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Interpretation of Cone Penetration Tests
Part I (Sand) and Part II (Clay)

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INTERPRETATION OF CONE PENETRATION TESTS - PART I (SAND)

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ABSTRACT

Significant advances have been made in recent years in research, development, interpretation and application of cone penetration testing. The addition of pore pressure measurements during cone penetration testing has added a new dimension to the interpretation of geotechnical parameters.

The cone penetration test induces complex changes in stresses and strains around the cone tip. No one has yet developed a comprehensive theoretical solution to this problem. Hence, the cone penetration test provides indices which can be correlated to soil behaviour. Therefore, the interpretation of cone penetration data is made with empirical correlations to obtain required geotechnical parameters.

This paper discusses the significant recent developments in cone penetration testing and presents a summarized work guide for practicing engineers for interpretation for soil classification, and parameters for drained conditions during the test such as relative density, drained shear strength and deformation characteristics of sand. Factors that influence the interpretation are discussed and guidelines provided. The companion paper, Part II (Clay), considers undrained conditions during the test and summarizes recent developments to interpret parameters for clay soils such as, undrained shear strength, deformation characteristics of clay, stress history, consolidation characteristics, permeability and pore pressure. The advantages and use of the piezometer cone are discussed as a separate topic in Part II (Clay). The authors personal experiences and current recommendations are included.

Key Words: Static cone penetration testing, in-situ, interpretation, shear strength, modulus, density, stress history, pore pressures.

INTERPRETATION OF CONE PENETRATION TESTS - PART I (SAND)

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INTRODUCTION

The cone penetration test is becoming increasingly more popular as an in-situ test for site investigation and geotechnical design. As a logging tool this technique is unequalled with respect to the delineation of stratigraphy and the continuous rapid measurement of parameters like bearing and friction.

Recent publications have provided vast amounts of information about cone penetration methods and their interpretation (ASCE Symposium on Cone Penetration Testing and Experience, 1981 and the 2nd European Symposium on Penetration Testing, 1982). In addition, much experience has been developed at the University of British Columbia (UBC) over the past 5 years in research, development, interpretation and application of cone testing from many service to industry projects as well as thesis research.

About the only recent publication to summarize interpretation of cone penetration results was produced by John Schmertmann (1978a) in a report to the U.S. Department of Transportation, which is now out-of-print. Schmertmann's report was prepared principally for interpretation of mechanical cone data although much of the report is applicable to electric cone data. Also, Schmertmann's report was prior to the development of the piezometer cone.

This paper is limited to a discussion of the recent advances in interpretation of cone penetration test data to obtain soil parameters in an attempt to update the 1978 report by Schmertmann.

In the space available, it is not possible to discuss in detail all recent developments. However, an attempt has been made to discuss the significant developments, present a summarized work guide for practicing engineers and to provide the reader with a comprehensive list of references that will provide further details.

The paper has been divided into two parts. Part I (Sand) deals with the general topics of soil classification, stratigraphy and interpretation of soil parameters in drained soil. Part II (Clay) deals with interpretation of soil parameters in undrained soil and the interpretation of pore pressure data from a piezometer cone.

Equipment and Interpretation in General

The use of the Dutch mechanical friction cone is gaining wider popularity in the U.S. Unfortunately, its initial low cost is more than offset by its relatively slow incremental procedure, ineffectiveness in very soft soils, requirement for moving parts, labor intensive data handling and presentation, and generally poor accuracy and shallow depth capability. While electric cones have an initial higher cost, they reap benefits in terms of a more rapid procedure, continuous recording, potential for automatic data logging, reduction and plotting, and high accuracy and repeatability. The electric cone also has allowed the addition of pore pressure measurements during penetration. The continuous measurement of pore pressures along with bearing and friction has enhanced the electric cone penetrometer as the premier tool for stratification logging of soil deposits. However, mechanical cones will continue to have a usefulness because of their lower cost and simplicity of operation.

The cone is best suited for stratigraphic logging and preliminary evaluation of soil parameters. Other more specialized in-situ test methods are better suited for use in critical areas, as defined by the logging method, where more detailed assessments may be required of specific soil parameters, which of course may include undisturbed sampling and laboratory testing.

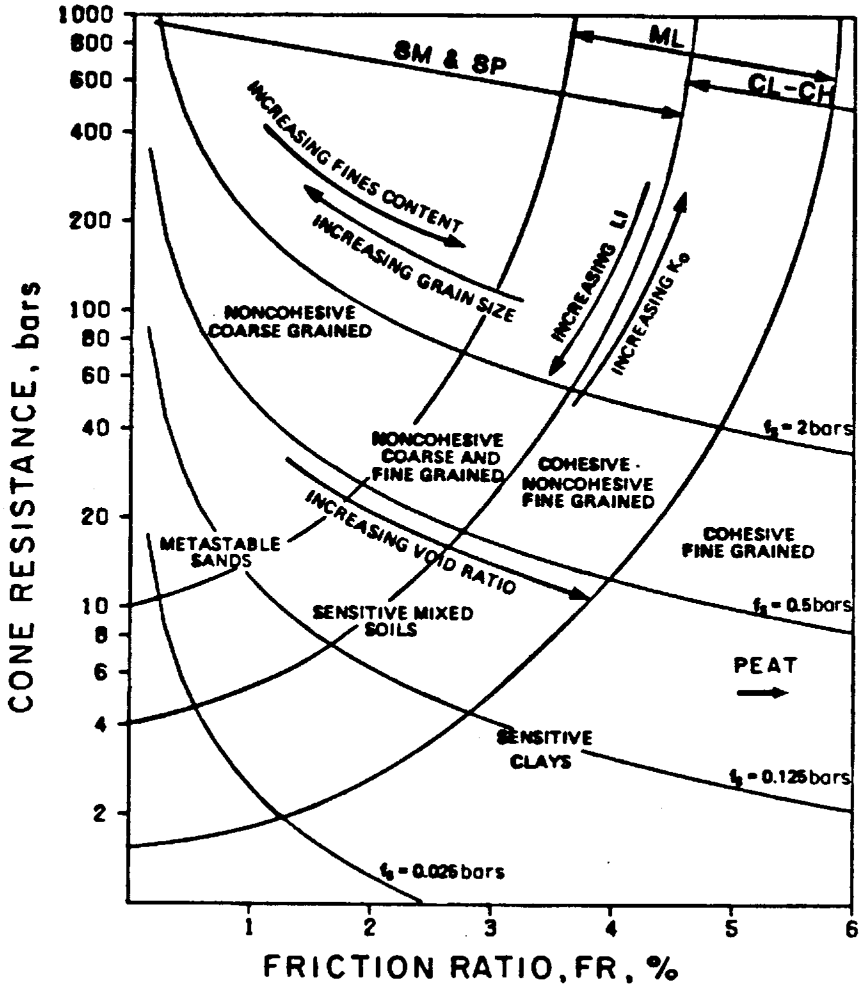
Recent publications by de Ruiter, (1982) and Campanella and Robertson, (1981 & 1982), have highlighted the importance of equipment design and procedure related to accuracy and repeatability of results obtained using the electric cone. Equipment and procedure standards are given in ASTM (D3441, 1979) and European Standard (ISSMFE, 1977).

The cone penetration test induces complex changes in stresses and strains around the cone tip. No one has yet developed a comprehensive theoretical solution to this problem. Therefore, the interpretation of cone penetration data is made with empirical correlations to obtain required geotechnical parameters.

SOIL CLASSIFICATION

The most comprehensive recent work on soil classification using electric cone penetrometer data is that by Douglas and Olsen (1981). A copy of their proposed soil-behaviour type classification chart is shown in Fig. 1. The chart shows how cone penetration test data has been correlated with other soil type indices, such as those provided by the Unified Soil Classification System. The correlation was based on extensive data collected from areas in California, Oklahoma, Utah, Arizona and Nevada (USGS Open-File Report No. 81-284, 1980). Figure 2 is a simplified working version for identifying soil type.

The usual progression of site investigation using cone penetration test (CPT) is to perform the CPT soundings, develop detailed site profiles



1 bar = 100 kPa ≈ 1 kgf/cm^2

FIG.1: SOIL CLASSIFICATION CHART FOR STANDARD ELECTRIC FRICTION CONE.

(After Douglas and Olsen, 1981).

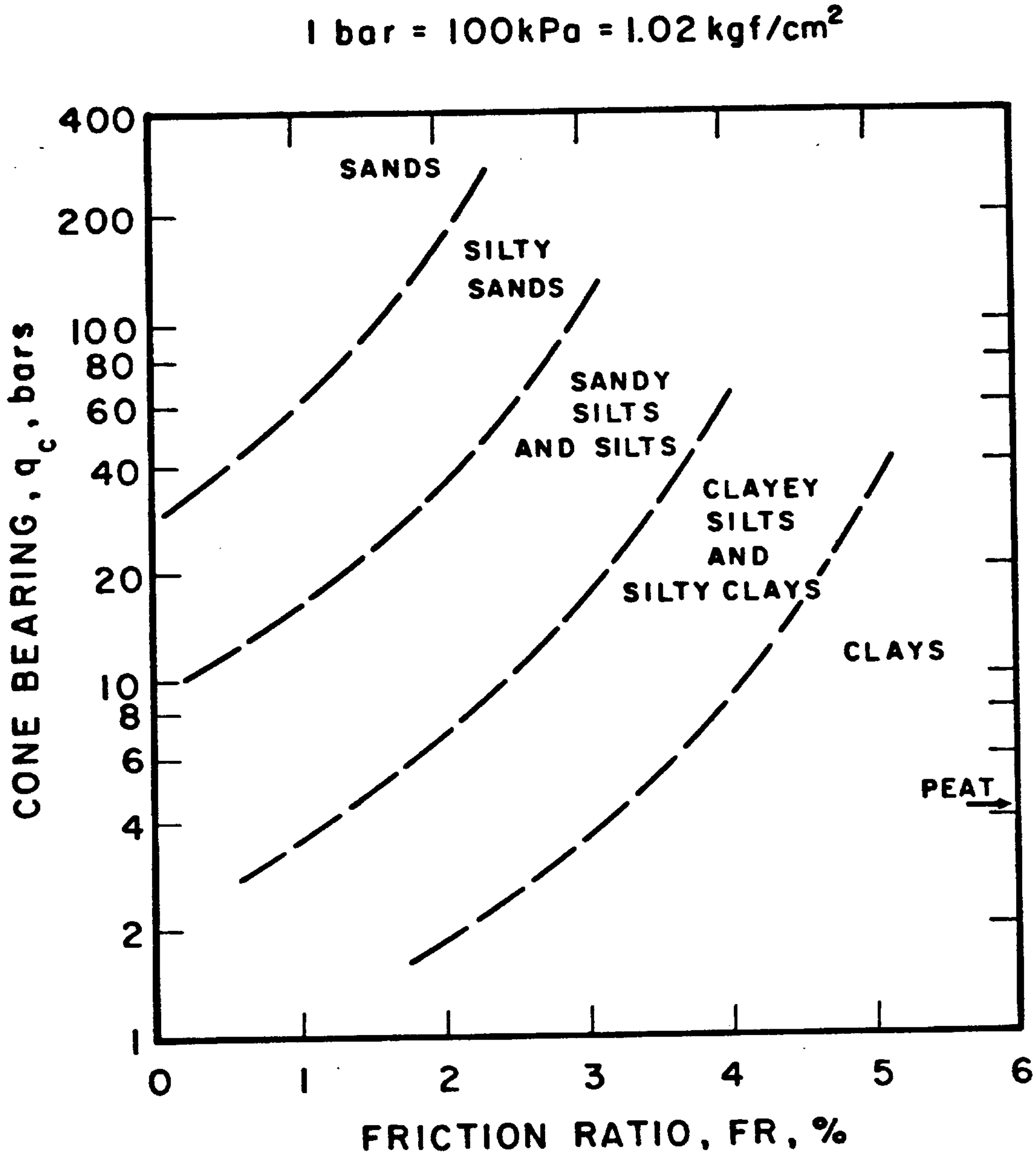
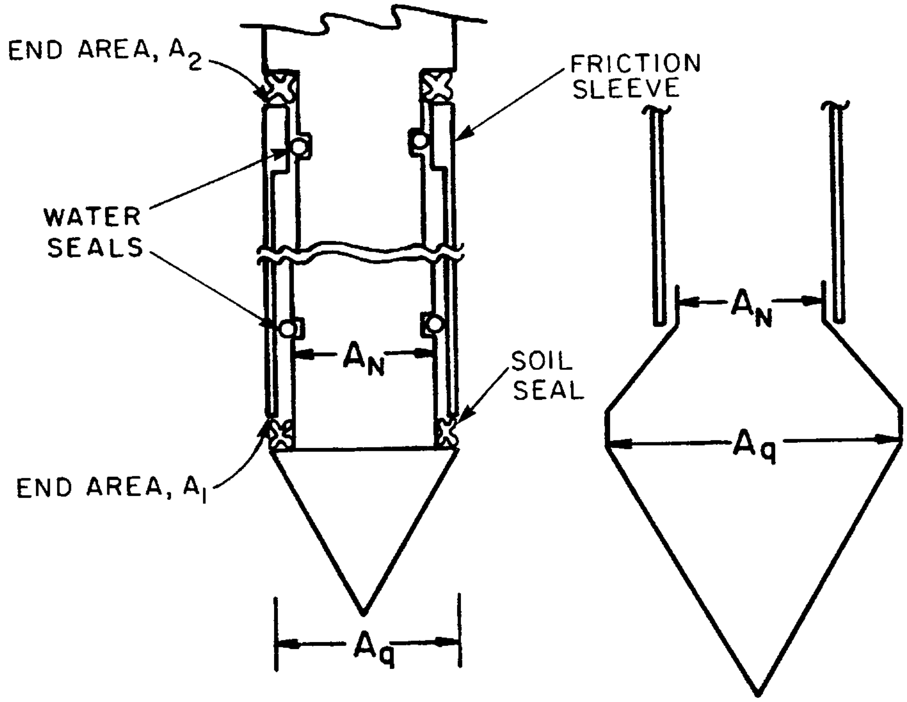


FIG. 2: SIMPLIFIED SOIL CLASSIFICATION CHART FOR STANDARD ELECTRIC FRICTION CONE.

with the soil classification charts (Figs. 1 and 2), and then selectively sample and test to provide any additional information regarding ambiguous classifications. With local experience this latter step is often not necessary.

Recent work by Campanella, Robertson and Gillespie (1983) has illustrated the importance of cone design and the effect that water pressures have on the measured bearing and friction due to unequal end areas (Fig. 3). An all around water pressure causes a thrust on the friction sleeve which in turn causes a zero offset on the load cell output. The friction sleeve offset can be calculated as the total water pressure during penetration multiplied by the difference in end area (A_1-A_2) and can be either positive or negative depending upon which end area is larger. With respect to bearing, the tip should measure total stress or intergranular pressure plus water pressure. However, the water pressure is only partially sensed by the tip or bearing load cell because the area A_N is for current designs always less than A_q (see Fig. 3). A discussion of bearing corrections is given in Part II under unequal end area effects. Thus cones of slightly different designs will give different bearing, friction stress and friction ratios. With proper calibration and measurement, the effects of unequal end areas can be corrected. A detailed discussion concerning cone design is also given by Schaap and Zuidberg (1982).

The data used to compile the classification chart (Fig. 1) used bearing and friction values that had not been corrected for pore pressure effects, since, in general, pore pressure measurements were not made. It appears that there is little difference between corrected and uncorrected friction ratios for most soil types except for those soils that classify in



Bearing Net Area Ratio = A_N / A_q
Friction Unequal End Area, $A_1 \neq A_2$

FIG. 3: INFLUENCE OF UNEQUAL END AREAS.

the lower left portion of the chart (Fig. 1). These soils usually generate large positive pore pressures during penetration and have very low measured bearing (q_c<10 bars) and small friction values where corrections become very significant. These soils also tend to have high liquidity index values, as noted by Douglas and Olsen (1981). Also, in off-shore investigations where significant hydrostatic water pressures exist, it may be important to account for these effects for most soil types. Most standard electric cone data does not include pore pressure measurements and the measured bearing and friction values are therefore not corrected for pore pressure effects. For this type of data the chart in Fig. 1 can be used directly to provide a reasonable estimate of soil type. If pore pressure measurements are included and the necessary corrections applied to the data, Figs. 1 and 2 should be used with caution, especially for soft saturated soils, and should always be adjusted to reflect local experience. Some electric cones, like the one developed at UBC, have equal end area friction sleeves where the friction measurement requires no correction. Increased use of these cones will eventually lead to further improvements in the soil classification chart shown in Fig. 1. Unfortunately, there is currently no cone design which eliminates the need to correct bearing for net area ratio.

