Girishkumar KolibyluNanaiah: Interpretation of Small Scale Field Test Data for Geotechnical Engineering Practice in South- East Queensland

Project Brief

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Title of Project: Interpretation of Small Scale Field Test Data for Geotechnical Engineering Practice in South- East Queensland

Background:

Due to the extensive construction activities in Queensland, it is important to analyse the available in-situ small scale field test data and establish useful correlations of the engineering properties. Such correlations are more accurate than the properties obtained from laboratory tests which are subjected to sample disturbance and other testing errors.
In this study, the extensive test data available from cone penetration tests and standard penetration tests will be analyzed from the major projects in Surfers Paradise within Gold Coast. Such tests are often carried out for the design of shallow and deep foundations. As such the correlations to be established in this research work will be of academic value as well as of appropriate use to the private sector in foundation engineering.

Objectives:

The main objectives of this research are to:

- **Interpret the cone penetration and standard penetration tests data.**
- **Obtain useful engineering parameters for shallow and deep foundation practice in the estimation of**
  - **Allowable bearing capacity and**
  - **Settlements**

Steps involved in achieving these objectives:

- **A critical literature review on the relevance of in-situ tests in foundation engineering.**
- **Collection of in-situ tests data on cone penetration tests and standard penetration tests.**
- **Interpretation of these data and establishment of relevant correlations**

Abstract

Piezocone has now become a versatile tool for site investigation works on major projects which involve deep foundations in Gold Coast and road embankments for expressways in Southeast Queensland. Thus a detail review was made on the evaluation of geotechnical parameters in sand and clays and the available methods on soil profiling. In addition twenty one piezocone tests at Broad Beach in Gold Coast and further three tests on Port of Brisbane Motorway were analysed. The sand at Broad Beach was found to be medium dense to dense from soil profiling and dense to very dense from the estimated angle of friction which ranged from 40 to 50 degrees. The soft clay at the POB site was found to have strength to effective overburden ratio of 0.25 and the coefficient of horizontal consolidation to generally range from 1 to 2 m^2/year, while these values also became more than ten fold in many tests. It is recommended that a major study programme be carried out with the piezocone test and their applications and to relate the findings with the dynamic cone tests which are popular in smaller and medium size projects.

CHAPTER 1

INTRODUCTION

1.1 General Introduction

Rapid urban development is taking place in Gold Coast and Southeast Queensland. Thus the School of Engineering at the Griffith University Gold Coast Campus has a major research program on the engineering behaviour of the subsoils in Gold Coast and Southeast Queensland. This project work forms part of this major research activity and is on the analysis of piezocone penetration tests at two sites one in Broad Beach, Gold Coast called Site A and the other in the vicinity of the Port of Brisbane called Site B. Initially, the research work in the School of engineering at the Griffith University Gold Coast Campus was on soft clays and their improvement (see Ross, 2003; Eddie (2003), Shuttlewood, 2003; Braund, 2004; Scott, 2004and Surarak, 2005. Also, Thames (2003) and Whittemeir (2004) concentrated on the subsoils in Gold Coast and their engineering properties. Armstrong (2004) worked on the Emerald Lake Development project where acid sulphate soil is encountered. Additional work on soft clays and their improvement was also undertaken by Baker (2005), Eke (2005), Scar (2005) and Botha (2005). The current project is undertaken with a view to investigate the potential of CPTu tests and their applications in deep foundations and ground improvement works, especially with surcharge and vertical drains in soft clays. A complete detail literature review was conducted with this aim in mind, however the full analysis of the vast amount of the available CPTu tests data as collected are somewhat handicapped by the limited time available for the study. It is expected that this project work would encourage further research in detail in this important in-situ test which has virtually replaced other form of in-situ tests such as the SPT and vane tests in Southeast Queensland.

The cone penetration tests (CPT) and the piezocone tests (CPTu) have now become the most popularly used in-situ test in Brisbane, Gold Coast and other parts of Southeast Queensland. As pointed out by Meigh (1987) , cone penetration test has three main applications: (1) to determine the soil profile and identify the soils present; (2) to interpret ground conditions between control boreholes and (3) to evaluate engineering parameters of the soils and to assess bearing capacity and settlement. The CPT and CPTu have two main advantages over the usual site investigation practice with boring, sampling and standard penetration testing. It provides a continuous or almost continuous record of the subsoil conditions. Additionally, it eliminates the disturbance of the ground generally caused by boring and sampling and in particular with those due to the SPT testing. The main geotechnical parameters derived from CPT and CPTu test data are the angle of shearing resistance and deformation moduli in cohesionless soils and undrained strength, deformation moduli, overconsolidation ratio and coefficient of consolidation in cohesive soils.

In CPT and CPTu the identification of soils is achieved by means of empirical correlations between the soil type and the ratio of local side friction to cone resistance (skin friction ratio) considered in relation to the cone resistance. During a cone penetration test, complex changes take place in stresses, strains and pore pressures, thus theoretical analysis is most difficult and empirical correlations are preferred in engineering practice. Thus, the estimation of geotechnical parameters for engineering calculations as based on empirical correlations. The results can also be used directly to estimate the bearing capacity and the settlement on an empirical basis.

In this study 21 CPTu tests as obtained from Site A in Broad Beach and an additional three test from the Port of Brisbane were analysed. Site A in Broad Beach is mainly of sand with a sub-layer of peat which needs to be located as this layer is most undesirable in foundation works. Site B in Port of Brisbane consists of a deep layer of soft clay and thus it was possible to interpret the CPTu test data both in Sand and Soft clays.

The data from Broad Beach is mainly used in the design of deep foundations. The skin friction measurements are directly used in the estimation of the skin friction in driven and bored piles, while the cone resistance is used in determining the end bearing capacity. In this study CPTu tests carried out in a grid pattern is used to interpret the sub-soil profile at Broad Beach. The method adopted are Searle (1979), Douglas and Olson (1981) and Robertson and Campanella (1983). These methods are designated in this report as Method-1 (Searle, 1979), Method-2 (Douglas and Olson, 1981) and Method-3 (Robertson and Campanella (1983).

At Site B in Port of Brisbane the CPTu tests were conducted to estimate the engineering properties of the soft clay with respect to the design and construction of road embankments.

1.2 Objective and scope

As stated before, the soil encountered in Broad Beach is mainly sand and those at Port of Brisbane is mainly soft clays. The CPTu tests are the latest development in penetration testing both in sand and soft clays, where undisturbed sampling is most difficult for laboratory tests. Thus the objective and scope of this project work are

i) A review of the development in the cone penetration tests and how their data can be used in soil profiling and in estimating engineering parameters of both sand and soft clays.

ii) To use three methods developed by well known researchers in soil profiling and to determine the sub-soil profile at Site A in Broad Beach Gold Coast.

iii) Obtain the relevant engineering parameters for the soft clays at Site B in Port of Brisbane as pertinent for the design of road embankments.

1.3 Layout of thesis

Following the introduction in Chapter1, the developments of cone penetration tests and the testing procedure are presented in Chapter 2. This chapter also deals with the measurements of the relative density, D_r and the angle of internal friction Φ for granular material such as sand while the undrained shear strength, s_u and the coefficient of consolidation (c_v, c_h) for clays as well as the estimation of the pre-consolidation pressure p_c and the over consolidation ratio. (OCR). Deformation moduli such as G_{max} and the coefficient of volume decrease, m_v can also be estimated. Chapter 3 is entirely devoted to the available methods of using CPT and CPTu tests data in soil profiling. Chapter 4 deals with the bearing capacity estimation and settlement computations from CPT and CPTu tests. Finally, the interpretation of the test data from Broad Beach and Port of Brisbane are contained in Chapter 5 and the conclusions are summarised in Chapter 6.

CHAPTER 2

LITERATURE REVIEW

2.1 General Introduction

This chapter is mainly devoted to the interpretation of the Piezocone tests (CPTu) data and the associated theoretical developments and the estimation of the relevant geotechnical parameters. In the piezocone, the penetration tip is normally a cone with an apex angle of 60 degree and a base area of 1000 mm^2. The friction sleeve located immediately above the cone base has an area of 15000 mm^2. In addition a standard pore water pressure (u_{max}) measurement system during the penetration of the cone is incorporated with a fine porous stone element and a pressure transducer. During a CPTu
(see Fig.2.1) penetration test, continuous measurement of the tip resistance ($f_{s)}$ and the generated excess pore pressure (u_{max}) are made.

The test is carried out by pushing the piezocone continuously in the sub-soil layer, at a rate of 0.02 m/s. Since the penetration of the cone cause complex change in the stress and the strain conditions around the cone tip, the measured data (Fig.2.2) such as q_c, f_s and u_{max} are directly used to obtain the relevant geotechnical parameters. In addition to q_c, f_s , the pore pressure parameters B_q (Senneset and Janbu, 1982) can also be determined. Thus the parameters measured are given in Table 2.1. Inside the cone penetrometer, a piezoelectric element converts digital coded data at the tip to an acoustic signal that is transmitted along the rods to a microphone mounted on the rig. The microphone converts the signal to data that is transmitted to a personal computer which presents both analog and digital information. The wireless CPTu system provides a rapid and cost effective way to obtain subsurface data. Continuous and simultaneous information collected includes: Cone tip resistance, $q_{c,}$ local cone friction, f_s and pore pressure, $u_{max.}$ From the measured data , geotechnical and hydrological parameters can be obtained from empirical correlations (see Table 2.2).

2.2 Cone Penetration Test

This section relates to aspects of the cone penetration test which relate directly to the study of the spatial variability of the soil properties. These include: equipment, test procedure, applications and data interpretation, determination of the soil profile, undrained shear strength, extent of the failure zone and accuracy of the test itself. As mentioned before the cone penetration tests are used in this study to interpret the sub-soil conditions at Broad Beach, Gold Coast. The Piezocone penetration test (as distinct from the mechanical cone penetration test, and commonly referred to by the shorter name of cone penetration test, CPT) essentially involves pushing a steel cone and rods of standard dimensions in the soil subsurface and monitoring the mobilized resistance to penetration of the soil.

Since it was first developed in Holland in 1965, the CPT has continued to gain wide acceptance in many countries throughout the world. De Ruiter (1981) attributed its increased worldwide use to three main factors:

1) The electric cone penetrometer provides more precise measurements and the improvements in the equipment allow deeper penetrations, particularly in dense materials.
2) The need for penetration testing as an in situ technique in offshore foundation investigations, in view of the difficulties in achieving adequate sample quality in marine environments.
3) The addition of other simultaneous measurements to the standard friction penetrometer, such as pore water pressure and soil temperature.

De Ruiter (1981) stated that the CPT is the only available routine technique that provides an accurate continuous profile of soil stratification.

2.3 Equipment

The electric cone penetrometer essentially consists of two strain gauge load cells, one being attached to the cone tip and measuring the cone tip resistance, q_c and the other connected to the side surface or sleeve of the cone penetrometer and measuring the sleeve friction, f_s .

The load cells contain a number of electrical resistance strain gauges which are arranged in such a manner that automatic compensation is made for bending stresses and only axial stress is measured (de Ruiter, 1971). The push rods, used to advance the electric cone penetrometer into the subsurface profile, are usually of a standard length of one meter with a tapered thread, male at the lower end and female at the upper. In addition, the rods have a hollow core so that the cone penetrometer cable can pass through each rod enabling the electronics of the cone to be connected to the recording instruments located at the ground surface. The recording devices generally consist of both analogue and digital types. These instruments measure q_c and f_s , and in some cases, depth of the cone penetrometer.

The equipment and procedure of the CPT vary throughout the world. Over the years, many committee have been formed in an attempt to establish a consistent worldwide standard for the CPT. The most recent of these being at the First International Symposium on Penetration Testing (ISOPT-1) (De Beer et al., 1988). In addition, some countries have established individual standards for the CPT, the most relevant of these for this research being the American Standard, *ASTM D3441* (American Society for Testing and Materials, 1986), and the Australian Standard, *AS 1289.F5.1* (Standards Association of Australia, 1977). In general, these three standards agree on the fundamental aspects of the CPT equipment and procedure. Relevant details of the CPT equipment standard procedure as specified in these standards are summarised briefly below:

- The standard cone has a base diameter of 35.7 mm and an apex angle of 60° resulting in a projected area of 1000 mm^2 (10 cm^2). The gap between the cone and other elements of penetrometer shall not be greater than 5 mm.

- The diameter of the standard friction sleeve is 35.7 mm and has a surface area of 15,000 mm^2 (150 cm^2). The friction sleeve is located immediately above the cone.

- Both the cone and sleeve shall be made from steel of a type and hardness suitable to resist wear due to the abrasion by the soil. The cone shall have a roughness of ≤ 1 μm and the friction sleeve shall have a roughness of 0.5 μm ± 50%.

- The thrust machine shall have a stroke of at least one meter and shall push the rods into the soil at a constant rate of penetration. The thrust machine shall be anchored such that it does not move relative to the soil surface during the pushing action.

The position of the filter for the measurement of the pore pressure is not standardized but the International Reference Test Procedure suggests behind the cone (u2) as the preferred location (Figure 2.3). Other locations are on the cone (u_1) or behind the friction sleeve (u_3). Piezocones which measure pore water pressures at two or three locations are denoted dual element or triple element piezocones, respectively. The measurement of reliable pore water pressure is not easy and requires greater care in instrument preparation than that for standard friction cone CPT testing.

Different arrangements are used by different manufacturers for measurement of the cone resistance, q_c, and the sleeve friction, f_s, on electrical strain gauge load cells. Figure (2.4) illustrates the three main design types. In Figure (2.4a) cone resistance and sleeve friction are measured by two independent load cells both in compression. In Figure (2.4b) the sleeve friction compressive load cell of Figure (2.4a) is replaced by one in tension. In Figure (2.4c) the sleeve friction load cell, in compression, records the summation of the loads from both the cone resistance and the sleeve friction, the sleeve friction being obtained from the difference in load between the friction and the cone resistance load cells. This cone is often referred to as the

"Subtraction Cone". The main advantage of this design is the overall toughness of the penetrometer (Schaap and Zuidberg, 1982). In early subtraction cones, problems were encountered in the accuracy of the friction sleeve measurement with this arrangement. However, recent designs and improvements in manufacturing details have led to improved accuracy in the measurement of the sleeve friction.

All current CPT/CPTU devices consist of seals and/or 0-rings in order to stop the entrance of both soil and water into the body of the device during testing. Great care must be given to the design of these so that they work effectively without hindering the ease of movement of the mechanical parts with resultant negative effects on the calibration performance of the CPT/CPTU.

Measuring pore pressures during cone penetration requires careful consideration of the probe design, the choice and location of the porous element and the probe saturation. For a high frequency response the pore pressure measurement system must have a small fluid- filled cavity, low compressibility and viscosity of the fluid, a high permeability of the porous filter, a large area to wall thickness ratio of the filter (Smiths, 1982) and a rigid or low compliance pressure transducer. To measure penetration pore water pressures rather than filter compression effects, the filter should be rigid. However, to maintain saturation, the filter should have a high air entry resistance, which requires a finely graded filter and/or high viscosity of the fluid. A balance is required between a high permeability of the porous filter to maintain a fast response time and a low permeability to have a high air entry resistance to maintain saturation. Clearly, not all these requirements can be combined.

2.4 Pushing Equipment

The pushing equipment consists of push rods, a thrust mechanism and a reaction system. The rigs used for pushing the penetrometer normally consist of hydraulic jacking and reaction systems. They are usually specially built for this purpose, but sometimes the pushdown of an anchored drill rig is used. The thrust capacity needed for cone testing generally varies between 10 and 20 tonnes (100 and 200 kN), although 5tonnes and 2tonnes capacities (50 kN and 20 kN) are also common for use in soft soils. The maximum allowable thrust on the standard 35.7 mm diameter high-tensile steel push rods is 20 tonnes (200 kN). Exceeding that load can result in damage and buckling of the test rods, either in the rig or in the soft upper layers of the soil.

Land-based rigs are often mounted in heavy duty trucks that are ballasted to a total deadweight of around 15 tonnes (150 kN) or more. Screw anchors can be used to develop extra reaction. The power for the hydraulic jacking system is usually supplied from the truck engine. With a double rear axle and both rear and front wheel drive, the trucks can operate off the road in most terrain conditions. Sometimes, all-terrain vehicles are used for work in marshy or soft areas.

