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USE OF CPTU DATA IN CLAYS/FINE GRAINED SOILS

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Abstract: The Piezocone test (CPTU) continues to gain in popularity as a result of its potential, particularly in clay soils. In order to have confidence in the interpretation of CPTU results in clay soils, it is vital that the test results are accurate and representative of the in situ soil conditions. This can be achieved by using equipment and procedures following the new International Reference Test Procedure (IRTP) (and in future the forthcoming ISO document on CPTU). A particular feature of the IRTP is the introduction of Accuracy Classes according to what the results are to be used for. This aspect is discussed in some detail. Testing on land is normally done by pushing from ground level until refusal or to a predetermined depth. A new and very interesting method whereby CPT's can be carried out while drilling is advanced facilitates continuous testing to large depths.

The rest of the paper covers the use of CPTU for engineering purposes in clays soils. The aspects that are dealt with are: layering and identification of soil type, stress history, lateral stress ratio, undrained and remoulded shear strength, small strain shear modulus, coefficient of consolidation and in situ pore pressure.

1. INTRODUCTION

In the authors' opinion that there is a clear and increasing trend for the CPT, and especially the CPTU, to be used is one of the more important and fundamental parts of site investigation and ground characterisation; not only as a profiling tool but also for deriving soil parameters for foundation design, even in soft clays. For example, as recently as 7–8 years ago the vane test was the main tool used in Norway for in situ measurement of undrained shear strength, the CPTU was only used occasionally. However, the current trend is now to use the CPTU for determining soil parameters in soft clays; the vane test is being used less and less. The main reasons for this change are thought to be: the continuous information obtained from the CPTU, the cost effectiveness and recent improvements in the operation and interpretation of the CPTU in terms of soil design parameters. In the UK, there is evidence that the reliance on the SPT is waning and the CPTU is continuing its gain in popularity.

In order to obtain reliable soil design parameters in clays, especially soft clays, from CPTU there are two aspects that need to be considered: firstly the test results must be accurate and representative of the ground conditions, and secondly sound and proven interpretation principles must be used. This paper will address these two aspects and make a summary of today's status with particular emphasis on recent developments.

The paper will cover some aspects of interpretation of the test results in terms of soil design parameters but will not cover direct use of the results for foundation design.

2. OBTAINING RELIABLE TEST RESULTS

2.1. THE INTERNATIONAL REFERENCE TEST PROCEDURE (IRTP)

In 1999, the International Society of Soil Mechanics and Geotechnical Engineering (ISSMGE) technical committee TC16 published a new International Reference Test Procedure (IRTP) which gave very specific requirements for equipment and procedures for the CPTU (ISSMGE [13]). In addition to the basic requirements, the document also includes a number of notes with advice on details of how to carry out and report the CPTU. Frequent reference will be made in the following to this document. The IRTP has also formed the basis of the new CEN standard on CPTU due for completion in the 2006; it is likely that this CEN document will later become an ISO standard.

2.2. EQUIPMENT

2.2.1. STANDARD PENETROMETER WITH PORE PRESSURE MEASUREMENTS (CPTU)

Figure 1 shows a standard cone penetrometer measuring cone resistance and sleeve friction and also pore pressure. The diameter of the standard 60-degree cone is 35.7 mm (giving a cross-sectional area of 1000 mm^2) and the area of the friction sleeve is 15000 mm^2 (these sizes form the standard sizes in the IRTP and CEN documents). Cone resistance and sleeve friction are normally measured via strain gauged load cells. Figure 1 shows three locations where the pore pressure can be measured on various cones, but the IRTP refers to the *preferred location* being just behind the cone, i.e. the u_2 position as shown. In most cases, the penetration pore pressure is measured through a filter which, in the u_2 position, should have the same, or slightly larger, diameter than the cone. The pore pressure measurement system consists of a small pressure chamber leading to a pressure transducer. The pore pressure measurement system should be designed in such a way that it is easy to saturate and to keep saturated. It should be stressed that the success of a CPTU profile depends very much on good saturation of the pore pressure measurement system particularly in clays (pore pressure is used not only to aid profiling but also for correcting other measured parameters as will be discussed later in this paper).

