SETTLEMENT OF FOUNDATIONS ON SAND

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ABSTRACT

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In this major techical report the problem of settlement of foundations on sand is summarized with major emphasis on methods of predicting settlement on sand by in-situ testing.

The factors affecting settlement of sand deposits are investigated and the evaluation of sand compressibility characteristics is treated.

Major emphasis is given to methods of predicting settlement by data obtained through the Standard Penetration Test and the Dutch cone static penetration test. The different established methods of predicting settlement, based on the above tests are examined with their advantages and limitations.

Finally correlations between the data from the two tests are given.

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TABLE OF CONTENTS

Table of Contents (cont'd)

LIST OF FIGURES

FIGURE PAGE

List of Figures (cont'd)

List of Figures (cont'd)

LIST OF TABLES

NOTATIONS

a_o	Slope of settlement curve for a 1 ft. square plate loading test
a_B	Slope of settlement curve for a footing of width B
B	Width of a footing
C	Constant of compressibility
C_D	Correction factor for depth of embedment
C_N	Correction factor for overburden pressure influence
C_T	Correction factor for incompressible layer presence
C_W	Correction factor for the ground water table effects
C_1	Correction factor for the depth of embedment effect
C_2	Correction factor for time effects
D	Depth of footing embedment
D_f	Depth of footing embedment
D_R	Relative density
D_W	Depth of water table
E, E_s	Young's modulus of elasticity
e	Void ratio
H	Thickness of a sand layer
I_z	Vertical strain influence factor
L	Length of a footing
M	Modulus of compressibility
m	Slope factor of contact area
m'	Relative slope factor
N	Blow count of S.P.T.
N_B	Corrected N-values for overburden pressure

N_e, N_c	Corrected N-value for overburden pressure
N'	Measured N-value needing correction
n	Length to width ratio of a footing
P	Average applied pressure
p	Effective vertical overburden pressure
P_o	Effective vertical overburden pressure at a certain depth
q	Contact pressure
q_c	Point resistance of static penetrometer
S	Settlement
S_B	Settlement of a foundation having width B
S_o	Settlement of a 1 ft. square plate loading test
T	Depth of incompressible layer
U_o	Influence factor reflecting the influence of embedment and layer thickness
U_1	Influence factor reflecting the influence of footing dimensions
γ	Unit weight of sand
γ_d	Unit weight of sand at a certain density
ν	Poisson's ratio
σ	Overburden pressure
$\bar{\sigma}$	Effective overburden pressure
Δh	Vertical compression of a sand layer having constant C
ΔH	Thickness of a layer having constant C

Δp	Increment in pressure
ϕ	Angle of internal friction
ϵ	Strain
R.D.	Relative density
S.P.T.	Standard Penetration Test

CHAPTER I

INTRODUCTION

1.1 General

Structures impose stresses on the supporting soil causing deformations in the soil mass and settlement of the structure. In the design of foundations, two requirements must be satisfied. First the load must be sufficiently less than the ultimate bearing capacity of the foundation to insure foundation stability, second the total and differential settlement must be small enough so that the superstructure will not be damaged by the foundation movements. If the stability of the foundation is ensured with an adequate margin of safety, the design of foundations is governed largely by the requirement that the foundation movements must be within limits that can be tolerated by the superstructure. Thus, reliable and accurate methods for estimating foundation settlement are essential for the design of foundations.

The simplest and most widely used procedure for estimating settlement in cohesionless soils employs the use of insitu tests such as penetration tests. The most commonly used penetration tests are the Standard Penetration Test and the Dutch cone Penetration Test. The penetration resistance has been correlated to footing settlement by empirical relations. However, such empirical correlations are only approximate and are influenced by a number of factors. These are the water presence, the stress history of the sand deposits and the sand characteristics.

Due to these factors and because the interpretation of the Penetration test data is not unique, several methods of predicting settlement have been established. Some of these methods are not original but are modifications of the originally developed in order to achieve a better agreement between the estimated and measured settlement of foundations.

Although the penetration tests have many shortcomings they have the great advantage of giving the in-situ engineering characteristics of sand deposits. Also both are relatively simple to perform and their cost is small. Hence, the methods of predicting settlement of foundatios on sand, based on penetration test data are the most commonly used and possibly the best applicable for estimating the settlement of foundations on sand.

1.2 Settlement of foundations on sand

In soil mechanics literature the terms settlement and consolidation (or long-term) settlement are used. This dinstinction is based on the rate of settlement and depends on the loading conditions simulation. The settlement problem may be simulated by one of the two following conditions. The drained and the undrained-drained. The first is widely used for foundations on cohesionless soils. These soils usally have such high permeability that no excess hydrostatic porewater pressure develops during normal construction operations, so the drained condition holds. With soils consisting of clay the permeability is so small that very little dissipation of excess hydrostatic porewater pressure can occur during construction, hence, the loading corresponds to the undrained condition. After the end of construction,the load may remain nearly constant, and the excess hydrostatic porewater pressure dissipates by consolidation. This sequence is called the undrained-drained condition.

In cohesionless soils the immediate-settlement predominates with possibly some creep effects. In sands, as in all fine grained soils the settlement takes place as soon as the load is applied and is completed in less than about 7 days, because of the applied loads and site vibrations.

The settlement of a loaded area is usually nonuniform,hence it is necessary to distinguish between the maximum settlement and the differential settlement. The differential settlement is often more important as it produces distortions of the structure and other surface installations. Terzaghi and Peck (1967) have recommended that a maximum differential settlement of 0.75 in. be adopted for ordinary structures.

Because of difficulties encountered in the sampling and testing of undisturbed samples of cohensioless soils there have been only a few cases in which the immediate settlement could be predicted on the basis of theory. From a practical viewpoint a very important characteristic of immediate settlement is that the strains in the soil from which the settlement is produced, occur in the zone immediately underneath the loaded area The stresses are highest immediately beneath the load and decrease rapidly with depth. At a depth egual to twice the width B of the load,the vertical stress is only about one fourth of the applied pressure. Hence in general it can be said that the settlement is primarily a result of the strains in a zone down to a depth of 2B. It follows then, that the soil properties in this zone exert the largest influence on the immediate settlement.

It is of interest to note that the available procedures for settlement computations are based on many assumptions that are often not satisfied in reality. Between the many difficulties encountered, there is the basic difficulty of obtaining undistributed sample apart from the question of what area the sample is represantative. Studies in natural sand deposits confirm the fact that most deposits of sand are highly variable in their properties. So in the case of an erratic subsoil, such as a sand deposit the variations in soil properties are likely to be very great, and under such conditions settlement calculations become meaningless until the general pattern of the variations in soil properties can be determined. These difficulties in calculating the settlement in cohesionless soils have led to the development of a number of semi-empirical methods. These methods correlate the settlement with the relevant properties, by means of in-situ tests.

As a matter of fact, it appears that settlement predictions of foundations on cohesionless soils are usually conservative with current methods. This is probably due to the high degree of conservatism in the semi-empirical methods by which the prediction is made.

CHAPTER 2

FACTORS AFFECTING FOUNDATION SETTLEMENT IN SAND

2.1 General

A proper evaluation of settlement in sand can only be obtained when the factors affecting settlement or compressibility have been properly investigated. The tendency of sand to settle or compress under load is influenced by two groups of characteristics of the sand deposits. The first consists the factors associated with the quantitative sand characteristics alone and the second encompasses the in-situ characteristics of the sand deposits. Factors which are included in the first category are the mineralogy of the sand, the grain size distribution and also the cohesive admixtures in the sand. Factors associated with the second category are the relative density of the sand (which is also associated with the characteristics of the first category), overburden pressure, water table level, and stress history of the sands deposits. Of the above mentioned factors the most marked influence have the second group characteristics and between them the relative density and overburden pressure. The influence of the above factors can affect settlement as discussed in the following section.

2.2. Factors associated with quantitative sand characteristics.

The settlement in sand deposits is associated with a rearrangement of the sand particles . The grain shape affects the particle rearrangement because the degree of roundness influences the tendency towards rearrangement and the closer packing of the particles. Particles ride up on one another during rearrangement so that the more rounded the grains the greater the likehood that this riding up will occur. With increased angularity a greater amount of particle interlocking occurs inhibiting particle rearrangement and thus decreasing compressibility. On the other hand an increased angularity would perhaps result in a more open structure to the fabric of the sand deposits which would be reflected in a greater difference between the maximum and minimum void ratio. This opportunity for a greater change in void ratio result in an increased compressibility. So the two effects of a more open structure with angular grains and thus increased compressibility and a decreased tendency towards the rearrangement of grains due to interlocking and less riding up would both be present. Schultze and Moussa (1961) suggest that the former predominates and that increased angularity results in a net increased compressibility. The roundness may be a more domonant effect in finer material.

The rearrangement of sand particles is a result of grain crushing and also of elastic deformations which occur at inter-particle contact points under load. The mineralogy of sand is of importance because it is associated with the strength of sand particles which affects the crushing and deformation of grains. As an example, quartz grains are more resistant to crushing and elasticdeformations than the weaker feldspar grains. The higher the streses at the contact points the more crushing would occur. At low confining stress levels angular, rough grains result in higher compressibility due to crushing at the contact points. This effect may be less, however, the finer the material. The separate and varying effects of roundness, angularity and grain crushing would seem to exhibit a combined effect of generally increased compressibility with increased angularity.

The grain size distribution also affects the tendency of sand settling. The greater the number of different sizes of particles represented, the greater is the likehood that voids formed by larger particles are filled in with smaller particles. The result is a decreased void ratio (increased relative density) and thus decreased compressibility.

The presence of other admixtures, and especially cohesive ones, affects the sand compressibility. The compressibility of clean sands is reduced by the addition of fine grained materials as has been pointed out. The compressibility is reduced by approximately 2.5 times with the addition of 4% to 9% of silt. Similar quantities of clayey silt may reduce the compressibility by about 7.5 times, because the fine grain materials fill the void space and effectively prevent the sand particles from moving closer together. There is a limit to this effect however. With greater quantities of clay and silt the sand particles are generally separated and the mixture behaves more like a cohesive soil. The compressibility of the mixture is then influenced by the consolidation characteristics of the clay.

2.3. Factors associated with in-situ sand characteristics.

The relative density and overburden pressure are the most important factors of the in-situ characteristics of sand deposits. The relative density is an indication of the voids in the sand. It has been observed that as the initial void ratio increases, the settlement at a given pressure also increases. So as the initial relative density of a sand deposit decreases the settlement of the sand increases under a given load. This has been observed by many investigators such as Schultze and Moussa (1961).

An increased overburden pressure has the effect of increasing the confining pressure on an element of sand and reducing the lateral strain. The greater the restriction on lateral strain the more dominant is the vertical strain relative to volumetric compression, so that at increased overburden pressure, compressibility is decreased. This effect of overburden pressure in decreasing compressibility occurs even though relative density may decrease. The investigations of Gibbs and Holtz (1957) show an increased resistance to penetration with increasing overburden pressure even though relative density is decreasing.

The sand compressibility is also affected by the effect of precompression or preloading of a sand deposit. The effect of precompression is to decrease the compressibility. The preloadind has the effect of interlocking and prestressing the sand particles. This action is more pronounced in statically preloaded deposits than in dynamically preloaded deposits.

The water content also affects compressibility. Upon saturation there is no apparent change in the compressibility of a sand deposit, but at moisture contents less than full saturation the compressibility is decreased. This is due to capillary forces acting in partially saturated sands, retarding the rearrangement of grains. The resistance to penetration is observed to decrease by an average of 15% relative to values above the ground water level. Some authors believe that the effect of ground water level is reflected in penetration resistance, while others believe that this effect must be taken into consideration by means of correction to penetration resistance.

CHAPTER 3

METHODS OF SETTLEMENT PREDICTION

3.1 General

Settlement analysis is frequently made for projected construction. The objective is to predict whether excessive settlement is likely to occur and cause damage. Different procedures and methods have been proposed for solving the problem of settlement prediction. Settlement computations are particularly difficult because they tend to be a problem in elasticity and the soils are elastic in only very small strains and also it is very difficult to obtain the elastic properties of a soil on the site. In spite of the above shortcomings, it is convenient to treat the soil as an elastic isotropic, homogeneous mass for estimating settlement. A knowledge of the compressibility characteristics of the soil is required for the calculation of settlement. This can be obtained directly by laboratory testing of samples or indirectly by means of in-situ tests.

Methods used for calculating the settlement on sand layers, produced by additional loading of the subsoil may be classified in four quantitatively dinstinct groups of solutions depending on the preferred kind of solution.

The first group includes solutions based on the theory of elasticity. These methods include mathematically exact methods of the elasticity theory which satisfy the equilibrium conditions, the compatibility equations and the boundary conditions. It can be applied linear or non linear solutions and the soil may be treated as homogeneous or non-homogeneous. The most refined of these methods, the theory of elastic homogeneous isotropic and linear half space, is mathematically exact but its constitutive equations do not usually correspond to real foundation soils, hence the results obtained by this means are at considerable variance with reality. Due to the effort required for these solutions, to problems of obtaining the proper constants of elasticity and to limited accuracy of the predictions due to inherent difficulties, these methods are not preferable by engineers.

The second group encompasses engineering methods which are most widely used for settlement calculations. In this group which makes use of indirect methods of calculations, may be included methods as the stress-path method, the Skempton- Bjerrum method, and the oedometric compression method. This group of methods makes use of the elasticity theory only for stress calculations. The stress-strain relations needed are dirived from experimental data. This is the reason why these methods in contrast to the former direct methods, are called indirect. The stress field is supposed to be invariant to a considerable degree with respect to the constitutive relations as well as to the mechanical properties of the subsoil. The methods of this group readily introduce the non-linearity of the stress-strain relations of a real subsoil, and this fact is the reason why the calculated and the actual settlements are generally in good agreement.

In the third group are included empirical and semi-empirical methods. Empirical or semi-empirical formulae are used to predict the results of loading or mechanical tests for actual foundations. Methods included in this group are the plate loading test method, the standard penetration test methods, the static penetreation test methods and the pressuremeter method. Due to many advantages inherent in these methods such as the convenience of calculations and the necessity of performing in site sounding tests, and also the rather conservative but accurate results, the methods of this group are the most accepted and widely performed methods of settlement calculations.

In the forth group are included methods based on numerical methods and demanding the application of computers. Methods included are the finite element method and the method of finite differences. These methods are characterized by the introduction of real non-linear stress-strain relations, and possibly of anisotropy and non-homogeneity and the accuracy of the solution from the mathematical standpoint is limited only by the capacity of the computer.

In the following the semi-empirical methods and especially the methods based on penetration test data will be examined.

CHAPTER 4

SEMI-EMPIRICAL METHODS

4.1 General

The heterogeneous nature of natural sand deposits, the many difficulties of obtaining undisturbed samples of cohesionless soils and also the major difficulties in establishing the in situ compressibility of sands by laboratory tests have forced as a practical expedient, the adoption of settlement prediction methods known to be approximate, empirical, or of limited applicability. These methods start from the results of some type of field tests of the mechanical properties of the foundation soil, which cannot uniquely be interpreted theoritically, and use empirical correlations to predict the settlement of an actual foundation. They have been classified as semi-empirical because of their origin which is absolutely empirical and the partial theoretical interpretation of the results.

The most common types of field tests used are small scale plate loading tests, and penetration tests in which the resistance of the soil to penetration is measured. First to be introduced were the plate loading tests to evaluate the soil bearing pressure. The main problems with this type of field tests were obtaining an accurate interpretation of the results, and the number of tests which must be contacted in order to achieve represantative soil conditions between the model and the prototype footing. The interpretation of results and hence the correlation between the prototype footing settlement and that of the plate, as an approximate and empirical relationship involved a significant scattering from the average. Also representative subsurface conditions between the zone below the prototype and that of the plate were rarely achieved because minor density variations in the zone below the plate were of significant importance for the plate load test, but were insignificant for the prototype footing. Hence the plate loading tests were an expensive and time-consuming operation, compared with the penetration testing developed, and also gave less accurate predictions.

The practical alternative to conducting plate load tests is to use penetration tests, since this technigue is also used for the routine subground soil exploration. The penetration test consists of driving a metal bar or cone into the soil, using a constant driving effort. Penetrometers can be classified as either dynamic or static, depending on the used method for driving the penetrometer into the soil. In the dynamic method the penetrometer is driven into the soil using a constant dynamic driving effort as hammering, while the static penetrometer is forced slowly into the ground by means of a nondynamicaly applied force.

Of the many penetrometer types developed the most commonly used and accepted is the Standard Penetrometer Test (S.P.T.), and the Dutch Static Cone Test. The first is a dynamic penetrometer and was developed in U.S.A while the second is a static one and was developed in the Down Countries.