Several recent publications have suggested soil classification based on pore pressure and bearing data (Jones et al., 1981, Jones and Rust, 1982, Baligh et al., 1980). Significant improvements in classification are made if all three parameters, namely, pore pressure, bearing and friction ratio are used (Campanella, Robertson and Gillespie, 1983). This is especially true for partially drained soils like fine sands, silts and clayey silts. However, a classification chart that combines all three has not yet been developed.

A comprehensive classification chart for use with a mechanical cone was proposed by Searle (1979). The chart is similar to that proposed by Schmertmann (1978a) although considerably more information is contained on Searle's chart.

Recently the CPT has also been used for classification and interpretation of unconventional soils such as carbonate sediments (Searle 1979; and Power, 1982).

Experience gained by the writers suggest that the friction ratio for some fine grained soils may decrease with increasing effective overburden pressure. Thus, the accuracy of Figs. 1 and 2 may be reduced somewhat for CPT data from very deep soundings. Of course there is no substitute for improvements to Figs. 1 and 2 based on local experience.

STRATIGRAPHY

The cone penetration tip resistance is influenced by the soil properties ahead and behind the tip. Chamber studies by Schmertmann (1978a) showed that the tip senses an interface between 5 to 10 cone diameters ahead and behind. The distance over which the cone tip senses an interface increases with increasing soil stiffness. Since stiffness increases with increasing overburden pressure, the distance over which the cone tip senses an interface tends to increase with depth (Treadwell, 1975). For interbedded relatively shallow deposits, the thinnest layer for which the measured cone bearing represents a full response (i.e. q_c to reach full value characteristic for the soil within the layer) is about 10 to 20 cone diameters. For the standard 10 cm^2 electric cone, the minimum layer thickness to ensure full or plateau value of tip resistance is between 36 cm to 72 cm. Since the cone tip is advanced continuously, the tip resistance will sense much thinner layers, but not fully. This has significant implications when interpreting cone bearing, for example, relative density determination in sand. If a sand layer is less than about 70 cm thick and located between, say, two soft clay deposits, the cone penetration resistance may not reach its full value within the sand because of the close proximity of the adjacent interfaces. Thus, the relative density in the sand may be severely underestimated.

The continuous monitoring of pore pressures during cone penetration can significantly improve the identification of soil stratigraphy (Campanella, Robertson and Gillespie, 1983). The pore pressure responds to the soil type in the immediate area of the cone tip. To aid in the identification of very thin silt or sand layers within clay deposits, some researchers (Torstensson, 1982) have proposed and successfully used thin (2.5 mm) pore pressure elements located immediately behind the cone tip.

DRAINED SOIL

Density

For cohesionless soils the density, or more commonly the relative density, is often used as an intermediate soil parameter. Recent research has shown that the stress-strain and strength behaviour of cohesionless soils is too complex to be represented solely by the relative density of the soil. Several papers in ASTM (1973) have discussed difficulties associated with determination of maximum, minimum and in-situ densities as well as problems in correlating relative density with measured soil properties. However, because many engineers continue to use relative density as a guide in design some discussion is given here on recent work relating cone penetration resistance to soil relative density.

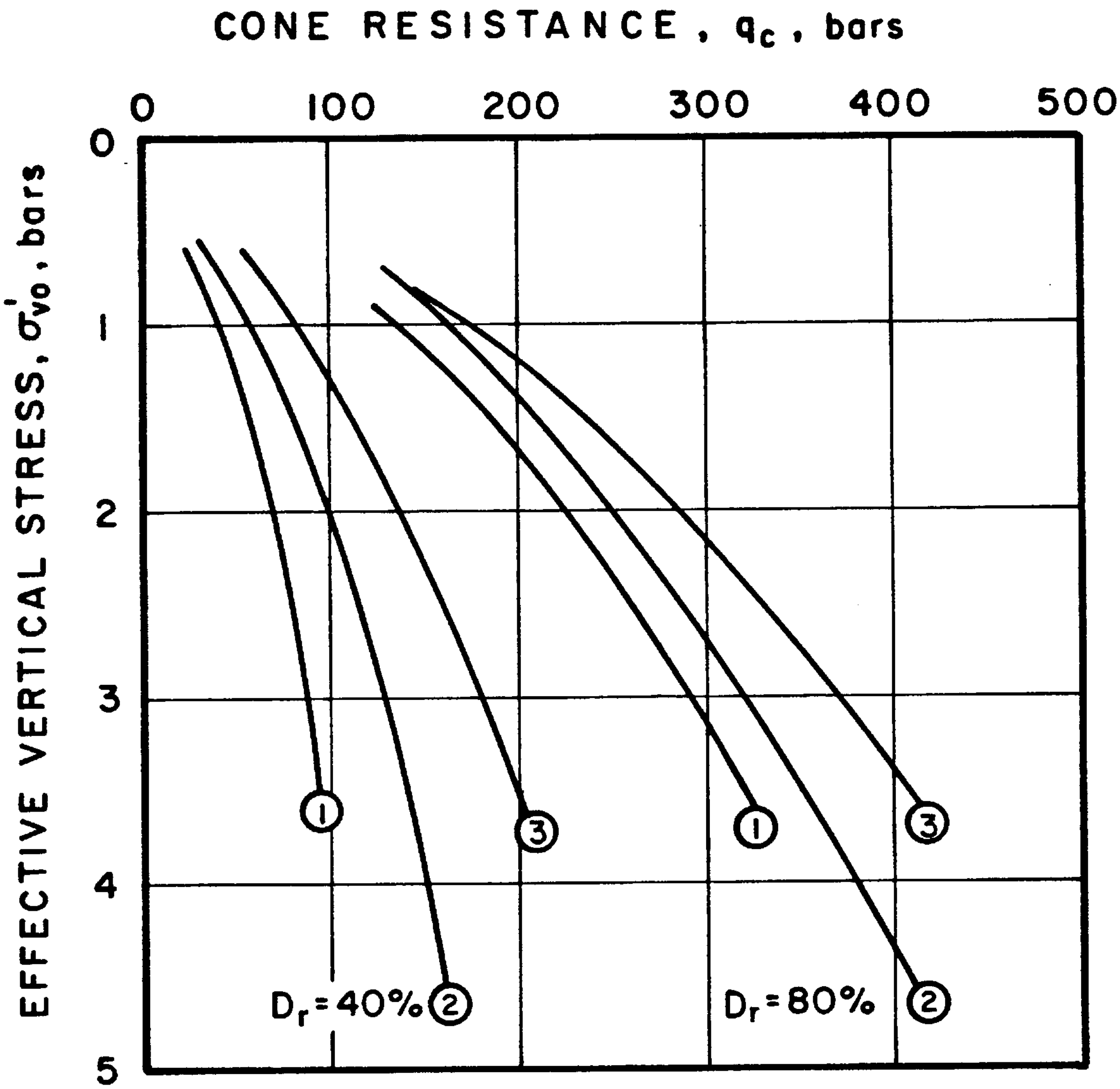
Recent work in large calibration chambers (Veismanis, 1974, Chapman and Donald, 1981, Baldi et al. 1981, Parkin et al., 1980 and Villet and Mitchell, 1981) has provided numerous correlations of cone resistance (q_c) with soil relative density (D_r). Most of these works have also shown that no single unique relationship exists between relative density, in-situ effective stress and cone resistance for all sands. Recent work by Parkin and Lunne (1982) has also shown that the measured relationships between relative density and cone resistance is influenced by the small calibration chamber size, particularly at higher densities.

It is not surprising that no unique relationship exists between cone resistance, in-situ effective stress and relative density since other factors such as soil compressibility also influence cone resistance.

A review of the numerous calibration chamber tests performed on a variety of different sands shows a significant range of D_r versus q_c relationships. However, on detailed inspection part of the variation can be accounted for due to differences in chamber size and boundary conditions. All the chamber test results, however, show that the curves are all similar in shape and most show that the cone resistance can be more uniquely related to relative density, for any given sand, if correlated with the in-situ initial horizontal effective stress (σ'_{ho}). If the horizontal effective stress is used the relationship can be expected to apply to both normally and overconsolidated sand. Fig. 4 shows a comparison between the curves proposed by Schmertmann (1978b), Villet and Mitchell, (1981) and Baldi et al. (1981) for two levels of relative density. All the curves have been corrected for chamber size. Details of the sands used in the calibration chamber studies are given in Table 1.

The calibration test data (Fig. 4) shows the importance of sand compressibility. The curves by Schmertmann (1978b) represent the results of tests on Hilton Mines sand, which is a highly compressible quartz, feldspar, mica mixture with angular grains. The curves by Villet and Mitchell (1981) represent results on Monterey Sand which is a relatively incompressible quartz sand with subrounded particles. Schmertmann (1978b) also performed tests on Ottawa sand, which is also an incompressible quartz sand with rounded particles, and obtained curves almost identical to those of Villet and Mitchell (1981). Thus, it appears that sands with a low compressibility have a D_r-q_c relationship similar to that shown by Villet and Mitchell (1981) and sands with a high compressibility have a relationship similar to that shown by Schmertmann (1978b). The sand used by Baldi et al. (1981) (Ticino Sand) was a quartz, feldspar, mica mixture with subangular particles. The Ticino Sand appears to have a moderate compressibility somewhere between the two extremes of Hilton Mines and Monterey Sand.

A large portion of CPT work is often carried out in sands where the grain minerals are predominately quartz and feldspar. These are sands similar to those tested in most of the calibration chamber work. Research has shown that there is relatively little variation in the compressibility for most quartz sands, although this depends on the angularity of the grains (Joustra & de Gijt, 1982). Angular quartz sands tend to be more compressible than rounded quartz sands. If an estimate of relative density is required for a predominantly quartz sand of moderate compressibity, the writers recommend that the relation given by Baldi et al. (1981 & 1982) be



① SCHMERTMANN (1976) Hilton Mines Sand - High Compressibility
② BALDI et al. (1982) Ticino Sand - Moderate Compressibility
③ VILLET & MITCHELL (1981) Monterey Sand - Low Compressibility

FIG.4: COMPARISON OF DIFFERENT RELATIVE DENSITY RELATIONSHIPS.

Reference	Sand Name	Mineralogy	Shape	Gradation (mm)		Porosity	
				D_{60}	D_{10}	n_{max}	n_{min}
Baldi et al., (1981, 1982)	Ticino	Mainly quartz 5%* mica	Subangular to angular	0.65	0.40	0.50	0.41
Villet & Mitchell (1981)	Monterey	Mainly quartz some feldspar	Subrounded to subangular	0.40	0.25	0.45	0.36
Schmertmann (1978b)	Ottawa #90	quartz	Rounded	0.24	0.13	0.44	0.33
—— " ——	Hilton mines	quartz + mica + feldspar	Angular	0.30	0.15	0.44	0.30
Parkin et al (1980)	Hokksund	35% quartz 45% feldspar 10%* mica	Rounded to subangular	0.5	0.27	0.48	0.36
Veismanis (1974)	Edgar	Mainly quartz	Subangular	0.5	0.29	0.48	0.35
—— " ——	Ottawa	Quartz	Subangular	0.54	0.45	0.42	0.32
Holden (1971)	South Oakleigh	Quartz	Subangular	0.19	0.12	0.47	0.35
—— " ——	—— " ——	Quartz	Subangular	0.37	0.17	0.43	0.29
Chapman & Donald (1981)	Frankston	Mainly Quart	Rounded to Subangular	0.37	0.18	----	----

* Percent mica by volume

TABLE 1: Properties of Sand Tested in Calibration Chamber Studies

used.

Fig. 5 shows Baldi's relationship between relative density (D_r) vertical effective stress (σ'_{vo}) and cone resistance (q_c), corrected for chamber test size (Baldi et al., 1982). The relationship is for normally consolidated, where $K_o = 0.45$, uncemented and unaged sands. If overconsolidated or aged sands are encountered, the horizontal effective stress (σ'_{ho}) should be used instead of σ'_{vo}. However, the application of this relationship to overconsolidated sands appears, at present, very difficult because of the inherent difficulties in measuring or choosing an appropriate σ'_{ho} in-situ and assessing the stress history of natural sand deposits.

The writers suggest that Fig. 5 should be used only as a guide to in-situ relative density, but can be expected to provide reasonable estimates for clean normally consolidated moderately compressible quartz sands. A visual classification of the grain characteristics would significantly improve the choice of relative density correlation. Care should be exercised in interbedded deposits where the cone resistance may not have reached the full value within a layer.

In an effort to overcome some of the above problems, Villet and Mitchell (1981) extended the bearing capacity theory developed by Durgunoglu and Mitchell (1975) so that cone resistance, relative density, vertical stress curves for any sand could be constructed based on a knowledge of the soil friction angle (ϕ) and its variation with stress level and a current lateral earth pressure coefficient (K_o). However, this theory takes no account of the soil compressibility. It would also seem likely that detailed information concerning the frictional strength of the soil and the lateral earth pressure (K_o) is not often available, and if

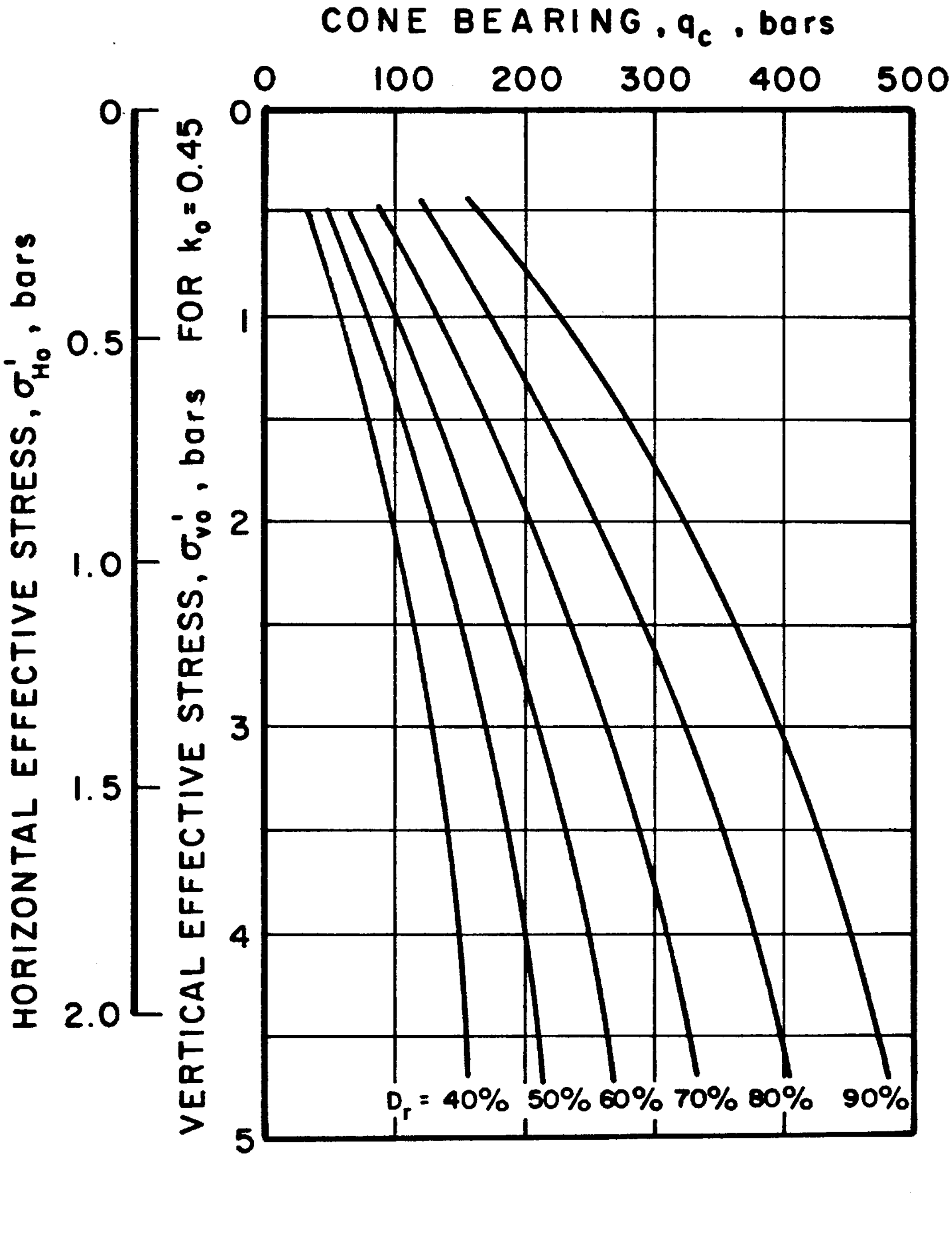


FIG.5: RELATIVE DENSITY RELATIONSHIP FOR UNCEMENTED AND UNAGED QUARTZ SANDS. (After Baldi et al. 1982).

sufficient information were available, the requirement for a knowledge of relative density would probably not exist.