The load of the hydraulic ram is transferred either by a thrust head on top of the push rods or by a clamping system that works by friction on the outside of the upper rod or by a notch cut into the rods. An automatic mechanical clamp saves time in the operation as the next rod can be screwed on, while the rig is pushing down the previous one. The standard cone rods have special tapered threads and are one meter in length.
Typically, the rods are pushed in 1 m strokes and the hydraulic rams then retracted ready for the next stroke. Systems do exist, however, whereby the rods can be pushed into the ground without

any pause to reset the system. Borros manufactured a rig that uses two synchronized hydraulic cylinders.

The enclosure of a truck provides ideal space for the installation of all electronic equipment for data acquisition. Climate control gives added comfort for personnel and preservation of electronics.

The penetrometer rig can also be placed on a light trailer equipped with earth anchors. A high-production truck mounted rig can produce up to 250 meters of penetration testing in one day, as compared to about 120 meters for a trailer-mounted rig, both under favorable site conditions. The most time-consuming part of the trailer-mounted operation is the setting of screw anchors, which are usually required to provide additional reaction because of the lack of deadweight. An intermediate solution is to mount the rig on a heavy trailer or heavy-duty truck frame that can be ballasted. A CPT can also be performed using drilling rigs, but pushing capacity can be often limited to about five tonnes without anchors. Use of a drilling rig can have the added advantage of improved cost and flexibility. Some drill rigs, especially auger rigs, are suited to rapid installation of anchors for testing with reaction of up to 20 tonnes.

A 20-tonne (200 kN) thrust will normally result in penetration depths of around 30 meters in dense to medium dense sands and stiff clays. In weaker soils penetration to depths in excess of 100 meters may be achieved provided verticality is maintained. Gravel layers and boulders or heavily cemented zones can, of course, restrict the penetration severely and deflect and damage cones and rods.

2.5 Cone penetration Test Procedure

2.5.1 Set up Procedure

The set up usually consist of the following steps:

i. De-air and saturate the pores stone
ii. Insert cone cable through all rods and connect the cable to the data acquisition system
iii. Set up the hydraulic pushing machine and attach the depth wheel to the pushing frame
iv. Check the recording system before starting the test
v. Adjust the speed of the penetration to within the acceptable range 0.015-0.020 m/s

2.5.2 Testing Procedure

The procedure shall be as follows:

a) Using a dummy cone to push a few meters (1 to 3m) into the ground to clear the top crust. Then the dummy cone is replaced by the piezocone. Remove material such as crushed rock or gravel, which will be too hard to penetrate with the penetrometer or could damage the equipment. Measure the depth from the surface level to the upper surface of the layer to be tested to the nearest 10 mm, and record.
b) Hold the penetrometer vertical with the point of the cone on the surface of the layer to be tested and, if necessary, gently tap the hammer on the anvil until the widest part of the cone has started to penetrate the surface.

c) Drive the penetrometer into the ground by raising the hammer to the stop and allowing it to fall freely onto the anvil.
d) Set up the thrust machine for a thrust direction as near vertical as practicable.
e) Connect the penetrometer and the first length of rod assembly, and place them in the thrust machine, inserting the penetrometer in the rod guide at the base of the machine.
f) Where the test is being carried out in a cased borehole, connect sufficient lengths of rod assembly for the penetrometer to reach the soil surface. Add spacers at 1.5 m intervals for lateral support.
g) Advance the penetrometer to the required level by applying sufficient thrust to the outer rods. The rate of penetration is not critical for this operation. Record the test level or depth.

For electrical devices, allow the tip of the cone to remain in this position until it has reached the ground temperature. A period of 5 to 10 min should normally be sufficient. Withdraw the penetrometer and take initial readings of the electrical transducers, with the penetrometer hanging freely in air and protected from sunlight. Advance the penetrometer to the recorded test level or depth.

2.5.3 Simplified cone penetrometer

Cone penetrometer procedure shall be as follows:

a) Force the cone into the soil to the full extent of its travel at a constant rate of 10 to 20 mm/s, by applying thrust to the inner rod. Record the load reading at specific points in the travel and the depth of the point.
b) Force the outer rod down behind the cone and take a second reading of the force.
c) Continue thrusting on the outer rods and take a third reading of the force as the collapsed penetrometer tip is advanced to the next test level. Record the new test level or depth.
d) Repeat steps (a) to (c) as penetration proceeds. Once the thrusting mechanism has reached the end of its travel, disconnect it from the rod assembly, raise the mechanism and insert an additional length of rod assembly.
e) On completion of the test to the required depth, fit the withdrawal mechanism to the rod assembly and withdraw the rods in stages.

2.5.4 Friction-cone penetrometer

The procedure shall be as follows:

a) Force the cone alone into the soil to the full extent of its independent travel at a constant rate of 10 to 20 mm/s by applying thrust to the inner rods. Record the force reading at a specific point in the travel.
b) Continue thrusting on the inner rods, to engage the friction sleeve and force the cone and friction sleeve into the soil to the full extent of their travel, to give the total cone-plus-sleeve resistance.
c) Force the outer rods down behind the sleeve and cone, to collapse the penetrometer tip and advance it to the next test level.
d) Repeat steps (a) to (c) as penetration proceeds.

e) Once the thrust mechanism has reached the end of its travel, disconnect it from the rod assembly, raise the mechanism and insert an additional length of rod assembly.

f) On completion of the test to the required depth, fit the withdrawal attachment to the rod assembly and withdraw the rods in stages until the penetrometer tip is hanging in air.

2.6 Undrained shear strength of clays

Basically, the undrained shear strength of clay is estimated from the cone resistance, the effective overburden stress and an empirical cone factor N_K. The various expressions available for the estimation of the undrained shear strength, s_u are summarized in Table 2.3.
The N_K factor can be determined theoretically, but in practice is obtained by correlating the cone resistance to the undrained shear strength as measured from the vane shear apparatus or from other laboratory test. N_K generally lies between 5 and 30 as presented by (Aas et al, 1988). The variability in the values of N_K can be reduced by using a piezocone and using the corrected q_t instead of q_c. Larochelle et al (1988) presented N_K values when correlated with the uncorrected vane shear strength. The N_K values so estimated by Larochelle et al (1988) did not vary much with the plasticity index and also not affected by the over consolidation ratio. This observation by Larochelle et al contradicts with the data presented by Aas et al, (1986). In view of the significant variation in N_K values, Seah (1995) recommended that the N_K values be calibrated with the laboratory values of shear strength or those obtained from vane tests.

The N_K and N_{KT} values generally used for soft clays (which are close to the normally consolidated state) are found to be not reliable for stiff clays in the overconsolidated state when affected by the pressure of fissuring and other fabric features. No correlation was found to exit between N_{KT} with the pore pressure parameter B_q. thus the expression for the undrained shear strength presented in Table 2.3 by Robertson and Campanella (1986) used the piezocone factor N_{KE} as correlated with B_q.

Larsson and Mulabdie (1991) used the liquid limit values (as determined in the laboratory) to estimate the undrained shear strength. In this approach the constants a and b are obtained by using vane shear strength or the laboratory values of the undrained shear strength. The expression by Larsson and Mulabdie (1991) for Swedish clays in Table 2.3 contains values of a and b.

Larsson and Mulabdie (1991) also used the measured excess pore pressure values in the piezocone tests for the estimation of the undrained shear strength in clays. In these expressions which involve the measured excess pore water pressure instead of the cone resistance q_c and q_t , the value of the excess pore pressure can be determined at the face (designated Δu_{STD}). The incorporation of the excess pore pressure can accommodate the dependency of the undrained shear strength on the overconsolidation ratio.

2.7 Overconsolidation ratio

A number of empirical approaches have been developed for the estimation of the OCR values in clays. In these approaches, the location at which the excess pore pressure is measured is also important.

Robertson and Campanella (1983) recommended that the excess pore water pressure be measured with the porous element located behind the cone tip as this position is the most sensitive one in measuring the stress history of soils. This was also studied by Jamiolkowski et al. (1985). Earlier several correlations were established to estimate OCR as measured from standard cone tests. Schmertman (1978) noted that q_c varied almost linear with the depth and the OCR values can be estimated. However there was no systematic correlation between the OCR values so estimated from the cone tests and those values determined in the laboratory by testing undisturbed sample of clay. Tavenas and Leroueil (1979) argued that the cone resistance can be an indication of the limit state stress condition and thus a measure of the preconsolidation pressure as well.

With the advent of the piezocone, several methods were proposed for the estimation of the OCR values from piezocone tests data. Table 2.4 is assembled by the author for the estimation of OCR values in clays from piezocone tests data.

Sully et al (1987) suggested a method in which OCR values can be estimated from normalized pore pressure as measured at the tip and at the base of the cone. Wroth (1988) recommended the estimation of OCR from values of q_t, σ_{VO} and σ_{VO}'. q_t is the total cone resistance, σ_{VO} is the total overburden pressure and σ_{VO}' is the effective overburden pressure. Powell et al (1988) incorporated the liquid limit of the clays which is also a reflection of the stress history. In the method of Powell et al (1988) the axial load on the cone is used, instead of the cone resistance. Sandven (1990) used q_t, σ_{VO}' and the liquid limit, w_l. Houlsby (1988) further incorporated the measured values of u_{STD}, the excess pore pressure as measured at the cone tip.

Additionally, the following points need to be noted:

1. There is no relationship between B_q and OCR (Battahlis et al, 1986, Jamiolkowski et al. 1985, and Wroth, 1984).
2. The pore pressure parameter PPD gives a good relationship with OCR for clays when OCR is less then ten. For this a special cone with two porous elements, one at the tip and the other located just behind the tip is needed. Such equipment is not commonly available (Cheng, 1998).
3. Incorporation of the liquid limit of the clays seems to minimise the wide variation in OCR values as determined purely from the cone resistance and the effective overburden stress.
4. Accounting pore pressure measurement in overconsolidated soils is difficult and thus the use of the axial load seems a better approach.
5. A number of researchers, Wroth (1988) and Crooks et al (1988) among others, have advocated the horizontal stress σ'_{H0}, or the mean principal stress, $p = (\sigma'_{VO} + 2\sigma'_{H0})/3$ instead of σ'_{vo} for the use in the correlations. This is not controversial but rather difficult to carry out in practice. The horizontal stress is rarely measured and also possible measurements are subject to uncertainties. When the various correlations using the vertical in- situ stress are normalized against the plasticity of the soil, possible effects of stress anisotropy may also be expected to be indirectly taken into account.

2.8 Coefficient of consolidation and permeability in the horizontal direction- c_h and k_h

Pore pressure dissipation tests are preformed with the piezocone to measure the c_h values- coefficient of consolidation in the horizontal direction. In the interpretation of these dissipation tests the decay in excess pore pressure with fines is analysed. Several simplified approaches are adopted and these have their origin to the work of Tortensson (1975, 1977). In this work the cavity expression theory was used and an uncoupled analysis is performed on the consolidation process. The initial excess pore pressure distribution is estimated by a one dimensional analysis with cylindrical and spherical cavity expansion. The usual approach for the estimation of the coefficient of consolidation as based on the time factor T is used as

$$c = \frac{T}{t} R^2 \text{ -------------- (2.1)}$$

where:

T: time factor
t: time
R: the equivalent cavity radius

However, the initial pore pressure response is also depended on the rigidity index I_R of the soil defined as

$$I_R = \frac{G_u}{S_u} \text{ -------------- (2.2)}$$

where:

G_u: undrained shear modulus and is assumed as G_{u50}
s_u: undrained shear strength

Table 2.5 summarises the various available expression for the calculation of the coefficient of consolidation c_h. In these solutions the following points be noted

1. The original Terzaghi – Rendulic solution is the basis for the subsequent developments.

2. Baligh and Levodoux(1980) incorporated the recompression ratio RR and compression ratio CR. Teh and Houlsby (1991) incorporated the rigidity and I_R.

The following points could further add to the value of the understanding of the c_h determination

1. The Tortensson (1975) solution incorporates the rigidity of the soil by using the ratio G_u/s_u of the undrained Young's modulus and the undrained shear strength. A stiff soil will extend a much layer zone influence than the soft soil during the piezocone penetration.

2. Levadoux and Baligh (1980) used the "strain path method" which takes into account of the two-dimensional axi-symmetric nature of the cone penetration test. The effect of coupling between the total stress and the pore pressure is small except at the early state of consolidation for pore pressure decay less than 20 percent. The pore pressures are computed near a cone with an apex angle of 18 degrees. The uncoupled solution provided nearly accurate prediction of the pore

pressure dissipation. The Levadoux and Baligh (1980) work additionally revealed that from the two-dimensional consolidation analysis around the cone, the dissipation rate is mainly controlled by c_h and even a tenfold change in c_v has negligible influence on the shape of the pore pressure isochrones.

3. The cone penetration produces undrained shearing of the soil with pore pressure increase and reduction of the effective stresses. When pore pressures start to dissipate, the soil surrounding the cone is subjected to an increase of effective stresses under conditions of reloading, and only after some dissipation has taken place do the effective stresses equal those existed before the tip penetration. Only from this point onward, the consolidation proceeds along the virgin compression curve.

4. Baligh and Levadoux (1980) have postulated that c_h obtained from the early stage of dissipation (less than 50% of consolidation) is relevant for the reloading conditions reflecting therefore the behaviour of OC soils. They proposed a method to obtain c_h from the dissipation data by using linear uncoupled Terzaghi-Rendulic solution .

5. In order to obtain the coefficient of consolidation in the NC range, the following equation is suggested:

$$c_{h(NC)} = \frac{RR}{CR} . c_{h(OC)} \text{ ----------------------- } (2.3)$$

where:

RR : recompression ratio
CR : virgin compression ratio

6. The above analysis of CPTU dissipation tests by Baligh and Levadoux (1980) does not include any possible influence of smear generated by the penetration process, which may lead, if not taken into consideration, to an underestimate of the computed c_h.

7. Teh and Houlsby (1991) observed that the theoretical solution depend very much on the soil rigidity index. They modified the solution by including soil parameter, I_r as :

$$T^* = \frac{c_h t}{R^2 \sqrt{I_r}} \text{ -------------------------------} (2.4)$$

Where

T* : the modified time factor.

8. It was observed that the simplified solutions by Tortensson (1977) provides essentially the same values as the most recent solution by Teh and Houlsby (1991). However, since the solution by Tortensson (1977) are based on cavity

expansion (cylindrical and spherical), they are unable to clearly define the different response curves for different pore pressure element locations.

9. Robertson (1992) recommended that the theoretical solutions proposed by Teh and Houlsby (1991) provide reasonable estimate of the in-situ coefficient of consolidation in the horizontal direction.

10. The solution given by Teh and Houlsby (1991) represent the most recent and comprehensive theoretical study of the CPTU as well as piezocone dissipation test. (Robertson, 1992).

11. Campanella and Robertson (1988) recommended that the applicability and meaning of the theoretical solution is complicated by several phenomena, such as the importance of the vertical as well as the horizontal dissipation, effect of soil disturbance, uncertainly over distribution, level, and changes in total stress. Despite these limitations, Campanella and Robertson (1988) suggested that the CPTu dissipation test provides an economic and useful means of evaluating approximate consolidation properties, soil macro- fabric, and related drainage paths of natural, fine grained soil deposits.