Terzaghi and Peck in 1948 were the first to correlate plate bearing tests results with S.P.T. results and this was a basic factor in the development of the method of calculating settlements using S.P.T. values. After that many methods were presented of calculating settlement based on both S.P.T. and static Penetration test values.

Despite many disadvantages and limitations penetrometer tests are relatively simple to perform and cost less than plate bearing tests. Also an amount of data of limited accuracy can be accumulated quickly and economically.

Another method used for predicting settlement is based on the data obtained in the field by a pressuremeter. The pressuremeter was developed by Menard (1956, 1969) and consists of two main portions, a probe and a pressure-volumeter connected by plastic tubes through which water and gas are applied. The required pressures and volume changes are measured, giving a type of stress-strain test in place. Since this is a relatively new method it is not so well known, and hence has a very limited applicability.

CHAPTER 5

METHODS BASED ON STANDARD PENETRATION TEST

5.1. The Standard Penetration Test

The Standard Penetration Test (S.P.T) was developed in the U.S.A presumably for use in the assessment of the compactness of soil. The origin of the S.P.T. can be found in the beginning of this century when the practice of driving a 1 inch open-end pipe into the soil to recover dry samles of soils in making borings, marked the end of the wash boring process and the beginning of the dynamic sampling of soils. The standardization of this technique was developed in the late 1920's, when the 140 lb weight falling 30 in. and the 2 in. split sample spoon were introduced. This type of penetration test which became known as the Standard Penetration Test was developed by the Raymond concrete and Pile Co. for use on pile and caisson projects. Since then, only minor modifications have been adopted, and the S.P.T. has been a widely used test for the evaluation of properties of soils in situ.

The S.P.T. has been defined by the A.S.T.M. test method designation D1586-63T. It consists of driving a sampling spoon, with an ouside diameter of 2 in., by means of a 140 lb (63.5 kg) drop hammer falling from a height of 30 in. (76.2 cm) and is carried out in a cased hole. The borehole is cleaned out with the casing tube advanced slightly below the bottom of the boring. The penetrometer is lowered to the base of the borehole, is driven an initial 6 in. and is then driven a further

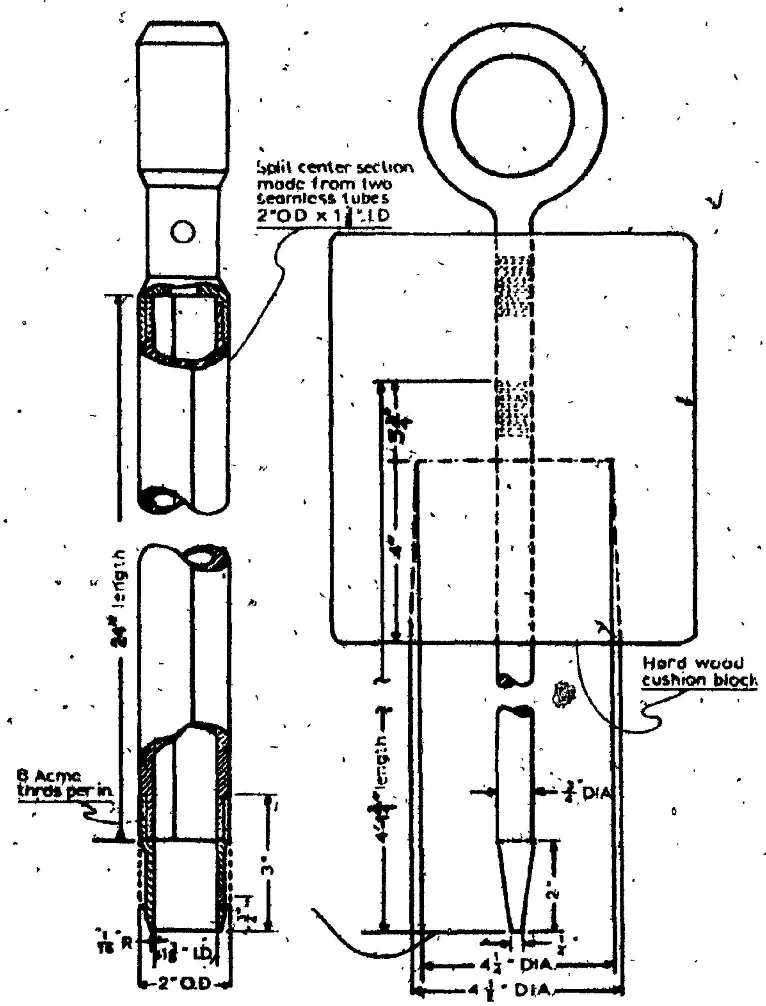
12 in. The sampling spoon dimensions are shown in Figure (1).

The resistance to penetration is measured and is expressed by the number of blows N, required for the penetration through the 12 in. length from the initial 6 in. to 18 in. Hence, the S.P.T. is a measure of the dynamic strength of the soil being penetrated. The driving resistance is made up of frictional resistance on the inside and outside walls of the sampler and a type of end-bearing resistance around the cutting edge of the sampler.

The interpretation of the S.P.T. results is based on the development of correlation between the blow-count N and the engineering properties of soils.

In saturated, fine or silty, dense or very dense sands the N-values may be abnormally great because of the tendency of such materials to delate during shear, under undrained conditions. In such soils the results of S.P.T. should be interpreted conservatively. Also the N-values in cohesionless soils are influenced to some extent by the depth at which the test is made. Because of the greater confinement caused by increasing overburden pressure, N-values at increasing depths may indicate larger relative densities than actually exist.

Many investigations have been carried out to determine what variables affect the N-values and to what extent they are affected. The major findings of these investigations have been that blowcount is influenced primarily by sand density and effective stress state. The particle size does not appear to have a major influence provided that gravel sizes



(a) SPLIT SAMPLE SPOON (b) 140 LB. DROP WEIGHT

Fig. 1 Apparatus for Standard Penetration Test.

are not present. The Bureau of Reclamation of U.S.A. has carried out extensive research to investigate the extent to which the N-values are influenced by the above factors. It was concluded that any correlation between the S.P.T, relative density and effective overburden stress can only be approximate.

Because the S.P.T.is a measure of the sand's resistance to dynamic penetration and this resistance is dependent on a host of variables that include particle shape, size and gradiation in addition to density and stress state, the interpretation of the S.P.T. N-values is not unique but depends on the in-situ sand characteristics.

Altough the S.P.T. has many shortcomings and cannot be regarded as a refined and completely reliable method of investigation, experiance over the years has confirmed that the S.P.T. can be used to interpret, at least qualitatively the character of inplase sand deposits. The test provides an indirect method of evaluating to a certain extent, the mechanical properties of subsurface soils. It has been established that it has fairly reliable application to granular cohesionless soils but has less application to cohesive soils which are appreciably influenced by moisture content and clay mineral characteristics.

5.2 Terzaghi and Peck method (1948, 1967)

Terzaghi and Peck (1948) were the first who proposed a correlation between blow count N and settlement of footings on sand. Originally the method was intended to serve as a conservative design tool in predicting the allowable bearing pressure for a given footing and for one inch of foundation settlement, because according the authors "the maximum settlement of the foundation should not exceed one inch". Since the method was developed mainly as a design tool, and not for estimating the actual settlements of various footings at a given site, it was important to make the method rather conservative to include the possible effects of heterogeneity in the sand deposits.

When the method was introduced Terzaghi and Peck then stated that the settlement of a footing on dry or moist sand depends primarily on the relative density of the sand and the width of the footing. The basic equation for determining the degree of relative density is:

$$R.D.\% = \frac{e_{max} - e}{e_{max} - e_{min}} \times 100 = \frac{\gamma_d - \gamma_{min}}{\gamma_{max} - \gamma_{min}} \times \frac{\gamma_{max}}{\gamma_d} \times 100 \quad (1)$$

where :

e = void ratio for variable densities

e_{max} = void ratio at maximum density

e_{min} = void ratio at minimum density

γ_d = variable densities

γ_{max} = maximum density

γ_{min} = minimum density

The direct determination of relative density of sands is difficult and time consuming.So the N-values of S.P.T. were used as an indirect determination of relative density. The N-values of S.P.T.depent not only on the relative density of the sand, but also on numerous other factors, such as the shape of the grains and the grain size distribution. These all contribute to the many uncertainties in the interpretation of the results. In table (1) is presented the correlation between N-value and the relative density of sand as was suggested by Terzaghi and Peck.

The authors developed the method based on S.P.T. results and results of plate load tests. The method involves use of a chart (Fig. 2) relating the allowable bearing stress P, blow count N, and footing width B for a footing settlement of one inch. The allowable bearing stress given by the chart will limit the settlement of the largest footing to one inch, even if it is located over the loosest part of the sand deposit, by means of the suggested procedure.

The curves in Fig. 2 can be apprximated by the expression.

$$S = \frac{3P}{N} \left(\frac{2B}{B+1} \right)^2 \tag{2}$$

where :

S is the settlement (inches)

P is the average bearing stress (tons/sq.ft)

B is the footing width (ft)

This relationship is sensibly independent of the shape of the footing. Equation (2) is valid for footing widths greater than about 4ft.

Terzaghi and Peck make specific suggestions conserning the value of N to be used in the chart. Between the level of the base of the footings and a depth B below this level, one S.P.T. should be performed for every 2.5 feet of depth. The average value of N in this depth is representative of the sand density in the zone of influence of the footing. Several borings should be made, depending on the size of the structure. If the average N varies between borings, then the lowest average value should be used for determining the allowable bearing stress

TABLE 1

Relative density and S.P.T. values in sand

Description of compactness	Relative density	S.P.T. values (N)
Very loose	< 0.2	< 4
Loose	0.2 - 0.4	4 - 10
Medium dense	0.4 - 0.6	10 - 30
Dense	0.6 - 0.8	30 - 50
Very dense	0.8 - 1	> 50

(Terzaghi and Peck 1948)

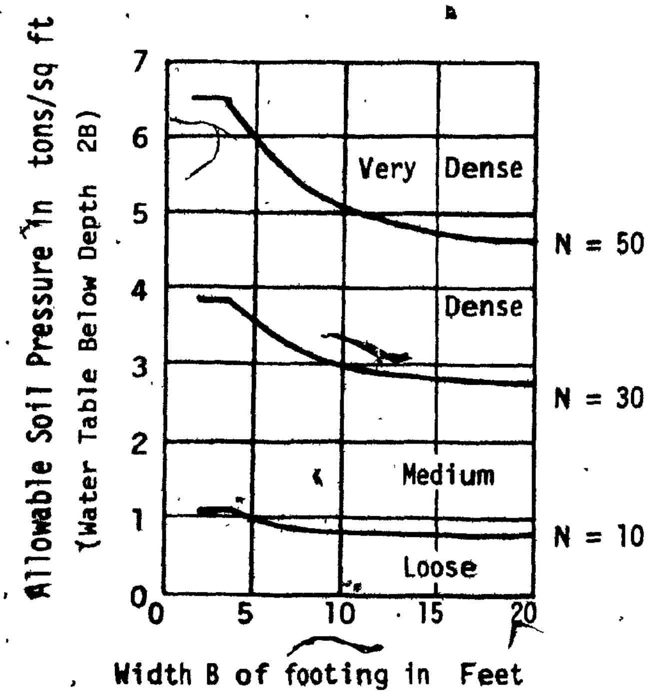


Fig. 2 Chart for estimating allowable soil pressure for footings on sand on the basis of results of S.P.T.

(Terzaghi and Peck 1948)

The Terzaghi and Peck chart was developed for cases where the ground water level is more than 2B below the base of the footing, where the ground water effects are negligible. For all other cases the ground water level location must be taken into account because it has a great influence on the results. The depth of the footing embedment below the ground surface must also be taken into account.

To take these effects into account two corrections were applied to Equ. 2 or on the results as obtained from the chart. These two corrections in the form of correction factors are one for the effects of ground water level position with respect to the footing (C_w), and the other for the depth of embedment of the footing below the ground surface (C_D). After the introduction of these two correction factors Equ.2 becomes:

$$S = C_w C_D \frac{3P}{N} \left(\frac{2B}{B+1}\right)^2 \qquad (3)$$

As already mentioned, where the ground water level is more than 2B below the base of the footing no correction is necessary or $C_w = 1$. Also $C_w=1$ when the ground water is located between the footing base and a depth of 2B below the base of the footing. However for the case where the ground water table is located at the base of the footing or above, Terzaghi and Peck estimated that the footing settlement would be twice the settlement that would occur if the water level were 2B below the footing or $C_w = 2$.

When the footings are embeded below the ground surface, due to the increased confinement provided by the surrounding soil, a smaller settlement occurs than for footings placed on the surface. Terzaghi and Peck recommended that no correction is necessary or C_D=1 when the footing is at the surface ($D_f = 0$). But when the footing is placed at such a depth that the depth to base ratio is close to unity ($\frac{D_f}{B} = 1$) the allowable bearing stress is increased by a factor 4/3 and so $C_D = 0.75$.

Concerning the value of N to be used in the chart, Terzaghi and Peck make specific recommendations because N is affected by the water table and also by the kind of sand. They suggested (1948) that, for loose very fine or silty submerged sand, positive pore-water pressures might develop in the soil during the S.P.T. due to the dynamic application of the load and the low permeability of the soil. These positive pore-water pressures would reduce the shearing resistance of the soil which opposes the penetration of the sampling spoon, hence the N value of these loose soils would decrease upon submergence. On the other hand for dense,very fine or silty submerged sand, the S.P.T. might produce negative pore-water pressures which would increase the resistance to penetration and hence increase the N value. To take these effects into consideration, Terzaghi and Peck suggested that, for very fine or silty submerged sand with a S.P.T. value N" greater than 15, the relative density would be nearly equal to that of a dry sand with a S.P.T. value N where:

$$N_c = 15 + 0.5\ (N - 15) \tag{4}$$

If the sand is very loose with N values less than about 5, footings should not be used unless the sand is compacted. If the soil consists of sand containing gravel, the N value would indicate incorrectly a high degree of compactness. Terzaghi and Peck recommended excavating test pits to determine the density of gravelly soils and then selecting an allowable bearing sress equal to that of a sand having the same relative density. Terzaghi and Peck stated that if the method was applied to gravelly soil it would give conservative results.

5.3 Criticism on the Terzaghi and Peck method, modifications of the original method.

Since the correlation between blow count N and settlement the first was introduced by Terzaghi and Peck in 1948 a lot of research work has been performed on the subject. The settlement predictions were compared with those obtained from plate loading tests and with measured settlements of actual foundations. From the comparison, it became clear that the settlement predictions were too conservative. The discrepancies which were found between predicted and observed settlements led to a critical examination of the origin of the method and of the various factors entering into it.

According to Bazaraa (1967) the chart was developed primarily on the basis of plate load test data and intuition, with a very minimum amount of observed foundation behavior relating S.P.T.to footing settlement. Apparently Terzaghi and Peck had at their disposal a number of cases where plate load tests and S.P.T. were conducted at the same location. From these data a correlation was made between plate load tests settlement and blow count N measured just below the loading plate. Such a correlation is approximate and involves a great deal of scatter. Next Terzaghi and Peck correlated the settlement of small plate tests to the settlement of larger footings. This correlation was achieved primarily by using rather limited data available in the literature at the time. The recommended corrections for groundwater level and depth of foundation embedment were based primarily on judgment. It is clear that any correlation made from these fragmentary data must be very approximate. Terzaghi and Peck used a great deal of their considerable experience and judgment in constructing the chart. It is quite certain that they built a good deal of conservatism into the design method because of their own uncertainties and the absence of suppotring data.

Introducing the method Terzaghi and Peck clearly stated that their correlations did not take into account the geologic origin and environment of the sand deposits. Analysis of extensive laboratory tests as well as field data indicated that the N-value is not only a function of the relative density but is also related to the overburden pressure and position of the groundwater level and to a lesser extent to the soil type, the total weight of the rods and other factors. It was pointed out that is of greater significance the infleunce of effective overburden pressure which was not taken into account in the original method.

The work which has been carried out since Terzaghi and Peck presented their method gives some guidance on the degree of conservatism in the method. Several authors have pointed out that it is overly conservative and a number of modifications and refinements have been proposed to the basic Terzaghi and Peck approach in order to give better agreement between predicted and observed settlements on sand deposits.

In discussing a paper D' Appolonia et al (1970) stated that "Terzaghi and Peck must have recognised the chart to be a 'temporary expedient' to be replaced as evaluated case studies became available".

In their conclusion the writers recommended that use of the original Terzaghi and Peck method for designing footings on sand should be discontinued, since other suggested methods give better results.