Drained Shear Strength of Sand

Many theories and empirical or semi-empirical correlations for the interpretation of drained shear strength of sand from cone resistance have been published. The theories can be divided into two categories; namely those based on bearing capacity theory (Janbu and Senneset, 1974, Durgunoglu and Mitchell, 1975) and those based on cavity expansion theory (Vesic, 1972).

Work by Vesic (1963) has shown that no unique relationship exists between friction angle for sands and cone resistance, since soil compressibility influences the cone resistance. The curvature of the Mohr-Coulomb failure envelope for granular soils has been observed repeatedly by numerous investigators and is présently recognized as a typical material behaviour. Most of the available bearing capacity theories on deep penetration neglect both the curvature of the shear strength envelope and the compressibility of the soil. The increasing influence of these two factors tend to reduce the tip resistance.

Based on cavity expansion concepts, Vesic (1972) developed a theory for tip resistance taking account of soil compressibility and volume change characteristics. Baligh (1976) developed this further to incorporate the curvature of the strength envelope. The comprehensive calibration chamber test work by Baldi et al. (1981) showed that the cavity expansion theory appeared to model the measured response extremely well. The cavity expansion theory by Vesic, however, cannot incorporate volume expansion. At first this appears to be a major disadvantage since almost all sands dilate (expand) during shear. Work by Vesic and Clough (1968) showed that at high stresses (greater than 50 bars) dense sand (D_r = 80%) will compress during shear. The stresses immediately in front of the cone tip during penetration into sand often exceeds 200 bars. Thus it seems reasonable that the high stresses developed during cone penetration in sands cause a compressive punching failure beneath the tip. This agrees well with the observed behaviour from model tests of deep penetration (Robinsky and Morrison, 1964; Mikasa and Takada, 1974). The cavity expansion analysis, however, is complex and requires considerable input data regarding compressibility and shear strength. Calibration chamber results illustrate the complex nature of cone penetration in sands and show that simple closed form solutions to derive the shear strength are not possible. In addition, chamber tests provide valuable insight into the relative importance of the various factors that influence cone penetration in sands.

In general, it would be expected that the bearing capacity theories, which cannot take account of soil compressibility, could not provide reliable predictions of friction angle. However, the work by Villet and Mitchell (1981) showed that the bearing capacity theory developed by Durgunoglu and Mitchell (1975) provided reasonable predictions for a variety of different sands. The chamber test study by Baldi et al., (1981) showed that Durgunoglu and Mitchell's theory gave excellent agreement with measured friction angle at a failure stress level approximately equal to the average stress around the cone. The average stress around the cone was assumed to be about 9 times the in-situ horizontal stress (Baldi et al., 1982). Thus, due to the curvature of the strength envelope, the Durgunoglu and Mitchell theory appears to underestimate the friction angle at the in-situ stress level by about 2 degrees.

As discussed earlier, the two main parameters that control penetration resistance in sands are shear strength and compressibility. Work by Al-Awkati (1975) showed that, for the predominantly quartz sands he tested, shear strength had significantly more influence on cone resistance than compressibility. This is probably due to the fact that for most natural quartz sands the variation in compressibility is not that large, especially when compared to the possible variation of shear strength. This observation is particularly important when one considers that a large portion of natural sands encountered in the northern hemisphere consist predominantly of quartz and feldspars with small amounts of mica. These sands are similar to those tested in the calibration chamber studies (see Table 1). Thus it has been possible to use bearing capacity theories, in which the influence of compressibility is neglected, and produce reasonable estimates of friction angle. It is interesting to note that such theories will give conservatively low estimates of friction angle for more compressible sands (i.e. carbonate sands).

A review of the calibration chamber test results was carried out to compare the measured cone penetration resistance to measured friction angle from drained triaxial tests. The friction angle values were obtained from triaxial tests performed at confining stresses approximately equal to the horizontal effective stress in the calibration chamber before cone penetration (i.e., in-situ σ'_{ho}). The results of the comparison are shown on Fig. 6. Details of the sands used in the studies are given in Table 1. The scatter in the results illustrate the limited influence of soil compressibility. Also shown in Fig. 6 are the theoretical relationships proposed by Janbu and Senneset (1974) and Durgunoglu and Mitchell (1975). The Durgunoglu and Mitchell method includes the effect of in-situ horizontal stresses. The difference between the normally consolidated state, where $K_o = 1-\sin\phi$, and the overconsolidated state (OCR ≈ 6), where $K_o = 1.0$, is less than 2 degrees, as shown on Fig. 6.

Since the solution by Janbu and Senneset (1974), for $\beta = 0$, (see Fig. 6) tends to slightly over-estimate ϕ and Durgunoglu and Mitchell tends to under-estimate ϕ, an average empirical relationship is proposed by the writers, as shown on Fig. 6. If the average relationship is taken, a useful design chart for estimation of friction angle from cone penetration resistance can be developed, as shown in Fig. 7. The proposed chart in Fig. 7 can be expected to provide reasonable estimates of friction angle for normally consolidated, moderately incompressible, predominantly quartz sands, similar to those used in the chamber studies. For highly compressible sands, the chart would tend to predict conservatively low friction angles. Durgunoglu and Mitchell's theory shows that there is little change in predicted friction angle for relatively large changes in stress history. It is important to note that the friction angle predicted from Fig. 7 is related to the in-situ initial horizontal stress level before cone penetration.

It is interesting to note that the friction ratio for sands increases with increasing compressibility. Many compressible carbonate sands have friction ratios as high as 3 percent (Jonstra and de Gijt, 1982) whereas, typical incompressible quartz sands have friction ratios of about 0.5 percent. Thus, the presence of compressible sands may be identified using the friction ratio.

The writers recommend that, for sands where the friction ratio is about 0.5%, the peak friction angle can be estimated using Fig. 7. In overconsolidated sands, Fig. 7 may slightly overestimate the friction angle

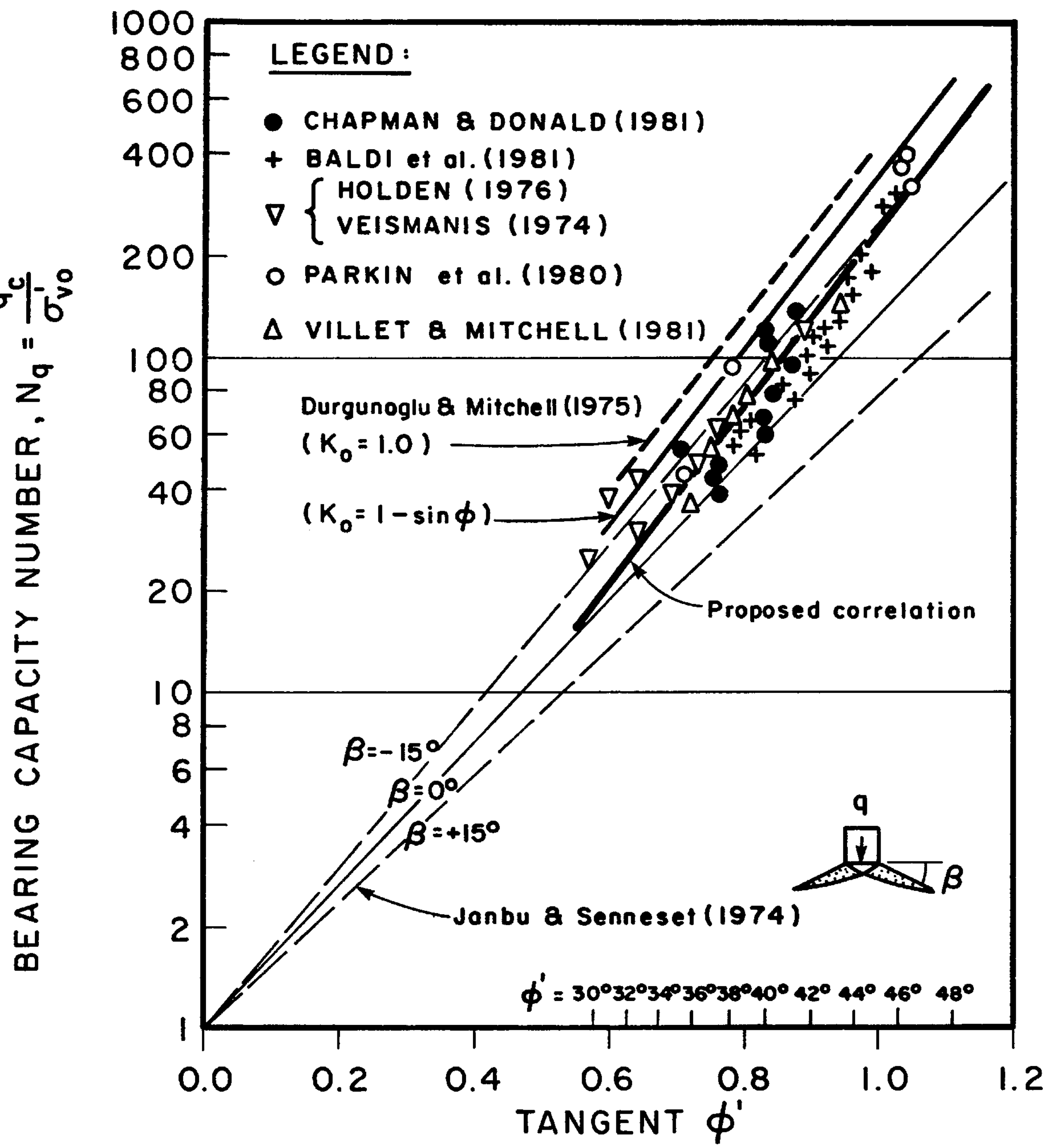


FIG.6: RELATIONSHIP BETWEEN BEARING CAPACITY NUMBER AND FRICTION ANGLE FROM LARGE CALIBRATION CHAMBER TESTS.

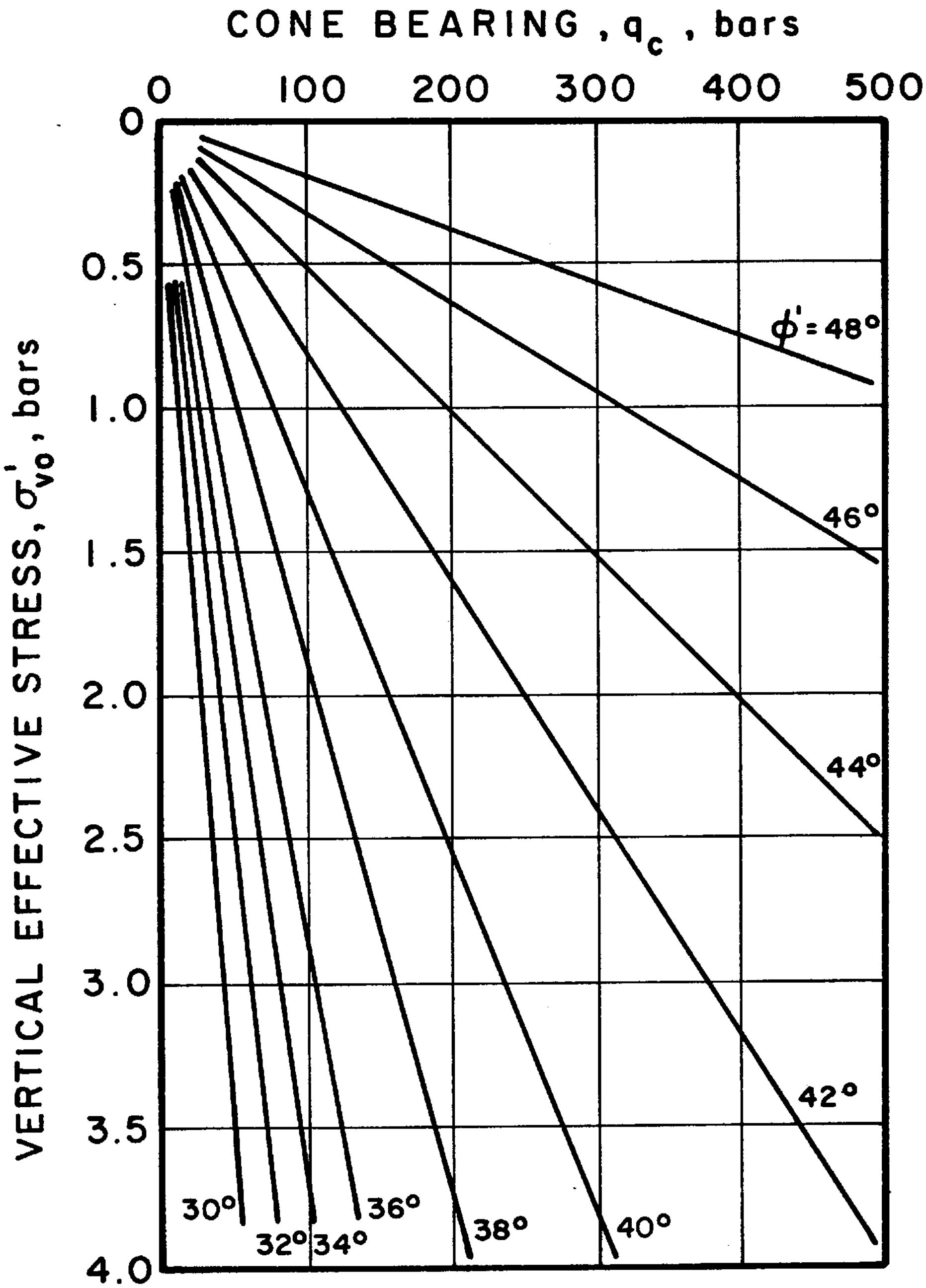


FIG.7: PROPOSED CORRELATION BETWEEN CONE BEARING AND FRICTION ANGLE FOR UNCEMENTED, QUARTZ SANDS.

(see Fig. 6). Care should be exercised in interbedded deposits where the cone resistance may not have reached the full value within a layer.

Deformation Characteristics of Sand

Constrained Modulus - As already discussed, the cone penetration resistance in sand is a complex function of both strength and deformation properties. Hence, no generally applicable analytical solution for cone resistance as a function of deformation modulus is available. Instead, many empirical correlations between cone resistance and deformation modulus have been established. Mitchell and Gardner (1975) made a comprehensive review of the existing correlations for sand. The correlations generally take the form

$$M = \alpha\, q_c \qquad (1)$$

where M is the drained constrained modulus (equal to $1/m_v$ from oedometer tests). The factor α is generally recommended in the range of 1.5 to 4.0.

Considerable confusion appears to exist as to whether or not α should remain constant with depth. Vesic (1970) proposed $\alpha = 2(1+D_r^{\,2})$, where D_r = relative density. Dahlberg (1974) found α to increase with q_c based on M values obtained from screw plate tests for precompressed sand. Other references by Mitchell and Gardner use decreased α values when q_c exceeds a certain limit.

Review of calibration chamber tests (Lunne and Kleven, 1981) are shown in Table 2. Results indicate that $\alpha = 3$ should provide the most conservative estimates of one-dimensional settlement. The choice of α value depends on judgement and experience.

Considerable insight into the relationship between one dimensional deformation modulus and cone resistance can be obtained from a careful review of calibration chamber tests. Baldi et al. (1981) report tangent moduli corresponding to the last load increment for normally consolidated samples, and apply them to the empirical formula proposed by Janbu (1963):

$$M_t = k_m \, Pa \left(\frac{\sigma'_{vo}}{Pa}\right)^n \qquad (2)$$

where M_t = tangent constrained modulus

k_m = modulus number, which varies with relative density

n = modulus exponent, which may be approximately 0.4

σ'_{vo} = vertical effective stress

Pa = reference stress, usually taken as 1 bar or 100 kPa.