2.9 Horizontal Coefficient of Permeability

Approximate estimates of the horizontal coefficient of permeability k_h can be obtained from the expression of Baligh and Levadoux (1980) as :

$$k_h = \frac{\gamma_w}{2.3.\sigma'_{vo}}.RR.c_{h(OC)} \quad \text{-----------------------------(2.5)}$$

where
σ'_{vo} : Effective overburden stress
γ_w : Unit weight of water
RR : recompression ratio

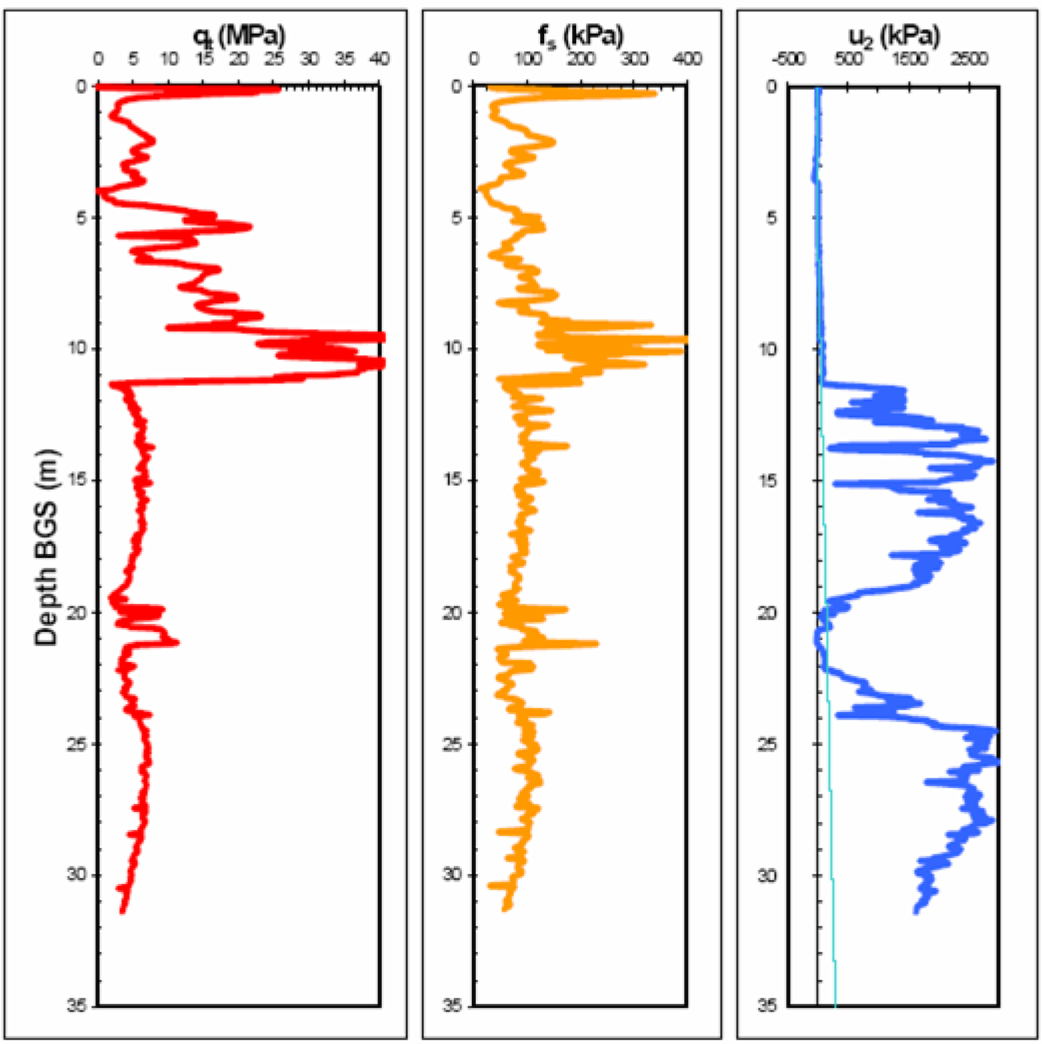
In Table 2.6 the author has presented a modified version of the contribution of Mayne (1991) on the various parameters relevant to the estimation of OCR from the piezocone tests data. Similarly, Table 2.7 summarise the available contributions as related to the estimation of the coefficient of consolidation c_h. in these approaches cylindrical and spherical cavity types are used. The soil behaviour is either non-linear elastic or elasto-plastic. Both 1-D and 2-D consolidation phenomena are incorporated in the analysis.

Table 2.8 summarise the work done in estimating the drained shear strength. Generally the bearing capacity and cavity expansion approaches are used. The main assumption made on the soil behaviour is either elastic, rigid plastic or elastic and perfectly plastic. Some approaches take into account of the compressibility of the soil while most of them ignore. How ever if the rigid plastic and perfectly plastic assumptions are made then the compressibility will be zero. In most instances a linear strength envelope is assumed. Also in the majority of the methods the in-situ

effective vertical stress is used while the cavity expansion theories also employ the in-situ effective horizontal stress. The other data employed are the coefficient of earth pressure at rest, K_0; relatively density D_r; rigidity modulus G; compressibility parameter λ_{ss}; volumetric strain, ε_v; and the perimeter, B.
Concluding Remarks

Penetration testing has had rapid developments in the last three decades or so. The standard penetration test (STP) has been widely used, yet the test is not continuous in nature and course disturbance around the location where the test is performed. Cone penetration test offered a means of assessing the penetration resistance on a continuous basis and also to separate the cone resistance and sleeve friction. Also, the piezocone allows to determine the pore pressure parameter B_q, which gives an indication of the permeability of the soils. The most important part is bot CPT and CPTu needs to be calibrated locally to allow for the variation in soil properties and also similar to STP to allow for the testing errors involved by the various organizations who perform these tests.

Figs: Chapter 2



Cone tip stress Sleeve Friction Pore water pressure

Figure 2.1 Measured data from CPTu test

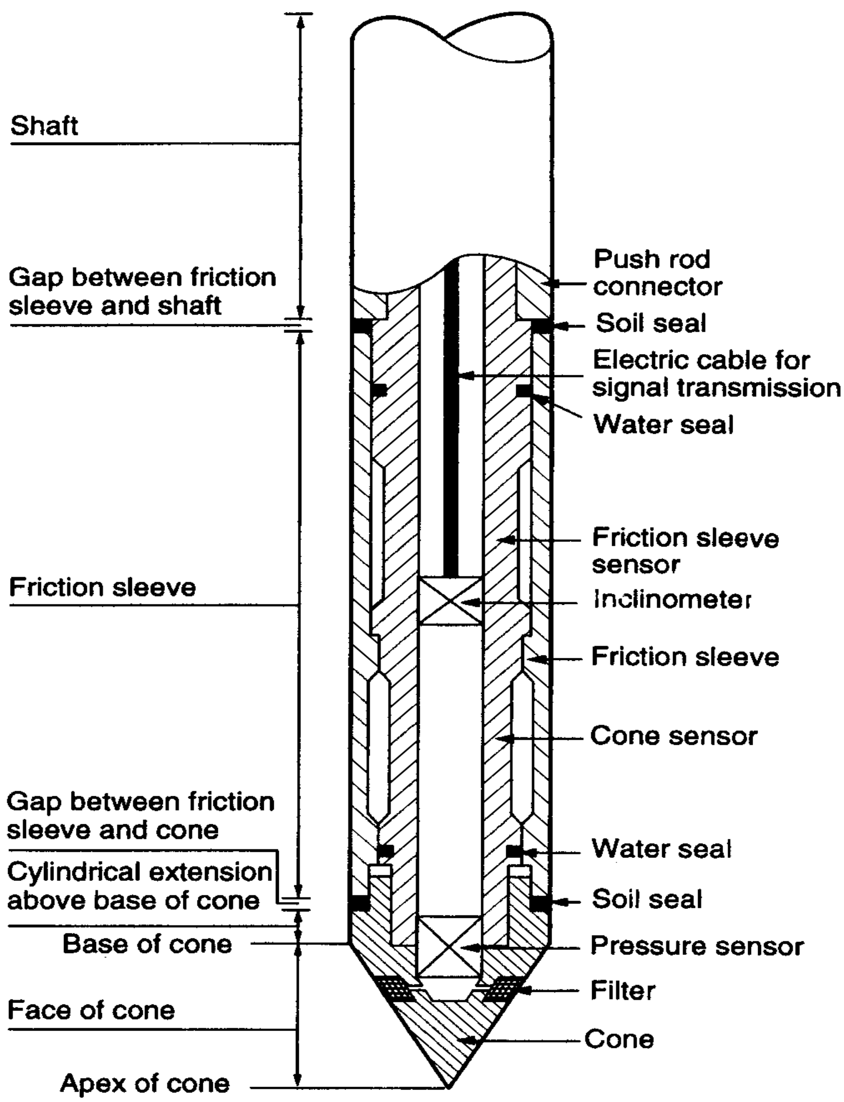


Figure 2.2 Piezocone (after Zuidberg, 1988)

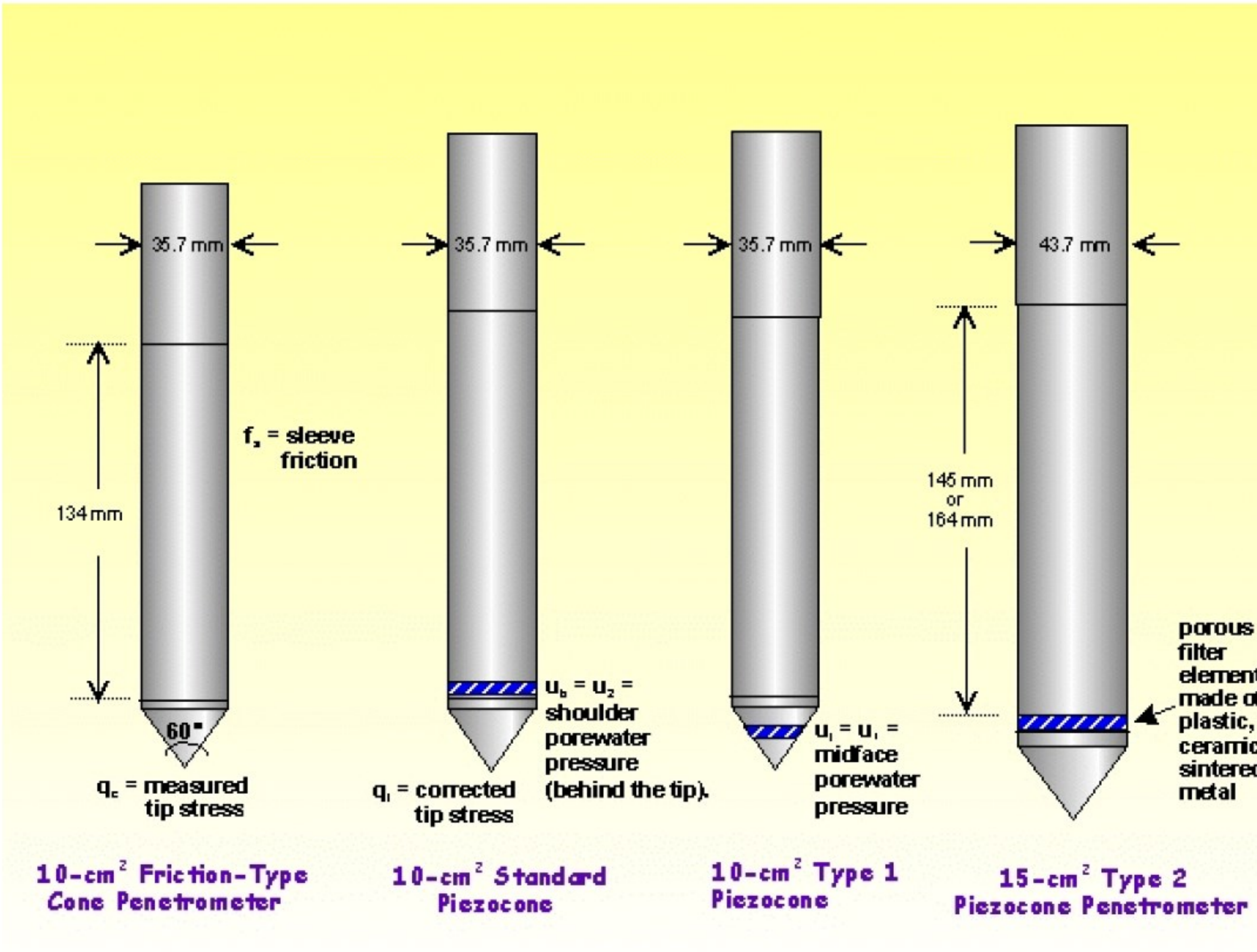


Figure 2.3 Three different position of pore measurement

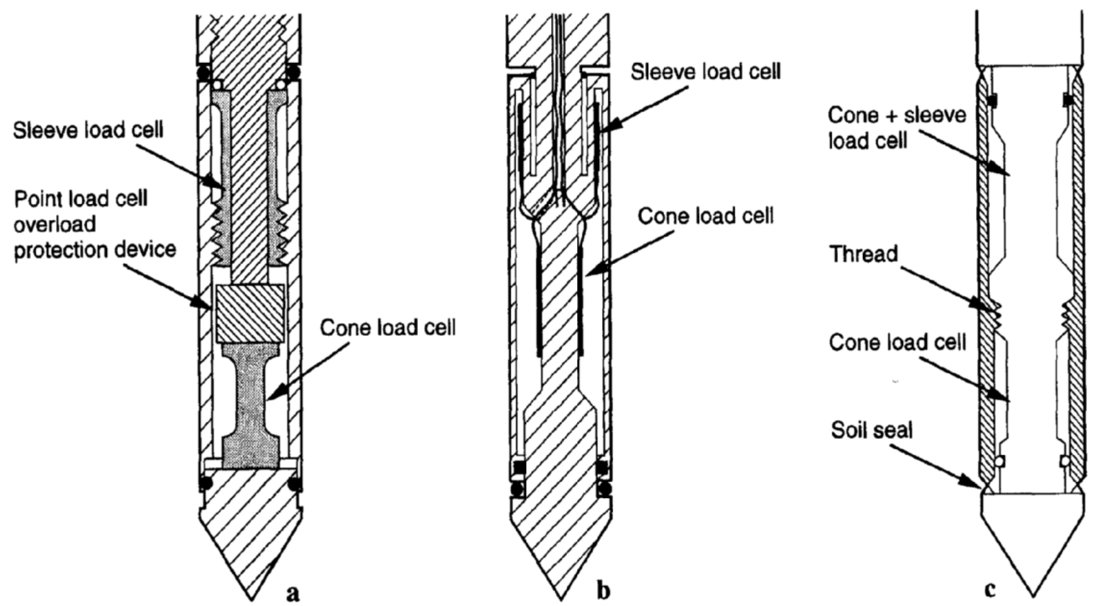


Figure 2.4 Cone penetrometers.
(a) Load cells for cone resistance and sleeve friction measurement (compression mode)
(b) load cell in compression and sleeve friction load cell in tension. (tension mode)
(c) Subtraction type cone penetrometer

CHAPTER 3

SOIL PROFILING USING CONE PENETRATION TEST DATA

3.1 General Introduction

In this chapter the available soil profiling methods using cone penetration tests are reviewed. Begemann (1965) presented the first rational soil profiling method. The other researchers who contributed for soil profiling are Sanglerat et al., (1974) Schmertmann (1978), Searle(1979), Douglas and Olsen (1981), Jones and Rust(1982),Robertson and Campanella (1983), Robertson et al (1986), Senneset et al., (1989), Robertson (1990) and Eslami and Fellenius (1997).

3.2 Method of Begemann (1965)

Begemann pioneered the work on soil profiling from the CPT, showing that, while coarse-grained soils generally demonstrate larger values of cone resistance, q_c, and sleeve friction, f_s, in comparision to the fine-grained soils, the soil type is not a strict function of either cone resistance or sleeve friction, but of the combination of the these values. Fig. 3.1 presents the Begemann soil profiling chart, showing (with linear scales) q_c as a function of f_s. Begemann showed that the soil type is a function of the ratio between the sleeve friction and the cone resistance (the friction ratio, R_f). The friction ratio is indicated by the slope of the fanned-out lines. The Begemann chart was derived from tests in Dutch soils with the mechanical cone. The chart is site-specific, i.e., directly applicable only to the specific geologic locality where it was developed. However, the chart has important general qualitative value.

3.3 Method of Sanglerat et. al. (1974)

Sanglerat et al. (1974) proposed the chart shown in Fig. 3.2, presenting data from an 80 mm diameter research penetrometer. The chart plots the cone resistance (logarithmic scale) versus the friction ratio (linear scale). This manner of plotting has the apparent advantage of showing the cone resistance as a direct function of the friction ratio and, therefore, of the soil type. However, plotting the cone resistance versus the friction ratio implies, falsely, that the values are independent of each other; the friction ratio would be the independent variable and the cone resistance the dependent variable. In reality, the friction ratio is the inverse of the ordinate and the values are not independent. That is, the cone resistance is plotted against its own inverse, multiplied by a variable that ranges, normally, from as low as 0.01 to as large as 0.07. The plotting of data against its own inverse values will predispose the plot to a hyperbolically shaped zone ranging from large ordinate values at small abscissa values through small ordinate values at large abscissa values. The resolution of data representing fine-grained soils is very much exaggerated as opposed to the resolution of the data representing the coarse-grained soils. Simply, while both cone resistance and sleeve friction are important soil profiling parameters, plotting one as a function of the other distorts the information.