In the following different suggested methods and procedures based on the S.P.T. N-values will be presented. Some of the methods are modifications of the original Terzaghi and Peck method, and some reflect a different approach to the problem of calculating settlement. The methods vary in their treatment of the measured N-value, the overburden pressure, the depth of embedment and the depth of ground water level.

Due to the different interpretation of the relationship between penetration resistance and compressibility and the factors affecting them, the various methods give different answers for the magnitude of settlement. Most methods tend to give a maximum value of settlement, some are more conservative and one or two give the maximum probable settlement more often than others.

5.3.1. Modifications based on Gibbs and Holtz research.

The United States Bureau of Reclamation carried out a series of tests under controlled laboratory conditions using fine and coarse sands placed at known relative densities. The purpose of these tests was to determine and evaluate the influence of moisture and overburden pressure on penetration resistance. In 1957 Gibbs and Holtz reported on the results of the performed tests and demonstrated that penetration resistance increases with an increase in either relative density or overburden pressure. Hence for a sand at a given relative density the number of blows required per foot of penetration increased substantially as the effective overburden pressure increased.

For saturated sand the results indicated some reduction in penetration resistance for the coarse sand and appreciable reduction for the fine sand. The penetration resistance results for sands of different moisture content are shown in Fig. 3. Because the results for saturated sand were in contradiction to the previous literature (Terzaghi and Peck 1948), and because the representation of natural water table conditions was doubtful, the authors suggested that the results for dry and moist sand should be applied to saturated sands, as these would be on the conservative side.

The suggested correlation between the S.P.T. blow count N, relative density and effective overburden pressure, indepentenly of moisture content is shown in Fig. 4. In this figure a dashed line correlating the penetration resistance versus relative density, according the Terzaghi and Peck suggestions, has also been plotted for a compatison with these by the Gibbs and Holtz proposed. Gibbs and Holtz concluded that the Terzaghi and Peck correlations are conservative for shallow footings since the relative density of sands near the ground level is underestimated.

The reduced penetration resistance below the ground water table was confirmed by Schultze and Menzenbach (1961), who found that on the average, penetration resistance fell by 15% when the sand was completely saturated. They also confirmed the influence of the effective overburden pressure on penetration resistance and their results give a reasonable

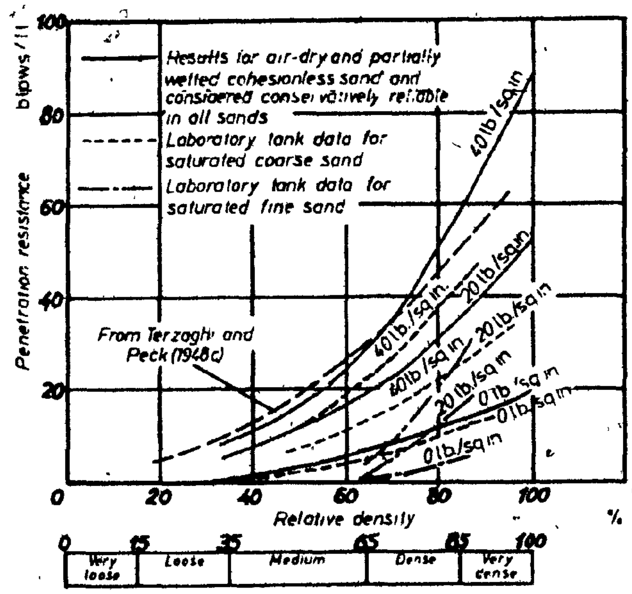


Fig. 3 General relationship between penetration resistance and relative density for cohesionless sand of different moisture content.

(Gibbs and Holtz 1957)

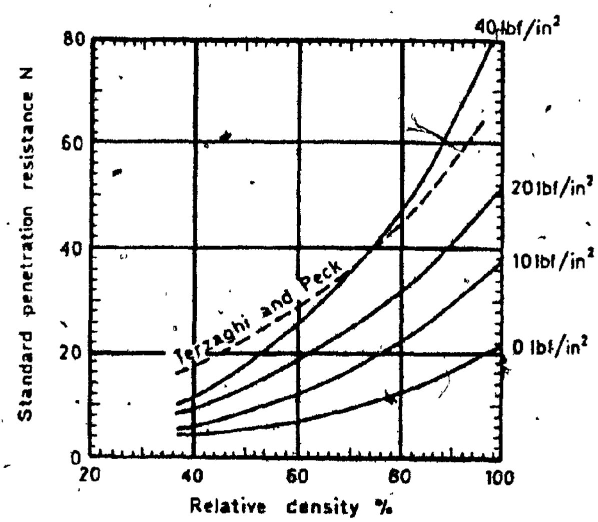


Fig. 4 Gibbs and Holtz correlation between S.P.T., relative density, and effective overburden pressure.

agreement with those of Gibbs and Holtz as is shown in Fig.5 . Mansur and Kaufman (1958), Philcox (1962) and Zolkov and Wiseman (1965) also confirmed from field work that the effective overburden pressure has a substantial influence on the penetration resistance of sands.

Several authors on the basis of the work of Gibbs and Holtz have suggested that the N-value should be corrected in order to reflect the influence of effective overburden pressure to penetration resistance. The method of correction is based on the approximate coincidence of the 40 lbf/sq.in. effective overburden pressure curve in Fig. 4 with the relationship between blow count and relative density implied from the Terzaghi and Peck chart.

Thus Sutherland (1963) suggested that the correction should be made by means of Fig. 4 as follows :

i) For a given S.P.T. value N at a certain depth calculate the effective overburden pressure correspoding to the depth of penetration.

ii) On Fig. 4 find the point corresponding to measured N, and effective overburden pressure.

iii) By a perpendicular through this point meet the Terzaghi and Peck curve and project horizontally through this point to obtain N_e, the corrected value of N.

iv) Use this corrected value N_e in Fig. 2 to obtain the required data.

Thorburn (1963) suggested a correction to the measured N values by means of Fig. 6 which was compiled by the author, based on Gibbs and Holtz work and on a limited number of plate loading tests. To find the corrected N-values the procedure is similar to that of Sutherland. The

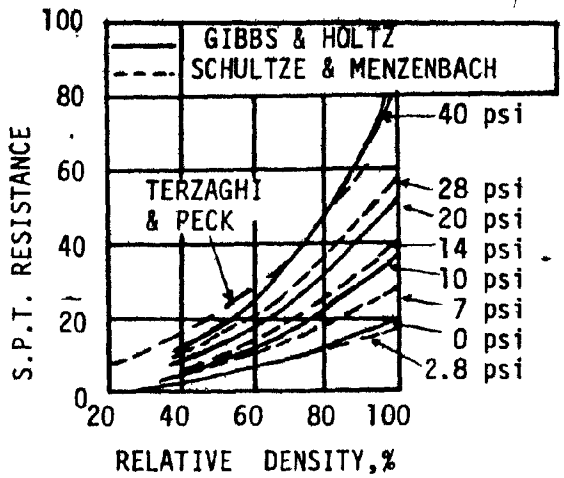


Fig. 5 Influence of effective overburden pressure on S.P.T. resistance. Comparison of Gibbs & Holtz and Schultze & Menzenbach.

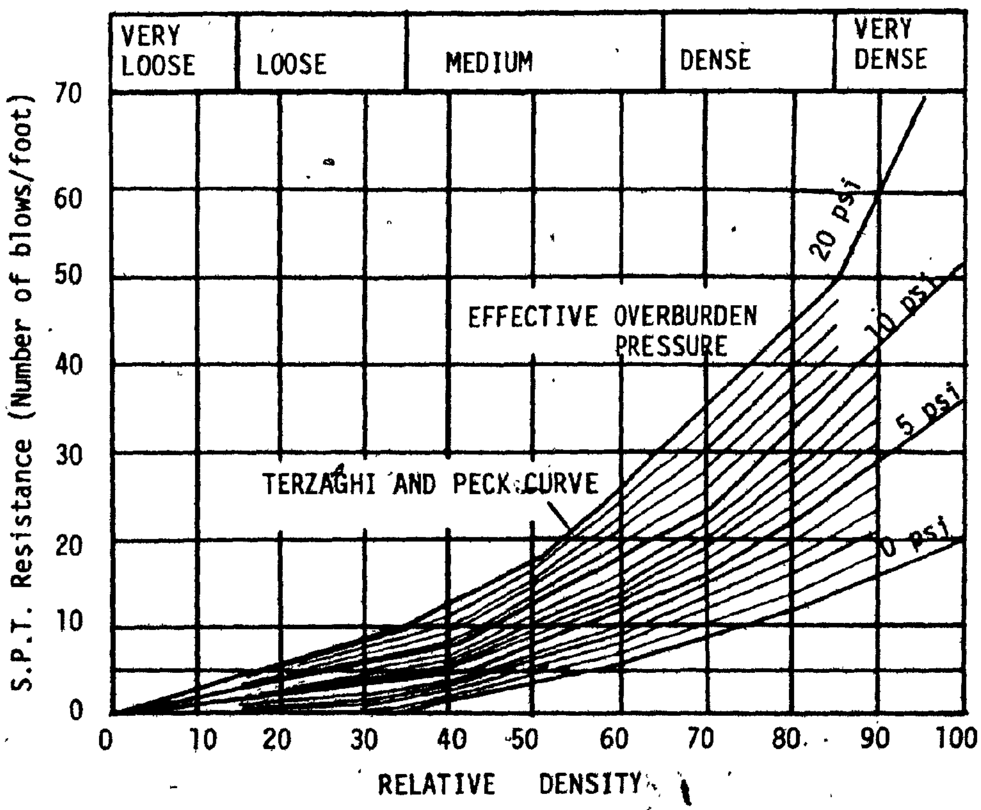


Fig. 6 Relationship between penetration resistance, overburden pressure and relative density for cohesionless soils. (Thorburn 1963)

point corresponding to the measured N-value and the effective overburden pressure is projected to intrsect the Terzaghi and Peck curve. The point of intersection is projected horizontally to give the corrected N_e value which is used in Fig. 2.

Coffman (1960) presented the data from the work of Gibbs and Holtz in another form in order to simplify the use of Fig. 4. The Coffman graph which correlates blow count N, relative density and vertical pressure is presented in Fig. 7. In this chart the lines of equal relative density are straight, by induction, and they essentially intersect at a point. This chart was constructed by interpolation of the curves published by Gibbs and Holtz.

Alpan (1964) presented his correction based on Coffman's work. The correction proposed by Alpan consists of the graph of Fig. 8 in which the Terzaghi and Peck curve is plotted together with the lines of relative density. In order to obtain the corrected N_e value the following method is applied :

i) In Fig. 8 find a point corresponding to measured N-value and a certain effective overburden pressure.

ii) This point corresponds to a certain line of relative density (or an interpolated one), as indicated in Fig. 8.

iii) The previous obtained line of relative density intersects the Terzaghi and Peck curve, in Fig. 8, to a certain point.

iv) Projecting this point perpendicular, the corrected N_e value is obtained .

Tomlinson (1969) proposed a correction based on a simple chart

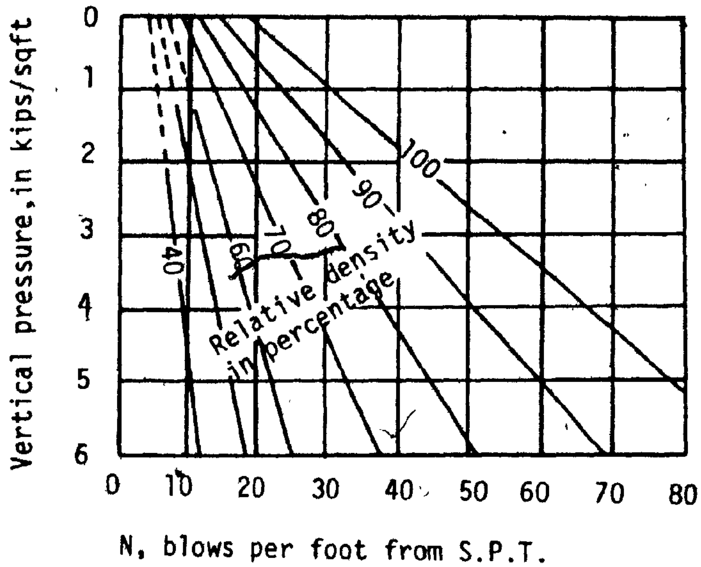


Fig. 7 General relationship between standard penetration, vertical pressure, and relative density for sands.
(Coffman 1960)

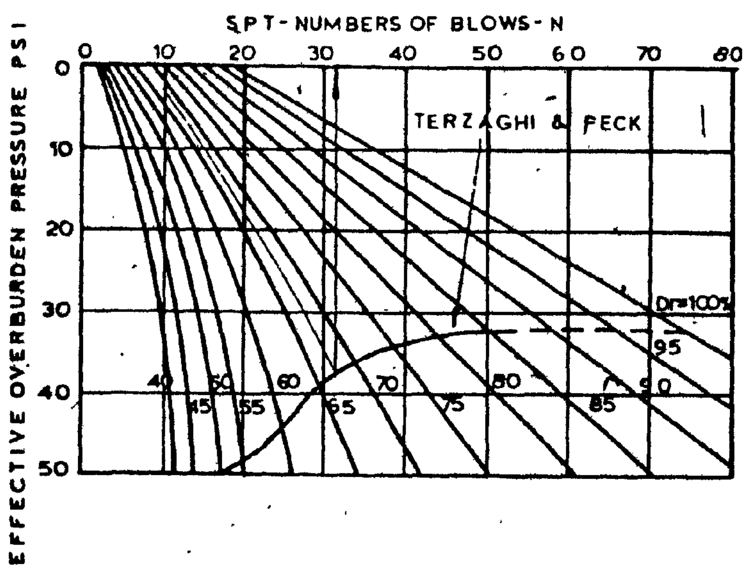


Fig. 8 Correlation between standard penetration resistance, overburden pressure and relative density.
(Alpan 1964)

shown in Fig. 9. The construction of this chart is based on the work of Gibbs and Holtz. It gives the correction factor corresponding to a given overburden pressure. The correction factor is greatest at shallow depths, the graph indicating that the measured N-value should be increased up to fourfold for shallow depths. However a correction of such magnitude should be applied with causion as is suggested by Sutherland (1974).

Thus based on the research of Gibbs and Holtz the first modification to the original Terzaghi and Peck method appeared. This modification reflects that penetration resistance is a function of effective stress as well as relative density. The measured N-values must be corrected for the overburden pressure effects. This may be done by means of the procedures suggested by Sutherland, Thornburn, Alpan or Tomlinson. For very fine or silty sand below the water table and for N greater than 15 a correction also must be applied. This correction is given by the equation :

$$N_c = 15 + 0.5(N_e - 15) \tag{4a}$$

where N_e is the measured N-value corrected for the overburden pressure effects. The treatment of settlement prediction is the same as the original method, except for the correction for the overburden pressure effects. The results obtained by this correction are much improved compared with those of the original method.

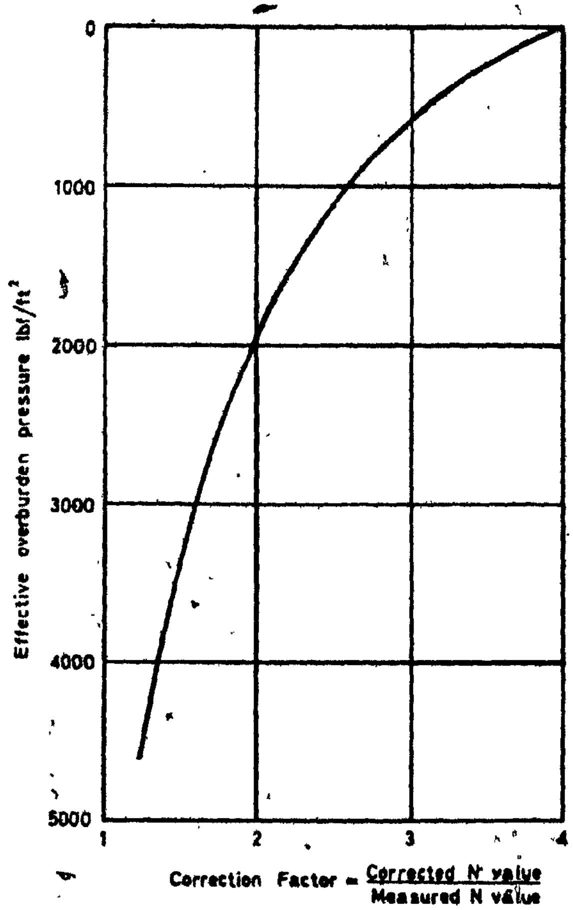


Fig. 9 Correction factor for influence of effective overburden pressure on S.P.T. N-values.