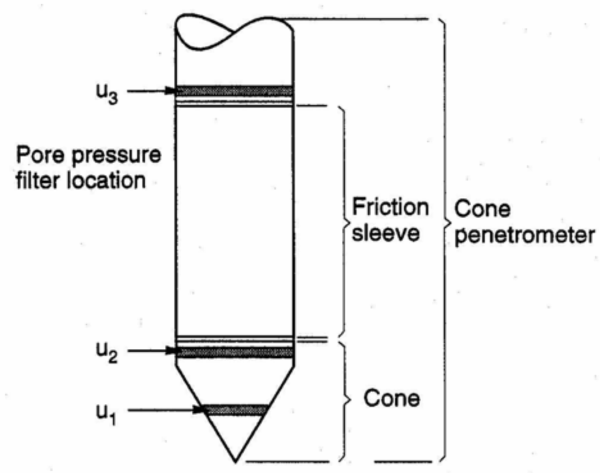


Fig. 1. Standard cone penetrometer
with pore pressure measurement (CPTU)

An alternative approach to using a filter is to use an arrangement called a slot filter. In this system, the pore pressure is measured by an open system with a 0.3 mm slot immediately behind the conical part (e.g. LARSSON [21]). Hence the porous element between the soil and the pressure chamber becomes redundant. The slot communicates with the pressure chamber, which is saturated with de-aired water, antifreeze liquid or other liquid, whereas the slot and channels are saturated with gelatine, silicon grease or similar. The use of a slot filter can reduce the time required for preparation of the probe. In addition, this pore pressure system also maintains its saturation better when passing through unsaturated zones in the soil. Despite the apparent simplicity of this system there are some indications that the pore pressure response is not always as good as the 'standard' type filter mentioned above.

The standard cone penetrometer should also measure inclination, the importance of which will be discussed later in this paper.

Figure 2 shows a typical profile of measured (cone resistance q_c, sleeve friction f_s, and pore pressure) and derived (friction ratio R_f) parameters from a CPTU profile.