3.4 Method of Schmertmann (1978)

Schmertmann (1978) proposed the soil profiling chart shown in Fig. 3.3. The chart is based on results from the mechanical cone data in “North Central Florida” and incorporates Begemann’s CPT data and indicates zones of common soil type. It also presents boundaries for loose and dense sand and consistency (undrained shear strength) of clays and silts, which are imposed by definition and not related to the soil profile interpreted from the CPT results. Schmertmann (1978) chart presents the cone resistance as a plot against the friction ratio that is the data plotted against their inverse themselves.

Schmertmann (1978) states that the correlations shown in Fig.3.3 may be significantly different in areas of dissimilar geology. The chart is intended for typical reference and includes two warnings: “Local correlations are preferred” and “Friction ratio values decrease in accuracy with low values of q_c”. Schmertmann also mentioned that the soil sensitivity, friction sleeve surface roughness, the soil ductility, and the pore pressure effects can influence the correlation provided in the chart. Notwithstanding this criticism, the Schmertmann chart is still commonly applied “as it is” in North American practice.

3.5 Method of Searle (1979)

Searle (1979) method of soil profiling is based on using the data from the Dutch mechanical friction sleeve penetrometer. The chart is shown in Fig 3.4 and the soil is classified by plotting the cone resistance against friction ratio. The chart identifies all types of soils ranging from very loose gravel to stiff clay. It differentiates the gravel as loose, medium dense or dense. How ever with low value of cone resistance and friction ration the chart identifies the soil as a very sensitive soil.

3.6 Method of Douglas and Olsen (1981)

Douglas and Olsen (1981) were the first to propose a soil profiling chart based on tests with the electrical cone penetrometer. The chart shown in Fig. 3.5 is based on the Unified Soil Classification system to distinguish the soil types. The chart also indicates trends for liquidity index and earth pressure coefficient, as well as sensitive soils and “metastable sands”. The Douglas and Olsen chart envelopes several zones using three upward curving lines representing increasing content of coarse-grained soil and four lines with equal sleeve friction. This way, the chart distinguishes an area (lower left corner of the chart) where soils are sensitive or “metastable”. While in the Schmertmann chart the soil type envelopes curve downward, in the Douglas and Olsen chart they curve upward. Zones for sand and for clay are approximately the same in both charts.

3.7 Method of Jones and Rust (1982)

Jones and Rust developed the soil profiling chart shown in Fig.3.6, which is based on the piezocone using the measured total cone resistance and the measured excess pore water pressure mobilized during the advancement of the cone in the soil. The chart presents the excess pore water pressure plotted against the net cone resistance (total overburden stress subtracted from total cone resistance). The chart is interesting because it identifies also the density (compactness condition) of the coarse-grained soils and the consistency of the fine-grained soils. However, the suggestion that high negative pore water pressures (indicating dilatancy) could be measured in very soft clays is surely a result of an overzealous desire for symmetry in the chart. Vermeulen and Rust (1995) present a large number of data plotted using this chart (with slight modification in the axes).

3.8 Method of Robertson and Campanella (1983)

Robertson and Campanella (1983) proposed a chart based on cone resistance and the chart is shown on Fig 3.7. The chart classifies soils based on the cone resistance and the friction ratio and subdivided the soil clearly as sand, silty sand, sandy silt, clayey silt or clay. It gives a rough estimation of the type of soil. For large value of the cone resistance from CPT tests the chart identifies the soil as sand.

3.9 Method of Robertson et al. (1986)

Campanella and Robertson (1986) were the first to present a chart based on the results of the piezocone with the cone resistance corrected for pore pressure at the shoulder according to Eq. 3.1.

$$q_t = q_c + u_2(1-a) \tag{3.1}$$

where q_t = cone resistance corrected for pore water pressure on shoulder

q_c = measured cone resistance

u_2 = pore pressure measured at cone shoulder

a = ratio between shoulder area (cone base) unaffected by the pore water pressure to total shoulder area

The Robertson et al. (1986) soil profiling chart is presented in Fig. 3.8. The chart identifies the numbered areas which separate the soil types in twelve zones.

A novel feature in the profiling chart is the delineation of Zones 1, 11, and 12, representing somewhat extreme soil responses thus enabling the CPTu to uncover more than just soil grain size. The rather detailed separation of the in-between zones, Zones 3 through 10, indicates a gradual transition from fine-grained soil to coarse-grained soil.
The Robertson et al. (1986) profiling chart introduced a pore pressure ratio, B_q, defined by Eq. 3.2, as follows.

$$B_q = \frac{u_2 - u_0}{q_t - \sigma_v} \quad \text{--------------------------------} \ (3.2)$$

where B_q = pore pressure ratio
u_2 = pore pressure measured at cone shoulder

u_q = in-situ pore pressure
q_t = cone resistance corrected for pore water pressure on shoulder
σ_v = total overburden stress

Directly, the Bq-chart shows zones where the u_2 pore pressures become smaller than the initial pore pressures (u_0) in the soil during the advancement of the penetrometer, resulting in negative Bq-values. Otherwise, the Bq-chart appears to be an alternative rather than an auxiliary chart; one can use one or the other depending on the preference. However, near the upper envelopes, a CPTu datum plotting in a particular soil-type zone in the friction ratio chart will not always appear in the same soil-type zone in the Bq-chart. Robertson et al. (1986) points out that "occasionally soils will fall within different zones on each chart" and recommends that the user study the pore pressure rate of dissipation (if measured) to decide which zone applies to the data under interpretation.

The pore pressure ratio, Bq, is an inverse function of the cone resistance, q_t. Therefore, also the Bq-plot represents the data as a function of their own self values, in conflict with general principles of data representation.

3.10 Method of Senneset et al. (1989)

Senneset et al. produced a soil classification chart based on plotting the corrected cone resistance, q_t , against the pore pressure ratio, Bq, as shown in Fig. 3.9. The chart is limited to the area where q_t is smaller than 16 MPa, i. e., the zone Robertson et al. (1986) denoted sensitive soil. It identifies limits of density and consistency (dense, stiff, soft, etc) that appear to be some what lower than those normally applied in North American practice, as for example, indicated in Fig 3.3. In comparing the chart to the Sanglerat chart shown in Fig. 3.2, it appears that the introduction of q_t and plotting against Bq, as opposed to Rf, avoids exaggerating the resolution in the clay region.

3.11 Method of Robertson (1990)

Robertson (1990) proposed a refinement of the Robertson et al. (1986) profiling chart, shown in Fig. 3.10, plotting a "normalized cone resistance", q_{cnrm}, against a "normalized friction ratio", R_{frnm} in a cone resistance chart. The accompanying pore pressure ratio chart plots the "normalized cone resistance" against the pore pressure ratio, B_q, defined by Eq. 3.2 applying the same B_q-limits as the previous chart (Zone 2 is not included in Fig.3.10).
The normalized cone resistance is defined by Eq. 3.3, as follows.

$$q_{cnrm} = \frac{(q_t - \sigma_v)}{\sigma'_v} \qquad \text{-----(3.3)}$$

Where q_t = cone resistance corrected for pore water pressure on shoulder
σ_v = total overburden stress
σ'_v = effective overburden stress
(q_t - σ_v) = net cone resistance

The normalized friction factor is defined as the sleeve friction divided by the net cone resistance, as follows.

$$R_{cnrm} = \frac{f_s}{q_t - \sigma'_v} \text{ ----------------------------- (3.4)}$$

where f_s = sleeve friction

The numbered areas in the profiling chart separate the soil types in nine zones (as shown in Fig. 3.10).

The two first ,and, the two last soil types are the same as those used by Robertson et al. (1986) and Types 3 through 7 correspond to former Types 3 through 10. The Robertson (1990) normalized profiling chart has seen extensive use in engineering practice (as has been the Robertson et al., 1986 chart).

The normalization was proposed to compensate for the cone resistance dependency on the overburden stress and, therefore, when analyzing deep CPTu tests (i. e., deeper than about 30 m) a profiling chart developed for more shallow tests does not apply well to the deeper sites. At very shallow depths, however, the proposed normalization will tend to lift the data in the chart and imply a coarser soil than is necessarily the case. Moreover, the effective stress at each depth is a function of the weight of the soil and, to a greater degree, of the pore pressure distribution with depth. Where soil types alternate between light soils and dense soils (soil densities can range from 1,400 kg/m^3 through 2,100 kg/m^3) and/or where upward or downward gradients exist, the normalization is unwieldy. For these reasons, it would appear that the normalization merely exchanges one difficulty for another.

For reference to the Begemann type chart, Fig. 3.11 shows the envelopes of the Robertson (1990) converted to a Begemann type chart. The ordinate is the same and the abscissa is the multiplier of the normalized cone resistance and the normalized friction factor of the original chart (the normalized sleeve friction is the sleeve friction divided by the effective overburden stress). Where needed, the envelopes have been extended with a thin line to the frame of the diagram. As reference to Figs. 3.4 and 3.6, Fig.3.11 also presents the usual Begemann type profiling chart converted from Fig.3.10 under the assumption that the data apply to a depth of about 10 m at a site where the groundwater table lies about 2 m below the ground surface. This chart is approximately representative for a depth range of about 5 to 30 m. Comparing the “normalized” chart with the “as measured” chart does not indicate that normalization would be advantageous.

3.12 Method of Eslami and Fellenius (1997)

Eslami and Fellenius **(1997)** developed a soil profiling method when investigating the use of cone penetrometer data in pile design. They compiled a database consisting of CPT and CPTu data associated with results of boring, sampling, laboratory testing, and routine soil characterization of soils from 18 sources reporting data from 20 sites in 5 countries. About half of the cases were from piezocone tests, CPTu, and include pore pressure measurements (u_2). Non-CPTu tests were from sandy soils and were used with the assumption that each u_2-value is approximately equal to the neutral pore pressure (u_0). Five main soil categories listed as below can be distinguished:

1. Sensitive and Collapsible Clay and/or Silt
2. Clay and/or Silt
3. Silty Clay and/or Clayey Silt
4. Sandy Silt and/or Silty Sand
5. Sand and/or Sandy Gravel

The data points were plotted in a Begemann (1965) type profiling chart and envelopes were drawn enclosing each of the five soil types. The envelopes are shown in Fig.3.12. The database does not include cases with cemented soils or very stiff clays, and, for this reason, no envelopes for such soil types are included in the chart.
Plotting an "effective" cone resistance defined by Eq.3.6 was found to provide a more consistent delineation of envelopes than a plot of only the cone resistance.

$$q_E = (q_t - u_2) \quad \text{----------------------------- (3.6)}$$

Where q_E = "effective" cone resistance
q_t = cone resistance corrected for pore water pressure on shoulder
u_2 = pore pressure measured at cone shoulder

The q_E-value was shown to be a consistent value for use in relation to soil responses such as pile shaft and pile toe resistances (Eslami 1996, Eslami and Fellenius, 1995; 1996; 1997). Notice that, as mentioned by Robertson (1990), the measured pore water pressure is a function of where the pore pressure gage is located. Therefore, the q_E-value is by no means a measurement of effective stress in conventional sense. Because the sleeve friction is a rather approximate measurement, no similar benefit was found in producing an "effective" sleeve friction. In dense, coarse-grained soils, the q_E-value differs only marginally from the q_t-value. In contrast, cone tests in fine-grained soils could generate substantial values of excess pore water pressure causing the q_E-value to be much smaller than the q_t-value.
Eslami and Fellenius (1996) proposed a pore pressure ratio, B_E, defined, as follows.

$$B_E = \frac{(u_2 - u_0)}{u_0} \quad \text{-- (3.7)}$$

where B_E = "Effective" pore pressure ratio

A diagram showing q_t versus B_E provides a more perceptible picture of the pore pressure induced by the cone and it does not violate the principles of plotting. The authors believe that research may show that the pore pressure ratio B_E will be useful for assessing liquefaction potential, degree of overconsolidation, and compressibility of sand and silt soils. It is also hypothesized that the B_E-ratio may show to be useful in predicting the magnitude of the increase (set-up) capacity of driven piles between initial driving and after the soils have reconsolidated.

The Eslami-Fellenius (1997) chart is simple to use and requires no adjustment to estimated effective stress and total stress. The chart is primarily intended for soil type (profiling) analysis of CPTu data. With regard to the boundaries between the main soil fractions (clay, silt, sand, and gravel), international and North American practices agree, but differences exist with regard to how soil-type names are modified according to the contents of other than the main soil fraction. The chart assumes the lower and upper boundaries for adjectives, such as clayey, silty, sandy to be 20 % and 35 %, “some” to mean 10 % through 20 %, and “trace” to mean smaller than 10 % by weight as indicated in the Canadian Foundation Engineering Manual (1985).

A soil profiling chart based on a Begemann (1965) type plot, such as the Eslami-Fellenius (1997) method can easily be expanded by adding delineation of strength and consistency of fine-grained soils and relative density and friction angle of coarse-grained soils per the user preferred definitions or per applicable standards. No doubt, CPTu test information from a specific area or site can be used to further detail a soil profiling chart and result in delineation of additional zones of interest. However, there is a danger in producing a very detailed chart in as much the resulting site dependency easily gets lost, leading an inexperienced user to apply the detailed distinctions beyond their geologic validity. Other early profiling charts were proposed by Searle (1979), Olsen and Farr (1986), Olsen and Malone (1988), Erwig (1988). CPTu charts are similar to that of Robertson (1990), Larsson and Mulabdic (1991), Jefferies and Davies (1991, 1993), and Olsen (??).

Concluding remarks

In –situ tests have long been used to substitute for boreholes by proper correlation. In this respect, the soil profiling methods are important while bearing in mind, it is important that boreholes are performed to confirm the profiles that can be established by CPT and CPTu tests. One of the important development is the use of the index tests such as liquid limit, plastic limit etc in fine grained soils to be used in combination with the CPT and CPTu tests in establishing the stress history of natural soil deposits and also the determination of the coefficient of horizontal consolidation. In the case of coarse grained soils, a suitable parameter similar to the relative density and to distinguish sand and gravel need to be established to compliment the interpretation with the CPT and CPTu tests.

Chapter 3 Figs

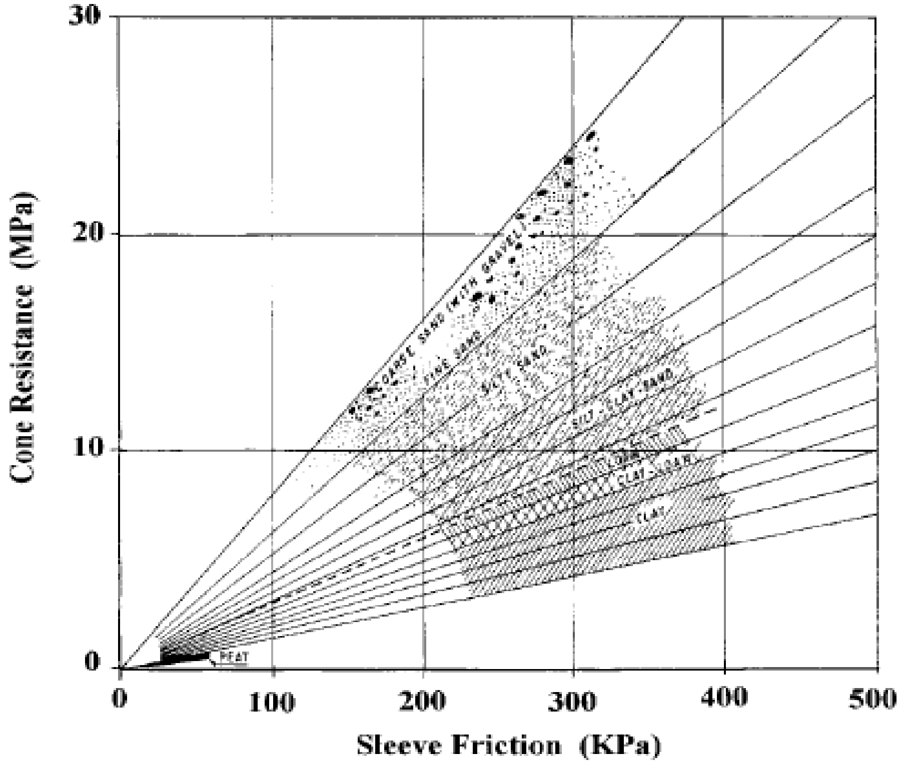


Figure 3.1: Soil profiling chart (Begeman, 1965)