(Tomlinson 1969)

5.4 Alpan method (1964)

Alpan (1964) presented a method for estimating settlement of foundations on sand, based on correlations between the elastic theory predictions and those from square plate loading tests combined with the S.P.T. N-value.

The method assumes that the loading is uniform, the soil is homogeneous and the linear range of the settlement curve extends roughly in proportion to the respective failure loads (Fig. 10).

Elastic theory indicates that the average settlement $S_{aver.}$ of a flexible square foundation of width B, on a semi-infinite homogeneous and isotropic elastic continum, trasmitting a contact pressure q is:

$$S_{aver} = m \left[(1 - \nu^2) / E \right] q B \qquad (5)$$

or

$$S_{aver} = a q B \qquad (6)$$

where $a = m \, (1 - \nu^2) / E$

and : m = slope factor of contact area

ν = Poisson's ratio of the continuum

E = Young's modulus of the continuum

Hence a is a constant and the relationship between the settlement and the width of a square foundation for a given unit contact pressure is linear. In the other hand the relation between actual foundation settlement and that of a square plate loading test as expressed by Terzaghi and Peck is:

$$S_B = S_o \left[\frac{2B}{B+1} \right]^2 \qquad (7)$$

which indicates that with increasing width B the settlement approaches a limiting value as indicated in Fig. 11.

Curves giving settlement as a function of the average pressure, for both plate loading tests and footings, have an initial shape whose slope for the linear range is :

$$a_o = \frac{S_o}{q} \qquad (8)$$

Combining equations (7) and (8) we obtain:

$$a_B = a_o \left[\frac{2B}{B+1}\right]^2 \qquad (9)$$

This equation proposed by Alpan indicates that the solution of the problem would require both the determination of the a_o for a representative loading test, and the determination of the limits of the linear relationship of settlement vs. load curve. These could be found from field tests but these are expensive and time consuming. Alpan proposed a solution based on the S.P.T. N-values by means of the chart of Fig. 12 in which the settlements of a loaded square plate with the N-values are correlated. It is supposed that entering the chart the N-values must have been corrected for the overburden pressure effects as already have been discussed in page 32.

From Fig. 12 the following approximate data can be extracted

a) The limits of the linear relationship between settlement and load.

b) Bearing capacities (for some of the curves).

c) Values of a_o (Equ. 8)

These data can be obtained from Fig. 13 also, as proposed by Alpan

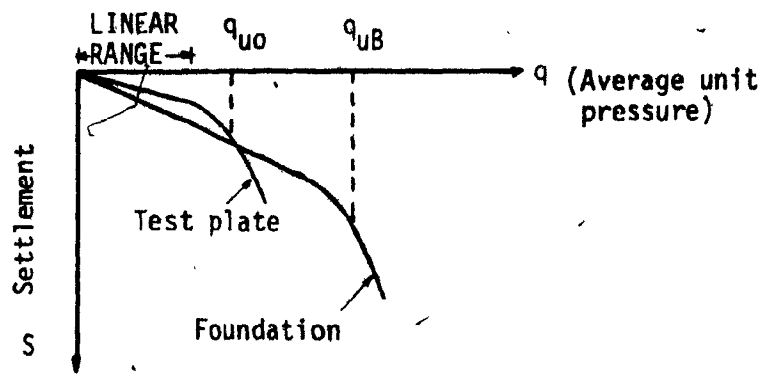


Fig 10 Settlement curve of an actual foundation compared with that of a plate loading test. (Alpan 1964)

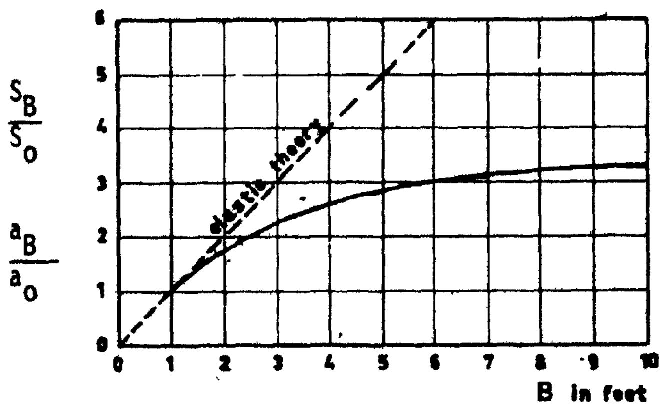


Fig. 11 Relation between B and S_B/S_o (Terzaghi 1941)

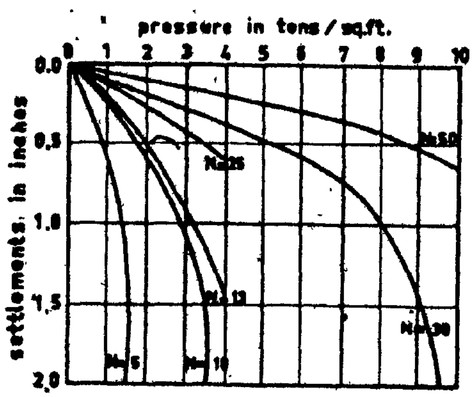


Fig. 12 Settlements after Alpan (1964)

and for the largest values of N, a_o can be obtained from Fig. 14.

So for a given square foundation of width B transmitting a pressure q to sand layer for which N is known the settlement can be predicted in the following way. The N-values must be corrected for overburden pressure Fig. 8. The corrected N-values, by means of Fig. 13 indicate a_o and the limit of linear range. Since the transmitting pressure is in the linear range the settlement will obey Equ. 9. From Fig. 11 and for the given B the value of a_B/a_o is found and hence a_B. Then the settlement will be found according to Equ. 6.

The settlements of rectangular footings are computed by the use of the relative shape factor m' and the following equation.

$$S'_B = m' \ S_B \qquad (10)$$

where S'_B is the settlement of the rectangular footing, S_B the settlement of a square footing having as width the least dimention of the rectangular. Values of m' can be found in Table II or in Fig. 15 where m' is a function of the length to width ratio n (n = L/B).

For the effect of ground water position Alpan suggests a 100% increase in the predicted settlement for small ratio of D_f/B and a 50% increase for D_f/B approximately equal to one. For very fine or silty sand of moderate density below the ground water table a correction for values of N greater than 15 is suggested, given by the following formula

$$N = 15 + 0.5 \ (N' - 15) \qquad (11)$$

where N is the corrected value and N' is the measured N-value already corrected for overburden pressure effects.

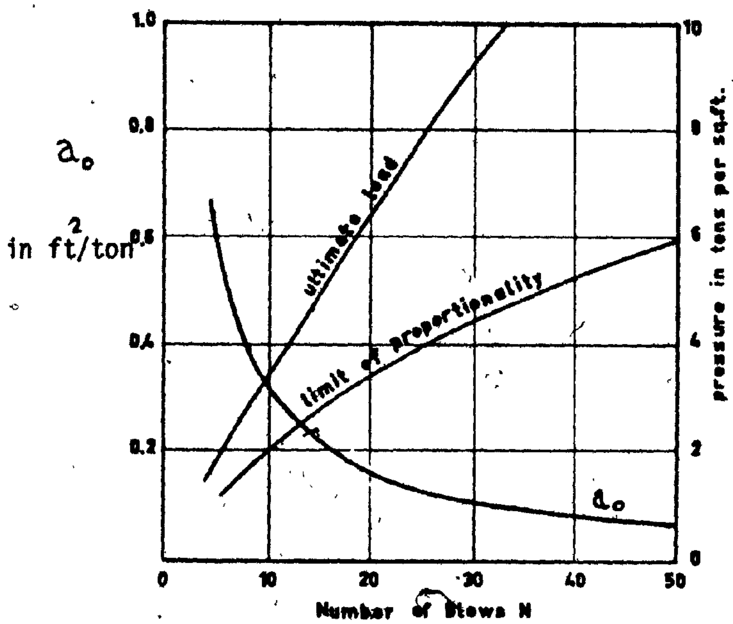


Fig. 13 Values of a_o as found by Alpan (1964)

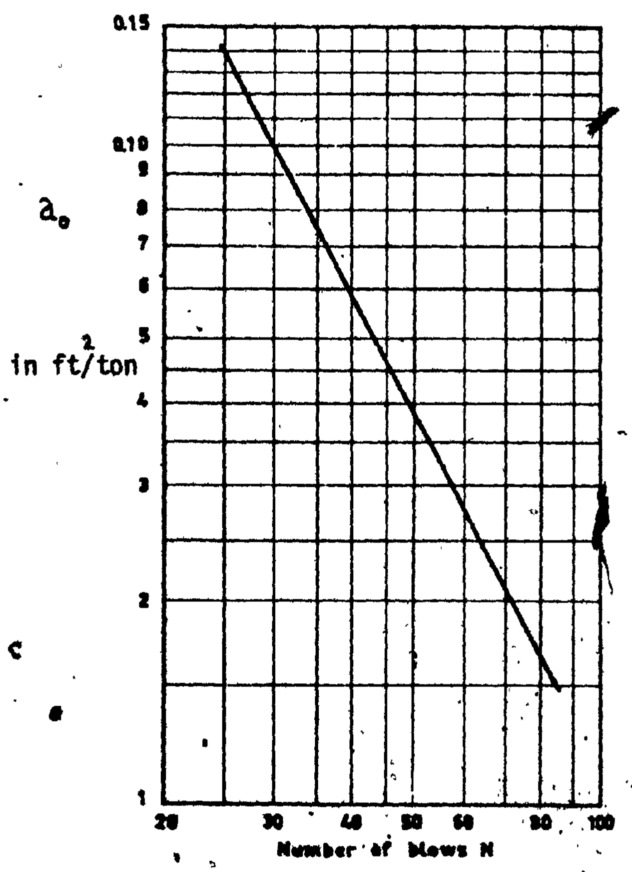


Fig. 14 Values of a_o (logarithmic scale) as found by Alpan (1964)

TABLE II

	CIRCLE	RECTANGLE with Length/Width Ratio n					
		square 1	1.5	2	3	5	10
m	0.96	0.95	0.94	0.92	0.88	0.82	0.71
m' (relative)	1	1	1.21	1.37	1.60	1.94	2.36

Values of the relative shape factor m'

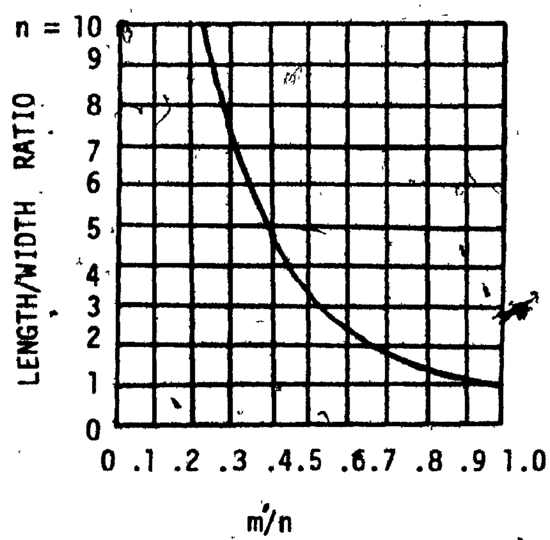


Fig. 15 Values of the relative shape factor.

5.5 Meyerhof modification (1965)

Meyerhof (1965) after studying a number of case records of actual settlement of buildings on sand and comparing the observed settlements with those estimated by the Terzaghi and Peck method, observed that the estimated values were 1.5 to 3 times greater than the measured values. He concluded that the method was rather conservative and recommended two modifications for more accurate results for practical purposes.

First he suggested that the allowable bearing pressure given by Terzaghi and Peck could be safely increased by 50 % . Thus Equ. 2 becomes.

$$S = \frac{2\ P}{N} \left(\frac{2\ B}{B+1} \right)^2 \tag{12}$$

Secondly for the effect of groundwater conditions, Meyerhof, recommended that no allowance for groundwater conditions need be made, because the effect of submergence is already reflected in the measured N values. This means a further increase of allowable bearing pressure up 100 % when the ground water table is at the level of the foundation base.

Meyerhof, as Terzaghi and Peck, ignored the effect of the influence of effective overburden pressure. He applied the same correction as Terzaghi and Peck for the influence of foundation depth of embedment.

For the eight reviewed case studies, even with these two proposed modifications, Meyerhof found that estimated settlements were 1.2 to 4 times greater than the observed settlements.

5.6 Bazaraa proposals (1967)

In 1967 at the University of Illinois, A.R.S. Bazaraa presented a thesis on the subject " Use of the Standard Penetration Test for estimating settlements of shallow foundations on sand ". This study is concerned with evaluating the results obtained from the original Terzaghi and Peck method and correcting the measured S.P.T. N-values for the effect of submergence and overburden pressure and with using the corrected N values to estimate settlement of shallow foundations on sand.

In order to evaluate the results of the original Terzaghi and Peck method, Bazaraa correlated the mathematical expression of the curves in Fig. 2 given by :

$$S_B = \frac{3P}{N}\left(\frac{2B}{B+1}\right)^2 \qquad (2)$$

and the expression correlating plate loading test settlement with that of an actual footing

$$S_B = S_o\left(\frac{2B}{B+1}\right)^2 \qquad (7)$$

From the above two equatons by substitution it is obtained

$$\frac{P}{S_o} = \frac{N}{3} \qquad (7a)$$

Then he plotted a large number of plate load tests in the form P/S_o versus N and observed that the relationship 7a was a very conservative interpretation of the data. He proposed that the relationship 7a would represent more closely a lower limiting condition (Fig. 16), hence the allowable bearing pressure would be increased by 50 % compared with that obtained from the Terzaghi and Peck chart.

The influence of overburden pressure was investigated and the important effect of vertical pressure on penetration values was clearly indicated from laboratory data. An increase in vertical pressure from 0 to 40 psi caused an increase in the S.P.T. values from 1 to 12 for a dry coarse sand. Bazaraa correlated the S.P.T. N-value for a sand at a relative density D_R and under an overburden pressure σ (kips/sqft) in the following way :

$$N = 20\ D_R^2(1+2\sigma) \qquad \text{for } \sigma = 1.5 \text{ kips/ sqft} \qquad (13)$$

$$N = 20\ D_R^2(3.25+0.5\sigma) \qquad \text{for } \sigma > 1.5 \text{ kips/ sqft} \qquad (14)$$

To correct the penetration value N for an overburden pressure σ , he assumed that the Terzaghi and Peck N-values corresponded to an effective overburden pressure of about 1.5 kips/sqft. Then from equations 13 and 14 it was found that:

$$N_B = \frac{4N}{1+2\sigma} \qquad \sigma \leq 1.5 \text{ kips/sqft} \qquad (15)$$

$$N_B = \frac{4N}{3.25\ \ 0.5\sigma} \qquad \sigma > 1.5 \text{ kips/sqft} \qquad (16)$$

where N_B is the corrected N-value for the effect of overburden pressure σ. He suggested that an average N-value should be determined for each boring for the sand between the level of the footing base and a depth B below this level. The smallest average value of N obtained, should be considered, and be corrected for the effect of overburden pressure according to Equations 15 and 16. The effective overburden pressure should be determined at a depth B/2 below the footing base.

