{mm} = 28.57$

$B = N_{tavg} = 28.57$

$$N_d = [\ 16 * 0.527 + (\ 1-0.527\ ) * 28.57\ ] = 28.57$$

Table 5.31. Comparison between results of algorithm, common criteria and a designer N value for case history No.18

Designer approach	* proposed equation					Common criteria			
Selected N-value	(1)	(2)	(3)	(4)	(5)	Nmin	Nmm	Nmax	Ntavg
20	36	18.78	18.78	45.8	28.57	10	28.57	65	28.57

* Number between brackets refers to the different combinations of parameters A and B in the proposed equation.

Case History No.19: A Reactor Building in Stetternich/Julich, Germany

Schultze (1963)

Soil Conditions

One soil boring was made beneath the foundation of this structure. The soil consisted of about 7 ft of sandy silt followed by 44 ft of sand and gravel and then by 18 ft of sandy clayey silt. The silt underlain by very dense fine sand that extended to the end of soil boring which is terminated at depth 82 ft below original ground surface. Figure (33) presents the results of standard penetration tests at the site of the structure. The ground water table was encountered at a depth of about 9 ft below ground surface.

Structure and Foundation

This is a 111.5 ft high reinforced concrete reactor building. It is built on a circular footing foundation 108 ft in diameter. The footing is constructed at a depth of 17 ft below ground surface.

Load And Settlement:

The total maximum applied pressure on the base of a raft was 2.2 tsf. The corresponding maximum measured settlement was 1.732 in.

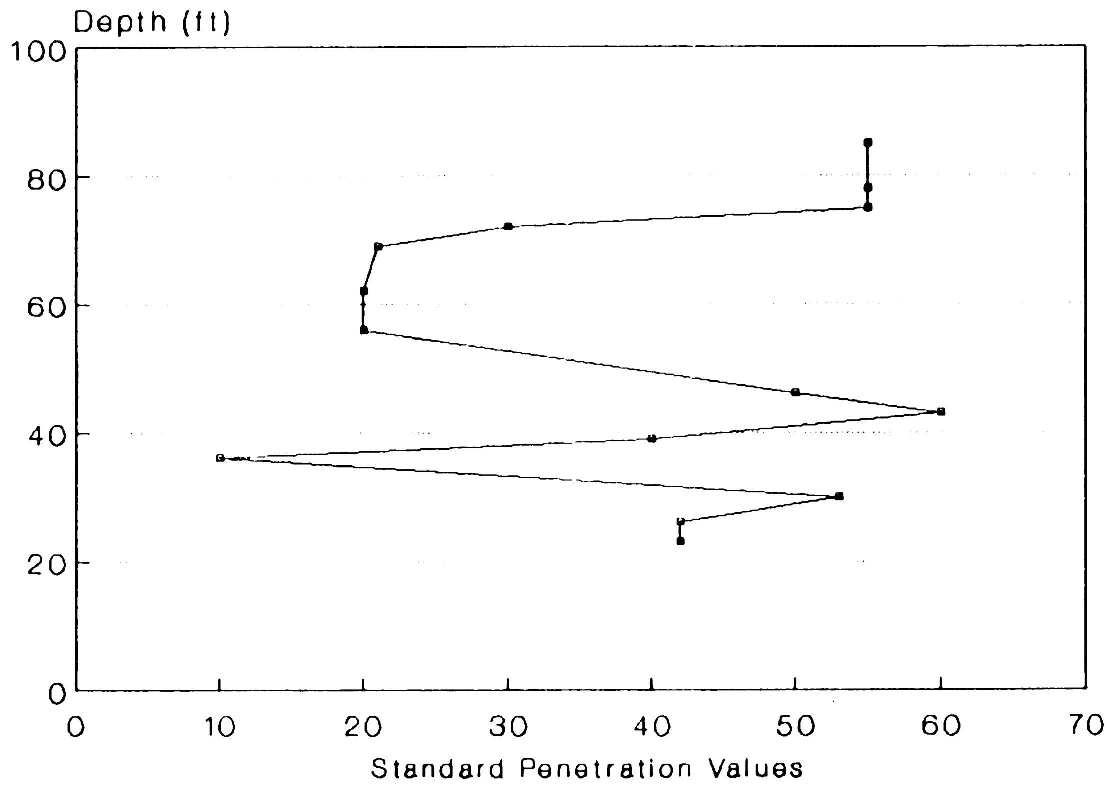


Figure 33. Results of Standard Penetration Test for a Reactor in Julich, Germany

Load and Settlement:

The total applied Dead load was about 2.45 tsf and the corresponding measured settlement was about 0.83 in. testing program consisted of building a stock piles of steel sheet coils on the surface of the silty sand layer that covers an area of 65 ft by 50 ft in plan. The applied load resulted in average static bearing pressure of 3.1 ksf or 1.55 tsf. The actual measured average settlement was 0.9 in.

Designer Approach:

Selected N value for the foundation design = 30
this was assumed to be the most representative
value for this foundation.

Proposed Algorithm:

1. SPT values within a depth of 2B from ground surface;

   The mean N values of all soil borings from ground
   surface to a depth of 2B = 36.71

   The Standard deviation = 16.127

   The Coefficient of variation (C) = 0.439

2. SPT values within a depth of 2B from the base of foundation;

   The mean N values of soil boring (Ntavg) = 36.71

The minimum N-value of soil boring (Nmin) = 10

The maximum N value of soil boring (Nmax) = 55

The minimum mean N value of soil boring (Nmm) = 36.71

The design N-value is then determined as ;

$$N_d = \frac{ERr}{60} * D_c * [\ A * C + (\ 1 - C\ ) * B\ ]$$

The parameter C is Coefficient of variation Figure (33).

The parameters A and B are assuming one of the following combinations.

First combination of parameters A and B

$A = N_{min} = 10$

$B = N_{max} = 55$

$N_d = [\ 10 * 0.439 + (\ 1\text{-}0.439\ ) * 55\ ] = 35.25$

Second combination of parameters A and B

$A = N_{min} = 10$

$B = N_{mm} = 36.71$

$N_d = [\ 10 * 0.439 + (\ 1\text{-}0.439\ ) * 36.71\ ] = 24.98$

Third combination of parameters A and B

$A = N_{min} = 10$

$B = N_{tavg} = 36.71$

$N_d = [\ 10 * 0.439 + (\ 1\text{-}0.439\ ) * 36.71\ ] = 24.98$

Fourth combination of parameters A and B

$$A = N_{mm} = 36.71$$

$$B = N_{max} = 55$$

$$\mathbf{N_d} = [\ 36.71 * 0.439 + (\ 1\text{-}0.439\ ) * 55\ ] = 46.97$$

Fifth combination of parameters A and B

$$A = N_{mm} = 36.71$$

$$B = N_{tavg} = 36.71$$

$$\mathbf{N_d} = [\ 36.71 * 0.439 + (\ 1\text{-}0.439\ ) * 36.71\ ] = 36.71$$

Table 5.32. Comparison between results of algorithm, common criteria and a designer N value for case history No.19

Designer approach	* proposed equation					Common criteria			
Selected N-value	(1)	(2)	(3)	(4)	(5)	Nmin	Nmm	Nmax	Ntavg
30	35.25	24.98	24.98	46.97	36.71	10	36.71	55	36.71

*** Number between brackets refers to the different combinations of parameters A and B in the proposed equation.**

Case History No.20: Student Building,
Aachen, Germany
Schultze (1963)