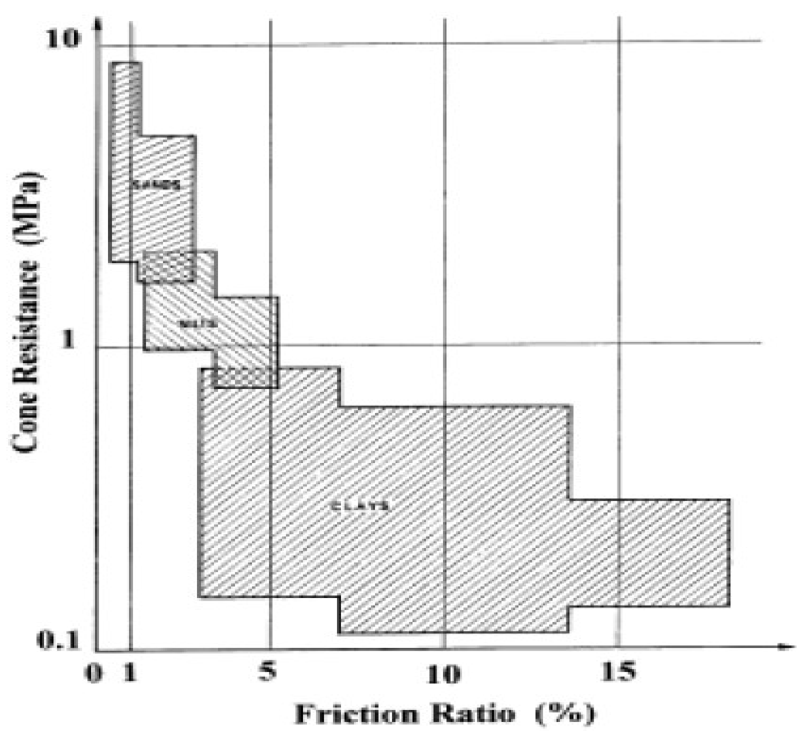


Figure 3.2: Soil profile chart (Sanglerat et al. 1974)

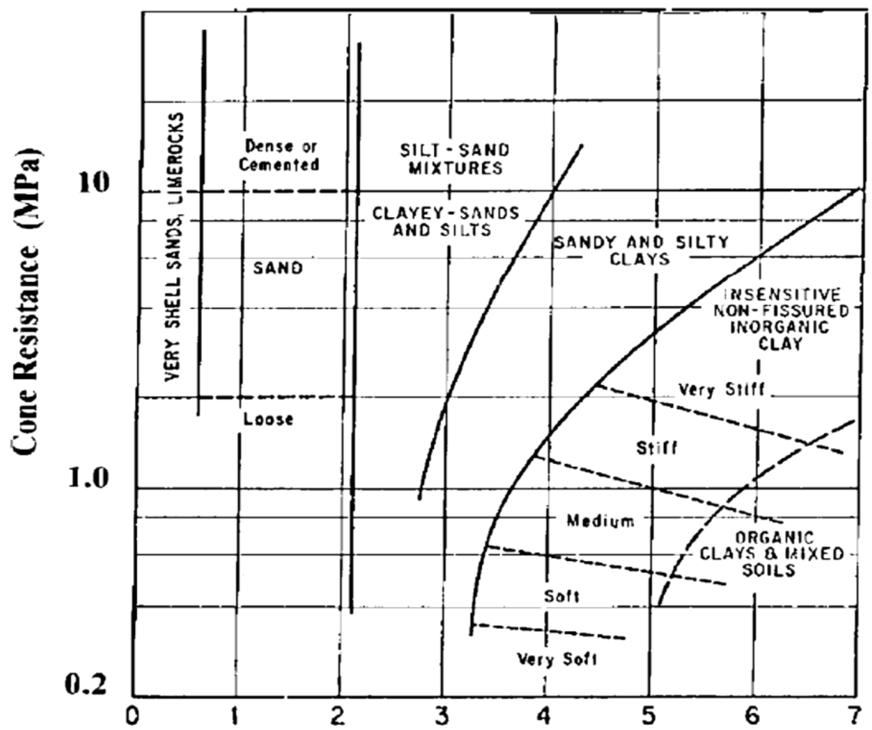


Figure 3.3: Soil profiling chart (Schmertmann 1978)

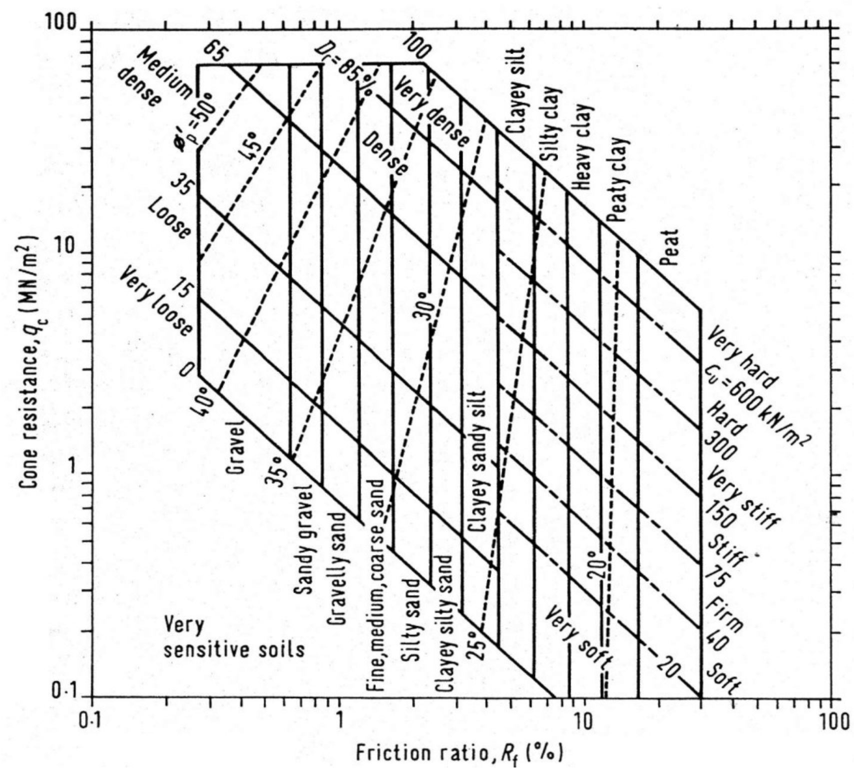


Figure 3.4: Soil profiling chart (Searle, 1979)

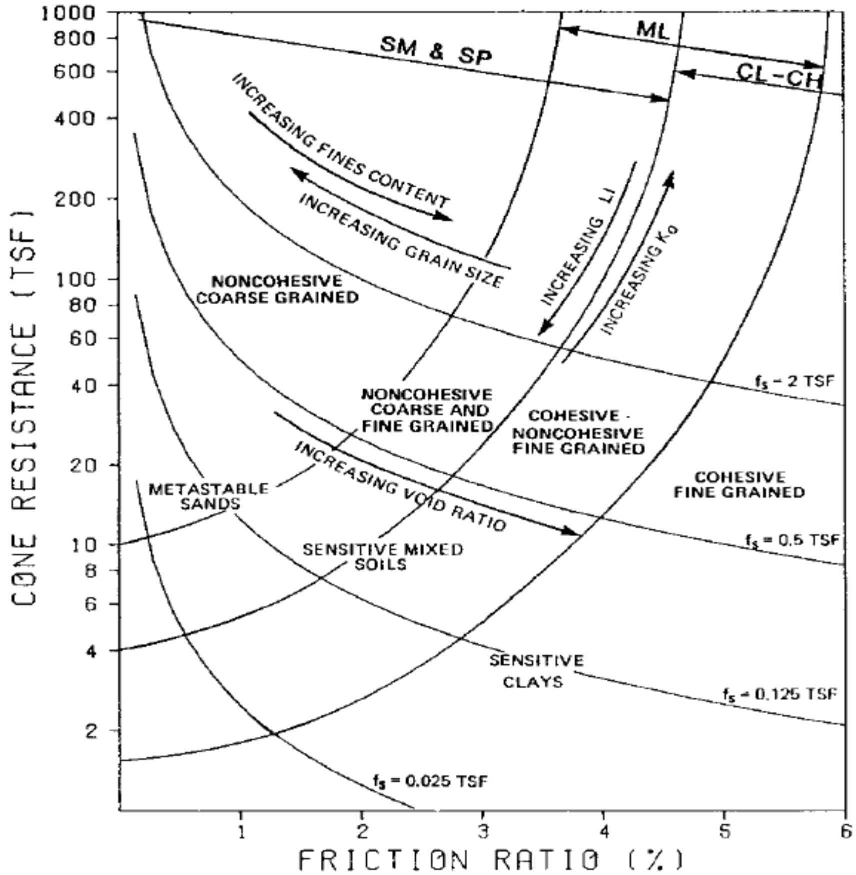


Figure 3.5: Soil profiling chart (Douglas and Olsen, 1981)

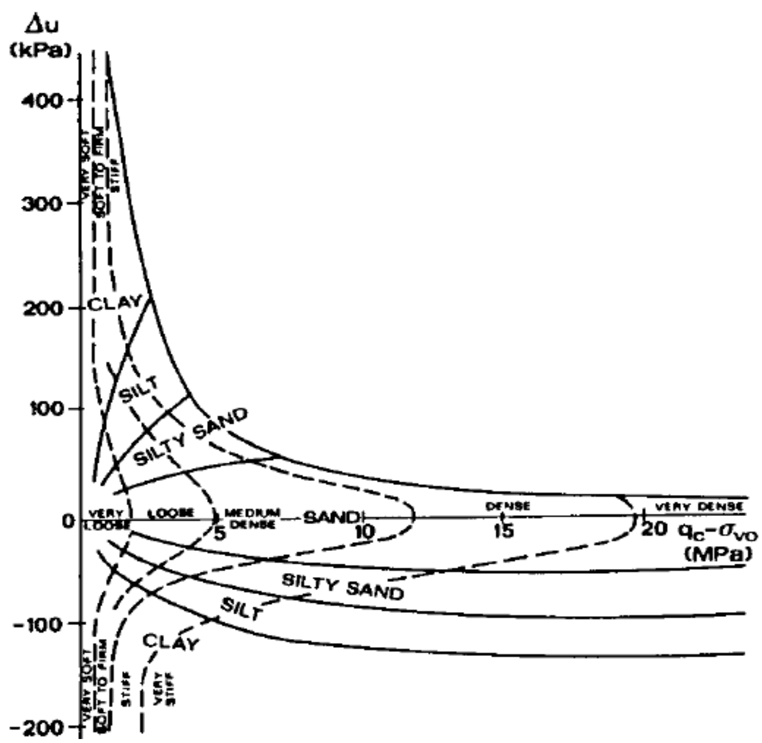


Figure 3.6: Soil profiling chart (Jones and Rust, 1982)

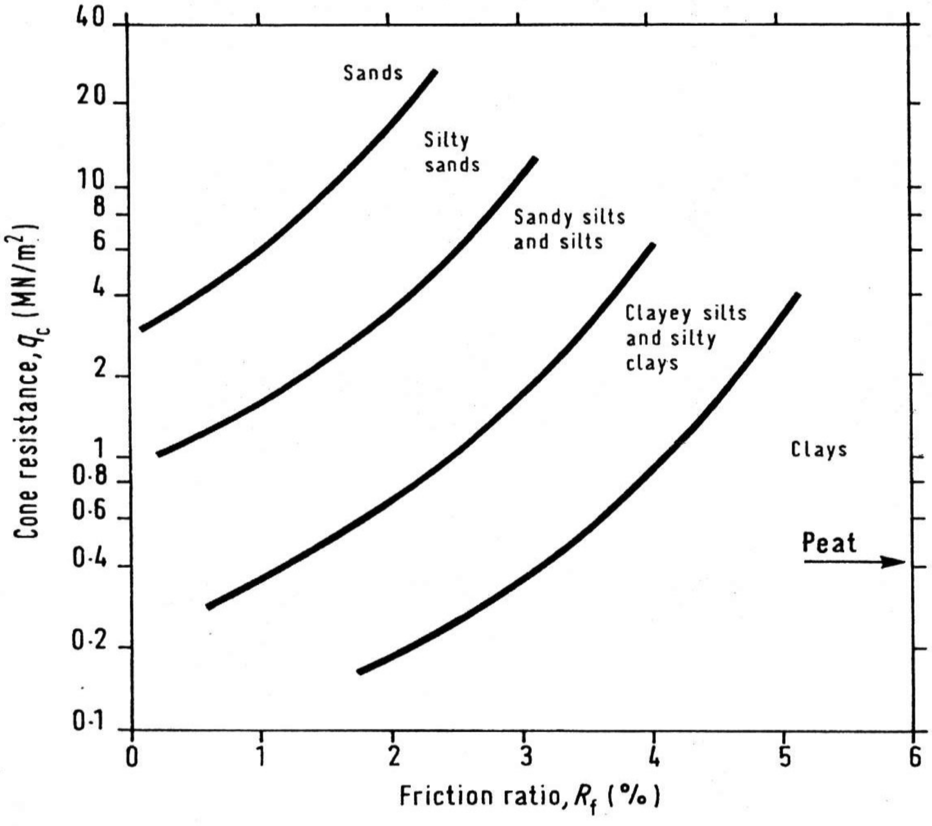


Figure 3.7: Soil profiling chart (Robertson and Campanella, 1983)

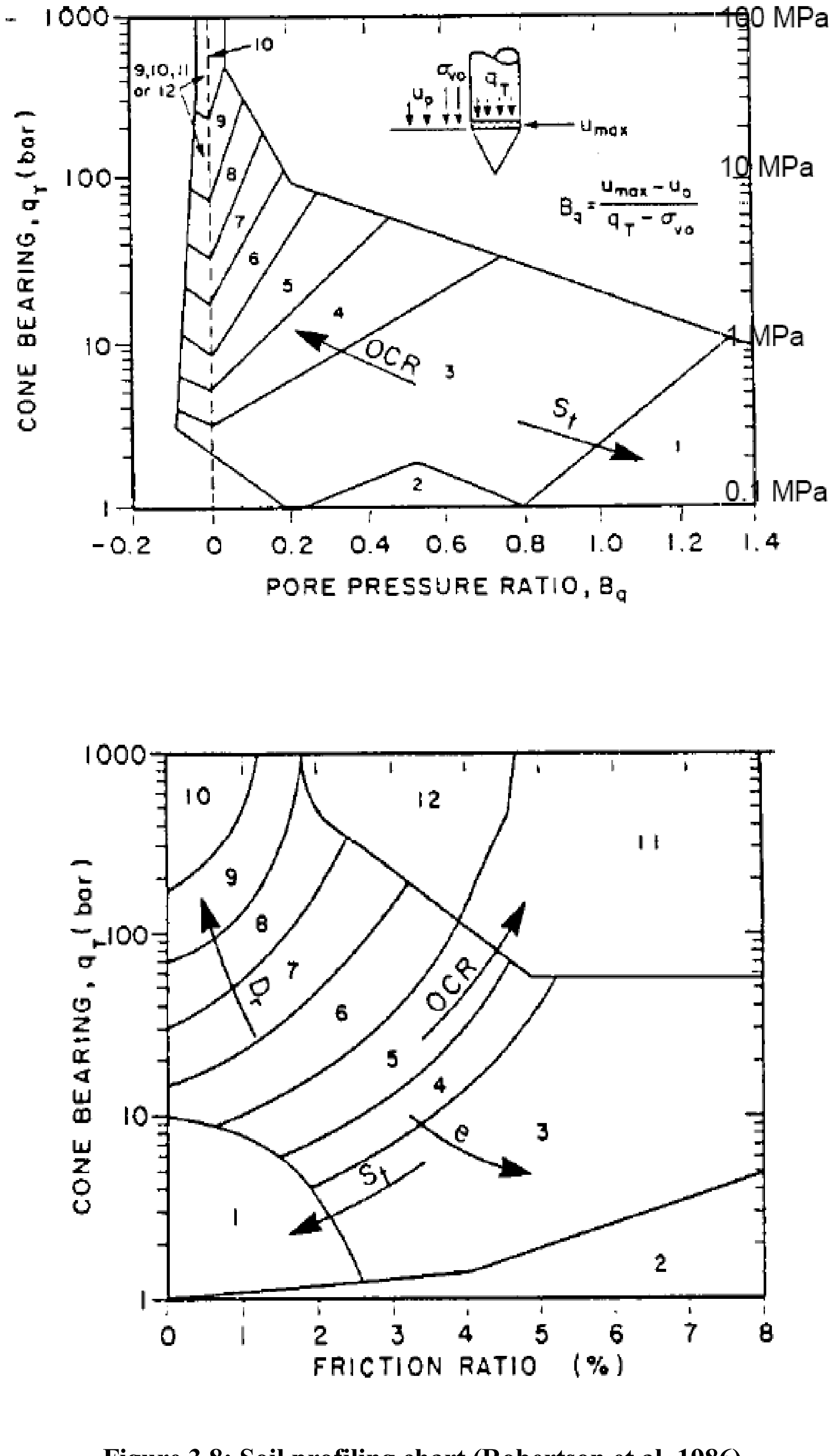


Figure 3.8: Soil profiling chart (Robertson et al.,1986)