For the influense of submergence, Bazaraa confirmed the previous suggestions that for fine to coarse sands and gravels the submergence

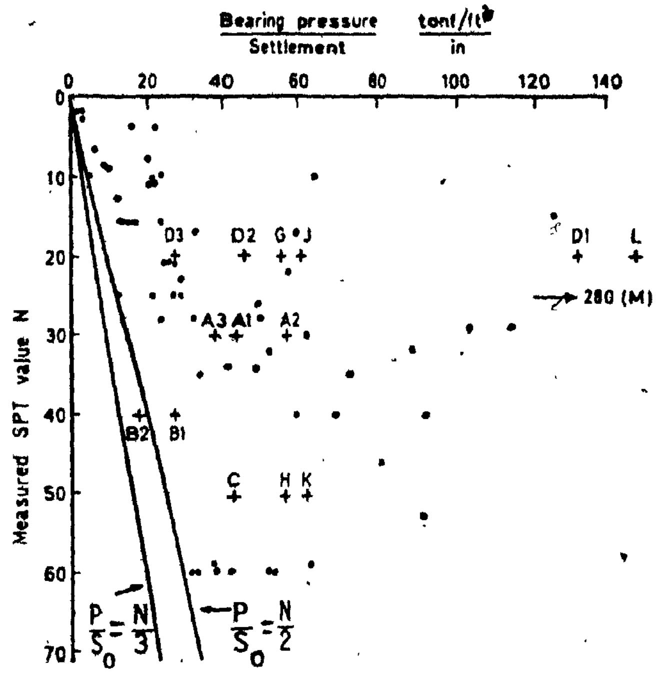


Fig. 16 S.P.T. versus bearing pressure settlement ratio from plate load tests. (Bazaraa 1967)

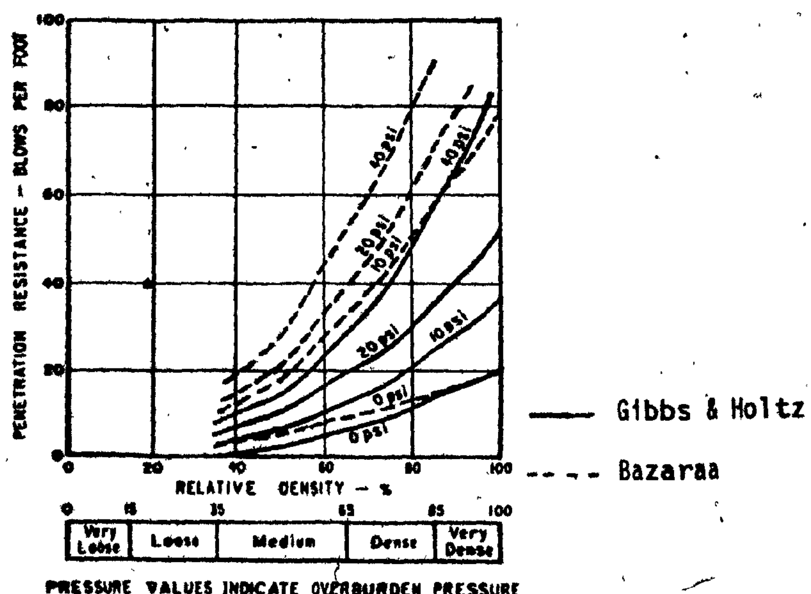


Fig. 17 Correlation between S.P.T., relative density and effective overburden pressure. Comparison between Gibbs & Holtz and Bazaraa curves.

did not have any significant influence on penetration resistance. But for very fine or silty sand with an average N-value of 15 or less within 3 ft above water table, the N-values were increased by a modal factor of about 1.8. For very fine or silty sands with N-values exceeding 15 within 3 ft above water table the N-values due to submergence were increased by a modal factor close to 1.3. Hence, the relative density of a submerged very fine sand with an S.P.T. value N' might nearly be equal to that of a dry sand with an S.P.T. value N where:

$$N = 0.6\ N' \tag{16a}$$

These results contradict the results of Gibbs and Holtz (laboratory results 1957), which indicate that submergence reduces the N-values for very fine and silty sands. Also the results are in contradiction with the suggestion by Terzaghi and Peck that submergence does not increase the N-value for very fine or silty sand except if these soils are relatively dense with an average N-value of more than 15 above water table

For the effect of footing embedment it was suggested that the following expression should be reasonable :

$$C_D = 10 - 0.4\left(\frac{\gamma D}{P}\right)^{1/2} \tag{17}$$

where :

C_D is the correction factor for the embedment

D is the depth of footing embedment below groud surface

γ is the total unit weight

P is the average vertical pressure applied to the footing

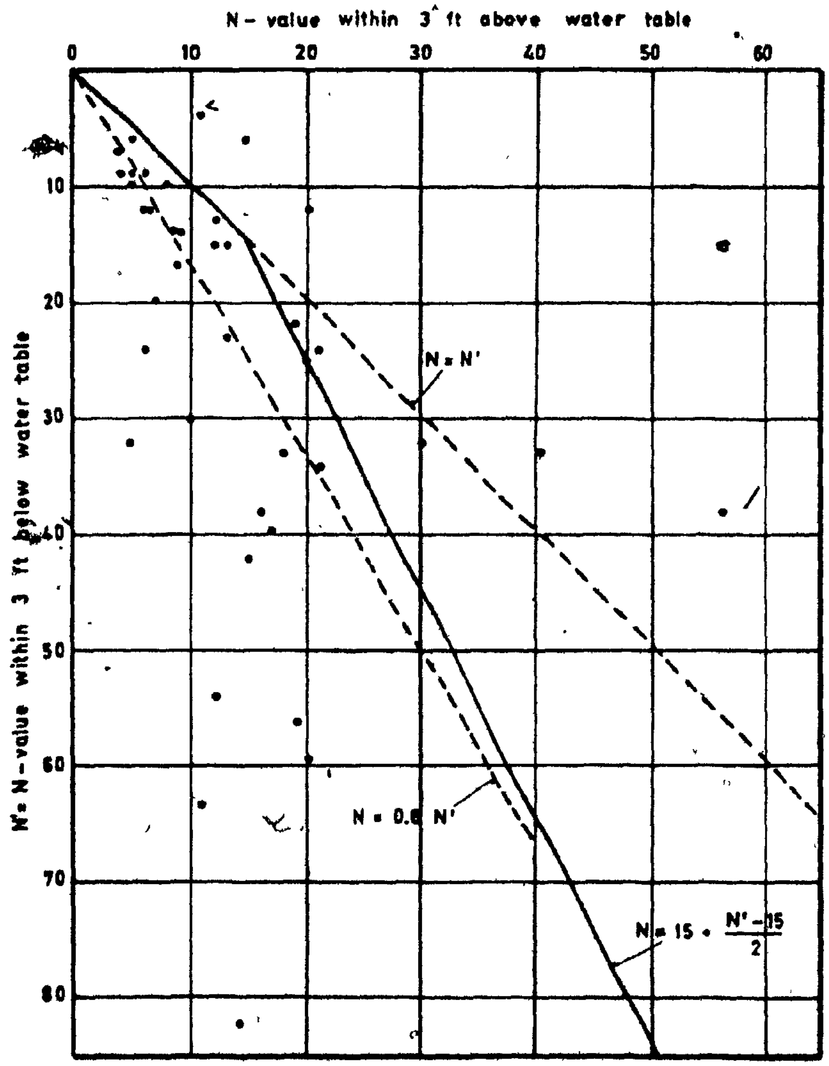


Fig. 18 Relationship between Standard Penetration Test values above and below water table for very fine and silty sand. (Bazaraa 1967)

5.7 Peck and Bazaraa method (1969)

In discussing a paper by D' Appolonia et al (1968) Peck and Bazaraa recognised that the original Terzaghi and Peck method was too conservative. They proposed a new method based on the Bazaraa research (1967)

The proposed method consisted of three modifications to the original Terzaghi and Peck method in order to achieve more accurate predictions. The first modification was a correction of the S.P.T. N-values for the influence of overburden pressure, by means of the empirical results of Gibbs and Holtz. They argued that application of the Gibbs and Holtz correction was, in most instances, unrealistic and led to an overcorrection. They suggested the correction proposed by Bazaraa by means of the following relations :

$$N_B = \frac{4N}{1 + 2\sigma} \qquad \text{for} \quad \sigma \leq 1500 \text{ psf} \qquad (15)$$

$$N_B = \frac{4N}{3.25 + 0.5\sigma} \qquad \text{for} \quad \sigma > 1500 \text{ psf} \qquad (16)$$

In which N is the S.P.T. value for a sand at a overburden pressure σ, and N_B the corrected value.

The second suggested modification was an increase by 50 % of the allowable bearing pressure given by the Terzaghi and Peck method as already had been proposed by Meyerhof.

The third proposal was a correction for the ground water position, because effects of submergence appear to exist even in coarse sands. They suggested that the settlement S, when the water table is at a distance D_w below the base of a shallow footing with width B may be estimated as:

$$S = K\, S_{dry}$$

where S_{dry} is the settlement of the same footing when the sand is dry and K is the ratio of the effective overburden pressure at depth B/2 below the base of the footing when the sand is dry to that at the same depth when the water table is present.

The above modifications combined with that suggested by Bazaràa for the effect of footing embedment can be summarized in the following way :

$$S = K\, C_D \frac{2P}{N_B} \left(\frac{2B}{B + 1} \right)^2 \qquad (18)$$

where $K = \frac{\sigma}{\bar{\sigma}}$ (groundwater effect)

$$C_D = 1.0 - 0.4 \left(\frac{\gamma D}{P} \right)^{1/2} \qquad (17)$$

5.8 Peck et al method (1974)

In the second edition of " FOUNDATION ENGINEERING " by Peck, Hanson and Thornburn another approach for the calculation of settlement of footings on sand was presented. The research of Bazaraa (1967) has been taken into account but a different detailed procedure has been proposed from that advanced by Peck and Bazaraa in 1969. The influence of the effective overburden pressure, depth of embedment and position of water table have been taken into account.

The authors based their proposal on the relation between the load per unit area correspoding to a given settlement as a function of the width of the footing. This relation is presented in Fig. 19 by the curved solid line. This curved line was replased by two straight lines namely ef and fg (dashed lines in Fig. 19). The results of this replacement was that the line fg makes the soil pressure corresponding to a settlement independent of the footing width, introducing an error for footing of usual dimensions less than 10 %, and that the line ef provides a margin of safety against a bearing capacity failure. Since the settlement can be affected by the sand properties and since the sand properties could be evaluated by the S.P.T. N-values, the above described procedure was repeated for different N-values in order to take into account the different sand deposits. Thus the design chart presented in Fig. 20 was constructed. This design chart correlates allowable bearing pressure with blow count N and footing width B for a 1 inch settlement, and the comparison of this correlation with the original Terzaghi and Peck correlation is shown in Fig. 21. The effect of embedment depth has been taken into account by means of the ratio depth of embedment to footing width (D_f/B). Three such ratios are included equal respectively to 1, 0.5 and 0.25.

Originally the chart was constructed for shallow footings(depth of embedment to width ratio less than 1) resting on uniform sand for which γ= 100 lb/cuft, and in which the water table is at a depth great enough to influence the behabior of the footing. Any variations of γ from the assumed value of 100 lb/cuft, in view of the other approximations, may be regarded as negligible.

It is suggested that the S.P.T. borings should be performed preferably at least one for every 4 to 6 footings. Values of N should be de-

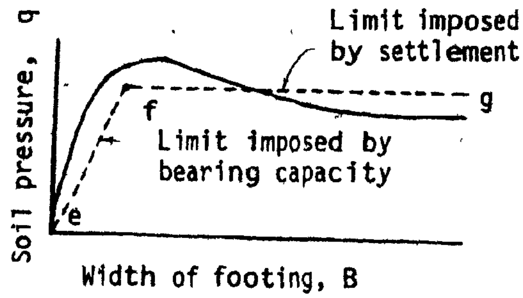


Fig. 19 Actual relation (solid line) between soil pressure and width of footing on sand for given settlement S_1, and substitude relation (dashed lines) used as basis for the design as proposed by Peck et al (1974)

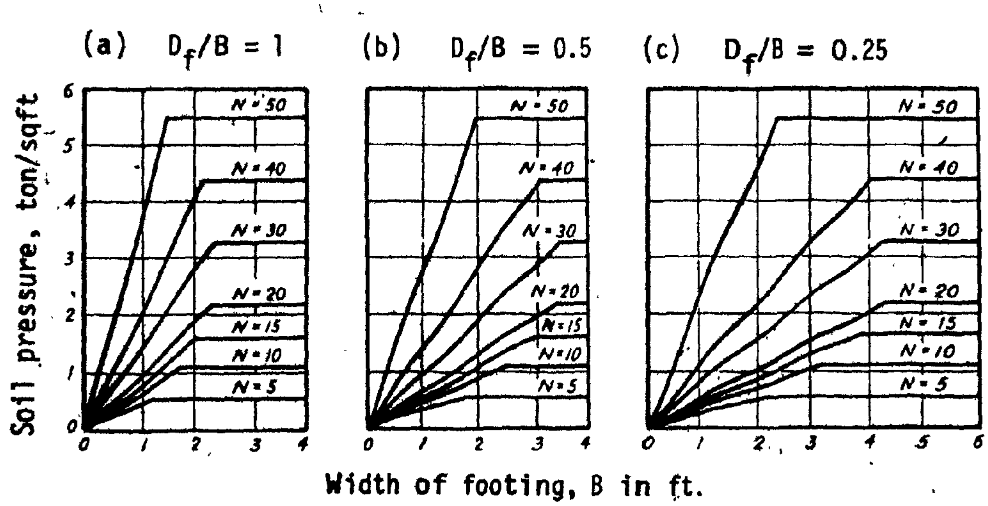


Fig. 20 Design chart for proportioning shallow footings on sand.

termined at 2½ ft vertical intervals between the level of the base of the footing and a depth B below this level. The average of the N-values for each boring should be computed, and the smallest average value of N obtained should be used to enter the chart. Before entering the chart the measured N-values should be corrected for the effect of overburden pressure and ground water location,were applicable. The correction for overburden pressure should be applied on each N-value and before the average borehole value is obtained, while the correction for the influence of ground water table should be applied on the allowable bearing pressure obtained from the chart.

The correction for the influence of overburden pressure is applied through a correction factor C_N to the blow count expressed as:

$$C_N = 0.77 \log_{10} \frac{20}{\bar{p}} \qquad (19)$$

where $\bar{p}$ is the effective vertical overburden pressure in tons/sqft at the elevation of the penetration test. This equation is valid for $\bar{p} \geqq 0.25$ tons/sqft. The blow count at a depth corresponding to an overburden of 1 ton/sqft (2000lbs/sqft) is taken as a "standard". When the overburden pressure differs greatly from 1ton/sqft, the N-values should be corrected. When the correction factor is within the range 0.8 to 1.2, corresponding to vertical pressures 0.5ton/sqft to 1.8tons/sqft (1000 lbs/sqft to 3600lbs/sqft)the corrections can be ignored without serious error. Alternative to Equ. 19 the chart of Fig. 22 may be used. For values of effective overburden pressure less than about 0.25 ton/sqft the correction should be taken from the chart, because Equ. 19 leads to un-

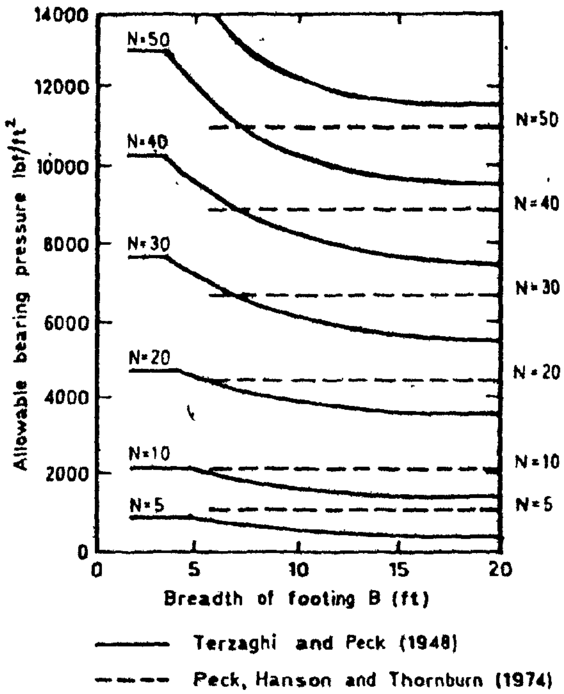


Fig. 21 Terzaghi and Peck, Hanson and Thornburn (1974) correlations of allowable bearing pressure for 1 in. settlement on sand with S.P.T. N-values.

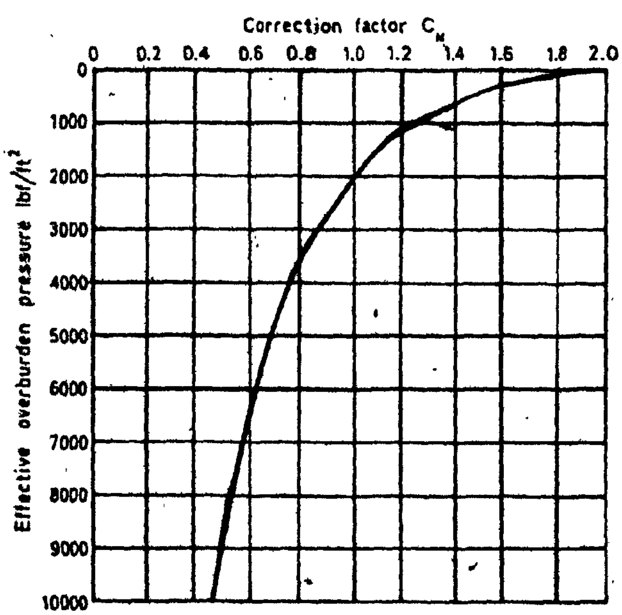


Fig. 22 Correction factor for influence of effective overburden pressure on S.P.T. (Peck, Hanson, Thornburn 1974)

resonably large correction factors. From Fig. 22 it can be seen that for $\bar{p} \leq 1$ ton/sqft the blow count is increased, while for $\bar{p} > 1$ ton/sqft the blow count is decreased.