The test results by Baldi et al (1981) on Ticino sand show a relationship between the modulus number, k_m and relative density, D_r as follows:

Medium dense,	$D_r = 46\%$	$k_m = 575$
Dense ,	$D_r = 70\%$	$k_m = 753$
Very dense ,	$D_r = 90\%$	$k_m = 815$

Similar values were reported by Parkin and Arnold (1977) and Byrne and Eldridge (1982).

If the correspondence between relative density and modulus number is used in cooperation with the correlation developed by Baldi et al. (1981), shown in Fig. 5, a series of curves relating tangent constrained modulus, M_t, to cone resistance, q_c, for different levels of vertical effective stress, σ'_{vo} can be developed, as shown on Fig. 8.

Reference	N.C. Sand		O.C. Sand	
	No. sands	α	No. sands	α
Veismanis (1974)	2	3 - 11	3	5 - 30
Parkin et al., (1980)	1	3 - 11	1	5 - 30
Chapman & Donald (1981)	1	3 - 4 3 absolute lower limit	1	8 - 15 (12 = average)
Baldi et al., (1982)	1	>3	1	3 - 9

TABLE 2: Summary of Calibration Chamber Results for Constrained Modulus Factor α. (After Lunne and Kleven, 1981)

Review of Fig. 8 illustrates the apparent reason for the wide range in α values reported in Table 2.

Some of the confusion concerning the use of CPT for interpretation of deformation modulus can be overcome if the following points are considered.

a) Soil is not linear elastic and modulus varies with both stress and strain level.

b) Modulus is often derived from or applied to non one-dimensional loading conditions.

c) Different theoretical methods were applied when obtaining correlations.

The simple fact that soil is not a linear elastic material makes the assumption of a constant modulus unrealistic. This is further complicated by the fact that many of the correlations where derived from non one-dimensional loading conditions for which "elastic" solutions were applied to back-figure a modulus. Thus, reasonable agreement can be expected only if the required problem involves similar boundary conditions and the same theoretical method is reapplied. Schmertmann's (1970) CPT method for predicting settlements in sand under spread foundations is a typical example. Schmertmann applied his strain influence elastic theory to analyse the results of screw plate tests. An equivalent Young's modulus (E_s) was calculated using a secant slope over the 1 bar - 3 bars (1 tsf - 3 tsf) increment of plate loading. This interval was chosen principally because real footing pressures commonly fall within this interval. Thus, Schmertmann's design method, where $E_s = 2\ q_c$ can be expected to produce good results if the proposed design problem has similar loading conditions to the screw plate (i.e. circular spread footing loaded from 1-3 bars) and

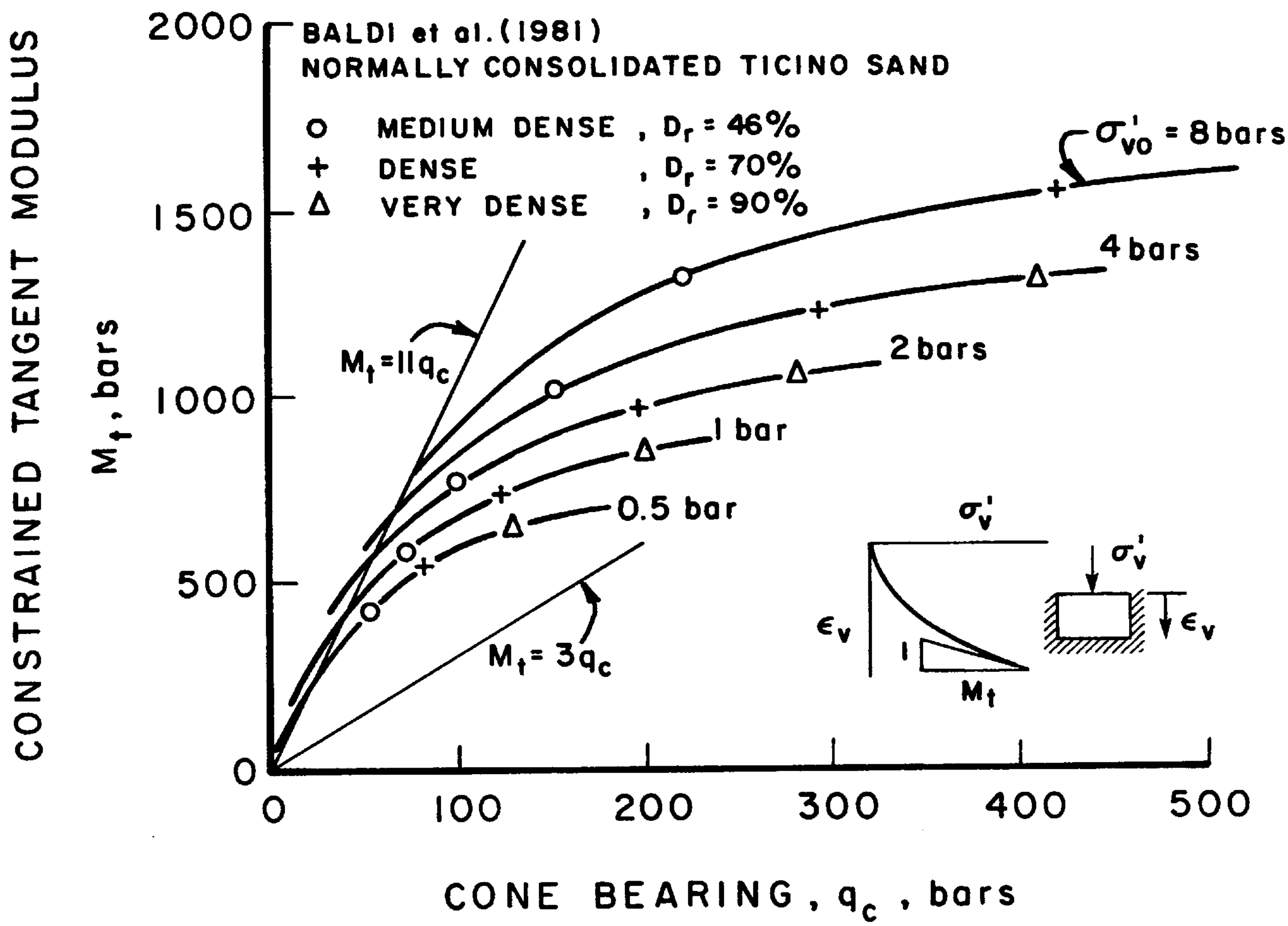


FIG.8: RELATIONSHIP BETWEEN CONE BEARING AND CONSTRAINED MODULUS FOR NORMALLY CONSOLIDATED , UNCEMENTED , QUARTZ SANDS. (Based on Data from Baldi et al.1981).

the same strain influence theory is reapplied.

Young's Modulus - A common problem, however, appears to be the use of the one-dimensional constrained modulus (M_t) applied to non one-dimensional loading conditions. For non one-dimensional cases an equivalent Young's modulus, as suggested by Schmertmann (1970), would appear to be a more logical parameter. A review, performed by the writers, of the calibration chamber results (Baldi et al. 1981) provides a relationship between the drained secant Young's modulus at the 50 and 25 percent failure stresses, E_{50} and E_{25}, respectively, and cone resistance, q_c, for different levels of vertical effective stress (Fig. 9). Since the overall safety factor against bearing capacity failure is usually around 4 for foundations on sand, the designer is usually interested in a Young's modulus for an average mobilized stress level around 25 percent of the failure stress. Thus, the calibration chamber results on normally consolidated sand give values of E_{25}/q_c varying between 1.5 and 3.0 which are in good agreement with the recommended value of 2 by Schmertmann (1970) for computation of settlements of shallow foundations on sand. Schmertmann (1978a) has changed the value to 2.5 and 3.0 to allow for the variation of shape factors for square and strip footings, respectively. A careful review of Fig. 9 shows that in Schmertmann's study the load increment of 1 to 3 bars was probably closer to the 50 percent failure stress level for loose to medium dense sands and closer to the 25 percent stress level for medium dense to dense sands. For very dense sands the load increment (1 to 3 bars) was only a small percentage of the failure stress.

Results from chamber tests suggest the ratio of E_{25}/q_c for overconsolidated sands is in the range of 3 to 6 times larger than those for

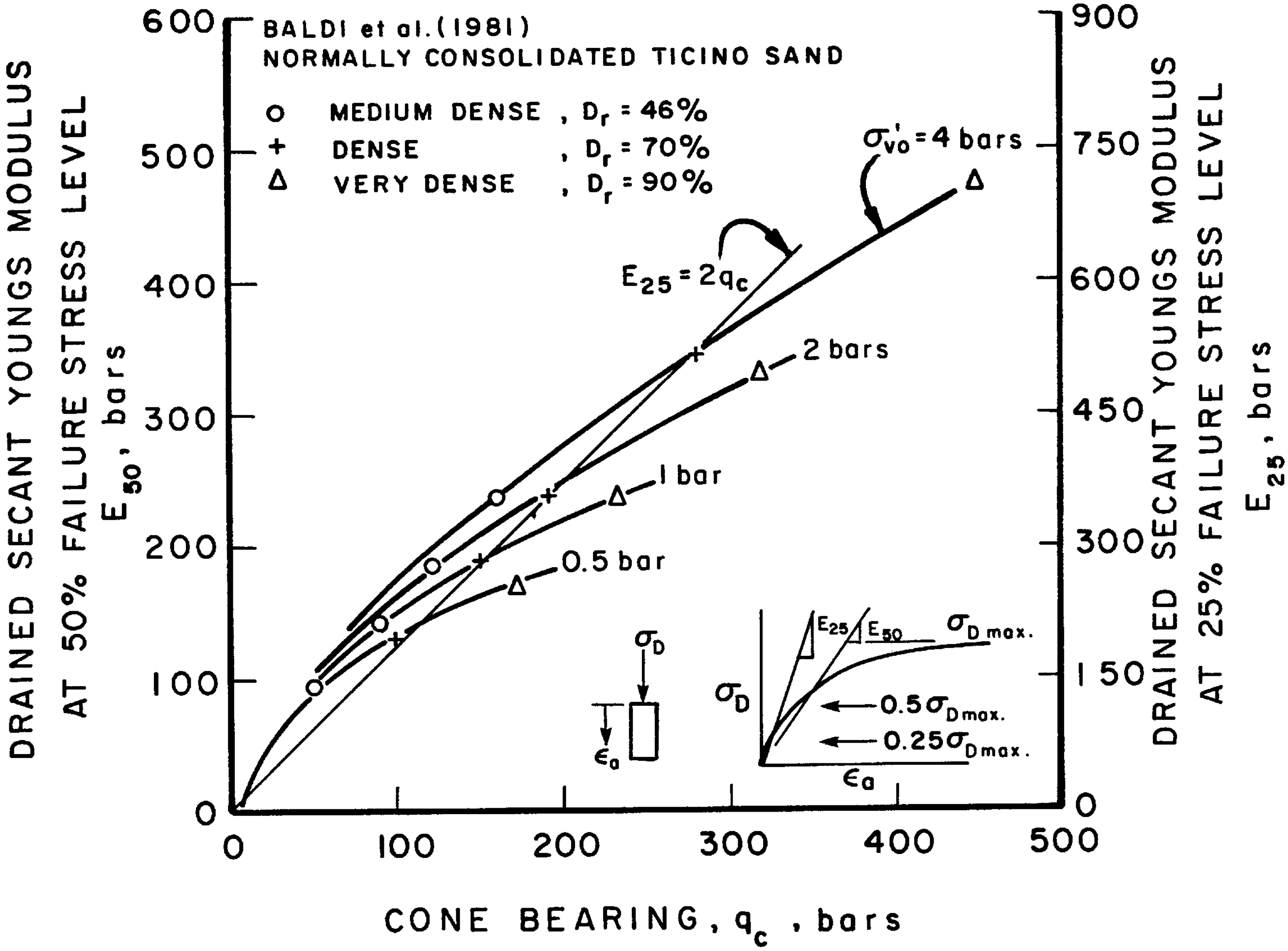


FIG.9: RELATIONSHIP BETWEEN CONE BEARING AND DRAINED YOUNGS MODULUS FOR NORMALLY CONSOLIDATED,UNCEMENTED,QUARTZ SANDS. (Based on Data from Baldi et al.1981).

normally consolidated sands (i.e. $6 < \alpha < 18$). However, the application of these larger factors to overconsolidated sands should be used with caution, since the increase is dependent on degree of overconsolidation and density (Baldi et al., 1982).

The use of Fig. 9 may underestimate the in-situ Young's Modulus because it is based on laboratory measured moduli using re-constituted samples. Many in-situ sand deposits have had some past stress history that can cause a significant increase in soil stiffness.

Shear Modulus - A similar approach can be applied to develop a correlation between cone resistance and shear modulus, G, for sands. Extensive laboratory work has been conducted by several researchers (Seed and Idriss, 1970, Handin and Drnevich, 1972) to relate dynamic shear modulus, G_{max} to soil index properties. When expressed in the form,

$$G_{max} = k_G \, Pa \left(\frac{\sigma'_m}{Pa}\right)^{0.5} \tag{3}$$

where k_G = modulus number

σ'_m = mean effective stress

Pa = reference stress (i.e. Pa = 1 bar)

the empirical equations can be compared, as shown on Fig. 10. If the proposed relationship for k_G shown in Fig. 10 is combined with the relative density, cone resistance relationship developed by Baldi et al. (1981) a series of curves relating G_{max} to q_c can be developed. This has been done by the writers and is shown on Fig. 11.

Once a correlation has been developed for the dynamic shear modulus it

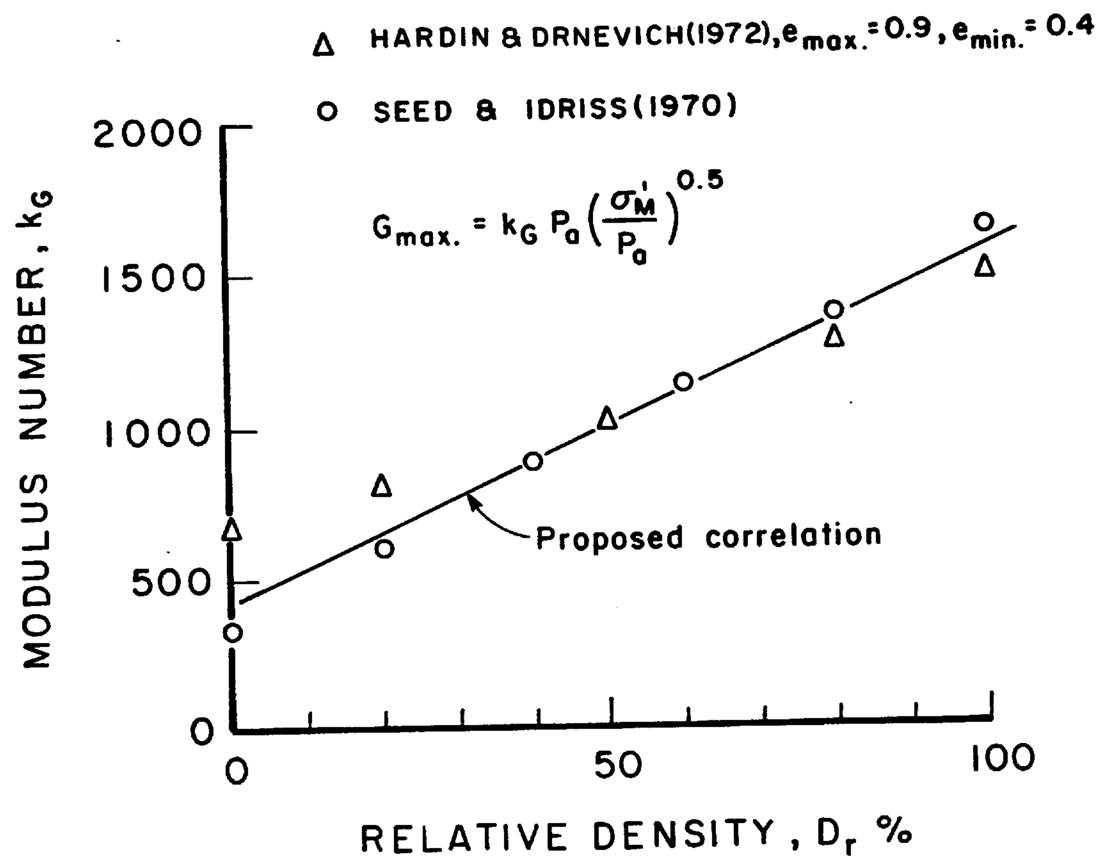


FIG.10: CORRELATION BETWEEN DYNAMIC SHEAR MODULUS NUMBER AND RELATIVE DENSITY.