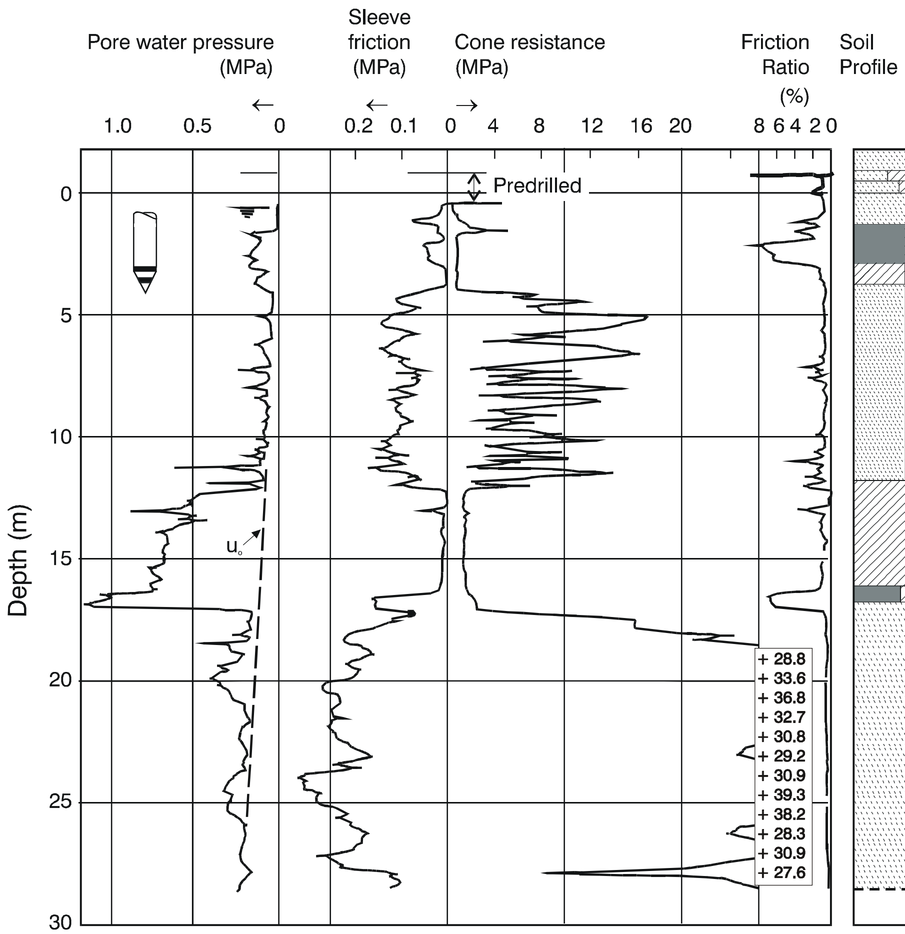


Fig. 2. A typical CPTU result

2.2.2. NON-STANDARD SIZE CONE PENETROMETERS

Traditionally in the site investigation industry cone penetrometers of a range of sizes are used. In the offshore industry for seabed testing, use of 15 cm^2 cone penetrometers has been accepted for many years whilst for downhole testing 10 cm^2 cone penetrometers have dominated. In addition, the use of minirigs has meant that cone penetrometers with cross-sectional areas as small as 1 cm^2 are now being employed. Similar situations apply to land where the 10 cm^2 has tended to dominate but the 15 cm^2 cone has been very popular in some countries (see POWELL et al. [37] for past UK practice). Previous studies on scale effects between cone penetrometers (e.g. LUNNE et al. [24] and TUMAY and LIMA [50]) have shown that some 15 cm^2 cones can give analogous results to the standard 10 cm^2 cone. POWELL and LUNNE [32] reported good general agreement between 10 and 15 cm^2 cones in UK clays. For cones with a cross-sectional area varying from 5 to 15 cm^2, De Ruiter (1982) reported that differences in the cone resistance and sleeve friction are not significant. The new IRTP (ISSMGE [13]) acknowledges this experience and allows some flexibility with regards to size: quote: *The cross-sectional area of cone shall nominally be 1000 mm^2, which corresponds to a diameter of 35.7 mm. Cones with diameters between 25 mm (A_c = 500 mm^2) and 50 mm (A_c = 2000 mm^2) are permitted for special purposes, without the application of correction factors. The recommended geometry and tolerances should be adjusted proportionally to the diameter.*

A recent study by TITI et al. [49] described a field testing programme to systematically compare results of cone penetrometers with cross-sectional areas of 15 and 2 cm^2. Parallel tests were carried out in soft clay, stiff clay and compacted clay in Louisiana, the USA. Based on a statistical study of the average results from 3 clays, Titi et al. concluded that the cone resistance q_t was 11% higher for the 2 cm^2 cone compared to the 15 cm^2 cone. On the other hand the sleeve friction was about 9 % lower for the 2 cm^2 cone relative to the 15 cm^2 cone penetrometer. Titi et al. then recommended using these numbers to correct the results of the 2 cm^2 cone penetrometer before interpretation in terms of soil parameters. One aspect that must be born in mind when evaluating the results of Titi et al. is that the soils they tested were to some extent layered. For layered soils the larger cone penetrometer needs a thicker layer to reach a steady cone resistance. Therefore in the thin layers the small cone may reach a "plateau" while this may not be so with the larger diameter cone penetrometer (see LUNNE et al. [25]). POWELL and LUNNE [32] concluded that when comparing cones ranging in size from 2 to 15 cm^2 in UK clays, then there was no systematic effect of cone or friction sleeve size on the results, they tended to form similar scatter bands. Figure 3 shows some results reported by POWELL and LUNNE [32] for a glacial clay till and a soft clay, in the glacial till a more spiky profile is observed in the 2 cm^2 cone profile as the cone resistance is more susceptible to the small stones encountered locally in the till (note that the data of figure 3 is in terms of the corrected cone resistance q_t, see section 2.6 below).