The correction for the water table position is given as :

$$C_w = 0.5 + 0.5 \frac{D_w}{D_f + B} \qquad (20)$$

where D_w is the depth of the water table from the surface of the surcharge surrounding the footing and D is the depth of the footing. For cases where the water table occurs and will remain at or below a depth $D_f + B$ beneath the ground surface, no correction is necessary. If the water table is located at or may rise to the ground surface the correction factor is $C_w = 0.5$. The occurence of these values of C_w means that the effect of the ground water table on settlement is considered to be less than originally proposed by Terzaghi and Peck, except for the case of a surface foundation with the ground water level at ground surface.

The proposed method in general gives allowable bearing pressures of the order of 50 % greater than the obtained from the original Terzaghi and Peck method, except for the cases where the relief of overburden pressure is greater than the standard of 1 ton/sqft. In such cases the corrected S.P.T. values being less than the original values, result in lower allowable bearing pressures than those obtained from the original Terzaghi and Peck method.

5.9 D' Appolonia et al method (1970)

In 1970 D. D' Appolonia, E. D' Appolonia and R. Brissete presented a different approach in applying S.P.T.data for estimating footing settlement on sand. The suggested approach is based on the theory of elasticity and on empirical correlations relating sand modulus to measured blow count .

Theory of elasticity for elastic displacement requires that :

$$S = \frac{P\,B}{M}\,I = P\,B\,\left(\frac{1-\nu^2}{E}\right) I \qquad (21)$$

where S is the footing settlement, P is the average applied pressure, B is the smallest footing dimension, M is the modulus of compressibility, I is an influence factor depending on footing geometry, depth of footing embedment and thickness of the comptessible layer, E is Young's modulus and ν is Poisson's ratio. Since Equ. (21) is based on theory of elasticity , it is strictly applicable to elastic homogenious and isotropic materials. The authors assumed it could be applicable also for nonhomogenious materials, as sand deposits, with reasonable accuracy provided that a representative average value of modulus M could be selected.

The authors, in order to determine representative values of sand modulus M correlated it to S.P.T. N-values. The S.P.T. N-values have been shown to be related to relative density, effective overburden stress and the grain size, shape and distribution. Sand compressibility modulus M is also related to the same variables, hence there is a correlation between sand compressibility and S.P.T. N-values. This correlation was developed by using the observed behavior of structures on sand and it was separated for preloaded or compacted sand and normally loaded sands. The correlation for preloaded sands was obtained from the results reported in their 1968 paper, dealing with settlement observations of structures on fine dune sand which had been compacted by vibrations. The correlation for normally loaded sand was based on less evidence and was obtained from field cases availiable in the literature. This correlation is shown in Fig. 23. The blow count used in this correlation is the average S.P.T. N-value in a depth equal to $(BL)^{\frac{1}{2}}$ below the base of the footing of length L and width B. The N-values are used without any correction. The presence of ground water level is ignored, if is assumed that its effects are reflected in the measured N-values. The linear ratio between M and blow count N in Fig. 23 can also be estimated from the following expression if the Poisson's ratio for sands is taken as:

$\nu = 0.25$

$$E(tsf) = 196 + 7.9N \quad \text{for a normally loaded sand} \qquad (22)$$

$$E(tsf) = 416 + 10.9N \quad \text{for a preloaded sand} \qquad (23)$$

There is no differentiation between heavily and lightly preloaded sands. The preload on the original site was approximately 5 tsf. Since the above correlation was developed for footing foundations on sand there are some restrictions on its use. This correlation should not be applied to raft foundations, nor should it be applied to silts and sandy silts.

The influence factor treatment was based on the relationship $I = U_o U_1$ the values of U_o, U_1, apply for rectangular footings and are based on elastic theory predictions. The influence factors U_o, U_1 reflect the influence of footing dimensions, embedment and layer thick-

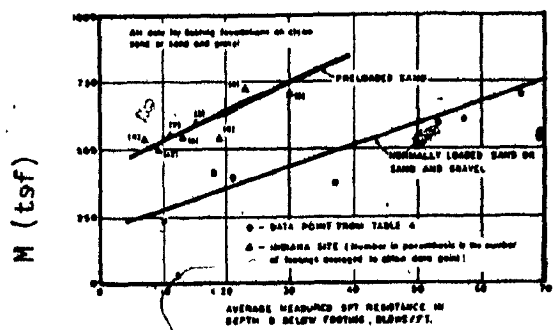


Fig. 23 Correlation between modulus of compressibility and average S.P.T. resistance. (D'Appolonia et al 1970)

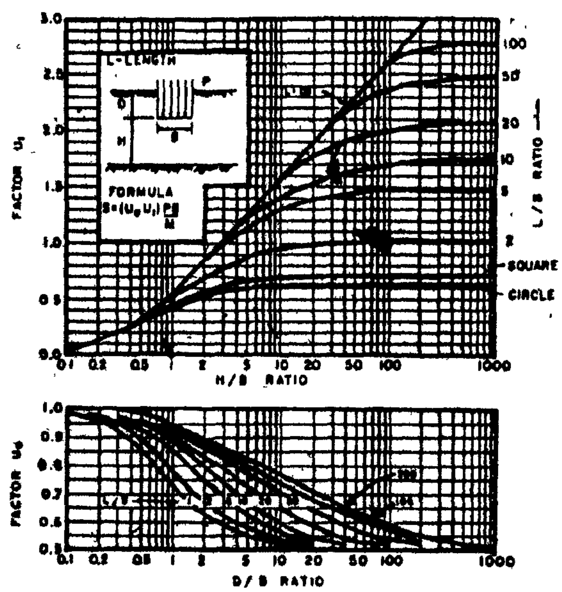


Fig. 24 Influence factors for computing settlement from elastic theory. (D'Appolonia et al 1970)

ness. These factors should be used with caution. They are under review.

D' Appolonia et al commenting on their method argued that an advantage of their method over the method based on the Terzaghi and Peck correlations is that the effects of footing embedment, varying foundation dimensions, and sand layer thickness can be taken into account. Also that it is a design method and not an estimating method as the Terzaghi and Peck method with its modifications. That means that the D' Appolonia method can be used to determine the settlement at a location where the blowcount is measured while the estimating methods can give an estimate of the probable maximum settlement.

5.10 Parry method (1971)

The method proposed by Parry (1971) is a modification of the classic elastic equation in which correction factors have been applied. It is based on direct correlation with S.P.T. Parry pointed out that the original Terzaghi and Peck approach does not allow taking into account stress changes in the ground, stress changes which could be due to site grading and foundation excavations.

The proposed equation is :

$$S = \frac{q\,B}{M}\, C_D\, C_W\, C_T \tag{24}$$

where :

$$M = \frac{E}{1 - \nu^2} \qquad \text{and} \qquad \nu = 0.25$$

and C_D, C_W, C_T are corrections factors for depth of excavation, water table location, and depth of incompressible layer respectively. These correction factors take into account stress changes which occur due to site grading or excavation and also allow a specific thickness of compressible sand layer to be considered.

The values of correction factors C_D, C_T are obtained from charts in Fig. 25 and Fig. 26 as a function of the ratio D/B and T/B respectively, where the symbols B,T,D are defined in Fig. 25. The correction for water table C_w is applied as follows.

For permanent excavations below the water table with a drawdown of the water level, the factor C_w is given as:

$$C_w = 1 + \frac{D_w}{D + 0.25B} \quad \text{for} \quad 0 < D_w < D \tag{25}$$

For permanent excavation with the water level below the base of excavation ($D < D_w$), the factor C_w is given as:

$$C_w = 1 + \frac{D_w (2B + D - D_w)}{2B (D + 0.75B)} \quad \text{for} \quad 0 < D_w - D < 2B \tag{26}$$

No correction is applied for ground water effects where surface footings are used or for footings in back-filled excavations provided the water table does not vary after the site investigation and during the life of the structure. In this case the obtained N-values must be reduced in direct proportion to the reduction in the effective overburden pressure due to water level.

The required value of sand compressibility E in Equ. 24 was obtained by correlating E to N. This correlation between blow count and sand compressibility was obtained not from actual structures but from a li-

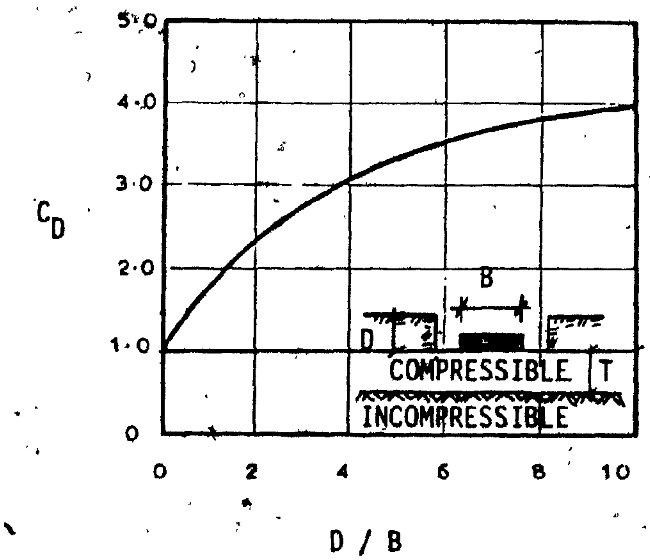


Fig. 25 Correction factor C_D for depth of excavation effects.

(Parry 1971)

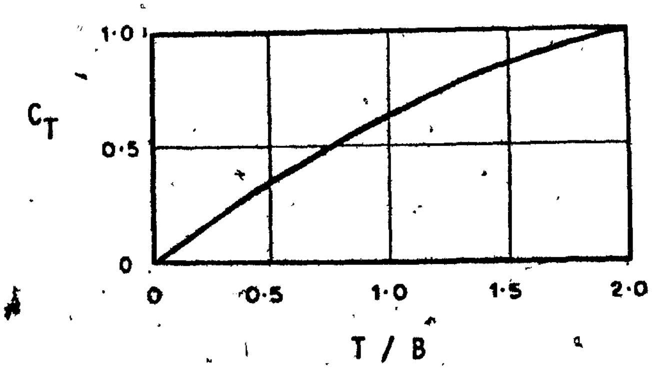


Fig. 26 Correction factor C_T for depth of incompressible layer.

(Parry 1971)

mited number of plate bearing tests published by other authors. The modulus of elasticity of the sand is given by:

$$E = 50\ N\ (kg/cm^2) \tag{27}$$

It is suggested that the appropriate N-values must be the average values at a depth 0.75B. The above expression for E gives much higher values than normal. A comparison of the E values obtained from Equ. (27) with those obtained from the values as were suggested by D' Appolonia et al (1970), indicate that Parry gives much higher values for the modulus particularly when $N > 20$.

Parry concluded that his method is suitable for feasibility studies and minor structures but recommended that the calculated values of settlement should be increased by 50 % for design purposes.

CHAPTER 6

METHODS BASED ON STATIC CONE PENETRATION TEST

6.1 The static cone penetration test

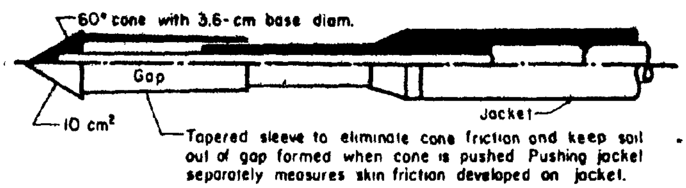
The static cone penetration test has a history of development in Europe, particularly in the Netherlands and Sweden, for more than 40 years. By definition it is a method of pushing a cone at a constant speed into the soil. The soil resistance is measured and so an indicator of the soil compressibility is obtained. The point resistance q_c of the cone is measured independently or if required, depending on the cone type, lateral friction and total resistance may also be reported.

The Dutch cone test is the most widely used type of static penetrometer test. In it a 60° cone with a cross-sectional area of 10 sqcm is forced into the ground with a constant rate of penetration and provision is made to measure independently the point ressistance and the resistance due to side friction. The constant rate of penetration must not exceed 125 m/min. The A.S.T.M. recommends a rate of 2 cm/sec

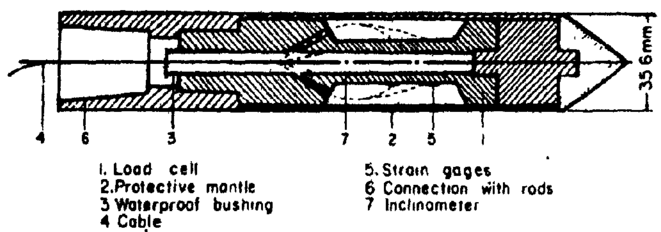
To use the Dutch cone sounding method,a hardened steel cone is forced vertically into the soil by a slow push or static thrust. This thrust, required to cause a bearing capacity failure of the soil surrounding the point, is measured and recorded, Such measurements are made every 8in. of sounding depth or 4in. if a more detailed profile is required. The measurements provide considerable detail for a bearing capacity profile and hence a shear strength profile of the soil penetrated. The cone point is specially designed to prevent soil contamination of the cone mechanism. It is advanced with a 2-rod system. The outer rod or casing provides structural strength and protects the inner rod from soil friction and buckling. The protected inner rod advances the point during a thrust measurement. The thrust is measured by a hydraulic load cell connected to a Bourdon gauge. Another type of cone used is the friction cone illustrated in Fig. 27. A special friction sleeve, attached above the point, permits the additional determination of static soil friction against the steel sleeve.

Using the friction cone the identification of soil strata penetrated by means of the recorded friction ratio is possible. This friction ratio, the dimensionless ratio of the unit sleeve friction to unit point bearing is possible. This approximate interpretation of the type of soil penetrated is possible, even though no samples are obtained, by means of corresponding a certain type of soil to certain values of friction ratios. In order to achieve such an interpretation a local calibration of the correspondance and a local experience with the friction cone are required.

The data are recorded in the form of a graph where the point resistance q_c in tons/sqft is plotted on the abscissa and the depths are recorded on the ordinate axis increasing downwards. In the recent types of apparatus the output is usually electronically made. When the friction cone is used also the friction ratio is reported. A typical output is illustrated in Fig. 29. Cohe bearing capacity and friction ratio were both used to make the interpretation of soil strata as indicated.



(a) Dutch cone modified to measure both point resistance C_R and skin friction



(b) Electric strain gage penetrometer (De Ruiter, 1971)

Fig. 27 Types of Dutch cone penetrometers.

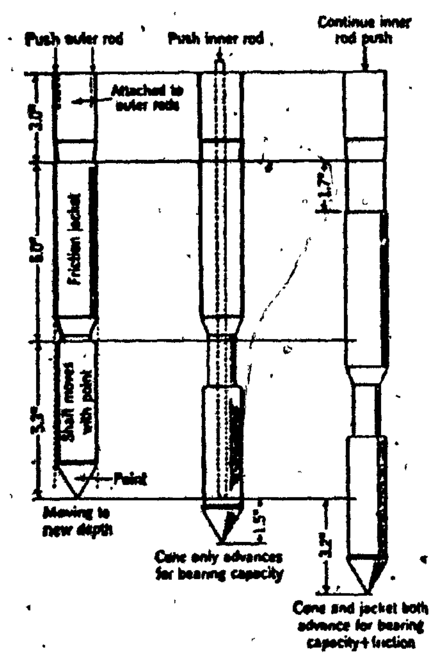


Fig. 28 Action of the friction cone.

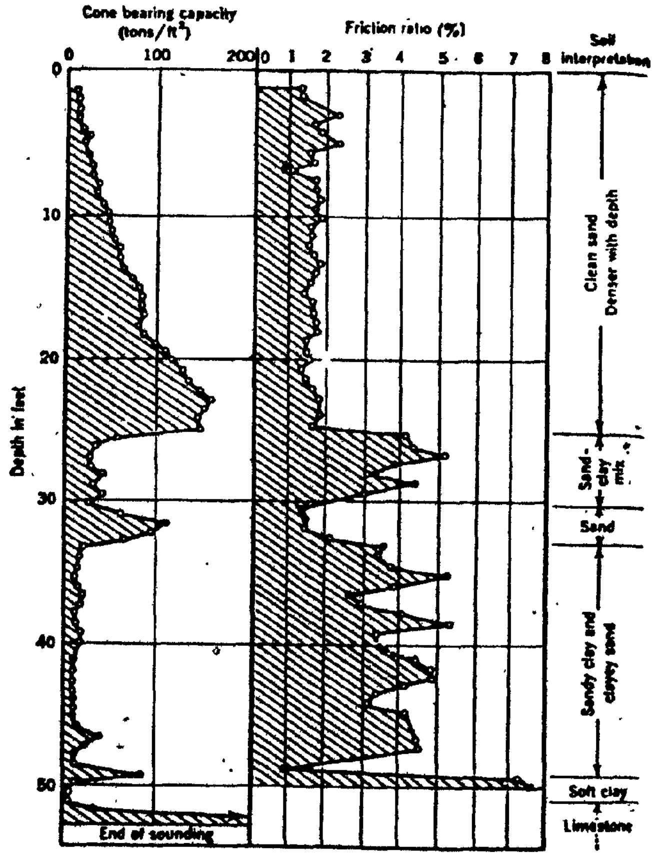


Fig. 29 Typical results from a friction cone penetrometer.