Soil Conditions

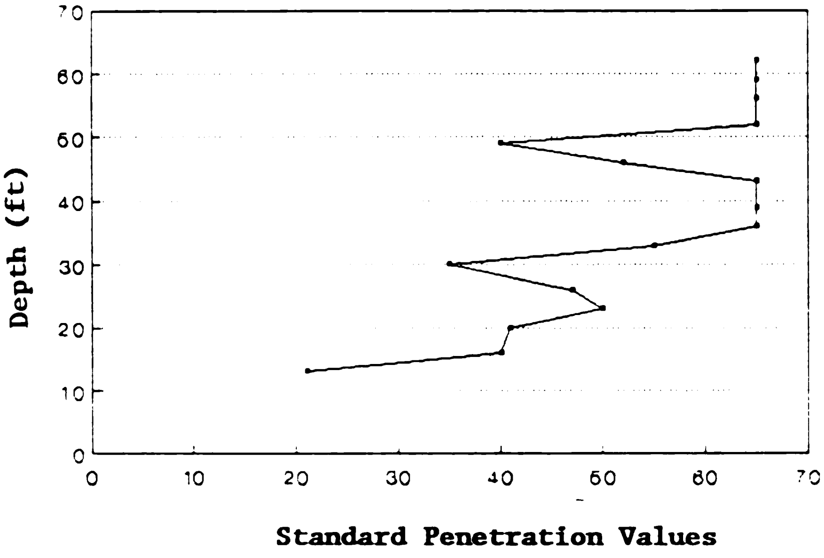
Soil foundation at the location of this structure was explored by two soil borings. The soil consists of about 5.0 ft of silt followed by 13-33 ft of dense sand and gravel then by alternating layers of silty sand and compact clayey silt. Figure (34), presents the results of standard penetration tests at the site of the structure. The ground water table was about 24.6 ft below ground surface.

Structure and Foundation

This is a 22 stories reinforced concrete building with steel skeleton. The building is 200 ft high and it contains two basements. It is constructed on 49.2 ft by 85.3 ft raft foundation. The base of foundation is at 19.68 ft below ground surface.

Load and Settlement:

The total maximum applied pressure on the base of a raft was 1.39 tsf. The corresponding maximum measured settlement was 0.465 in.



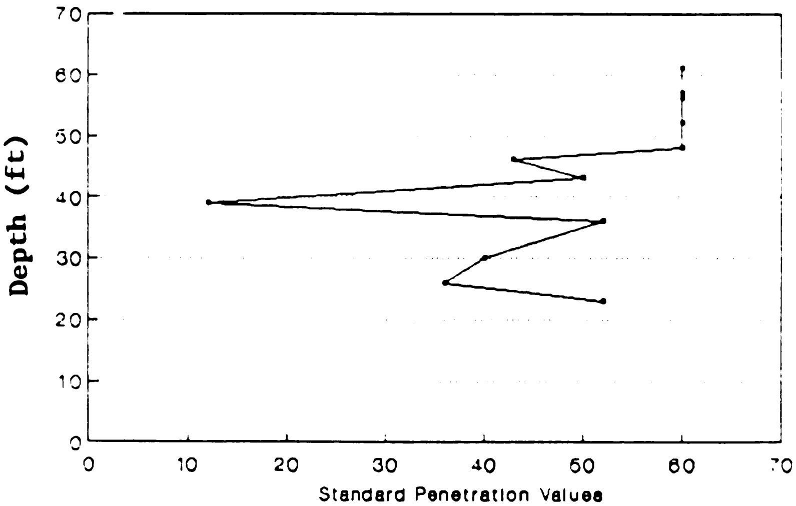


Figure 34. Results of Standard Penetration Test for a Student Center in Germany

Load and Settlement:

The total applied Dead load was about 2.45 tsf and the corresponding measured settlement was about 0.83 in. testing program consisted of building a stock piles of steel sheet coils on the surface of the silty sand layer that covers an area of 65 ft by 50 ft in plan. The applied load resulted in average static bearing pressure of 3.1 ksf or 1.55 tsf. The actual measured average settlement was 0.9 in.

Designer Approach:

Selected N value for the foundation design = 55
this was assumed to be the most representative
value for this foundation.

Proposed Algorithm:

1. SPT values within a depth of 2B from ground surface;

   The mean N values of all soil borings from ground surface to a depth of 2B = 50.75

   The Standard deviation = 13.933

   The Coefficient of variation (C) = 0.275

2. SPT values within a depth of 2B from the base of foundation;

   The mean N values of soil borings (Ntavg) = 50.75

The minimum N-value of soil borings (Nmin) = 12

The maximum N value of soil borings (Nxavg) = 57

The minimum mean N values of soil borings (Nmm) = 48.75

The design N-value is then determined as ;

$$N_d = \frac{ERr}{60} * D_c * [\ A * C + (1 - C) * B\ ]$$

The parameter C is Coefficient of variation Figure (34).

The parameters A and B are assuming one of the following combinations.