Figure 3c shows the level of repeatability that is achievable with a variety of 10 and 15 cm^2 cones when care is taken with all aspects of the testing.

One advantage of the smaller cone penetrometers is that in layered soils they will give better definitions of the layers. This was demonstrated in laboratory experiments by HIRD et al. [10] who also showed that by increasing the recording rate (e.g. 100 readings per second) much more effective use could be made of miniature piezocones to determine the fabric of soils. In some cases, this could be an important issue. De Groot et al. (2004) have recently shown similar behaviour in the field. Data in figure 3c was also gathered using a fast sampling rate but with a standard 10 cm^2 cone and helps give the level of detail shown there (see also POWELL and QUARTERMAN [35]).

In conclusion on cone size effects, it is suggested that in practice cone penetrometers ranging in diameters from 5 to 15 cm^2 will give very similar cone resistance values in most materials. For diameters outside this range it is recommended that the need for correction of the results should be considered, and then should preferably be based on site specific correlations.

2.3. PROCEDURES FOR PENETRATION OF CONE PENETROMETERS

2.3.1. GENERAL

The results of a cone penetration test are to some extent dependent on how the cone penetrometer is pushed into the soil. A quick overview of the different means of penetration is given below. For all methods the requirement to the speed of penetration is the same at 20 mm/s ± 5 mm/s. The Swedish Standard however requires even tighter tolerances to this rate at 20 mm/s ± 2 mm/s especially in soft clays and the highest accuracy classes (see 2.5 below). Generally the effect of variations in the rate of penetration requires significant changes to occur and will vary with soil type (see LUNNE et al. [25]).

2.3.2. PUSHING FROM GROUND LEVEL

The most common way of pushing the cone penetrometer into the soil is to use a rig at ground level and to push with a hydraulic cylinder in strokes of normally 1.0 m. In some soils the intermittent nature of carrying out the tests in this way causes some “unrepresentative readings”, every time the test is stopped and a new rod added. Offshore rigs with continuous pushing using rotating steel wheels to drive the cone rods have been used extensively since 1984. Onshore discontinuous pushing has continued to dominate, even if the Swedish company BORROS introduced a rig with continuous pushing more than 20 years ago. With the increased use of CPTU results to derive soil parameters for foundation design the use of continuous pushing should be considered more also for onshore practice.

2.3.3. RECENT DEVELOPMENTS

The Italian company SPG and the Swedish company ENVI have together developed a new alternative method for carrying out CPTU in a borehole (SACHETTO [43]). Figure 4 illustrates the method. A cone penetrometer protrudes in front of the drill bit during drilling in the same way as a corer. CPT data is stored in a memory unit (Memocone). At the same time as the CPT data is logged, drilling parameters (drill bit load, torque, rate of penetration and fluid pressure) are also recorded. If a hard layer is encountered, the CPT unit will be pushed into the drill bit and thus protected; the CPT system can also be retrieved using a wireline thus allowing cores to be taken. Figure 5 gives a typical example from a profile with this system. It is thought that the combination of both CPT and drill-

3
9
8
6

ing parameters will be a very powerful basis for interpreting the data. The advantage of this system compared to the downhole type CPTU described above is that much longer strokes than the normal 3 m can be made; in addition the information from the drilling parameters will be very useful, especially in hard formations where CPTU cannot be performed. Further studies are planned to verify that the results from CPTUs carried out this way are similar to CPTUs carried out in the traditional way. It is also expected that future development work will improve the method in terms of the procedures for carrying out the tests and also for interpreting the combined drilling parameters and CPTU logs. So far this method has only been used on land.