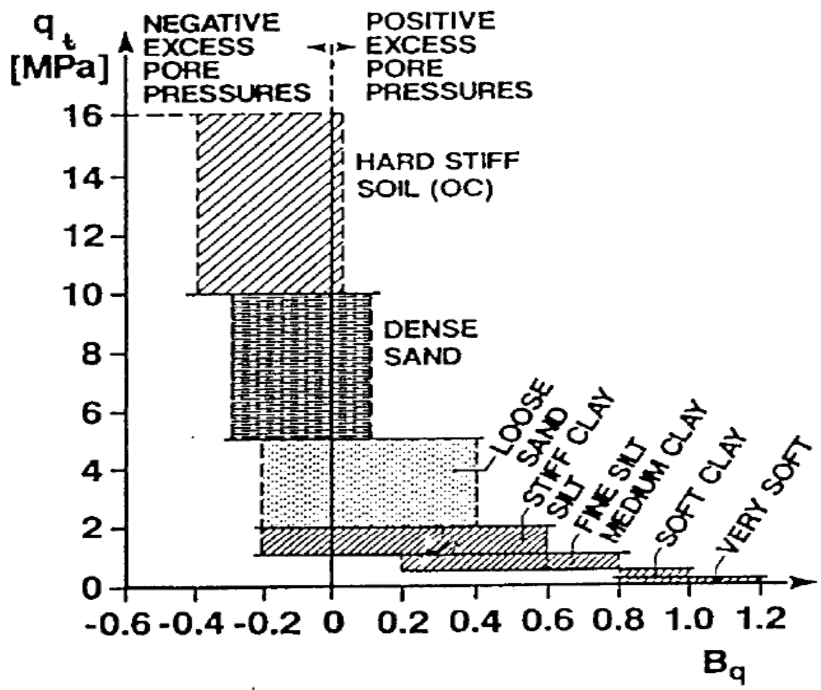


Figure 3.9: Soil profiling chart (Senneset et al., 1989)

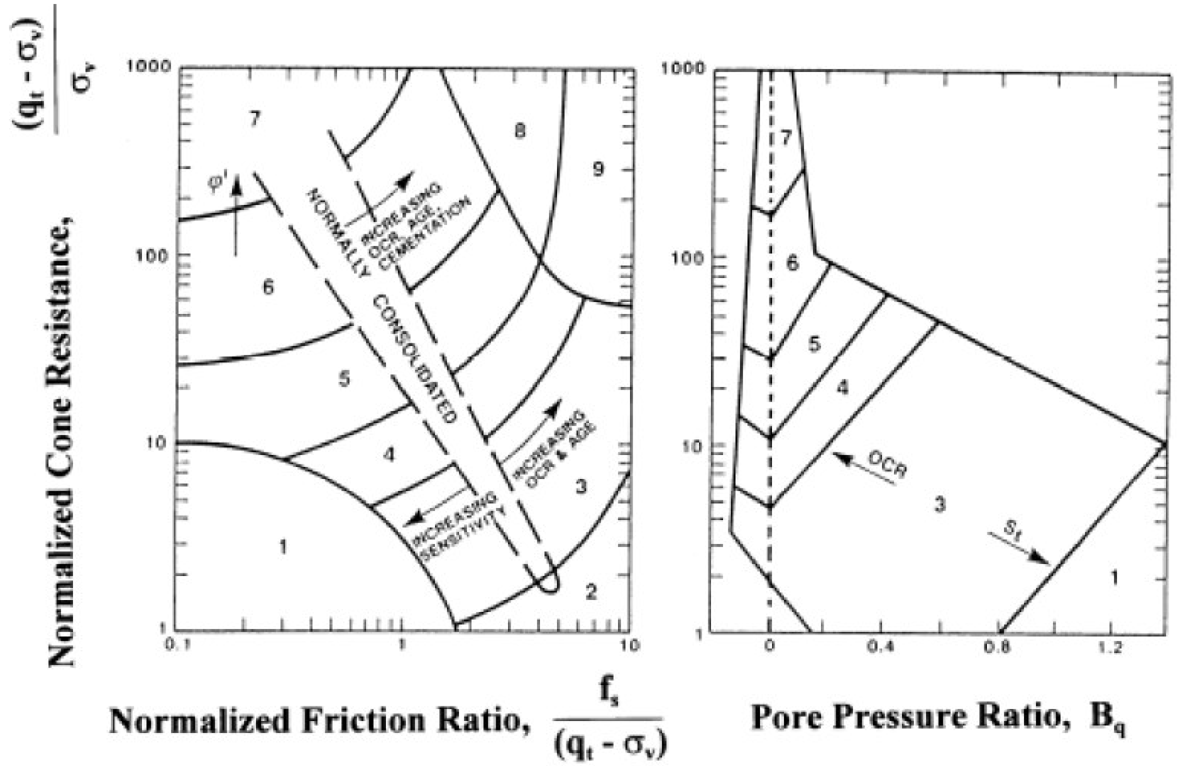


Figure 3.10: soil Profiling chart (Robertson, 1990)

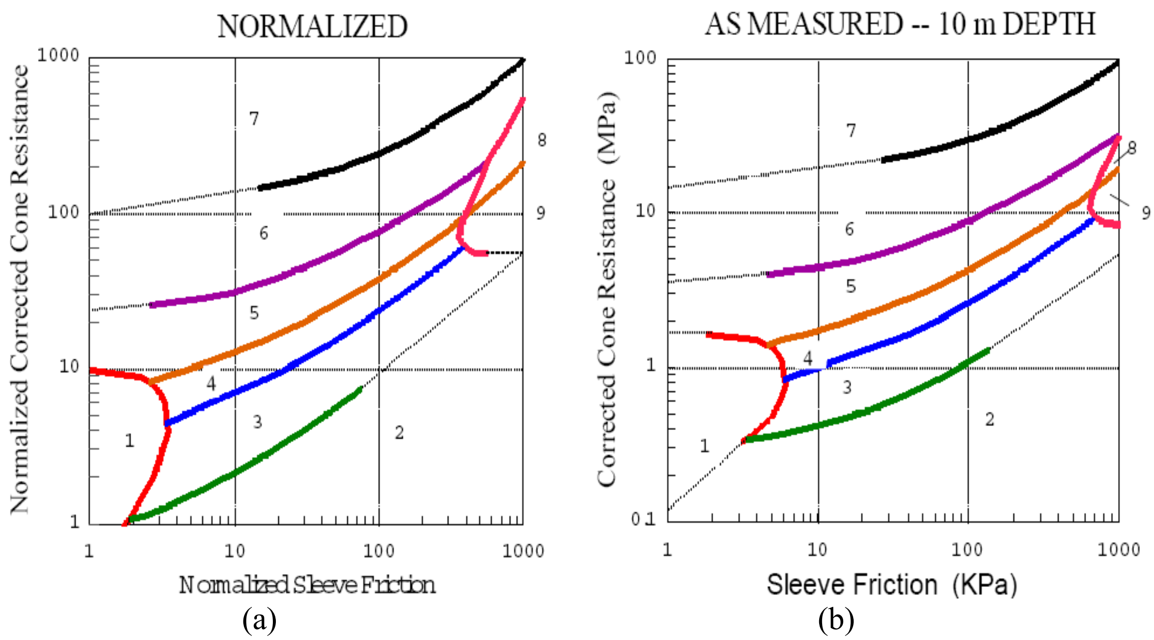


Figure 3.11: Soil profiling chart Robertson (1990) converted to Begemann type charts
a) Normalized cone resistance and sleeve friction
b) Corrected cone resistance versus sleeve friction

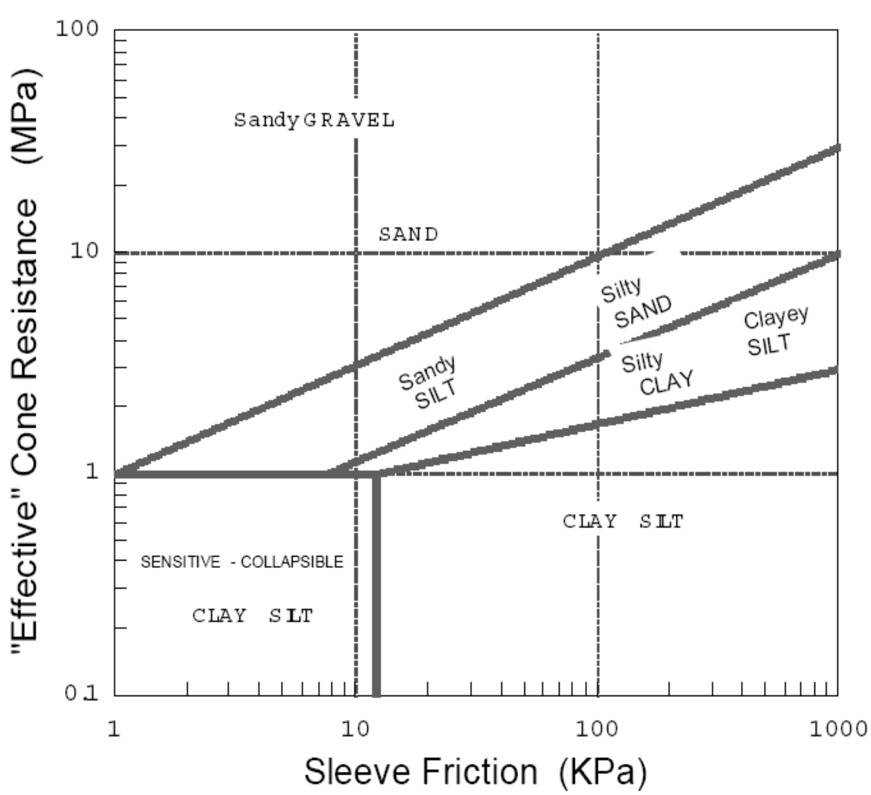


Figure 3.12 Soil profiling chart (Eslami-Fellenius)

CHAPTER 4

CONE PENETRATION TESTS AND FOUNDATION DESIGN

4.1 General Introduction

This chapter briefly summaries the application of cone penetration tests in both the shallow and deep foundation design. The estimation of the safe bearing capacity and the computation of settlements are presented both for foundations in sand and in clays.

4.2 Shallow foundations

As in all foundation design, it is necessary to consider both safe bearing capacity (i.e. the ultimate bearing capacity divided by a suitable factor of safety) and allowable bearing capacity related to tolerable settlements. The ultimate bearing capacity (and hence safe bearing capacity) can be calculated using the standard bearing capacity formulae (and the bearing capacity factors. For sand, values of the angle of internal friction can be obtained from correlations with the cone resistance. Except for narrow foundations on relatively loose sand, bearing capacity is seldom a problem, and the selection of an allowable bearing pressure is governed by settlement considerations.

Generally, the direct use of q_c in settlement calculations for foundations on sand is preferable to methods in which q_c is first converted to SPT blow count, N. However, there is a quick check method via SPT (Burland et. al. 1977) which is often useful to indicate the probable extent of a settlement problem. Suitable N value used for settlement prediction for both dense sand ($N > 30$) and medium dense sand ($10 < N < 30$).

In each case, the authors suggest that probable settlement can be taken as equal to half the upper limit value, and that maximum settlement does not normally exceed about 1.5 times the probable value. The upper limit for loose sands ($N < 10$) is regarded as tentative. Much of the data in the upper zone relates to very loose, slightly silty, organic sands, and the authors suggest that the upper limit values could be useful in the preliminary assessment of settlement of structures such as large oil tanks on loose sand.

Another method which has considerable merit, although it is indirect, is that of Burland and Burbidge (1985). This method is derived from an extensive review of case histories, mostly based on SPT but including some where the CPT was used.

A rapid conservative estimate of settlement of a footing on sand can be directly obtained from q, using the relationship proposed by Meyerhof (1974):

$$s = \frac{p_n B}{2\bar{q}_c} \quad \text{------------------------} \quad (4.1)$$

where

p = Applied loading

$\bar{q}_c$ = Average value of q over a depth equal to the footing width, B

s = settlement.

This is roughly equivalent to using a Young's modulus, E= 1.5q, compared with Schmertmann's value of 2.5q.

For footings and rafts in clays, the ultimate (and hence safe) bearing capacity can be calculated from undrained shear strength, s_u using standard formulae (e.g. Skempton, 1951). This requires values of N_k for use in the equation:

$$q_c = N_k . s_u + \sigma_{vo} \text{ -------------------------------(4.2)}$$

The value of N_k to be adopted is influenced by the factor of safety to be used, and other considerations. In normaly consolidated clays, a probable value for N_k is 15, and a more cautious value is 19. This may be very conservative in the case of clays of high sensitivity.

In De Ruiter and Beringen (1979) method for clays, the first step is to compute the undrained shear strength s_u from cone resistance q_c. Then pile side friction and end bearing are computed by applying suitable multiplying factors to s_u.
Regarding sands, de Ruiter and Beringen have found that the pile end bearing is governed by the cone resistance over a zone of 0.7 to 4 pile diameters below the pile tip. The procedure to compute q_p is depicted in Figure 4.2.

4.3 Pile Foundation

The ultimate bearing capacity of a pile, Q, is the sum of the ultimate end-bearing capacity, Q_b , and the ultimate shaft resistance, Safe bearing capacity, is then calculated by applying a factor of safety to Q_s or separate factors of safety to the components Q_b and Q_s. Q_b, dominates in sands, and Q_s in clays (except for the case of short piles with an enlarged base). Allowable bearing capacity depends on the settlement which can be tolerated.

4.3.1 Bearing capacity of Driven piles in sand

The methods described below are applicable to mainly quartz sands. They are not directly applicable to gravels, because the bearing capacity of a pile in gravel is less than indicated by cone resistance. The methods have only limited applicability in carbonate sands.

4.3.1.1 Ultimate end bearing in sand

In a uniform deposit of sand, below a certain depth a parallel-sided displacement pile achieves an ultimate bearing capacity equal to the cone resistance.

$$Q_b = q_p . A_b \text{-----------------------------(4.3)}$$

Where

$$q_p = \frac{q_{c1} + q_{c2}}{2} \text{ ------------------------------- (4.4)}$$

Figure 4.2 Procedure for determining composite cone penetration test value in evaluation of pile end-bearing capacity (after Schmertmann, 1978; and Heijnen. 1974)

Figure 4.2 Limit values of ultimate pile end-bearing capacity (de Ruiter and Beringen 1979)

The depth below which this occurs is known as the critical depth. It vanes with soil stiffness (and possibly with pile diameter), and it ranges between 4 and 20 pile diameters, the critical depth increasing with increasing soil stiffness. A typical value of 8 is often adopted. Sand deposits are seldom uniform, and, in practice, it is necessary to derive a composite q_c value, q_p , to take account of the variation of q above and below the pile toe, and a procedure for doing this (Heijnen, 1974) is shown in Figure 4.1. In evaluating q_{c2}, trials are made with a number of depths, below pile toe, between 0.7d and 4d, and the lowest resulting q_{c2} is adopted. Typical Dutch practice in assessing q is to limit the value of q used (normally to 30 MN/m2), and to limit the ultimate end-bearing capacity to a value not exceeding 15 MN/m2, which depends on OCR as shown in Figure 4.2 (Kemp, 1977). Some further reduction may be required if weaker layers exist between 4d and ld below pile toe level.

4.3.1.2 Ultimate shaft resistance from local side friction

Shaft resistance can be calculated from values of local side friction, f_s. However, measurement of cone resistance is often more accurate and more easily interpreted than that of f_s, so that shaft resistance is frequently based on q rather than on f_s. Shaft resistance for a driven pile in sand depends not only on the properties of the sand, but also on the extent to which the density of the sand is modified by pile driving. The ultimate shaft resistance is given by:

$$Q_s = \sum_0^L q_s \pi d \Delta L = S_1 \sum_0^L f_s \pi d \Delta L \text{ ----------------------- (4.5)}$$

where

q_s = ultimate shaft resistance
L = length of pile in the sand
S1= depends on the type of pile

4.3.1.3 Ultimate shaft resistance from cone resistance

Ultimate shaft resistance is given by:

$$Q_s = \sum_0^L q_s \pi d \Delta L = S_2 \sum_0^L f_s \pi d \Delta L \text{ -----------------------------(4.6)}$$

Schmertmann (1978), suggests values of S_2.