First combination of parameters A and B

$A = N_{min} = 12$

$B = N_{xavg} = 57$

$N_d = [\ 12 * 0.274 + (1\text{-}0.274) * 57\ ] = 44.67$

Second combination of parameters A and B

$A = N_{min} = 12$

$B = N_{mm} = 48.75$

$N_d = [\ 12 * 0.274 + (1\text{-}0.274) * 48.75\ ] = 38.68$

Third combination of parameters A and B

$A = N_{min} = 12$

$B = N_{tavg} = 50.75$

$N_d = [\ 12 * 0.274 + (1\text{-}0.274) * 50.75\ ] = 40.13$

Fourth combination of parameters A and B

$A = N_{mm} = 48.75$

$B = N_{xavg} = 57$

$N_d = [\ 48.75 * 0.274 + (\ 1-0.274\ ) * 57\ ] = 54.74$

Fifth combination of parameters A and B

$A = N_{mm} = 48.75$

$B = N_{tavg} = 50.75$

$N_d = [\ 48.75 * 0.274 + (\ 1-0.274\ ) * 50.75\ ] = 50.20$

Table 5.33. Comparison between results of algorithm, common criteria and a designer N value for case history No.20

Designer approach	* proposed equation					Common criteria			
Selected N-value	(1)	(2)	(3)	(4)	(5)	Nmin	Nmm	Nxavg	Ntavg
55	44.67	38.68	40.13	54.74	50.20	12	48.75	57	50.75

* Number between brackets refers to the different combinations of parameters A and B in the proposed equation.

Case History No.21: Steel Mill Building,
Indiana, U.S.A.
D'Applonia et al.,(1968)

Soil Conditions

The soils at this site is mainly of glacial origin. The soil consisted of a thick layer of fine beach and dune sand that has covered outwash and organic material. The outwash underlain by glacio-fluvial sand and then by a thick layer of about 75 ft of glacial till. The till material is underlain by bedrock. Figure (14), presents the variation in average standard penetration tests results for 96 soil borings that were made at the site of the structure. The ground water table was observed near the ground surface but it was lowered to a depth of about 18 ft below ground surface by a dewatering system at the time of footings constructions.

Structure and Foundation

This is a large steel mill building. The structural foundation consisted of a group of reinforced concrete rectangular footings. The footings supported columns that were spaced 50 ft centers. The area of footings varies from 150 ft^2- 450 ft^2. The base of footings varies in depth from 6-15 ft below the ground surface. This study concerns analysis of a footing with dimension of 11.2 ft by 17.7 ft.

Load and Settlement:

The total applied pressure at the base of each column footings was about 3.0 tsf. This includes the pressure from the weight of the footing columns,roof trusses,and a crane dead and live load together with live load from wind and snow and other miscellaneous live loads. However, this pressure was estimated assuming a uniform pressure distribution on the base of footings. No attempt,therefore,was made to account for eccentricity of the applied loads which icreases the toe pressure to 4.5 tsf. After four years of settlement monitoring, only few footings had settlement greater than 0.5 inch.

Designer Approach:

Selected N value for the foundation design = 15
this was assumed to be the most representative
value for this foundation.

Proposed Algorithm:

1. SPT values within a depth of 2B from ground surface;

   The mean N values of all soil borings from ground surface to a depth of 2B = 20

   The Standard deviation = 6.6

   The Coefficient of variation (C) = 0.33

2. SPT values within a depth of 2B from the base of foundation;

   The mean N values of soil borings (Ntavg) = 22

The minimum N-value of soil borings (Nmin) = 6

The maximum mean N values of soil borings (Nxavg) = 38

The minimum mean N values of soil borings (Nmm) = 8

The design N-value is then determined as ;

$$N_d = \frac{ERr}{60} * D_c * [\ A * C + (1 - C) * B\ ]$$

The parameter C is Coefficient of variation Figure (14).

The parameters A and B are assuming one of the following combinations.