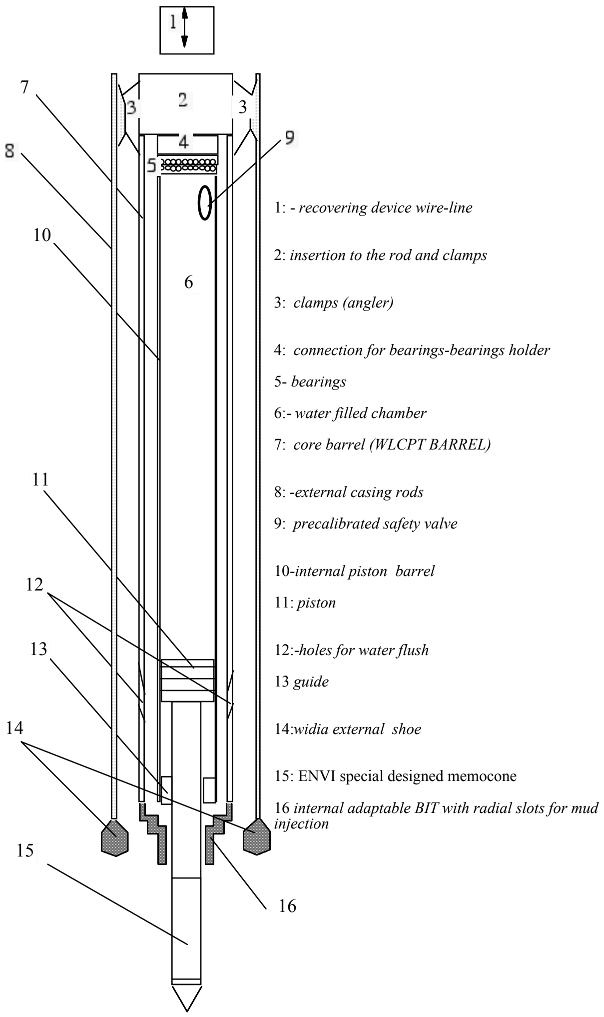


Fig. 4. Principle for new CPTU wd (SACHETTO [43])

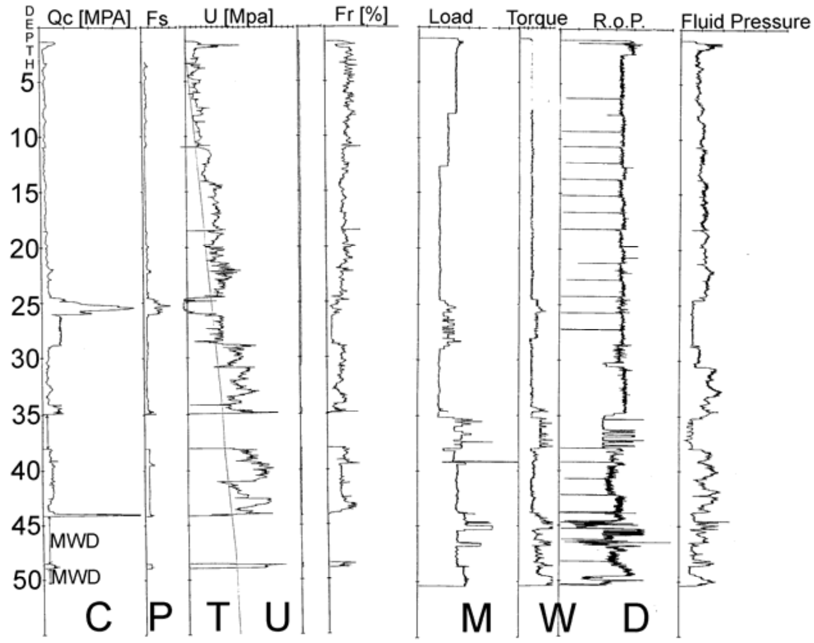


Fig. 5. Example result of CPTU wd (SACHETTO [43])