The static penetrometers such as the Dutch cone have the advantage of giving a continious record of penetration. The penetration resistance is not affected by the ground water level as in the Standard Penetration Test. The test itself does not allow the material penetrated to be visually identified and additional boreholes must be sunk to correlate strata descriptions with penetration resistance accurately.

The test was first devised to assess the bearing capacity of piles but is used to predict the settlement of structures on sand. The original prediction method was developed by Buisman, De Beer but more recently Schmertmann has proposed a different approach based on static cone penetrometer results.

6.2 Buisman, De Beer method (1957)

In 1957 De Beer and Martens presented a paper conserning a method of computation of an upper limit of the settlements of bridges founded on sand layers. The proposed method was the first utilizing the cone penetration results for the settlement calculations. The method takes into account the influence of the heterogeneity of sand layers. It consists of substituting for the real cone penetration diagrams, fictitious ones obtained by considering successively at each level, the maximum minimum and average values of the readings, since at least three cone penetration tests have been performed.

The method is based on the semi-empirical Terzaghi and Buisman formula for the calculation of settlement of foundations on soil given by:

$$\frac{\Delta h}{H} = \frac{1}{C}\ \ln\left(\frac{P_o + \Delta p}{P_o}\right) = \frac{2.3}{C}\log_{10}\left(\frac{P_o + \Delta p}{P_o}\right) \qquad (28)$$

in which Δh is the vertical compression of the sand layer having thickness H, P_o is the effective overburden pressure at the depth considered, Δp is the increment of pressure at the depth due to the foundation loading and C is the constant of compressibility of the sand layer. The settlement for the different sand layers can be computed as:

$$S = \int_o^H \frac{2.3}{C}\ \log\left(\frac{P_o + \Delta p}{P_o}\right) \Delta H \qquad (29)$$

where ΔH denotes the thickness of each layer having a constant C throught its thickness and H the total depth for which values of C are available. In Equ. 29 the variables that must be determined are the pressure increment Δp and the sand compressibility C.

The pressure increment Δp can be determined by a law of stress distribution with depth as has been proposed by Boussinesq, or Buisman. The law of Boussinesq, concerning the stress distribution with depth assumes a medium with a constant modulus of compressibility E and is given by :

$$\sigma_z = \frac{3}{2}\frac{P}{\pi z^2}\cos^2\theta \qquad (30)$$

This means that the above formula should be used when the cone penetration values q_c are nearly constant with depth. When the cone values increase with depth, which is generally the case even for homogeneous sands, the modulus of compressibility also increases with depth, thus instead of the law of Boussinesq the use of the law of Buisman which follows is suggested.

$$\sigma_z = 2 \frac{P}{\pi z^2} \cos^6\theta \qquad (31)$$

The choice between the two given formulas is quite simple since it can be based on the results of static cone penetration tests.

The sand compressibility C, is related to the cone penetration resistance q_c, by the relationship developed by Buisman.

$$C = 1.5 \frac{q_c}{p_o}$$

$$Cp_o = 1.5q_c = E \qquad (32)$$

Since the deformability characteristics of the sand layers by means of Equ. 32, and the stress distribution with depth by means of Equ. 31 have been determined, the settlement can be predicted using Equ. 29 for the total of the different sand layers for which C is assumed constant.

It was recommended that at least three penetration tests should be carried out and from these the maximum and minimum values of C be determined. An upper limit of the influence of the heterogeneity of the sand layers can be obtained by computing the settlement at the same point once with the minimum value of C and once with the maximum value of C for each layer considered. The extreme values of C are obtained by considering at each depth the minimum and maximum value of C, deduced from the penetration resistances, and by calculating the central value for a definite layer from all the minimum or maximum values of that layer.The average settlement and the limits of the settlement can then be calculated by means of the appropriate C values.A comparison was made by De Beer and Martens between calculated and observed settlements at sites where bridge piers were constructed and it was found that the average ratio of the predicted to calculated settlements was of the order of 1.9.

De Beer in 1965 made a distinction between normally loaded and overloaded sands for the application of the method. He stated that the previously described method applies only to normally loaded sands When the soil has already been previously loaded to higher pressures than will be imposed by the foundation, a reduction factor is applied to the settlement which was calculated as above. This reduction factor is obtained from cyclic loading tests carried out in an oedometer according to the following procedure.

Samples taken from the sand layer are subjected to a loading and unloading cycle in a oedometer at the same density as in the field. The loading cycle can be expressed by a log law as:

$$\Delta h = \frac{h}{C_{oed}} \ln \frac{p_1 + \Delta p + p_c}{p_1 + p_c} \tag{33}$$

where p_c is an introduced parameter. For the unloading cycle the same procedure is followed and a swelling constant A_{oed} is found:

$$\Delta h = \frac{h}{A_{oed}} \ln \frac{p_1 - \Delta p + p'_c}{p_1 + p'_c} \tag{34}$$

Hence the ratio A_{oed}/C_{oed} is obtained from the oedometer test, and the C values deduced from the cone resistances are multiplied by that ratio. So the value of the correction factor A is obtained:

$$A = C \frac{A_{oed}}{C_{oed}} \tag{35}$$

The settlement then can be calculated according to Equ.29 after the introduction of the correction factor A.

$$S = \int_{0}^{H} \frac{1}{A} 2.3 \log(\frac{p_o + \Delta p}{p_o}) \Delta H \qquad (36)$$

The fact that the sand has previously been subjected to higher loads than will be imposed by the foundation is obtained from geological evidence or from the special features of the work. Even so the obvious difficulty is that in many applications the degree of overconsolidation of a sand is not known and cannot be easily determined.

6.3 Meyerhof modification (1965)

Meyerhof proposed a modification of the prviously described method on the basis of a comparison of the observed settlements of 17 structures with those predicted by the method. He noted that the actual settlements were overestimated by a factor of 1.9 and recommended that 50 % greater bearing pressures could safely be used for the same calculated settlement. This modification leads, for the considered structures, to a range of predicted settlements between approximately 0.8 to 2 times the observed values, with an average of approximately 1.3.

Schmertman (1970) pointed out that the above modification by Meyerhof is roughly equivalent to increasing the modulus E encountered in Equ. 32 by 28 % which is equivalent of changing the Buisman relationship from

$$E_s = 1.5 q_c$$

to

$$E_s = 1.9 q_c \qquad (37)$$

6.4 Schmertmann theory (1970)

Schmertmann (1970) proposed a different approach to the use of cone penetration tests in the calculation of settlement of footings on sands. Instead of determining the induced vertical stress under the foundation due to the applied loading, as the Buisman, De Beer method, Schmertmann method is based on the distribution of vertical strain under the center of a footing lying on a uniform sand. Schmertmann pointed out from theoretical model studies and experimental and computer simulation results, that the vertical strain under shallow foundations on homogeneous sand, has a qualitatively different distribution than the distribution of the increase in vertical stress. This was in contradiction with the common assumption that the vertical strain distribution under a footing was qualitatively similar to the distribution of the increase in vertical stress, which had as a result that the greatest strain would occur immediately under the footing, where the stress increase is the greatest. According to the proposed theory, the depth at which the maximum vertical strain occurs is somewhat lower, at a depth approximately one-half the foundation width.The method can be summed up as follows.

From the elastic theory, the distribution of vertical strain within a linear elastic half-space subjected to a uniformly distributed load over some area at the surface can be described by:

$$\epsilon_z = \frac{\Delta p}{E} I_z \qquad (38)$$

in which Δp is the intensity of the uniformly distributed load, E is

Young's modulus of the elastic medium and I_z is a vertical strain influence factor given by :

$$I_z = (1 + \nu) \left[(1 - 2\nu) A + F\right] \tag{39}$$

where A and F are dimensionless factors depending only on the geometric location of the point considered and ν is the Poisson's ratio. Since Δp and E remain constant the vertical strain distribution is the same with the I_z distribution. The distribution of the vertical strain influence factor I_z for a uniformly loaded circular area at the surface of an elastic half-space is shown in Fig. 30 for Poisson's ratios $\nu = 0.4$ and $\nu = 0.5$ and for the results obtained by D' Appolonia and Eggestand.

Based upon this theoretical model study and on the results of a finite element analysis (Fig. 31) of deformations of a nonlinear material with assumed characteristics similar to sand, it was observed that the distribution of strain within such loaded cohesionless masses is very similar in form to that for a linear elastic medium. Hence, the distribution of vertical strain within a mass of cohesionless material could be expressed by Equ. 38 in which the Young's modulus might vary from point to point. For practical purposes the distribution of the strain influence factor was simplified and a triangular distribution shown in Fig. 30 by the heavy bold line was adopted.

This simplified distribution was called by Schmertmann the " 2B-0.6 distribution" because the maximum value of I_z is assumed to be 0.6 and occurs at a depth $z/B = 0.5$ or at B/2 and the magnitude of $I_z = 0$ at a depth $z/B = 2$ or 2B, where B is the least dimension of a rectangular footing.

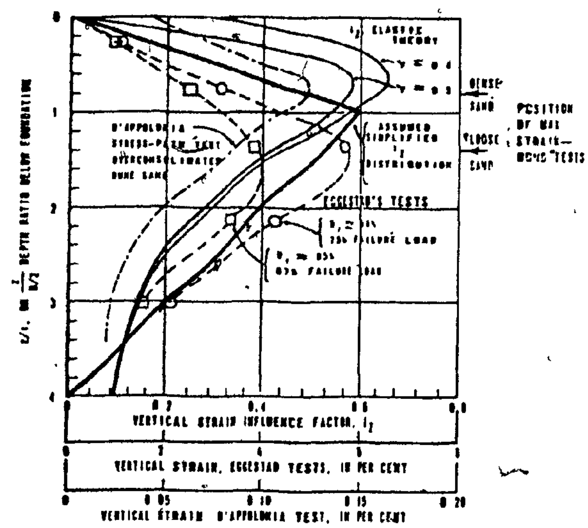


Fig. 30 Theoretical and experimental distribution of vertical strain below center of loaded area. (Schmertmann 1970)

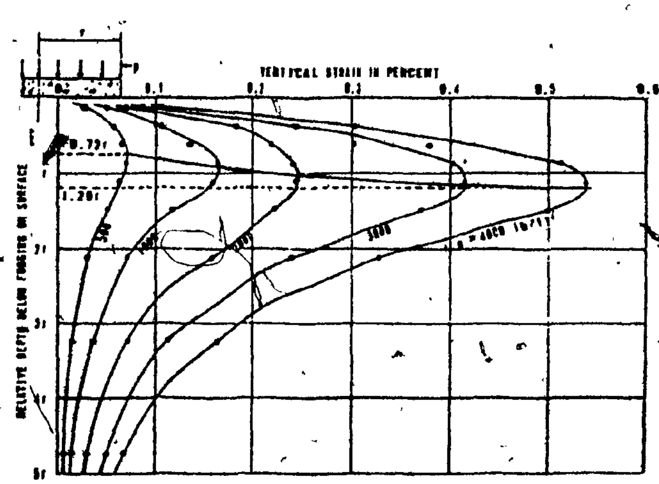


Fig. 31 Nonlinear, stress dependent finite element model prediction of vertical strains under center of 10-ft diameter, 1.25 ft thick concrete footing loaded on surface of normally consolidated sand with $\phi = 37^{o}$. (Schmertmann 1970)

Since the strain distribution is that of the strain influence factor multiplied by $\Delta P/E$ and the settlement is the area between the axis $I_z=0$ and the adopted simplified I_z distribution, the settlement can be computed by integrating the strains.

Hence :

$$S = \int_0^{\infty} \epsilon_z \, dz = \int_0^{\infty} \frac{\Delta P}{Z} I_z dz \simeq \Delta p \int_0^{2B} \frac{I_z}{E} dz \qquad (40)$$

Equ. 40 can be approximated still further as a summation of settlements of approximately homogeneous layers to account for soil layering. The settlement then becomes:

$$S = \Delta p \sum_{i=1}^{n} \left(\frac{I_z}{E}\right)_i \Delta z_i \qquad (41)$$

In proposing Equ. 41 Schmermann considered application of corrections in order to take into account the effects of foundation embedment, shape of loaded area, effects of adjacent loads, ground water effect, creep in sand and correction for soil layered by a rigid boundary layer. The proposed corrections and recommendations are given below.

For the foundation embedment effects it was considered that the foundation embedment can greatly reduce the settlement under a given load. Schmertmann proposed a correction factor based not on the basis of D/B ratio, but on the ratio $P_o/\Delta p$ where P_o is the effective in situ overburden pressure at the foundationdepth and $\Delta p = P - P_o$, is the net foundation pressure increase . The proposed linear correction factor was defined as:

$$C_1 = 1 - 0.5 \left(\frac{p_o}{\Delta p} \right) \geq 0.5 \qquad (42)$$

The correction factor introduced incorporates the effects of strain relief due to embedment, and yet retains simplicity for design purposes, assuming that the "2B-0.6" distribution of the strain influence factor is unchanged, but its maximum value is modified. The suggested proposal was based on experimental results and also on the elastic theory indication which has the restriction that C_1, should equal or exceed 0.5.

For the effect of the shape of the loaded area it was recommended that no correction was necessary, although the theory of elasticity indicates that the distribution of I_z should be modified according to the shape of the loaded area. While an approximation, Schmertmann suggested that such a correction would be a refinement which is unwarranted because, as the foundation shape changes from approximately axisymmetric to an approximately plane strain condition, the angle of shearing resistance increases as the stresses at a given depth also increase. These two effects tend to cancel each other, giving a strain distribution which is, perhaps, not very different over a wide range of length to width ratios. Of significance also to this interpretation was the fact that no such correlation is used with the S.P.T. empirical methods.

In considering the effect of adjacent load on settlement computations, there was a question of whether settlement was influenced from the interaction between adjacent foundation loading. Schmertmann suggested that if 45^o lines from the edges of two adjacent foundations intersect at a depth greater than $2B_2$, where B_2 is the width of a foundation placed next to an existing foundation of greater width B_1, the effects are negligible, and the foundations must be treated as isolated. The foundations, also must be treated as independent regardless of load seguence, if the 45^o lines intersect at a depth greater than B_1. On the other hand if the foundations are close enough and the distance between them is less than B_1, where B_1 is the smallest width, and are loaded simultaneously then they must be treated as a single foundation of width $B = B_1 + B_2 + B_{12}$ where B_{12} is the distance between them, loaded by the sum of the two loads distributed over the new width.

A correction must be made in the case of a rigid layer encountered within the intrval 0-2B. The simple I_z distribution remains the same but the soils below this boundary, to the depth 2B, are assumed to have a very high modulus. Vertical strains below such a boundary then become negligible and are assumed equal to zero. In this case the summation noted in Equ. 41 is carried out to the boundary only, rather than to 2B.

Schmertmann also applied a second correction factor to account for the time dependent increase in settlement which appears to occur even for foundations on cohesionless soils. In the past it had been rare to consider time effects in settlement problems involving free draining soils. But because many of the published settlement records, examined by Schmertmann, indicated settlement continuing with time in a manner suggesting a creep type phenomenon, and also because in the case studies carried out, the presence of time dependent effect were of significance, a correction factor was introduced, adopted and modified from the work of Nonveiler (1963).

This correction factor is given by :

$$C_2 = 1 + 0.2 \log_{10} \left(\frac{t}{0.1} \right) \tag{43}$$

in which time t is expressed in years.

Applying the correction factors Equ. 41 becomes :

$$S = C_1 \; C_2 \; \Delta p \sum_{0}^{2B} \left(\frac{I_z}{E}\right)_i \; \Delta z_i \tag{44}$$

which is the final expression as it was suggested by Schmertmann.

In the above expression C_1, C_2 are correction factors given by Equ. 42 and 43 respectively, Δp is the net load intensity at the foundation level, I_z is the vertical strain influence factor as in Fig. 30 and E is the appropriate Young's modulus at the middle of the i-th layer , of thickness Δz_i.