First combination of parameters A and B

$A = N_{min} = 6$

$B = N_{xavg} = 38$

$N_d = [\ 6 * 0.33 + (1-0.33) * 38\ ] = 27.44$

Second combination of parameters A and B

$A = N_{min} = 6$

$B = N_{mm} = 8$

$N_d = [\ 6 * 0.33 + (1-0.33) * 8\ ] = 7.34$

Third combination of parameters A and B

$A = N_{min} = 6$

$B = N_{tavg} = 22$

$N_d = [\ 6 * 0.33 + (1-0.33) * 22\ ] = 16.72$

Fourth combination of parameters A and B

$A = N_{mm} = 8$

$B = N_{xavg} = 38$

$N_d = [\ 8 * 0.33 + (\ 1\text{-}0.33\ ) * 38\ ] = 28.1$

Fifth combination of parameters A and B

$A = N_{mm} = 8$

$B = N_{tavg} = 22$

$N_d = [\ 8 * 0.33 + (\ 1\text{-}0.33\ ) * 22\ ] = 17.38$

Table 5.34. Comparison between results of algorithm, common criteria and a designer N value for case history No.21

Designer approach	* proposed equation					Common criteria			
Selected N-value	(1)	(2)	(3)	(4)	(5)	Nmin	Nmm	Nxavg	Ntavg
15	27.44	7.34	16.72	28.1	17.38	6	8	38	22

* Number between brackets refers to the different combinations of parameters A and B in the proposed equation.

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In general, the compressibility of sand is not strongly related to SPT results. The latter depends on current effective stress level, but compressibility is strongly related to soil stress history and can be significantly affected by minor changes in stress history (Clayton, et al., 1985). Clayton et al.,(1988) showed that the relationship between SPT and soil compressibility can assume different forms, as shown in Figure (4), so that soils with the same penetration resistance cannot

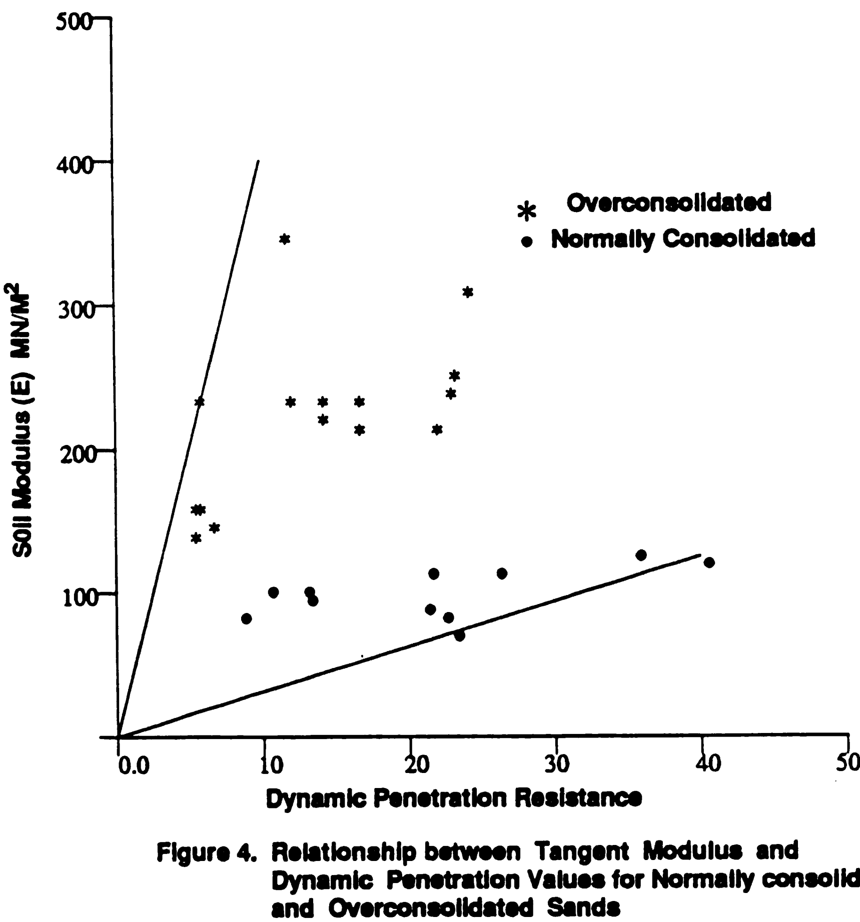

Figure 4. Relationship between Tangent Modulus and Dynamic Penetration Values for Normally consolidated and Overconsolidated Sands