2.4. DATA ACQUISITION

Modern electric cone penetrometers produce continuous signals that require relatively complex data collection and processing. Normally the signals are transmitted via a cable pre-threaded down the standard push rods. Acoustic transmission of signals is used to facilitate easier handling of the rods since no cable is required. These systems eliminate the need for a cable but still allow real time recording and can be less reliable than cabled systems and cannot operate at faster sampling rates. The most recent development in this area is the use of optical transmission of data (Van den Berg, 2004).

Other systems allow storage of data within the cone penetrometer for retrieval after the test. These systems also eliminate the need for a cable, although they do not allow real-time interrogation or review of the data. This could be a problem if the cone encounters a hard layer, which could damage the penetrometer without the operator's knowledge. To reduce this possibility the total thrust can be monitored by a load cell. Both the acoustic and memory cone systems have somewhat reduced flexibility but can be cost effective for routine work. In Norway, all commercial CPT/CPTU operations are based on cable-less systems. This development has come about due to the requirements for reduced cost.

2.5. ACCURACY CLASSES

Previously the required accuracy for a CPT was linked to the capacity of the measurement sensor. A significant improvement resulting from the new IRTP (and future CEN standard) is in adopting the system of accuracy (or application) classes which are related to the accuracy that can be expected in the measured parameter (previously introduced in the most recent Swedish, Norwegian and Dutch standards/guidelines).

Table 2 shows the accuracy classes detailed in the new IRTP. The accuracy class to be required for a certain project is to be chosen according to what the results of the test are to be used for (i.e. their application):

- Classes 3 and 4: Results should be used for general stratification and only for parameter evaluations in very stiff or dense soils.
- Class 2: Results can be used for stratification and soil type and may be acceptable for parameter interpretation in stiff clays and sands.
- Class 1: Situations where results will be used for precise evaluations of stratification and soil type as well as parameter interpretation in profiles including soft or loose soils.

The implications of these classes for the accuracy required for the various parameters are given in table 1.

Table 1

Requirements according to ISSMGE [13]

Test class	The parameter measured	Allowable minimum accuracy	Maximum length between measurements
1	Cone resistance Sleeve friction Pore pressure Inclination Penetrated depth	50 kPa or 3% 10 kPa or 10% 5 kPa or 2% 2° 0.1 m or 1%	20 mm
2	Cone resistance Sleeve friction Pore pressure Inclination Penetrated depth	200 kPa or 3% 25 kPa or 10% 25 kPa or 2% 2° 0.2 m or 2%	20 mm
3	Cone resistance Sleeve friction Pore pressure Inclination Penetrated depth	400 kPa or 5% 50 kPa or 15% 50 kPa or 5% 5° 0.2 m or 2%	50 mm
4	Cone resistance	500 kPa or 5%	100 mm

	Sleeve friction Penetrated length	50 kPa or 20% 0.1 m or 1%	

The allowable minimum accuracy of the measured parameter is the larger value of the two quoted. The relative or per cent accuracy applies to the measurement rather than to the measuring range or capacity. The method for calculation of penetration depth from penetrated length and measured inclination is given by ISSMGE [13] and discussed below.

Figure 6 shows results from a recent study on the comparison of different cone penetrometers and their ability to satisfy the above accuracy classes (Lunne et al., 2005). They found that whilst all the cones appeared to satisfy the accuracy class 1, the most marked scatter was in sleeve friction both within individual cones but more importantly between cone types and this resulted in friction ratios varying from less than 1% to nearly 4% at any one depth. The reasons for this range of friction and resultant friction ratio are not yet fully understood but have significant implications when used in soil classification charts or other correlations (see below).