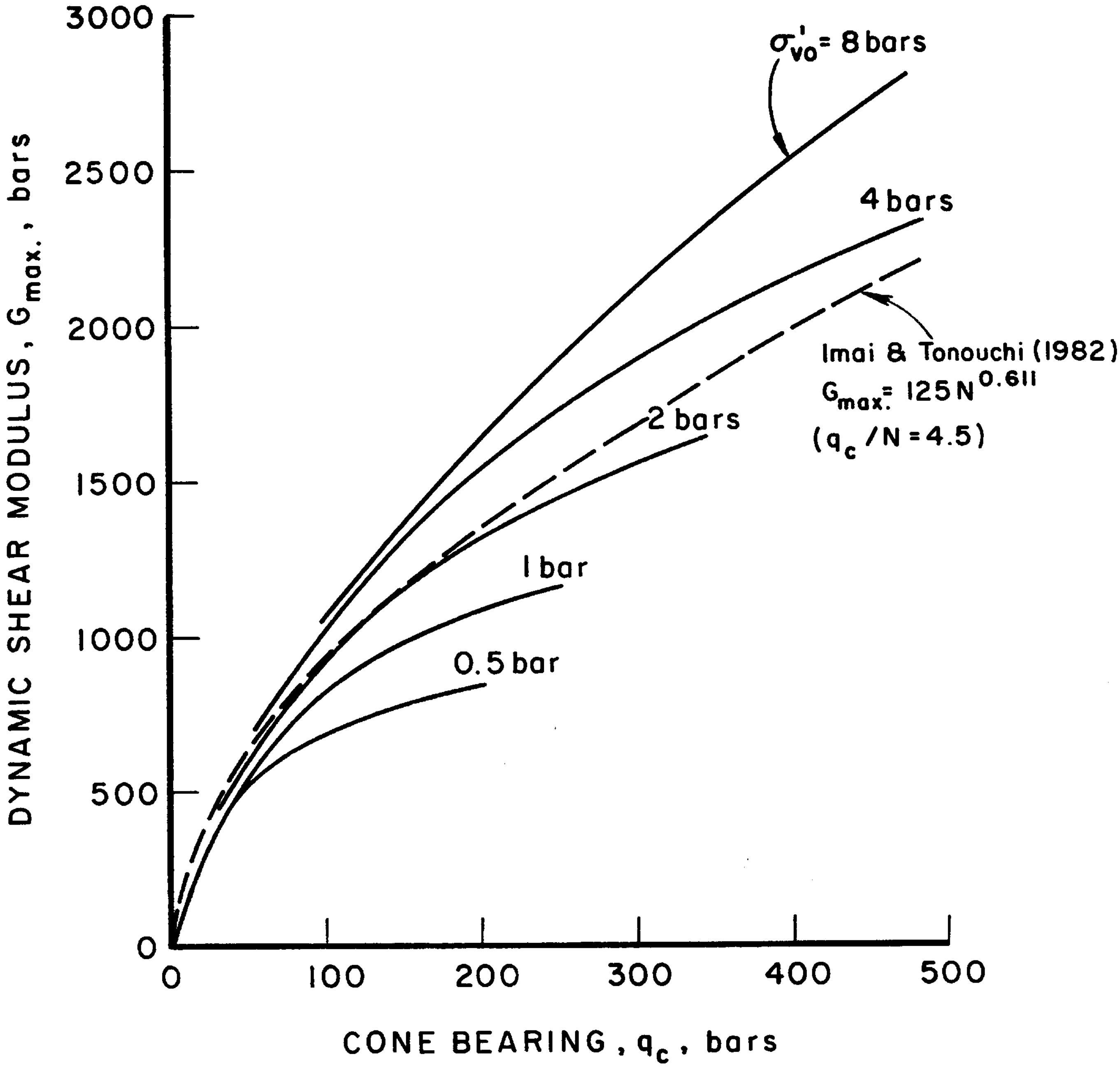


FIG. 11: RELATIONSHIP BETWEEN CONE BEARING AND DYNAMIC SHEAR MODULUS FOR NORMALLY CONSOLIDATED, UNCEMENTED, QUARTZ SANDS.

should be possible to estimate the shear modulus at any strain level by using the reduction curves suggested by Seed and Idriss (1970). Byrne and Eldridge (1982) suggest that the initial tangent modulus under static loading conditions is about 1/5 the dynamic modulus. This is because of the combined effect of strain level and repeated loading associated with the resonant column tests to obtain G_{max}.

Also shown on Fig. 11 is the relationship developed in Japan (Imai and Tononchi, 1982) between dynamic shear modulus and SPT N value for sands. The SPT N value has been converted to cone bearing, q_c, using the relationship for sands (Robertson et al., 1982),

$$\frac{q_c}{N} = 4.5 .$$

The writers suggest the use of Figs. 8, 9 and 11 for estimating M, E and G from CPT data in sands. However, it should be recognised that the correlations are applicable to normally consolidated, moderately incompressible, predominantly quartz sands. The compressibility of the sands can be expected to significantly effect any correlation between cone bearing and modulus. The correlations presented may significantly underestimate the moduli for overconsolidated sands. The use of Fig. 9 may underestimate the in-situ Young's Modulus because the correlation is based on laboratory measured moduli.

Stress History

Unfortunately, it is not possible to distinguish the stress history from cone penetration data during drained penetration. Sometimes, an indication of high horizontal stresses, i.e. high OCR, can be obtained from the relative density correlation (Fig. 5). If Fig. 5 is used with the vertical effective stress, σ'_{vo}, it is possible to predict relative densities in excess of 100% (D_r >> 100%). This, is usually a sign of high horizontal stresses or cementation.

Sometimes the presence of high horizontal stresses can produce high friction sleeve values, f_s. However, to quantify the stress level, it is necessary to know the friction sleeve value of the same sand under normally consolidated conditions.

A discussion of how the piezometer cone can be used to estimate stress history is given in the companion paper, Part II. Unfortunately, in sandy soils the pore pressures tend to dissipate almost as fast as they are generated resulting in a measure of the in-situ equilibrium water pressure.

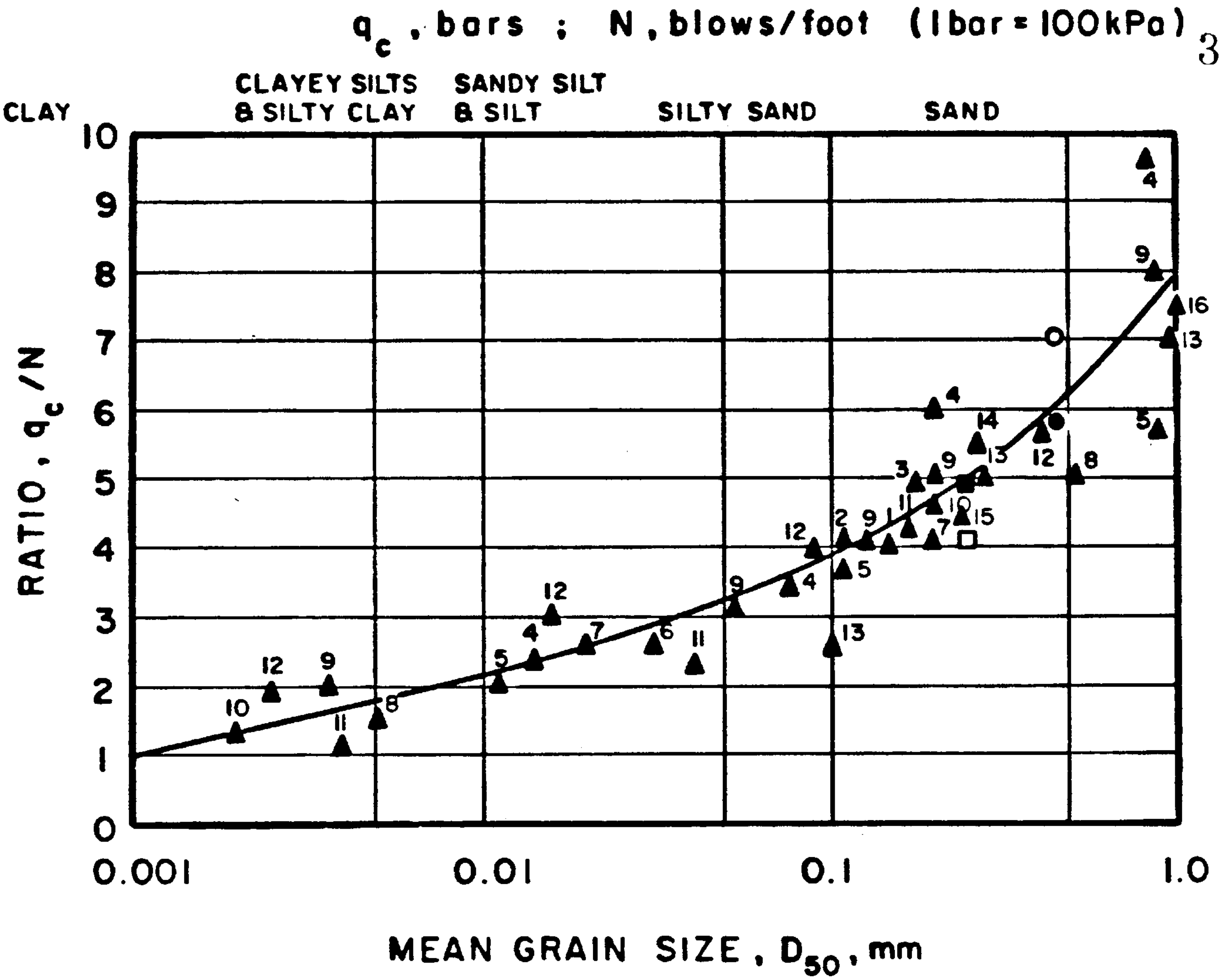
SPT-CPT CORRELATIONS

The Standard Penetration Test (SPT) is still the most commonly used in-situ test in North America. However, despite continued efforts to standardize the SPT procedure there are still continued problems associated with its repeatability and reliability. Many geotechnical engineers have developed considerable experience with design methods based on local SPT correlations. With time, direct CPT design correlations will also be developed based on local experience and field observation. However, with the initial introduction of CPT data there is a need for better SPT-CPT correlations so that CPT data can be used in existing SPT data based design correlations.

A considerable number of studies have taken place over the years to quantify the relationship between SPT N value and CPT cone bearing resistance, q_c. A wide range of q_c/N ratios have been published leading to much confusion. The variations in published q_c/N ratio can be clarified by reviewing the derived q_c/N ratios, as a function of mean grain size (D_{50}), as shown in Fig. 12. It is clear from Fig. 12 that the q_c/N ratio increases with increasing grain size. The scatter in results appears to increase with increasing grain size. This is not surprising since penetration in gravelly sand ($D_{50} \simeq 1.0$ mm) is significantly influenced by the larger gravel sized particles, not to mention the variability of delivered energy in the SPT data. Also sand deposits in general are usually stratified or non-homogeneous causing rapid variations in CPT tip resistance. There is also some difficulty in defining the D_{50} from some of the references.

A recent publication (Robertson et al., 1982) has discussed how the q_c/N ratio varies with the amount of energy delivered to the drill rods. Kovacs et al. (1981) and Robertson et al. (1982) have shown that the energy delivered to the rods during a SPT can vary from about 20% to 90% of the theoretical maximum, 475 J (4,200 in.lb.). The energy delivered to the drill stem varies with the number of turns of rope around the cathead and varies with the fall height, drill rig type, hammer and anvil type, and operator characteristics.

When using the rope and cathead procedure with two turns of the rope, the typical energy delivered from a standard donut type hammer is about 50% to 60% of the theoretical maximum (Kovacs and Salomone, 1982). Schmertmann (1976) has suggested that based on limited data, an efficiency of about 55% may be the norm for which it can be assumed that many North American SPT correlations were developed. Most of the data presented in Fig. 12 was



1. Meyerhof (1956)
2. Meigh and Nixon (1961)
3. Rodin (1961)
4. De Alencar Velloso (1959)
5. Schmertmann (1970)
6. Sutherland (1974)
7. Thornburn & MacVicar (1974)
8. Campanella et al. (1979)
9. Nixon (1982)
10. Kruizinga (1982)
11. Douglas (1982)
12. Muromachi & Kobayashi (1982)
13. Goel (1982)
14. Ishihara & Koga (1981)
15. Laing (1983)
16. Mitchell (1983)

TILBURY ISLAND SITE
- □ SPT N, ER_i = 47%
- ■ SPT N_c, ER_i = 55%

UBC SITE, McDonalds Farm
- ○ SPT N, $ER_i \approx$ 65%
- ● SPT N_c, ER_i = 55%

FIG.12: Variation of q_c/N Ratio with Mean Grain Size. (After Robertson, Campanella & Wightman, 1982).

obtained using the standard donut type hammer with a rope and cathead system.

Robertson et al. (1982) presented energy measurements on SPT data that indicate that the average energy ratio of 55% may represent the average energy level associated with the q_c/N correlation shown in Fig. 12.

Fig. 12 can therefore be used to convert CPT data to equivalent SPT N values. To estimate the mean grain size from CPT data use can be made of the classification chart shown in Fig. 2. The classification chart in Fig. 2 should be used as a guide to grain size. The addition of pore pressure measurements during cone penetration would significantly improve the soil classification (see Part II). For mechanical cone data use can be made of classification charts by Schmertmann (1978a), Searle (1979) or Muromachi and Atsuta (1980).

If local design correlations have been developed based on SPT data obtained using alternative procedures such as a trip hammer or procedures other than the rope and cathead technique, the q_c/N ratios shown in Fig. 12 may be slightly in error. If a trip hammer was used it is likely that the energy level would be higher than the average 55% level by a factor of about 1.4 (Douglas, 1982). Thus q_c/N ratios would be slightly higher than those shown in Fig. 12.

SUMMARY AND CONCLUSIONS

It is important to realize that the correlations presented here for interpretation of CPT data are empirical. An attempt has been made to discuss the factors that influence the interpretation and to provide guidelines for the practicing engineer.

The CPT is a fast, economic logging test that can provide continuous measurements of parameters like bearing, friction and dynamic pore pressure. These parameters can be used to provide a preliminary estimate of geotechnical parameters based on the correlations presented here. The continuous soundings enable detailed site profiles to be developed and critical areas or zones to be identified. These critical areas may then require further testing with specific test methods that may include sampling and laboratory testing.

In interbedded deposits, it is important to remember that the cone resistance may not obtain its full value within a layer if the layer has a thickness less than about 70 cm.

The main correlations presented in Part I of the paper are applicable to cone penetration test data in drained soil. The continuous measurement of pore pressure during cone penetration can significantly aid in the assessment of the applicability of these correlations to different soil types.

The correlations for density and moduli are approximate and should be used as a guide. The density correlation can be improved significantly if the compressibility of the sand can be assessed from the grain characteristics. It should be possible in the near future to quantify the grain characteristics by performing simple compressibility tests on disturbed bulk samples and comparing the results with similar tests on the chamber test sands.

The moduli correlations are likewise influenced by variations in grain characteristics because the parameter is a small strain measurement. The cone resistance, on the other hand, is a large strain measurement. Thus, the peak friction angle correlation appears less influenced by grain characteristics because it also is a large strain parameter. Therefore, when interpreting CPT data in sands more reliance can be placed on the estimated friction angle than density or moduli.

The equivalent SPT N value can be estimated from CPT data using Figs. 2 and 12. If local design correlations have been developed based on SPT data obtained using alternate procedures with resulting different average energy levels (i.e. $ER_i \neq 55\%$), Fig. 12 should be adjusted to reflect local practice and experience.

ACKNOWLEDGEMENTS

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INTERPRETATION OF CONE PENETRATION TESTS - PART II (CLAY)

P.K. Robertson and R.G. Campanella

ABSTRACT

This paper is the second of two parts and presents a summarized work guide for practicing engineers for interpretation of parameters for undrained conditions during the cone penetration test such as, undrained shear strength, overconsolidation ratio and deformation characteristics of clay. The advantages, use and interpretation of the piezometer cone are also discussed. Factors that influence the interpretations are discussed and guidelines provided. The companion paper, Part I (Sand), considers drained conditions during the test and summarizes interpretation of parameters such as relative density, friction angle and deformation characteristics of sand. The authors personal experiences and current recommendations are included.