In Equ. 44 the key soil-property variable which should be determined is the equivalent Young's modulus for the static compression of sand. In order to determine E Schmertmann used a form of plate bearing load test used in Norway, known as the screw-plate test, and the data obtained were correlated with the static cone bearing capacity q_c . He noticed that a good correlation existed between compressibility and cone bearing capacity q_c in sands. Actually about 90 % of the accumulated data fell within the factor of 2 band as shown in Fig. 32. With this band as a guide, Schmertmann chose a single correlation line for design in ordinary sands as shown in Fig. 33. This correlation is also expressed as :

$$E = 2q_c \tag{45}$$

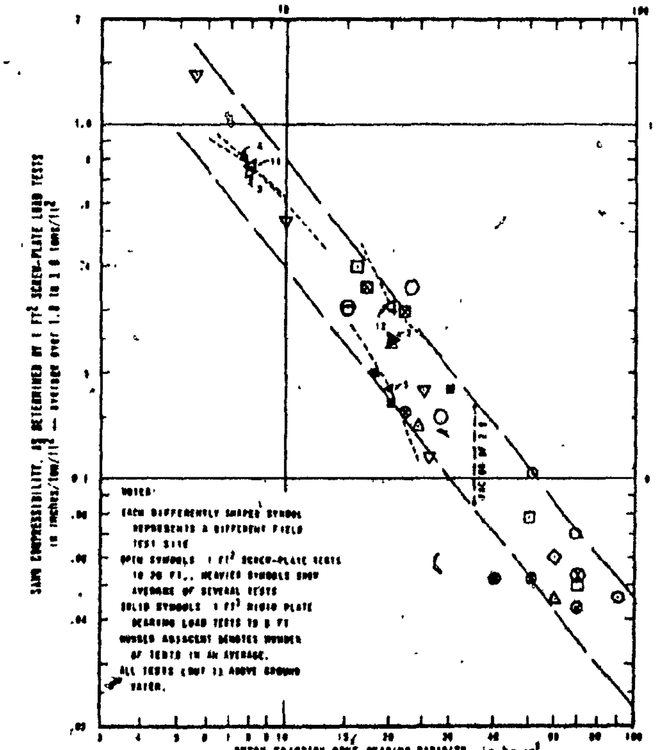


Fig. 32 Experimental correlation between Dutch cone bearing capacity and compressibility, under in-situ screw-plate load test, of some fine sand in Florida. (Schmertmann 1970)

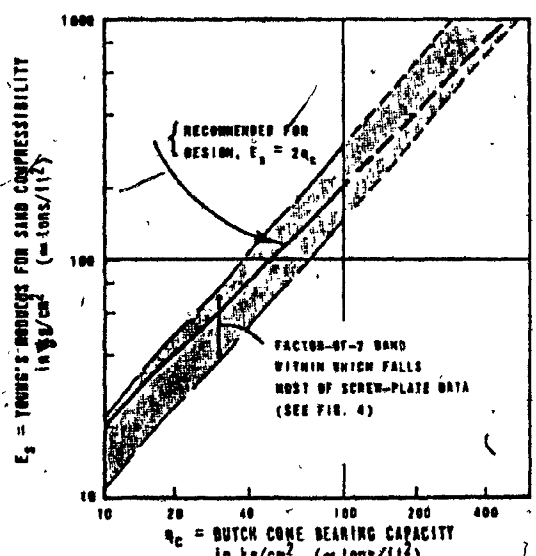


Fig. 33 Correlation between q_c and E_s recommended for use in ordinary design by Schmertmann (1970)

Having established the correlation between Young's modulus E and the static bearing capacity q_c and having all the parameters which enter in Equ. 44, determined, the static settlement of isolated, rigid, shallow foundations over granular soils can be computed in the following procedure as was suggested by Schmertmann :

i) Determine the static cone bearing capacity q_c profile over the depth interval from the proposed foundation lavel to a depth below this of 2B, or to a boundary layer that can be assumed incompressible, whichever occurs first. The penetration should be at a standard rate of 2cm/ sec and by means of a Dutch static cone equipment because the correlation of E has been established for such a system.

ii) The approximate unit weights of surcharge soils, and the position of water table if within D, where D is the foundation embedment, must be known. These data are needed to estimate p_0 for the evaluation of C_1 correction factor.

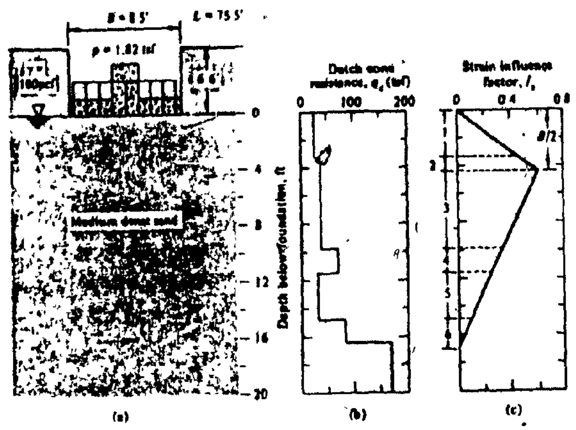
iii) Divide the q_c profile into a convenient number of layers, each with constant q_c over the depth interval 0-2B below the foundation. Each layer of constant q_c can be taken as applying to a depth interval Δz equal to the increment between successive q_c measurements.

iv) Draw the assumed 2B-0.6 triangular distribution for the strain influence factor I_z, along a scaled depth of 0-2B below the foundation. Locate the depth of the mid-height of each of the layers. Then determine the I_z value at each layer's mid-height.

v) The $(I_z/E)\,\Delta z$ is next determined, which represents the settlement of each layer. The sum of $(I_z/E)\Delta z$ multiplied by the appropriate correction factor C_1 and C_2 (Equ. 42 and 43) and by the appropriate Δp is the final settlement estimate for the time after loading assumed in the calculation of C_2. The above calculations may be further simplified by using a table as in Fig. 34.

The new method of Schmertmann is simpler than the Buisman ,De Beer method on computations.It does not require computation of the below foundation distribution of effective overburden stress and vertical stress increase. The method does not distinguish between overconsolidated and consolidated sands using the same correlation between q_c and E for both the cases, even though overconsolidated sands have less compressibility than simply consolidated. The above procedure for settlement estimation can also be used with S.P.T. data by means of tables correlating the S.P.T. N-values with q_c. Such a conversion is recommended only as a temporary expedient until cone data can be used directly.

Schmertmann concluded that his method leads to more accurate estimates of settlement than the Buisman,De Beer method. He based this claim on the examination of 16 sites and comparison of the settlement obtained by the two methods. On the average, the calculated Buisman, De Beer settlements were about 50 % greater than those obtained by his method.



(a) Schematic soil profile.

(b) Dutch cone bearing capacity.

(c) Assumed 2B-0.6 distribution and layers used in example.

(modified from Schmertmann)

TABLE III

Data for settlement estimate

(1) Layer No., i	(2) Δz (ft)	(3) Average q_c (tsf)	(4) Average E (tsf)	(5) Depth of Middle Below Foundation (ft)	(6) I_z	(7) $\left(\frac{I_z}{E}\right)\Delta z$
1	3.3	25	50	1.65	0.23	0.0152
2	1.0	35	70	3.8	0.53	0.0076
3	5.6	35	70	7.1	0.47	0.0376
4	1.6	70	140	10.7	0.30	0.0034
5	3.3	30	60	13.1	0.185	0.0102
6	2.3	85	170	15.9	0.065	0.0008
Total						$\sum\left(\frac{I_z}{E}\right)\Delta z = 0.0748$

Fig. 34. Application of Schmertmann method on an example.

CHAPTER 7

CORRELATION OF S.P.T. AND DUTCH CONE TESTS RESULTS

7.1 General

Many comparative studies have been carried out to correlate the N-values of the S.P.T. with the q_c values of the Dutch cone static penetration tests. The results of these studies differ in the proposed correlation ratio $n = q_c/N$ and also on the factors which may affect that ratio. These studies are empirical and are based on limited test data.

Meyerhof (1965) presented comparative data between the S.P.T. and the classical static penetration tests. The dynamic penetration resistance of a cone driven with an energy of 350 ft/lb is about twice that of the S.P.T. This may be partly due to the fact that the end area of the cone is approximately twice that of the spoon used in the S.P.T. For fine or silty medium dense to loose sands, it was shown by Meyerhof that the correlation may be expressed as follows:

$$\frac{q_c}{N} = 4 \qquad (46)$$

where q_c is in tons/sqft and N is the number of blows per foot of penetration of the S.P.T. The N value has to be corrected in the case of fine sand below the water table. Meyerhof findings are summarized in Table IV, in which are correlated the N and q_c values with the relative density and angle of internal friction. The lowest values of the angle of internal friction given in Table IV are conservative estimates for uniform, clean sand and they should be reduced by at least 5^o for clayey

TABLE IV

Approximate relationship between relative density of fine sand, the S.P.T. the static cone resistance, and the angle of internal friction

Compactness of fine sand	Relative density (D_r)	S.P.T. (N)	Static cone resistance (q_c in bar)	Angle of internal friction (φ in degrees)
Very loose	< 0.2	< 4	< 20	< 30
Loose	0.2 - 0.4	4 - 10	20 - 40	30 - 35
Medium dense	0.4 - 0.6	10 - 30	40 - 120	35 - 40
Dense	0.6 - 0.8	30 - 50	120 - 200	40 - 45
Very dense	0.8 - 1.0	> 50	> 200	45

(Meyerhof 1965)

TABLE V

Relationship between static point resistance q_c and N-values of S.P.T.

Soil type	q_c/N
Silts, sandy silts, and slightly cohesive silt-sand mixtures	2.0
Clean, fine to medium sands and slightly silty sands	3 - 4
Coarse sands and sands with little gravels	5 - 6
Sandy gravels and gravel	8 - 10

(Schmertmann 1970)

sand. These values, as well the upper values of the angle of internal friction which apply to well graded sand may be increased by 5^{o} for gravelly sand. The proposed relationship in Equ. 46 by Meyerhof does not take into account effects such as the particle size and particle size distribution and hence it is not applicable to all soils. Robin (1974) has reported that most researchers agree that particle size distribution is an important factor in such a correlation. For this reason the proposed correlation should be used with caution, particularly as the average size of particles increases. It was also pointed out that any proposed correlation should vary with the nature of the layers being tested.

A more detailed correlation was presented by Schmertmann (1970) and is given in Table V. These values have been obtained in order to transpose S.P.T. data to use the static penetrometer theories relating to bearing capacity and settlement. It is clear according to Schmertmann that the q_c/N ratio varies with grain size and perhaps with gradation. The finer grained the soil the smaller the q_c/N ratio, reaching as low as about 1 for some clays and as hight as 18 for some gravels. For the tests carried out in fine sand of Florida the mean values of q_c/N fall in the range of 4.0 to 4.5. But there was a great spread around the mean values. The writer assumed that these ratios are indepent of depth, relative density and water conditions. It was suggested that as many N-values as possible be obtained to minimize by averaging, the large correlation errors possible with only sparse data.

It is of importance to note that the proposed correlation values have been made to convert N-values to q_c values. This conversion should be conservative, with the q_c on the low side of the true values. Hence the proposed correlations are conservative when going from N to q_c and liberal for the opposite case.

REFERENCES

1. Alpan, I. (1964). "Estimating the Settlements of Foundations on Sand". Civ. Eng. and Public Works Rev., Vol. 59, Nov., 1415-1418

2. Bazaraa, A.R.S.S. (1967). "Use of Standard Penetration Test for Estimating Settlements of Shallow Foundations on Sand", Ph.D. Thesis, University of Illinois, Urbana- Illinois.

3. Coffman, B.S. (1960). "Estimating the Relative Density of Sands". J. Soil Mechanics & Foun. Div., A.S.C.E. Vol. 30, No. 10.

4. D' Appolonia, D.J., D' Appolonia, E., and Brisette, R.F. (1968) "Settlementof Spread Footings on Sand". J. Soil Mech. & Foun. Div., A.S.C.E., SM3, May, 735-760.

5. D' Appolonia, D.J., D' Appolonia, E. and Brisette, R.F. (1970). "Discussion on Settlement of Spread Footings on Sand". J. Soil Mech. & Foun. Div., A.S.C.E., SM2, March, 754-761.

6. De Beer, E. (1965). "Bearing Capacity and Settlement of Shallow Foundations on Sand". Proc. Sym. Bearing Capacity and Settlement of Foundations, Duke University, 15-33.

7. De Beer, E. and Martens, A. (1956). "Method of Computation of an Upper Limit for the Influence of Heterogeniety of Sand Layers in the Settlement of Bridges". Proc. 4th Int. Conf. Soil Mech. & Foun. Engng. London, Vol.1, 275-281.

8. Fletcher, G.A. (1965). "Standard Penetration Test- Its Uses and Abuses" J. Soil Mech. & Found. Div., A.S.C.E., Vol. 91, SM4, July, 67-75.

9. Gibbs, H.J. and Holtz, W.G. (1957). "Research on Determining the Density of Sands by Spoon Penetration Testing, Proc. 4th Int. Con. Soil Mech.

& Foun. Engng., London, Vol. 1, 35-39.

10. Mansur, C.I. and Kaufman,R.I. (1958). "Pile Tests, Low-sill Structure, Old River, Louisiana". Trans. A.S.C.E., Vol. 123, 715-748.

11. Meyerhof,G.G. (1956). "Penetration Tests and Bearing Capacity of Cohesionless Soils". J. Soil Mech. & Foun. Div., A.S.C.E., Vol. 82, SM1, Jan.

12. Meyerhof, G.G. (1965). "Shallow Foundations". J. Soil Mech. & Found. Div., A.S.C.E., Vol. 91, SM2, March 21-31.

13. Menard,L. (1956). "An Appatatus for Measuring the Strength of Soils in Place". M.Sc, Thesis, University of Illinois, Urbana-Illinois.

14. Menard,L. (1965). "Rules for the Calculation of Bearing Capacity and Foundation Settlement based on Pressuremeter Tests". 6th ICSMFE, Vol.2, 295-299.

15. Nonveiller,E. (1963). "Settlement of a Grain Silo on Fine Sand". Proc. European Conf. Soil Mech. & Found. Engng.,Wiesbaden, Vol. 1, 285-294.

16. Parry,R.H.G. (1971) "A Direct Method of Estimating Settlement in Sands from S.P.T. Values", Proc. Symp. Interaction of Structure & Found., Midlands Soil Mech. & Found. Engng. Soc., Birmingham, 29-37.

17. Peck,R.B., and Bazaraa, A.R.S.S., (1969). "Discussion of Paper by D'Appolonia et al", J. Soil Mech. & Found. Div. Proc. A.S.C.E., Vol. 95, SM3, 305-309.

18. Peck, R.B., Hanson, W.E., and Thornburn, T.H.,(1974), "Foundation Engineering", 2ed. John Wiley & Sons, Inc.

19. Philcox, K.T. (1962). "Some recent Developments in the Design of

High Buildings in Hong Kong", Struct. Engr., Vol. 40, No. 3, October.

20. Sanglerat, G. (1972). "The Penetrometer and Soil Exploration", Elsevier, Amsterdam.

21. Schmertmann, J. H. (1970). "The Static Cone to Compute Static Settlement Over Sand", J. Soil Mech. & Found. Engng. Div., A.S.C.E., Vol.96 SM3, 1011-1043.

22. Schultze, E. and Menzebach, E. (1961). "Standard Penetration Test and Compressibility of Soils", 5th ICSMFE, Paris, Vol. 1, 527

23. Schultze, E. and Moussa, A. (1961). "Factors affecting the Compressibility of Sand", 5th ICSMFE, Paris, Vol. 1, 335

24. Sutherland, H.B. (1963). "The Use of In Situ Tests to Estimate the Allowable Bearing Pressure of Cohesionless Soils", Struct. Engr., Vol. 41, No. 3, March, 85-92.

25. Terzaghi, K. and Peck, R.B. (1967). "Soil Mechanics in Engineering Practice, 2nd edition, John Wiley & Sons, New York.

26. Thornburn, S. (1963). "Tentative Correction Chart for the Standard Penetration Test in Non-cohesive Soils", Civ. Engng. and Pub. Works Rev., June, 752-753.

27. Tomlinson, M.J. (1969). "Foundation Design and Construction", 2nd edition, Pitman, London.

28. Zolkov, E. and Wiseman, G. (1965). "Engineering Properties of Dune and Beach Sands and the Influence of Stress Histiry", Proc. 6th Int. Conf. Soil Mech. & Found. Engng., Montreal, Vol 1, 134.

29. Rodin, S. (1961). "Experiences with Penetrometers with Particular Reference to the Standard Penetration Test", Proc. 5th Int. Conf. Soil Mech. & Found. Engng., Vol. 1, 517.