In Nottingham's method, with a continuously tapered or step-tapered pile, the pile length is divided into appropriate increments of constant-diameter length having the same total perimetral area, and the same procedure is used as for a parallel-sided pile. However, the k, applicable to each constant-diameter length is determined by using the l/d ratio at the bottom of each such length. Also, at each step of a step-tapered pile or at each imaginary step of a continuously tapered pile, an 'additional side friction' is assumed, equal to the average value of q, over the length, multiplied by the horizontal area of the step and a factor S3, as given in Table 8. If a step occurs within the upper length of 8d, the additional side friction for that step is multiplied by l/8d.

Figure 42 Nottingham's factor for calculating ultimate shaft resistance of a driven pile in sand (after Schmertmann, 1978)

4.3.1.4 Bearing capacity — the Poulos and Davis method

Poulos and Davis (1980) present a method of calculating the ultimate bearing capacity of a pile in sand which uses an idealized distribution of effective vertical stress adjacent to the pile, o, in which a is assumed to be equal to the effective overburden pressure at some critical depth.

Figure 45 Bearing capacity factor plotted against angle of shearing resistance (after Berezantzev. Khristoforov and Golubkov. 1961).

Figure 46 Shaft friction of piles in sand (after Poulos and Davis. 1980).

4.3.1.5 Settlement of piles in sand

At present, there is no direct method of calculating the settlement of a pile from CPT data. However, there are some indirect methods.
An approximate estimate of the settlement of a single pile in sand can be obtained from Meyerhof's (1959) equation:

$$s_1 = \frac{d_b}{30F} \text{ --------------------------- } (4.7)$$

where d_b = diameter of pile base
F = factor of safety on ultimate load (>3)

This may be sufficient in many cases, but if a more detailed analysis is required it is necessary to determine values of Young's modulus, E, and Poisson's ratio, v. Poisson's ratio for sands is usually between 0.25 and 0.35, and a value of 0.30 can be adopted without significant error. However, driving a pile into loose or medium dense sand gives rise to local increase in relative density and Young's modulus, so that values of E relevant to the settlement of a single pile may be higher than would be determined from Figure 17 (Section 5.4). Hence the resulting estimate of settlement is conservative.
Description of the methods of analysis is outside the scope of this Report, and reference should be made elsewhere (e.g. Poulos and Davis, 1980). These authors also describe methods of estimating settlements of pile groups in sand, for which a value of E is again required. Another useful reference is Vesic (1977).
Empirical relationships between settlement of a group of driven piles in sand and the settlement of a single pile are given by Skempton (1953) and by Meyerhof (1959).

4.3.2 Piles in clay

Although methods are available for calculating the bearing capacity of piles in clay in terms of effective stress parameters (e.g. Burland, 1973), it is more usual to use the undrained shear strength, c_u. This can be obtained from CPT results. At present, there are no commonly adopted procedures for determining pile bearing capacity in clay direct from CPT results, and other methods are preferred.

If CPT results are used to obtain values of c_u for calculation of the bearing capacity of piles in overconsolidated clay, it should be remembered that the values derived by the methods of 6.3 relate to shear strength back calculated from plate-loading tests. For end-bearing calculations, it is suggested that the derived values are directly used, without the customary reduction in ç made to allow for overestimation of c_u, measured on small laboratory specimens. However, for calculation of shaft resistance, where empirical values of the adhesion factor are based on undrained shear strength determined on small laboratory specimens, c_u, (PET) values derived from q have to be converted to laboratory values.

Figure 4.4: Analytical model for evaluating OCR from type piezocone data in clays.

Concluding remarks

In the earlier times, STP values N are used extensively in Foundation design. The CPT and CPTu tests, offered a means by which the penetration resistance can be separated as the end bearing cone resistance and the side friction. Undoubtedly, scaling factors need to be established when these data are used in the estimate of the capacity of piled foundations? In Southeast Queensland dynamic cone penetration tests are used very widely and it would be interesting to follow an approach similar to the CPT and CPTu tests in performing dynamic penetration tests in calibration chambers and to compare their performance with CPT and CPTu tests.

Chapter 4 Figures

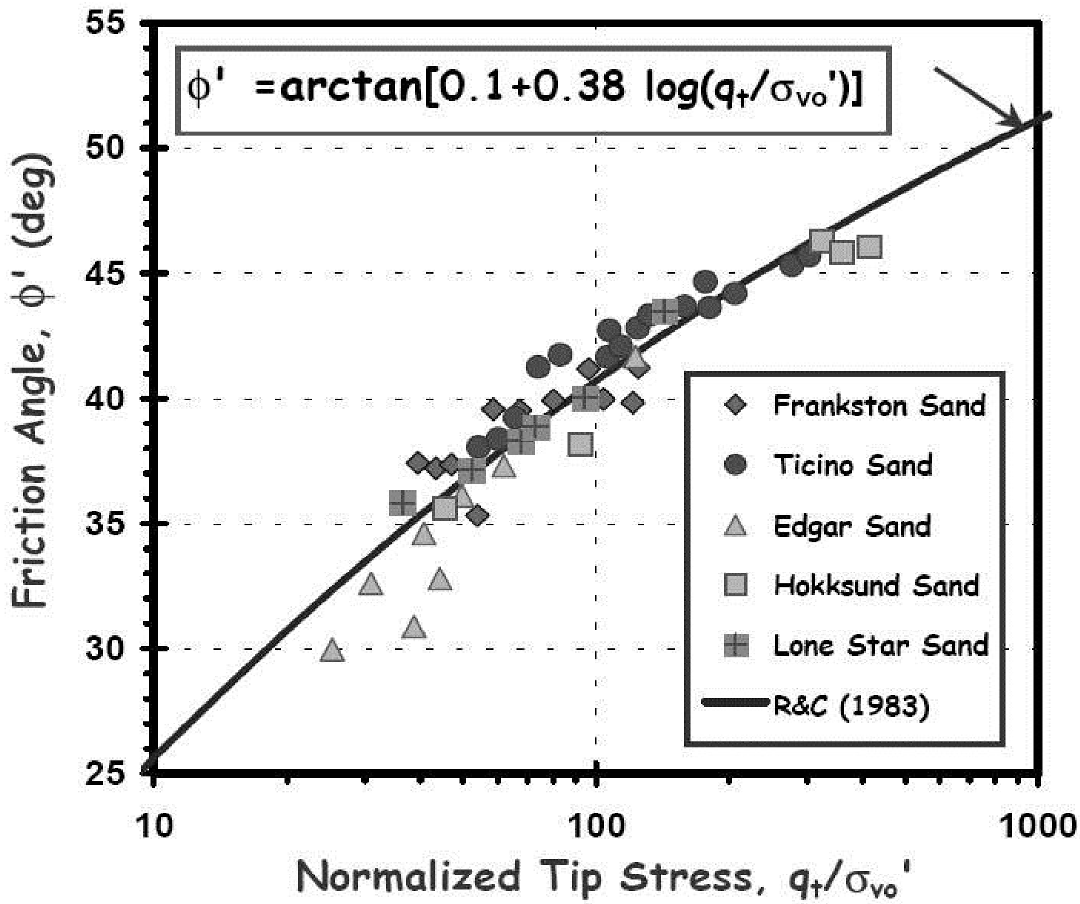


Fig. 4.1 Correlation of the angle of internal friction with the ratio of the cone tip resistance to effective overburden ratio

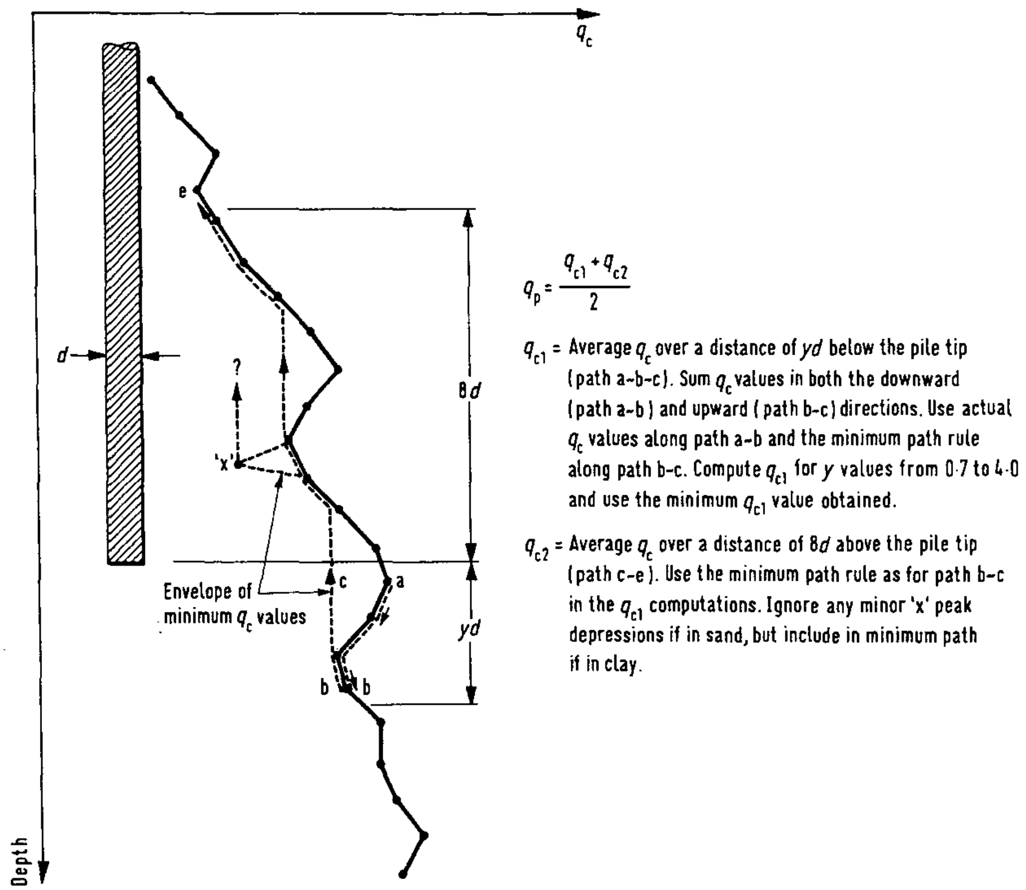


Fig. 4.2 Pile end bearing capacity determination from cone tests (After Schmertmann, 1978 and Heijnen, 1974)

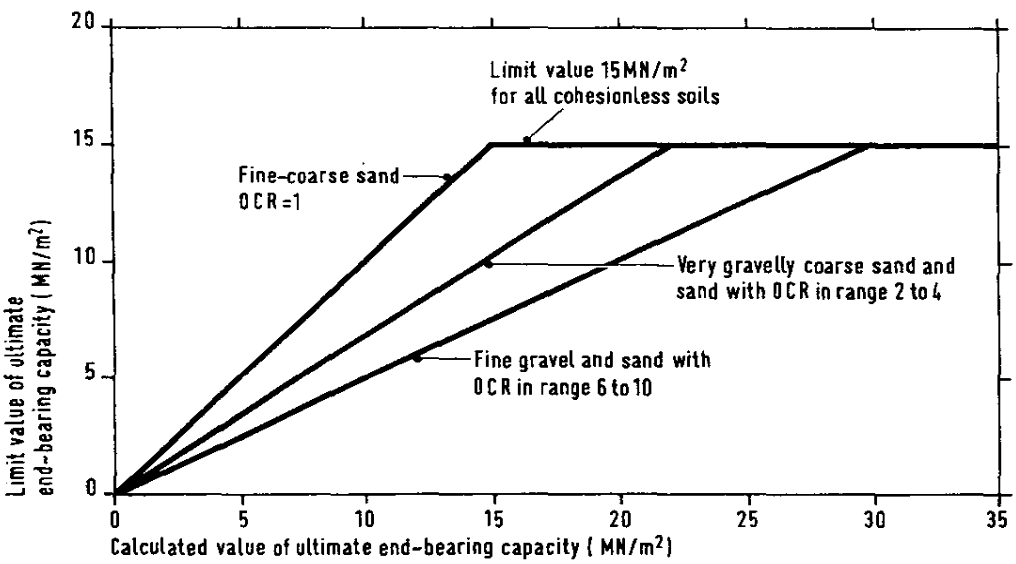


Fig. 4.3 Limit values of ultimate bearing capacity (de Ruiter and Beringen)

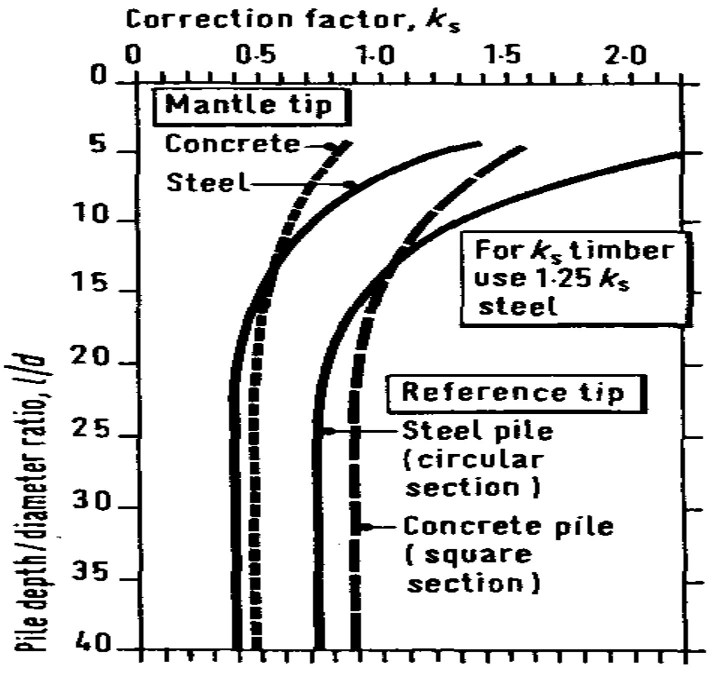


Fig. 4.4 Correction factor for ultimate shaft resistance of driven piles (After Schmertmann, 1978)

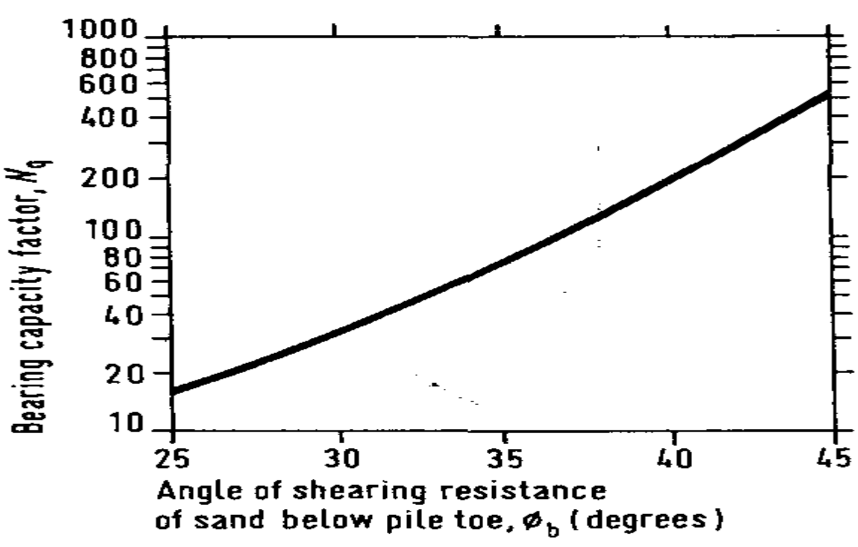


Fig. 4.5 Variation of bearing capacity factor, N_q with angle of internal friction (After Berezantev et. al., 1961)