Key Words: Static cone penetration testing, in-situ, interpretation, shear strength, modulus, stress history, pore pressures, permeability, consolidation.

INTERPRETATION OF CONE PENETRATION TESTS - PART II (CLAY)

P.K. Robertson & R.G. Campanella

INTRODUCTION

Part I (Sand) of this paper deals with the general assessment of soil type and the interpretation of CPT data in drained soil. Part II (Clay) of the paper deals with the interpretation of CPT data in undrained soil and the interpretation of pore pressure data from a piezometer cone. The need to measure and interpret equilibrium pore pressures to evaluate groundwater conditions is also discussed.

UNDRAINED SOIL

Undrained Shear Strength of Clay

One of the earliest applications of the cone penetration test was in the evaluation of undrained shear strength (c_u) of clays (Schmertmann, 1975). Comprehensive reviews of c_u evaluation from CPT data have been presented by Baligh et al. (1980), Jamiolkowski et al. (1982), and Lunne and Kleven (1981). Note that the undrained shear strength of clay is not a unique parameter and depends significantly on the type of test used, the rate of strain and the orientation of the failure planes.

Estimates of c_u from CPT results usually employ an equation of the following form:

$$q_c = c_u N_k + \sigma_o \tag{1}$$

where σ_o is the in-situ overburden total pressure,

N_k is the cone factor.

The contribution of the total overburden pressure (σ_o) has been interpreted as either the in-situ vertical total stress (σ_{vo}), or the in-situ horizontal total stress (σ_{ho}), or the in-situ octahedral stress $\left(\sigma_{oct} = \frac{1}{3}(\sigma_{vo} + 2\sigma_{ho})\right)$. Theoretical solutions for N_k have been based on bearing capacity theories (eg., Meyerhof, 1961) and more recently by use of cavity expansion theories (eg., Landanyi, 1967, and Vesic, 1972). Baligh (1975) combined these two approaches in approximate form to obtain the results shown in Fig. 1. The rigidity index (I_r) is the ratio of the undrained shear modulus to undrained strength and the vertical axis gives N_k similar in form to Eq. 1. Note the use of the in-situ total horizontal stress (σ_{ho}) rather than σ_{vo} and that the theory applies to the standard (Fugro) type electric cone. Fig. 1 clearly demonstrates that N_k should not be a constant for all clays and for the case of the standard cone, where $2\theta = 60°$, $N_k = 16 \pm 2$ over the full range of likely I_r values.

The solution by Baligh (1975) involves several simplifying assumptions, such as neglect of undrained strength anisotropy and strain softening behavior. The former can be adequately approximated by using the average of the vertical and horizontal strengths. Neglecting strain-softening, on the other hand, can lead to a serious error for sensitive clays, Landanyi (1972). Other factors such as cone type and rate of penetration may significantly affect the penetration resistance.

However, N_k is generally obtained from empirical correlations. The reference c_u is usually measured from field vane tests but sometimes half the unconfined compression test is used. The overburden pressure (σ_o) is usually taken as the in-situ total vertical stress (σ_{vo}) since the in-situ horizontal stress is usually not known.

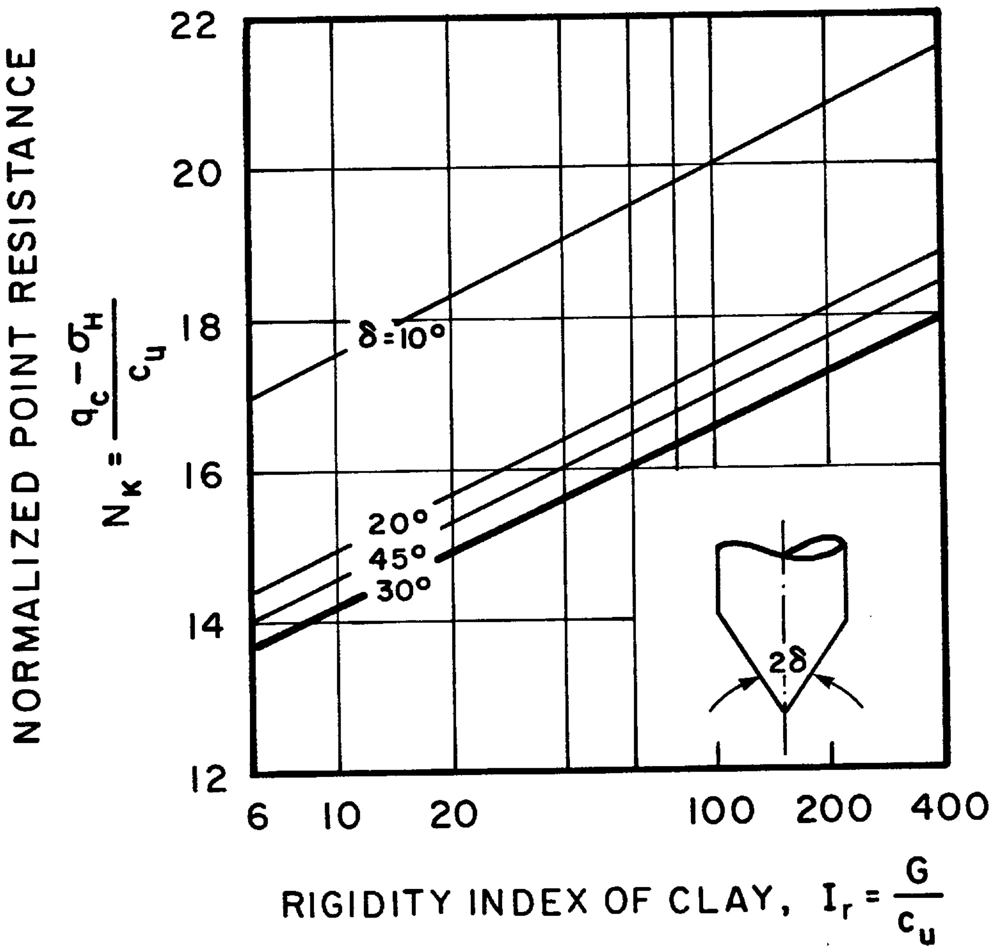


Fig.1: EFFECT OF RIGIDITY INDEX AND CONE ANGLE ON THE PENETRATION RESISTANCE OF CLAY. (After Baligh, 1975).

Data presented by Lunne and Kleven (1981) shows that for normally consolidated marine clays using corrected field vane strength (i.e. Bjerrum's, 1972, correction for plasticity index), the cone factor N_k falls between 11 and 19 with an average of 15. These results were obtained using a standard (Fugro) type electric cone at a standard rate of penetration of 2 cm/sec.

It is more difficult to establish similar correlations in stiff overconsolidated clays because of the effects of fabric and fissures on the response of the clay.

Investigations by Kjekstad et al. (1978) in non-fissured overconsolidated clays, indicate an average cone factor $N_k = 17$. In this case, the reference c_u was obtained by triaxial compression tests. The value of N_k appears to be independent of overconsolidated ratio.

The c_u value determined as a function of cone resistance (q_c) in highly overconsolidated clay deposits must be used with great caution since it is difficult to establish the extent fissures affect drainage and their effect on progressive failure.

Senneset et al. (1982) have recently suggested the use of effective bearing (q_c') to determine c_u from

$$c_u = \frac{q_c'}{N_c'} = \frac{q_c - u_T}{N_c'} \qquad (2)$$

where q_c' is defined as the cone resistance (q_c) minus the total measured dynamic water pressure (u_T). They propose $N_c' = 9$ with a likely variation of ±3. One of the major drawbacks of this method is the accuracy to which q_c' can be determined. In soft normally consolidated clays, the total dynamic water pressure generated on the tip during cone penetration is often as high as 90 percent or more of the measured cone resistance. Thus q'_c is an extremely small quantity. Because of cone designs (i.e. unequal end area effects, Fig. 3, Part I), the measured q_c is sometimes observed to be less than the measured u_T, which is physically impossible and makes the method by Senneset et al., (1982) unusable unless q_c is corrected. Campanella, Robertson and Gillespie (1983) suggest that all measured cone resistance, q_c, should be corrected for measured dynamic pore pressures using the net area ratio to give a true total stress measure, q_T. This point is more thoroughly discussed later in the paper. Research is currently underway to investigate the potential benefits of using effective bearing analysis.

It should also be possible to estimate c_u from the excess pore pressure (Δu) generated during penetration using cavity expansion theories. However, the location of the pore pressure element becomes extremely important. If the pore pressure is measured on the cone tip, the maximum excess pore pressure could be estimated using the spherical cavity expansion theory (Vesic, 1972) and would be in the range,

$$4 < \frac{\Delta u}{c_u} < 7 \quad \text{(spherical cavity).} \tag{3}$$

$$\text{where} \quad \Delta u = u_T - u_o, \quad \text{and} \tag{4}$$

$$u_o = \text{equilibrium water pressure.}$$

If the pore pressure is measured behind the cone tip, the maximum excess pore pressure could be estimated using the cylindrical cavity expansion theory, and would be in the range,

$$3 < \frac{\Delta u}{c_u} < 5 \quad \text{(cylindrical cavity)}. \qquad (5)$$

The range is dependent on the rigidity index ($I_r = \frac{G}{c_u}$) of the soil. The lower values apply to the more highly plastic soils (PI > 80) and the higher values apply to the low plasticity soils (PI ≈ 15). These values are only applicable to normally consolidated non-sensitive soils and tend to slightly overestimate c_u. The excess pore pressure tends to increase with increasing soil sensitivity. The semi-empirical solution proposed by Torstenssen (1977) and Massarch (1978) would enable this approach to be applied to overconsolidated clays, provided an estimate of Skempton's pore pressure parameter (A_f) could be made.

Schmertmann (1975) wisely comments that the best procedure is to make individual correlations for N_k based on c_u measurements for specific clays and CPT procedures. This, of course, requires a reliable estimate of the in-situ c_u appropriate to the particular design problem. Uncertainties involved in this assessment can be reduced somewhat by making reference to values of c_u back-figured from well documented case histories (eg. Bjerrum 1972) instead of using other in-situ tests or laboratory measured values.

The writers suggest using equation (1) with an N_k value of 15 for preliminary assessment of c_u. For sensitive clays, the N_k value should be reduced to around 10 or less depending on the degree of sensitivity. The overburden pressure can be taken as the total vertical stress. With local experience individual correlations for N_k can be developed for specific clays. The writers also recommend that N_k be defined for a specific method of evaluating c_u, such as by field vane corrected for P.I., since c_u is not a unique soil parameter.

Sensitivity

The sensitivity (S_t) of a clay which is the ratio of undisturbed strength to totally remolded strength can be estimated from the friction ratio (FR%) using,

$$S_t = \frac{10}{FR\%} \tag{6}$$

Equation (6), however, provides only a rough estimate of sensitivity of a clay based on the writers' experience. With an increased use of electric cones with equal end area friction sleeves, a new and more reliable estimate of sensitivity may be developed in the near future. Equation (6) implies that the friction measurement from the electric cone is close to the remoulded shear strength of a clay. It is essential, however, to determine through local experience if "10" is the most appropriate parameter to use in Eq. 6 for clays in a specific region.

Drained Shear Strength of Clay

Senneset et al. (1982) have suggested a method to determine the drained effective stress shear strength parameters (c', ϕ'), from the cone penetration resistance and the measured total pore pressures. However, their method, as with any method for determining effective stress parameters from undrained cone penetration data, can be subject to serious problems. Any method of analyses must make assumptions as to the distribution of total stresses and pore pressures around the cone.

Unfortunately, the distribution of stresses and pore pressures around a cone is extremely complex in all soils and has not adequately been modelled or measured to date except perhaps in soft normally consolidated clays. Also, an important problem, which is not identified by Senneset et al., (1982) is the location of the porous element, since different locations give different measured total pore pressures.

The authors feel that the present state of interpretation and analysis of CPT data has not yet reached a stage to allow reliable estimates of drained shear strength parameters from undrained cone penetration data. It should be pointed out, however, that the method by Senneset et al. (1982), does appear to give realistic effective stress strength values from CPT data in soft, normally consolidated clays.

A detailed discussion about limitations of the theories relating to interpretation of CPT data in clays is given by Tavenas et al. (1982). As mentioned previously, the subject of applying an effective stress interpretation to CPT data in undrained soil is a high priority for intense study by many researchers. The writers feel that it is this area where major advances should be made in the next decade.

Overconsolidation Ratio

An estimate of overconsolidation ratio and maximum past pressure may be obtained using the following method suggested by Schmertmann (1978a) and modified slightly by the writers:

i) estimate c_u from q_c;

ii) estimate vertical effective stress, σ'_{vo} from soil profile;

iii) compute c_u/σ'_{vo};

iv) estimate the average normally consolidated $(c_u/\sigma'_{vo})_{NC}$ for the soil using Fig. 2b. A knowledge of the plasticity index is required.

v) estimate OCR from correlations by Ladd and Foott (1974) and normalized by Schmertmann and reproduced in Fig. 2a.

If the plasticity index of the deposit is not available, Schmertmann (1978a) suggests assuming an average normally consolidated $(c_u/\sigma'_{vo})_{NC}$ ratio of 0.33 for most post-pleistocene clays.

It should also be noted that the shape of the tip resistance profile can also provide an approximate indication of a clays stress history. For normally consolidated clay deposits with hydrostatic groundwater conditions, the tip resistance is linearly increasing with depth. For most young clays where overconsolidation has been caused by erosion or desication, the OCR will decrease with depth until the deposit, at depth, is approximately normally consolidated. In these cases, the tip resistance profile will be approximately constant or even decrease with depth until the depth where the deposit is normally consolidated and will then increase linearly with depth. For aged clays where the OCR is constant with depth, the tip resistance may continue to stay approximately constant with depth.

DEFORMATION CHARACTERISTICS OF CLAY

Constrained Modulus and Compression Index - Mitchell and Gardner (1975) made a comprehensive review of the numerous correlations between cone resistance and constrained modulus, M. Most of these take the general form

$$M = \frac{1}{m_v} = \alpha\, q_c \tag{7}$$

where m_v = volumetric compressibility = $(\Delta v/v/\Delta p)$.

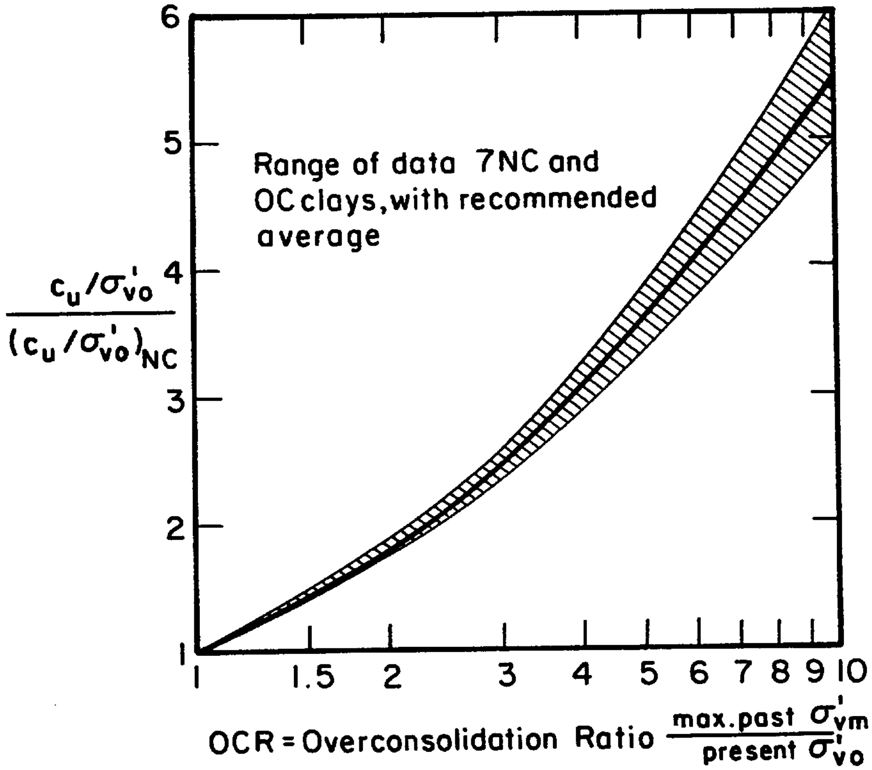


FIG.2a: NORMALIZED c_u/σ'_{vo} RATIO VS. OCR FOR USE IN ESTIMATING OCR. (After Schmertmann 1978a).