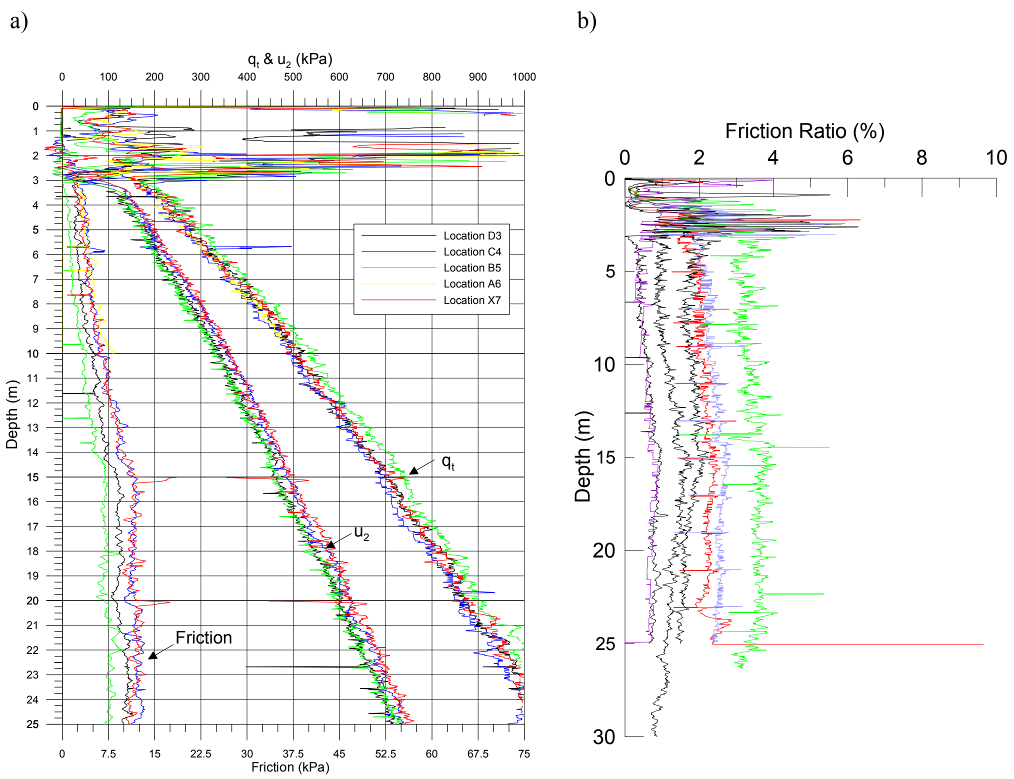


Fig. 6. Typical results showing repeatability for one cone (a),
friction ratio variations with cone type (b)

2.6. REQUIRED CORRECTIONS

The accuracy classes referred to above ensure that a given level of accuracy is achieved in the measured parameters. However, there are basic corrections that should be applied to the data in certain circumstances. CAMPANELLA et al. [6] and AAS et al. [1] showed that different cone penetrometers gave different results when used in the same deposit. This difference was shown to be the result of the pore pressures acting in the joints of the penetrometer affecting the measured results. This effect can be especially important in soft clays where it can make a significant difference to the results, depending on the geometry of the penetrometer. An important aspect of the piezocone is that the cone resistance can be corrected for these pore pressure effects.

This correction is expressed as

$$q_t = q_c + u_2\,(1 - a),$$

where:

q_t is the corrected cone resistance,

q_c is the measured cone resistance,

u_2 is pore water pressure measured just behind the cone,

a is the area ratio (that area affected by the pore water pressure (see LUNNE at al. [25]).

The new IRTP requires this correction to be made whenever possible and to do this reliably good pore water pressure readings are essential, as is good saturation of the system as mentioned earlier.