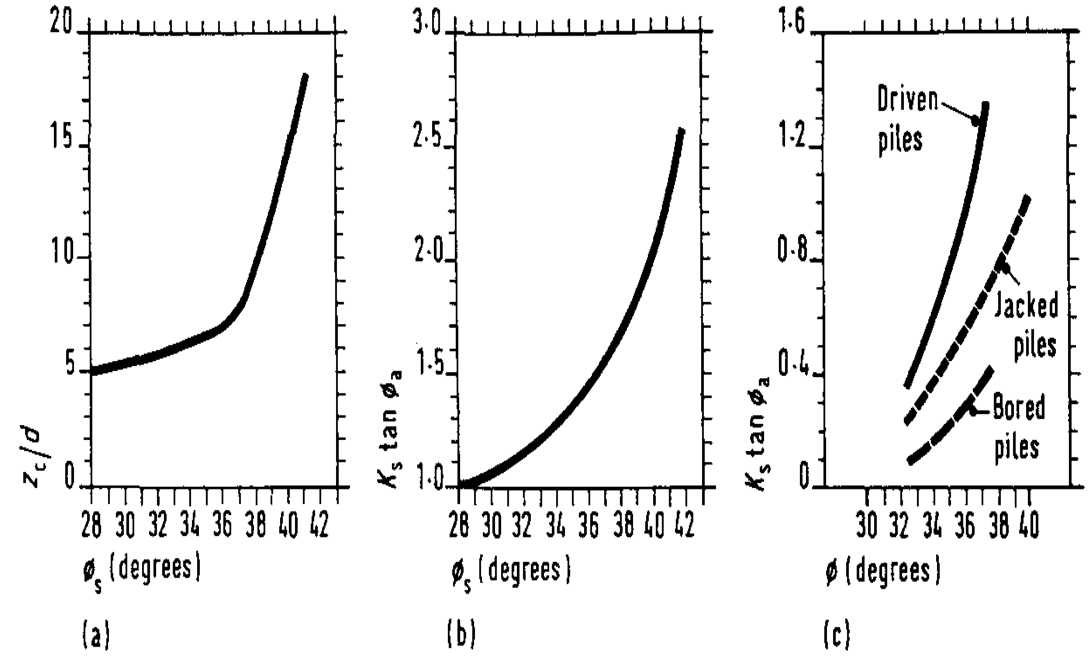


Fig. 4.6 Shaft friction of piles in sand (after Poulos and Davis, 1980)

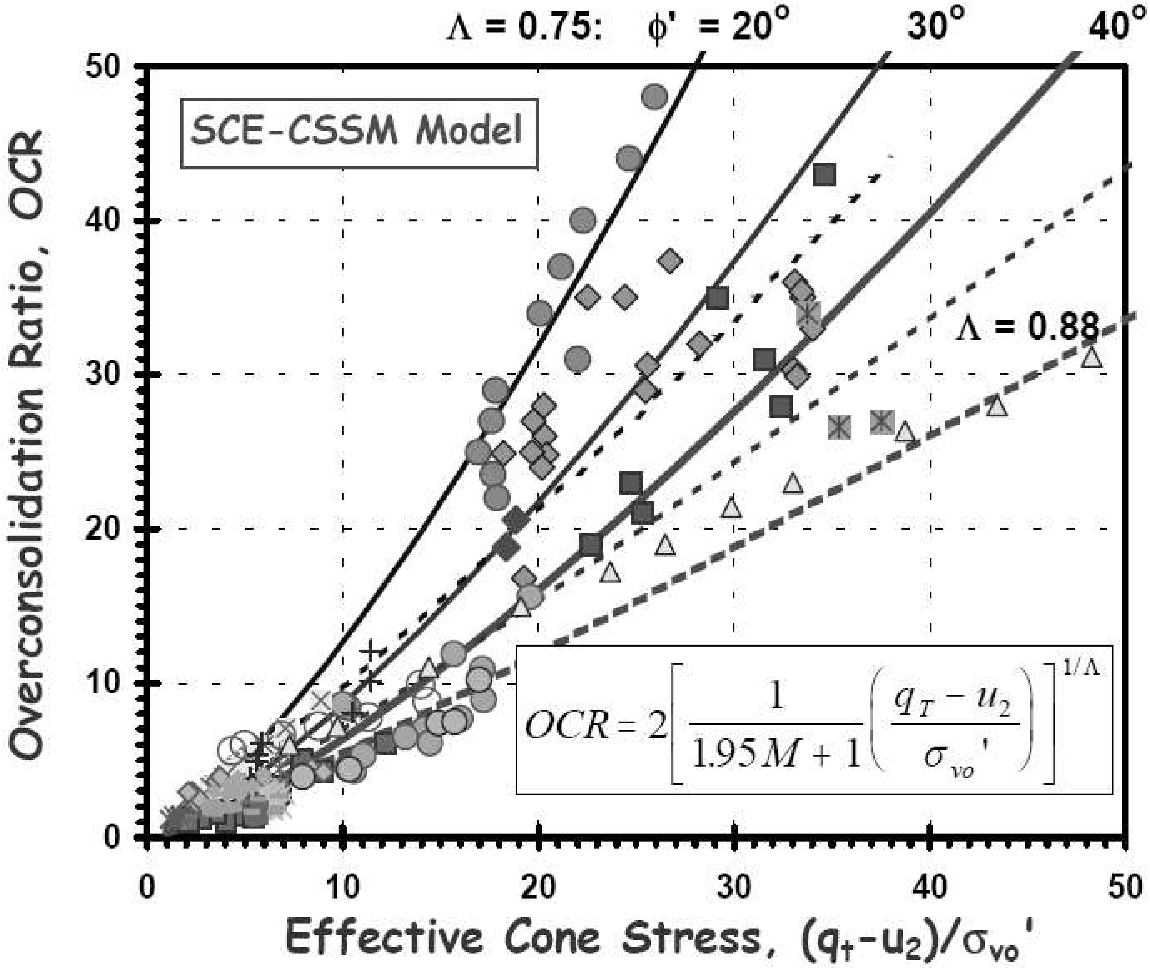


Fig. 4.7 OCR values as a function of $(q_t - u_2)/\sigma_{v0}'$

CHAPTER 5

INTERPRETATION OF CPTu TEST DATA AND ITS APPLICATIONS

5.1 GENERAL INTRODUCTION

The CPTu is the latest development of penetration testing and it is widely used in Southeast Queensland for site investigation works on major projects. At Gold coast, the CPTu measured cone resistance and skin friction are directly used in foundation design. Thus the use of CPTu data in foundation design is presented in Chapter 4. Often the angle of internal friction is estimated for sand from CPTu tests and in clays the undrained shear strength, the overconsolidation ratio and the in-situ horizontal coefficient of consolidation are measured. Due to the lack of time this report do not deal with the estimation of the various deformation moduli both for sand and clays. Various methods developed for soil profiling using CPT and CPTu are presented in Chapter 3.

In this chapter the interpretation of the CPTu tests data from two sites, one at Broad Beach, Gold Coast and other at the Port of Brisbane are presented. At the Broad Beach site 21 CPTu tests are performed to a depth of 20m, while 3 tests were interpreted at Port of Brisbane where the tests were conducted to a depth of 15m. At Broad Beach the sub-soil is mainly sand with a peat layer while the predominant sub-soil at the Port of Brisbane site is soft clay. At the Broad Beach site, soil profiling is carried out using the CPTu data and the angle of internal friction of the sand is also estimated. While for the soft clay at Port of Brisbane, the undrained shear strength, the coefficient of consolidation and the overconsolidation ratio are estimated.

5.2 BROADBEACH SITE AT GOLD COAST

Figure5.1shows the location of the CPTu test at this site. In order to explain the subsoil conditions, three cross sections are used along AA', BB' and CC'. Along AA' ten tests were incorporated while along BB' and CC' the number of tests are 14 and 18 respectively. The charts of Robertson and Campanella (1983), Douglas and Olsen (1981) and Searle (1979) were used in the interpolation of the subsoil profiles. Typical cone resistance (q_c), skin friction (f_s) excess pore water pressure and the friction ratio as measured at the site are shown in Figure5.2 (a) – (c).

5.2.1 Soil profiling at Broad Beach

A typical Table prepared in relation to the soil profiling for one cone test is shown in Table5.1. In this Table, the measured cone resistance, friction ratio and the pore pressures are tabulated. Also given are the type of soils as obtained from the methods of Searle (1979), Douglas and Olson (1981) and Robertson and Campanella (1983). Similar Tables for all the other CPTu tests are included in Appendix A. Plots of the soil profiles are given in Figs.5.3 (a) to (i) for all three sections by all three methods. In Fig.5.3 (a) a surface silt layer and a thin layer of interbedded silt layer are also revealed as per the method of Robertson and Campanella in Section AA'. This is not revealed in the plot of Douglas and Olson in Fig.5.3 (b) in Section AA'. The method of Douglas and Olson classifies the sand with USCS as SM and SP, that is silty sand and poorly graded sand. This type of identity is missing in the method of Robertson and Campanella. Also, in Fig.5.3 (b), the method of Douglas and Olson, suggest that a predominant part of the soil is non-cohesive coarse grained, implying that it can be sand and gravel as well. This phenomenon is clear in the plot of Searle in Fig.5.3 (c), where medium dense sand, gravel, sandy gravel and gravelly sand are revealed. Also, the presence of a layer of very sensitive soil is also indicated in the plot in Fig. 53 (c). The plots for Section BB' (Figs.5.3 d to f) are very similar to AA' as they both run parallel to each other. In Section CC' which is virtually perpendicular to AA' and BB' a thin layer of peat is noted at a depth of about 6m (see Fig.5.3 g)and such a peat layer is found to be troublesome in foundation practice in Gold Coast along the Surfers Paradise and other zones. Thus the method of Robertson and Campanella is possible to identify even such thin layers of peat interbedded between the predominantly sand and gravelly sand layers. The same is revealed in Fig.5.3 (h) of the Douglas and Olson plot.

Angle of internal friction φ' for Broad Beach sub-soils

.The angle of internal friction for the Broad Beach sand is estimated from the correlation with the normalized tip resistance as presented in Fig.4.1. From the measured values of the total cone resistance and the effective overburden pressure, the computed angle of internal friction is presented in Table 5.4 (a) and the variation is also presented below this Table in the form of a graph. The angle of friction ranges from 42 to 50 degrees except at the surface where it is about 38 degrees. This would imply that the sand and gravel at Broad Beach is dense to very dense. These values are very consistent for Sections BB' and CC' as given in Tables 5.4 (b) and (c). While in most foundation design as summarized in Chapter 4, the directly measured cone resistance and sleeve friction are used, it is also possible to estimate the bearing capacity factors for shallow and deep foundations from the estimated angle of internal friction.

Piezocone results at the Port of Brisbane Motorway site in soft clays

The QDMR have kindly made available to the Griffith University, the large number of piezocone test data on deep soft clay layers from several sites and in particular the Port of Brisbane Motorway site. The Author was only able to interpret a few of them due to the very limited time available in this project work
A typical profile of the piezocone test data at the Port of Brisbane Motorway site is shown in Fig 5.4 (a) and (b). From the measured data, the pore pressure ratio B_q is also computed now, and interestingly these values approach zero at times to indicate the presence of thin drainage layers of sand and silt.

Undrained shear strength

For soft clays, undisturbed sampling is quite difficult and also strength reduction can take place when the sensitivity is high. It is still in practice to use the Skempton and

Henkel (1953) correlation of the undrained strength with plasticity index. This correlation is as follows

$$\frac{s_u}{p'_0} = 0.11 + 0.0037 I_p \text{ ------------------------------------- } (5.1)$$

Where I_p is the plasticity index and s_u, the undrained strength and p'_0, is the effective overburden pressure. More recent work by Jamiolkowsy et al, 1985 and Mesri (1989) makes similar correlation with the preconsolidation pressure, p'_c.

A simple expression for the undrained shear strength from cone penetration tests is given by

$$s_u = \frac{(q_t - p_0)}{N_{kt}} \text{ --------------------------------------- } (5.2)$$

Where

q_t is the cone resistance corrected for pore pressure effects, and p_0 is the total overburden pressure and N_{kt} is the cone factor. Values of the cone factor ranges very widely from 10 to 30, but site specific values need to be calibrated with other empirical formulae and vane strength. Indicates that an s_u of a quarter of the effective overburden pressure forms a lower limit of the undrained strength as estimated from the piezocone.

Estimation of the horizontal coefficient of consolidation, c_h

Typical pore pressure dissipation tests as conducted in the piezocone at the Port of Brisbane site is shown in Fig.5.6.The following data is pertinent for the calculation of the c_h values.

Static pore water pressure = 74 kN/m^2

Maximum excess pore pressure = 290 kN/m^2

Pore pressure at the end of dissipation test = 190 kN/m^2

Duration of dissipation test 150 minutes

Time for 50 percent dissipation is 165 minutes

c_h values estimated by the method of Teh and Houlsby () is 1 to 2 m^2/year.These values seem reasonable, but there are instances where the values determined from the piezocone are larger than a tenfold magnitude of the above c_h values.

Concluding remarks

The CPTu is the latest development of penetration testing and it is widely used in Southeast Queensland for site investigation works on major projects. The major application of CPTu is in foundation design where the measured cone resistance and skin friction can be directly used. CPTu tests are also used to obtain geotechnical parameters in sand and clays. In the case of sand, the relative density, angle of internal friction and various kinds of deformation moduli are correlated with the cone resistance. In the case of clays, undrained shear strength, overconsolidation ratio and deformation moduli are correlated with the cone resistance and skin friction. Also the coefficient of consolidation in the horizontal direction is evaluated from pore pressure dissipation test in the CPTu. Various methods are also available for soil profiling.

In this chapter the interpretation of the CPTu tests data from two sites, one at Broad Beach, Gold Coast and other at the Port of Brisbane are presented. At the Broad Beach site 21 CPTu tests are performed to a depth of 20m, while 3 tests were interpreted at Port of Brisbane where the tests were conducted to a depth of 15m.The following conclusions are reached.

1. At the Broad Beach site in Gold Coast, the soil is mainly coarse grained and predominantly sand with layers of gravely sand and sandy gravel. A layer of interbedded peat is also revealed. The charts of Robertson and Campanella (1983), Douglas and Olsen (1981) and Searle (1979) were used in the interpolation of the subsoil profiles. The angle of internal friction as obtained from the CPTu tests ranged from 42 to 50 degrees for the major part and indicated the sand and gravely sand is dense to very dense. The soil profiling by the method of Searle (1979) indicates the coarse grained soil as medium dense.

2. At the Port of Brisbane site, the undrained shear strength of the soft clay and the coefficient of consolidation were estimated. The ratio of the undrained shear strength to effective overburden pressure is 0.25 and correlates well with those estimated from the index tests. The coefficient of consolidation in the horizontal direction is 1 to m^2/year, but these values also became more than ten fold in some of the tests.

Chapter 6

Conclusions and Recommendations

Conclusions

Piezocones are now widely used in Southeast Queensland as a major in-situ test in site investigation works including soil profiling and estimation of geotechnical parameters both in cohesionless and cohesive soils. A detail study is carried out

on 21 piezocone tests carried out in sandy soils at Broad Beach, Gold Coast and three tests in soft clays at the Port of Brisbane Motorway site.

(1) At the Broad Beach site, the test reveals

- **(a) the sand deposit is dense to very dense in terms of the angle of internal friction**
- **(b) the measured cone friction and the sleeve friction are directly used in deep foundation consisting of driven and bored piles.**
- **(c) Soil profiling was carried out by three different methods and all of them indicated the sub-soil layer as medium dense sand and gravel. A layer of interbedded peat is also revealed. Still the soil profile estimated from piezocone tests need to be verified by borehole data**

(2) At the Port of Brisbane Motorway site

- **(a) a deep layer of compressible soft clay layer is encountered**
- **(b) The strength to effective overburden ratio is about 0.25**
- **(c) The coefficient of horizontal consolidation was found to be generally 1 to 2 m^2/year, but increase more than ten fold in many other tests.**

(3) A critical literature review was conducted with a view to estimate the geotechnical parameters from all the tests which are very large in number, but due to the limited time available, such a study needs to be curtailed to fit in with the available time.

RECOMMENDATIONS

(1) It is recommended further detail study be made of all the possible interpretations of the CPTu test data both from Gold Coast and Brisbane in a systematic manner as an extension of the work presented here. This is important as CPTu is now very widely used in Southeast Queensland in all the major projects as the in-situ test.

(2) Dynamic cone penetration tests are used also extensively in smaller and medium size projects. Thus the dynamic cone data can be calibrated with CPTu tests and additional research could be done with the dynamic cone penetrometer in pressure chamber tests in the laboratories , similar to the early work on CPT and CPTu tests.

REFERENCES MISSING IN MY FOLDERS