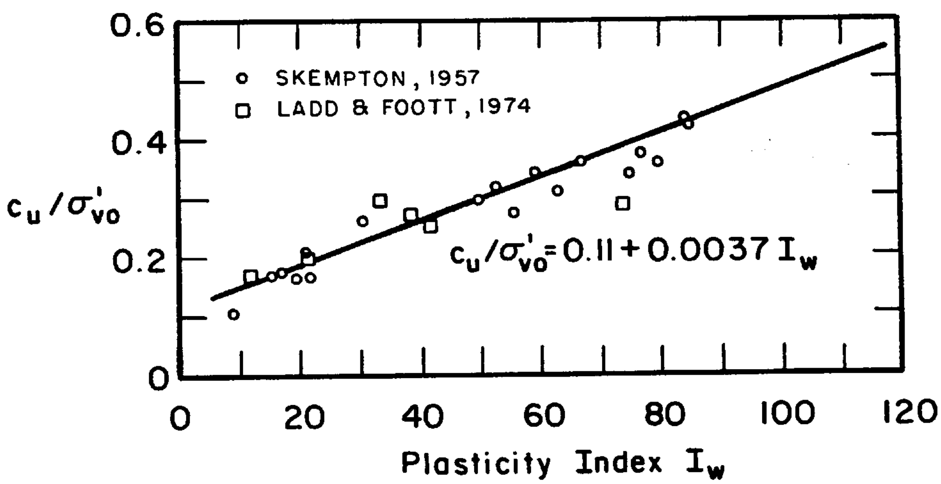


FIG.2b: STATISTICAL RELATION BETWEEN c_u/σ'_{vo} RATIO AND PLASTICITY INDEX, FOR NORMALLY CONSOLIDATED CLAYS.

Sanglerat et al. (1972) developed a comprehensive array of α values for different cohesive soil types with different cone resistance values. Mitchell and Gardner's (1975) summary of Sanglerat's α values are given in Table 1. Schmertmann developed a slightly more logical method that related the c_u/σ'_{vo} ratio to the overconsolidation ratio (OCR) and then to the one dimensional compression index of the soil, C_c, as shown on Table 2.

The volumetric compressibility (m_v) and the compression index (C_c) are related by:

$$m_v = \frac{0.435\ C_c}{(1+e_o)\sigma_{va}} \tag{8}$$

where e_o = initial void ratio,

σ_{va} = average of initial and final stresses.

These methods provide only a rough estimate of soil compressibility. The values by Schmertmann in Table 2 appear to give very conservative estimates of C_c. Additional data from Atterberg limit tests (PI) or undisturbed sampling and oedometer tests are required for more reliable estimates.

The estimation of drained parameters such as the one dimensional compression index, C_c, or compressibility, m_v, from an undrained test is liable to serious error, especially when based on general empirical correlations. Conceptually, total stress undrained measurements from a cone cannot yield parameters for drained conditions without the addition of pore pressure measurements. The predictions of volume change based on q_c

TABLE 1. Estimation of Constrained Modulus, M. (After Mitchell and Gardner, 1975)

$$M = \frac{1}{m_v} = \alpha\, q_c$$

$q_c < 7$ bars	$3 < \alpha < 8$	Clay of low plasticity (CL)
$7 < q_c < 20$ bars	$2 < \alpha < 5$	
$q_c > 20$ bars	$1 < \alpha < 2.5$	
$q_c > 20$ bars	$3 < \alpha < 6$	Silts of low plasticity (ML)
$q_c < 20$ bars	$1 < \alpha < 3$	
$q_c < 20$ bars	$2 < \alpha < 6$	Highly plastic silts & clays (MH, CH)
$q_c < 12$ bars	$2 < \alpha < 8$	Organic silts (OL)
$q_c < 7$ bars:		
$50 < w < 100$	$1.5 < \alpha < 4$	Peat and organic clay (P_t, OH)
$100 < w < 200$	$1 < \alpha < 1.5$	
$w > 200$	$0.4 < \alpha < 1$	

c_u/σ'_{vo}	approx. OCR	$C_c/(1 + e_1)$
0 - 0.1	less than 1	greater than 0.4 (still consolidating)
0.1 - 0.25	1	0.4
0.26 - 0.50	1 to 1.5 (assume 1)	0.3
0.51 - 1.00	3	0.15
1 - 4	6	0.10
over 4	greater than 6	0.05

TABLE 2: Estimation of Compression Index, C_c, from c_u/σ'_{vo} ratio (After Schmertmann, 1978a).

using either Table 1 or Table 2 may be in error by ±100%. However, with local experience individual correlations can be developed for specific soil types.

Young's Modulus - The estimation of undrained Young's modulus, E_u, is usually made using empirical correlations with the undrained shear strength, c_u, in the form

$$E_u = n\, c_u \qquad (9)$$

where n is a constant that depends on stress level, overconsolidation ratio, clay sensitivity and other factors (Ladd et al. 1977). As discussed earlier, because soil behaviour is non-linear, the choice of relevant stress level is very important. Fig. 3(a) presents data for normally consolidated soils from Ladd et al. (1977) that shows the variation of the ratio E_u/c_u with stress level for seven different cohesive soils, $(15 < PI < 75)$. Fig. 3(b), shows the variation of E_u/c_u with overconsolidation ratio (OCR) at two stress levels for the same soil types shown in Fig. 3(a).

The writers' recommended procedure for the estimation of the undrained Young's modulus (E_u) is to first estimate the undrained shear strength (c_u) from CPT profiles, as previously discussed, then estimate the stress history (OCR) using the ratio, c_u/σ'_{vo} (Fig. 2). Then, using Fig. 3, estimate E_u for the relevent stress level appropriate for the particular problem. A knowledge of the plasticity index (PI) would significantly improve the estimate.

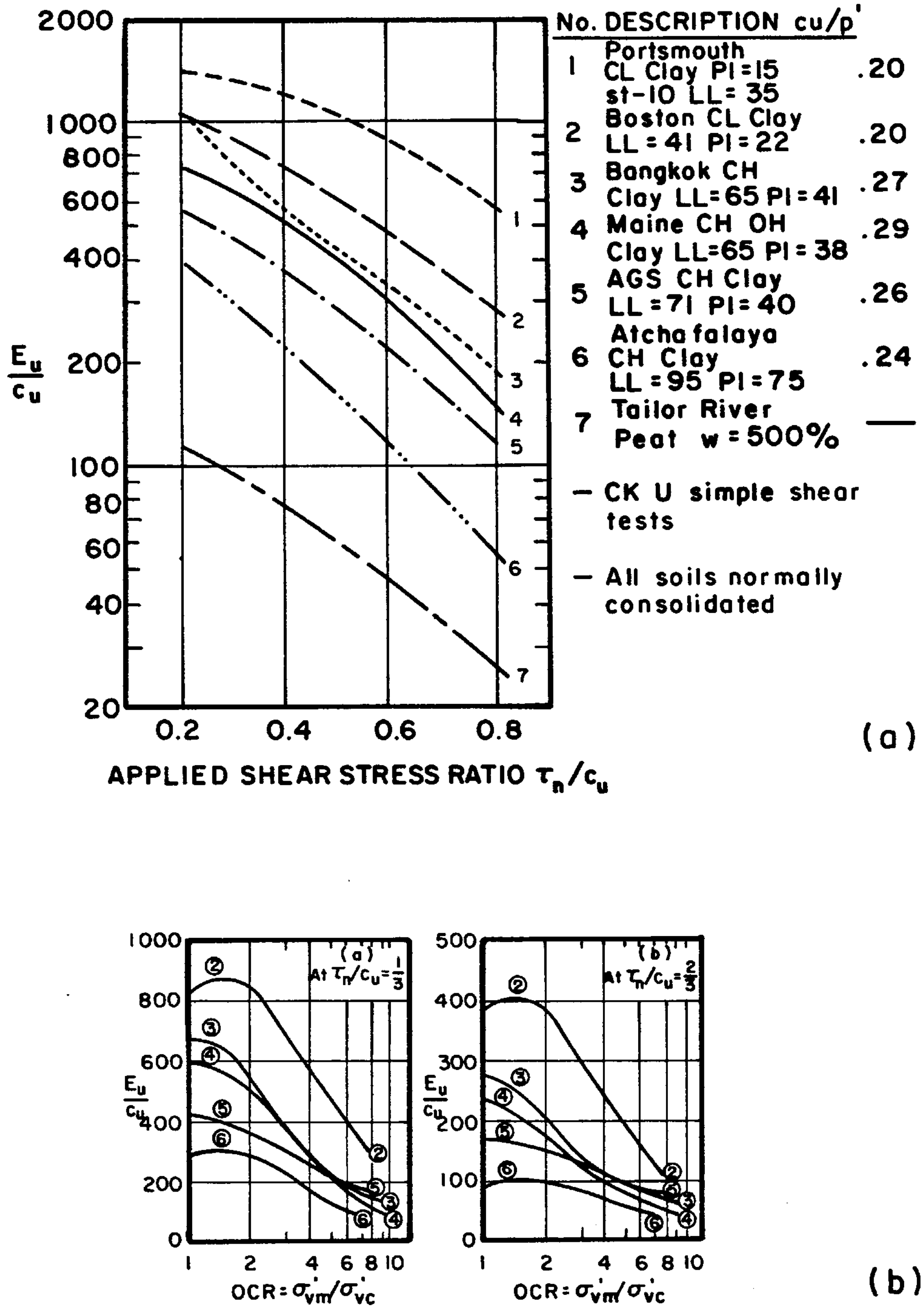


FIG. 3: SELECTION OF SOIL STIFFNESS RATIO FOR CLAYS.

(Adapted from Ladd et al. 1977).

PIEZOMETER CONE

The addition of pore pressure measurements during static cone penetration testing has added a new dimension to the interpretation of geotechnical parameters. The continuous measurement of pore pressures along with bearing and friction has enhanced the electric penetrometer as the premier tool for stratigraphic logging of soil deposits.

The excess pore pressure (Δu) measured during penetration is a useful indication of the soil type and provides an excellent means for detecting details in stratigraphy. The differential pore pressure ratio ($\Delta u/q_c$) also appears to be a good index of soil type and relative consistency and a rough indicator of stress-history. In addition, when the steady penetration is stopped, the excess pore pressure decay with time can be used as an indicator of the coefficient of consolidation. Finally the equilibrium pore pressure value (u_o), after complete dissipation is reached, provides important data on the ground water conditions.

These points will be discussed in more detail in the following sections.

Pore Pressure Measurements

During cone penetration, soils tend to generate pore pressures. For sandy soils these pore pressures dissipate almost as fast as they are generated and the high cone resistance (q_c) in sands generally gives differential pore pressure ratios ($\Delta u/q_c$) of essentially zero. Silty and clayey soils, because of their relatively low permeability, can generate significant excess pore pressures during cone penetration.

The volume change characteristics are a direct measure of a soils stress history. Normally consolidated silts and clays tend to develop large positive pore pressures during shear, whereas, overconsolidated silts and clays tend to develop smaller positive or even negative pore pressures during shear. Therefore, if the permeability of a soil deposit is relatively low such that drainage during cone penetration is small, the excess pore pressure (Δu) may be a direct measure of the soil deposit stress history. Thus, the excess pore pressure can give an excellent indication of both the volume change characteristics and relative permeability.

This logic applies equally well to sandy soils in that loose sands tend to generate positive pore pressures and dense sands negative pore pressures during shear. However, because of their relatively high permeability these pore pressures often dissipate as fast as they are generated and very little excess pore pressures are recorded.

Use of the pore pressure or differential pore pressure ratio is very dependent on the details of the cone design. The three significant aspects of cone design in relation to pore pressure measurements are:

i) Pore pressure element location,

ii) Unequal end area effects,

iii) Saturation of pore pressure measuring system.

i) Pore pressure element location - Because of the complex variation of stresses and strains around a cone tip, the location of the pore pressure element can significantly affect the measured pore pressure during cone penetration. In normally consolidated clays and silts, where large positive pore pressures are generated during shear, pore pressures measured on the face of the tip are generally about 10-20 percent larger than pore pressures measured immediately behind the tip (Roy et al., 1982, Campanella, Robertson and Gillespie, 1983). In overconsolidated clays and silts, and fine sands, where small positive or negative pore pressures are generated during shear, pore pressures on the face of the tip tend to be positive whereas pore pressures measured immediately behind the tip may be negative (Campanella, Robertson and Gillespie, 1983). This is because the area along the face of the cone tip is in a zone of maximum compression and shear. On the other hand, the area immediately behind the tip is in a zone of total stress relief. Pore pressures are generated in saturated soils because of both increases in normal stresses as well as shear stresses. Thus, the area behind the tip appears to measure a response dominated by the shear behaviour of the soil. Furthermore, because of the stress relief experienced by a soil element as it passes behind the tip, the pore pressure element behind the tip encourages the measurement of low or negative dynamic pore pressures. Thus, with the element located immediately behind the tip the differential pore pressure ratio appears to be a more sensitive measure of stress history since it tends to accentuate the soil behaviour during shear.

ii) Unequal area effects - Since the tip resistance is a total stress element, it should record a bearing stress equal to an all around applied pressure. This is never the case and the tip always records a stress less than an applied all around pressure because of unequal areas A_N and A_q at the tip (see Fig. 3 - Part I). Thus, every cone has a given bearing net area ratio associated with its design and dimensions. A detailed discussion concerning cone design is given by Campanella et al. (1983) and

Schaap and Zuidberg (1982). Most cones have bearing net area ratios of from about 0.6 to 0.8, but a bulbous cone tip like the one shown in Fig. 3 – Part I could easily have a net area ratio of less than 0.5. It is very important when using the pore pressure ratio that the bearing values be corrected to total stress, as follows

$$q_T = q_c + u_T(1-a) \tag{10}$$

where q_T is total stress, q_c is measured bearing, u_T is the total dynamic pore pressure $(u_o+\Delta u)$ and "a" is bearing net area ratio (A_N/A_q). This correction can not be eliminated except with a unitized, jointless design where the sleeve is strain gauged to measure tip load. Such a design is not yet available.

Also, not using total bearing, q_T, may account for some of the reported wide variations of calculated bearing capacity factor, N_k, required to determine undrained shear strength from cone bearing.

If the total tip resistance, q_T, is used, the differential pore pressure ratio, $\Delta u/q_T$, can be expected to relate more uniquely to the stress history of soil deposits. Some researchers prefer to use the ratio, u_T/q_T, which is not recommended, especially for offshore or underwater CPT where u_o values can be large.

iii) <u>Saturation of pore pressure measuring system</u> – It has been previously shown by Campanella and Robertson (1981) that complete saturation of the piezometer tip is essential for fast reliable response. Pore pressure response was compared for saturated and air entrapped piezometer cone systems. Both the maximum pore pressure and dissipation times are significantly effected by air entrapment. Unfortunately, it is not possible to check saturation before penetrating the soil.

Measuring dynamic pore pressures with the piezometer cone requires careful consideration of probe design, choice of the porous element and probe saturation. For a high frequency response, the design must aim at a small fluid filled cavity, low compressibility and viscosity of fluid, a high permeability of the porous filter and a large area to wall thickness ratio of the filter (Smits, 1982). The writers have found glycerin to work effectively as a saturating fluid. Glycerin has a compressibility about half that of water yet is completely miscible with water. Pure glycerin has boiling and freezing points at 290°C and -17°C and a viscosity about 50% larger than water. The glycerin combined with a relatively rigid polypropylene porous plastic filter develops a high air entry tension to prevent loss of saturation during use and penetration through soils above the water table. Details of the saturation procedure used by the writers is given in a paper by Campanella et al. (1983).

The importance of initial complete saturation is reduced somewhat once penetration in excess of about 5 m below the water table has been achieved. The resulting equilibrium water pressure is then often sufficient to put any minor air bubbles into solution. The importance of initial saturation is also reduced if only equilibrium piezometric readings are required during stops in the penetration.

Soil Type and Stress History

No clear correlation between soil type and pore pressure measurements during CPT has yet emerged. The recently suggested soil classification chart based on pore pressure and bearing data (Jones and Rust, 1982) shows considerable promise. Most researchers believe that it is just a matter of time before sufficient field correlations are obtained to allow a soil classification chart to be developed. Unfortunately, this can not be achieved until some standard is accepted for pore pressure element location. The writers believe that a location immediately behind the tip will soon become the standard.

Several researchers have recently shown that the pore pressure ratio can be related in a quantitative manner to OCR for clay soils (Smits, 1982, Tumay et al., 1982). However, there are several factors that influence any correlation. The first is standardization of the cone design and pore pressure element location, since these have a significant influence on measured cone bearing, q_c, and pore pressure, u_T. The second is standardization in definition of pore pressure ratio. The main definitions proposed to date are:

(i) $\frac{u_T}{q_c}$ Baligh et al., 1981

(ii) $\frac{\Delta u}{q_T}$ Campanella and Robertson, 1981

(iii) $\frac{\Delta u}{q_c - u_o}$ Smits, 1982.

(iv) $\frac{\Delta u}{q_c - \sigma_{vo}}$ Senneset et al., 1982, Jones and Rust, 1982, Jefferies and Funegard, 1983.

The latter two definitions are important for offshore CPT work. The writers believe that the last definition (iv) may become the standard provided the measured cone bearing, q_c, is corrected for unequal area effects (q_T). If q_T is used, definitions (ii) and (iii) are very similar for shallow cone profiles where u_o is small.

The third factor that will influence any correlation between stress history and pore pressure ratio is related to the fact that even for normally consolidated clays, the measured excess pore pressure (Δu) is not unique for all clays. The excess pore pressures depend on the rigidity index ($I_r = \frac{G}{c_u} = \frac{E}{3c_u}$). Generally, for the same OCR, I_r increases with decreasing plasticity index, PI (see Fig. 3). High excess pore pressures are generally generated by normally consolidated soils with a high rigidity index (i.e. a low PI). The rigidity index also tends to decrease as OCR increases (see Fig. 3). Thus, it may not be clear if a low excess pore pressure is related to a normally consolidated soil with a high PI or a soil with a high OCR. Therefore, any future correlation is likely to contain some relation to PI, similar to that shown in Fig. 2. The sensitivity of a soil may also complicate any correlation.

The previous discussion should provide a guide as to how soil permeability, stress history and cone design influence the measured pore pressures during cone penetration.

Coefficient of Consolidation

Upon the arrest of steady penetration, excess pore pressures generated during cone penetration immediately start to dissipate. The rate of dissipation depends upon the coefficient of consolidation of the soil for a homogeneous deposit. Tavenas et al. (1982) have shown that for Canadian clays, the rate of pore pressure dissipation is mainly governed by the consolidation characteristics of the intact clay away from the probe. By monitoring the rate of dissipation of the excess pore pressure, an estimate of the coefficient of consolidation of the soil may be obtained. Several theoretical solutions are available to obtain the coefficient from dissipation of excess pore pressures generated by cavity expansion.

A summary of these solutions are shown on Fig. 4, which highlights the major differences between solutions. In order to compare results of the different solutions, they have been non-dimensionalized and shown in Fig. 4. Fig. 4 shows the decay of excess pore pressure, Δu, plotted against a non-dimensional time factor, $T = c_h t/r^2$. Use of the time factor, T, allows a quick calculation for the coefficient of consolidation, $c_h t/r^2$. The solution by Baligh and Levadoux, (1980), and the cylindrical solution by Torstensson, (1977), yield essentially the same result. The solution by Randolph and Wroth, (1979), is not shown because of its similarity to that of Torstensson, (1977).

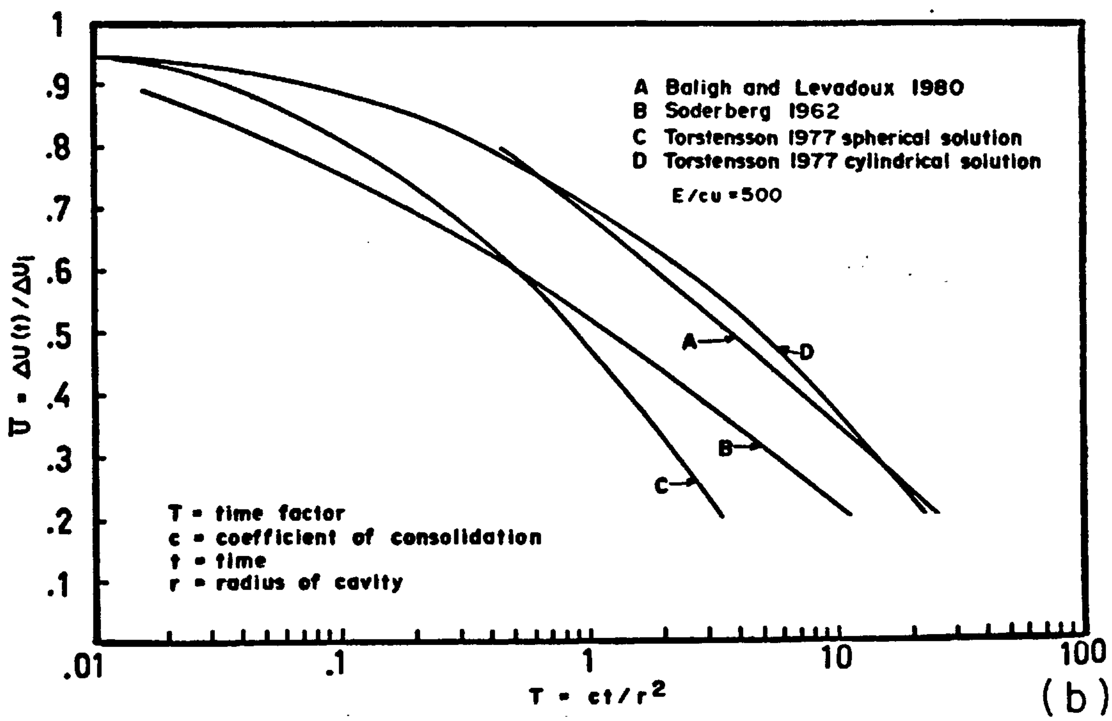
The solutions by Randolph and Wroth and by Torstensson require an estimate of the soil stiffness ratio or rigidity index (G/c_u). The reason for this is that a stiff soil will extend a zone of influence much larger than a soft soil. The result of a larger zone of influence is to decrease the rate of decay of excess pore pressures at the cone. The soil stiffness can be expressed as either, the undrained Youngs modulus, E_u, or the shear modulus, G, to the undrained shear strength, c_u. The undrained Youngs modulus and shear modulus are related by

$$G = \frac{E_u}{2(1+\mu)} \tag{11}$$

since $\mu = 0.5$ for undrained conditions.

Author	Cavity Type	Material Model	Initial Pore Pressure Distribution	Proposed Applications	Remarks
Baligh & Levadoux 1980	combined radial and spherical	non-linear Boston Blue clay	from F.E. studies using strain path method	consolidation characteristics	shows very small influence of spherical component of dissipation
Randolph & Wroth 1979	cylindrical	elastic-plastic	$\Delta u_i = 2\ cu\ \ln\left(\frac{R}{r}\right)$ $\frac{R}{r_o} = (G/cu)^{1/2}$	consolidation around piles pressuremeter analysis	analytical solution
Soderberg 1962	cylindrical	elastic-plastic	$\frac{u_r}{u_i} = \frac{r_i}{r}$	consolidation around piles	
Torstensson 1977	cylindrical	elastic-plastic	$\Delta u_i = 2\ cu\ \ln\left(\frac{R}{r}\right)$ $\frac{R}{r_o} = (G/cu)^{1/2}$	consolidation characteristics	proposes average of two results
Torstensson 1977	spherical	elastic-plastic	$\Delta u_i = 4\ cu\ \ln\left(\frac{R}{r}\right)$ $\frac{R}{r_o} = (G/cu)^{1/3}$	consolidation characteristics vertical drains	

(a)



(b)

FIG. 4: SUMMARY OF EXISTING SOLUTIONS FOR PORE PRESSURE DISSIPATION.

(Adapted from Gillespie, 1982).

$$G = \frac{E_u}{3} \quad (12)$$

Selection of an exact stiffness ratio is complicated by the variation in moduli with strain level, as shown in Fig. 3. With the complex variation in strains around the cone it seems reasonable to select a stiffness ratio at an intermediate stress level (say, G_{50} or E_{u50}). Although some doubt surrounds the selection of an appropriate stiffness ratio the solutions are not very sensitive to soil stiffness. For a four-fold increase in stiffness ratio, the predicted coefficient of consolidation changes by a factor of about 2. This is a relatively small variation for a parameter that can vary by several orders of magnitude.

Provided equilibrium pore pressures are not required, it is not necessary, for the purpose of obtaining consolidation characteristics, to wait past the 50 percent level of dissipation.

The applicability and meaning of the solutions is complicated by several phenomena. These phenomena include:

- the importance of vertical as well as horizontal diffusion,
- the effect of soil disturbance,
- uncertainty over the distribution, level and change of total radial stresses, and
- soil anisotropy and nonlinearity of soil compressibility.
- Non-homogeneity due to soil layering or nearness to a layer boundary (this problem is minimized when horizontal drainage dominates).

In spite of these limitations, the usefulness of the test procedure is encouraged by the repeatability of the test and the vast range in dissipation rates measured for various soils encountered.

The influence of vertical dissipation was shown by Gillespie and

Campanella (1981) to be insignificant and that horizontal dissipation appears to dominate the consolidation process, at least, for the pore pressure element located immediately behind the tip. Hence, cylindrical dissipation solutions, such as that by Torstensson (1977) can be expected to give reasonable results. Results from a study by Gillespie and Campanella (1981) showed that the theoretical solutions appear to give a coefficient of consolidation, c_h, in the horizontal direction for a soil in the slightly overconsolidated state (OCR ≈ 2). This result seems reasonable since the soil around the tip, especially behind the tip, has been preloaded due to the process of penetration.

The theoretical solutions are applicable only to soft, normally consolidated clays, where the initial pore pressure distribution around the cone is reasonably well defined. A detailed discussion about the limitations of the theories is given by Tavenas et al. (1982).

In spite of these limitations the dissipation test provides a useful means of evaluating approximate consolidation properties and macrofabric and related drainage paths of natural clay deposits. The test also appears to provide very important information in the design of vertical drains (Battaglio et al., 1981).

It is useful here to comment on the procedure used while recording the pore pressure dissipations. Some researchers have reported that they found it necessary to clamp the penetration rods at the ground surface while recording pore pressure dissipation. It appears that if the rods were not clamped a drop in the measured pore pressure would result when load was released from the tip. It appears the location of the sensing element explains the sensitivity of decay response to procedure used. When load is released, pore pressures at the tip immediately drop in response to the decrease in total stress. Whereas, behind the tip, in the zone of failed soil the stress level does not change significantly when load is released. It therefore appears that, for standard 60° cones, the location of the piezometer element behind the tip is less sensitive to the prodedure used. This is an important point because the amount of load applied to the tip, even with the rods clamped, will change with time due to stress relaxation.

Permeability

A crude estimate of permeability can be made from the soil type classification. A more reliable estimate of permeability, especially for fine grained soils, can be made from the consolidation and compressibility characteristics. Since:

$$\begin{aligned} k_v &= c_v \, m_v \, \gamma_w \\ k_h &= c_h \, m_h \, \gamma_w \end{aligned} \tag{13}$$

where k_v and k_h are the coefficient of permeability in the vertical and horizontal directions, respectively. Results of limited past experience suggests that soil compressibility can be regarded as approximately isotropic, $m_v = m_h$ (Mitchell et al., 1978; Ladd et al., 1977) for the purposes of estimating permeability.

Since an estimate of m_v can be made, then estimates of vertical permeability can be obtained. Estimates of m_v can be made using Table 1 or using an α factor based on local experience.

If it is assumed that soil compressibility is isotropic, then:

$$c_v = c_h \times \frac{k_v}{k_h} \tag{14}$$

An estimate of the ratio k_v/k_h can be obtained from Table 3, after Baligh and Levadoux, (1980). Evidence of the soil heterogeneity can be obtained from examination of the bearing, friction and dynamic pore pressure records.

Groundwater Conditions

The addition of pore pressure measurements during cone testing provides a direct measure of groundwater conditions. The equilibrium piezometric profile can be measured directly during a stop in the penetration. Experience gained by the writers has shown this to be an extremely important feature for the piezometer cone for penetration in both drained and undrained soils. It has been common practice to obtain the height of water in a borehole but rarely are the groundwater conditions hydrostatic. Often there is a slight upward or downward gradient of water pressures resulting from overall regional groundwater conditions. The ability to measure equilibrium piezometric pressures during a stop in the penetration is useful for evaluating consolidation conditions or unusual hydraulic gradients. Identifying the actual groundwater conditions can be extremely valuable for investigations of dams, embankments, tailings disposal areas, slopes and tidal areas.

The time required to reach full equilibrium pore pressure during a stop in penetration will depend on the soil permeability. For many investigations, it is sufficient to take equilibrium measurements at the end of the profile before pulling the rods and during rod breaks in any sand layers.

	Nature of Clay	k_h/k_v
1.	No evidence of layering	1.2 ± 0.2
2.	Slight layering, eg., sedimentary clays with occasional silt dustings to random lenses	2 to 5
3.	Varved clays in north-eastern U.S.	10 ± 5

TABLE 3: Anisotropic Permeability of Clays
(After: Baligh and Levadoux, 1980)

SUMMARY AND CONCLUSIONS

Many of the recommendations suggested by the authors are already in use by many engineers. Also many of these suggested correlations may require slight adjustment based on local experience for specific soil types.

The cone penetration test has traditionally produced excellent continuous profiles of undrained shear strength. The major problem, however, with the determination of undrained shear strength, (c_u), is the evaluation of the in-situ c_u appropriate to the particular design problem, since the c_u depends on the stress path followed during shear. The recommended general empirical correlation using an N_k value of 15 is related to the field vane strength. With local experience individual correlations for N_k can, and have been, developed for specific clays. With a measure of c_u from cone data, it is possible to estimate over consolidation ratio, (OCR), with information on σ_{vo} and PI.

The authors feel that the present state of the art is such that interpretation of CPT data has not yet reached a stage to allow reliable estimates of drained shear strength parameters from undrained cone penetration data. The concept of effective stress interpretation of CPT data is currently a topic of intense research.

The recommended predictions of volume change characteristics, such as compression index, which are based on undrained cone resistance may be in large error. However, with local experience significantly improved correlations can, and have been, developed for specific soil types.

The addition of pore pressure measurements during cone penetration testing has significantly improved the interpretation of CPT data to obtain geotechnical parameters. The continuous measurement of pore pressures has also enhanced the electric penetrometer for stratigraphic logging of soil deposits.

Procedures are recommended to estimate the horizontal coefficient of consolidation from pore pressure dissipation curves. The soil permeability can also be estimated after the soil compressibility is estimated from cone bearing.

The major problem facing the quantitative interpretation of the pore pressure measurements during cone penetration is the location of the porous element. A secondary problem is the saturation of the pore pressure measuring system. A very convincing argument can be made to standardize the location to provide a wide range of applications but yet maintain a practical location for saturation and protection. The authors believe that the pore pressure element should be located immediately behind the cone tip to meet these requirements (Campanella et al., 1983). It would appear logical that the overall cone design should also be made such that the porous element location can be changed in the field to allow for possible special soundings to be carried out to obtain specific pore pressure data. Although, in the authors experience, this is rarely necessary if the porous element is located immediately behind the tip.

If new pore pressure related correlations are to be developed and applied in engineering practice, a concensus is required as to the cone design and pore pressure element location. As a minimum, all pore pressure measurements from cone testing must clearly identify the location and size of the sensing element.

The authors also feel that the cone designs should be further standardized to include equal end area friction sleeves. Cone penetration resistance data should also be corrected, where possible, to total stress, q_T. It is only when all cone data is uneffected by water pressure effects that significant improvements can be made in the interpretation of CPT data. All cones, especially piezometer cones, should be fully calibrated for cross-talk effects, i.e., all channels should be recorded during each calibration of load, all around pore pressure and friction and that these should be reported for a given cone and its CPT data.

This paper deals principally with the interpretation of electric CPT data for the assessment of geotechnical parameters. The application of these parameters or the direct use of cone data to design problems, such as, design of shallow or deep foundations, is beyond the scope of this paper. The authors plan to publish a paper in the near future that discusses the various application techniques for CPT data.

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