A second correction that should be considered is that to depth measurement due to deviation of the cone from the vertical. Most electric cone penetrometers today have simple slope sensors incorporated in the design to enable a measure of the non-verticality of the sounding. This is useful to avoid damage to equipment due to sudden deflection and is particularly useful for very deep soundings where eventual penetrometer inclinations in excess of 45° are not uncommon, especially in stratified soil. As seen from table 2 all accuracy classes except 4 require inclination. The calculation is:

$$z = \int_0^l C_h \cdot dl\,,$$

where:

z is the penetration depth, in *m.*,

l is the penetration length, in *m.*,

C_h is a correction factor for the effect of the inclination of the cone penetrometer relative to the vertical axis.

Equations for the calculation of the correction factor C_h for the influence of the inclination of the cone penetrometer relative to the vertical axis are:

- for single axis inclinometer

$$C_h = \cos\alpha,$$

where α is the angle measured between the vertical axis and the axis of the cone penetrometer, in °,

- for bi-axial inclinometer:

$$C_h = (1 + \tan^2 \alpha + \tan^2 \beta)^{-1/2},$$

where:

α is the angle between the vertical axis and the axis and the projection of the cone penetrometer on a fixed vertical plane, in °,

β is the angle between the vertical axis and the axis and the projection of the cone penetrometer on a vertical plane that is perpendicular to the plane of angle α, in °.

The maximum depth for which a slope sensor can be omitted depends on the acceptable error in recorded depth, provided that obstructions do not exist. If detailed information is required for the depth of stratification, then records of inclination are important, e.g. depth of a sand layer for pile length.

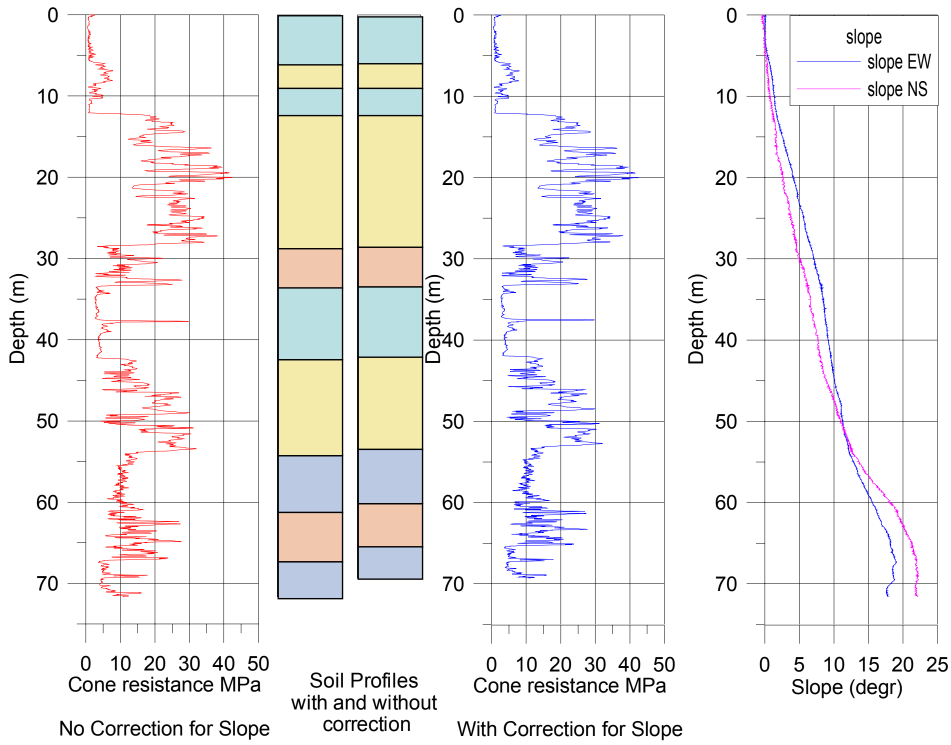


Fig. 7a. Effect of correcting penetration depth for inclination, deep profile

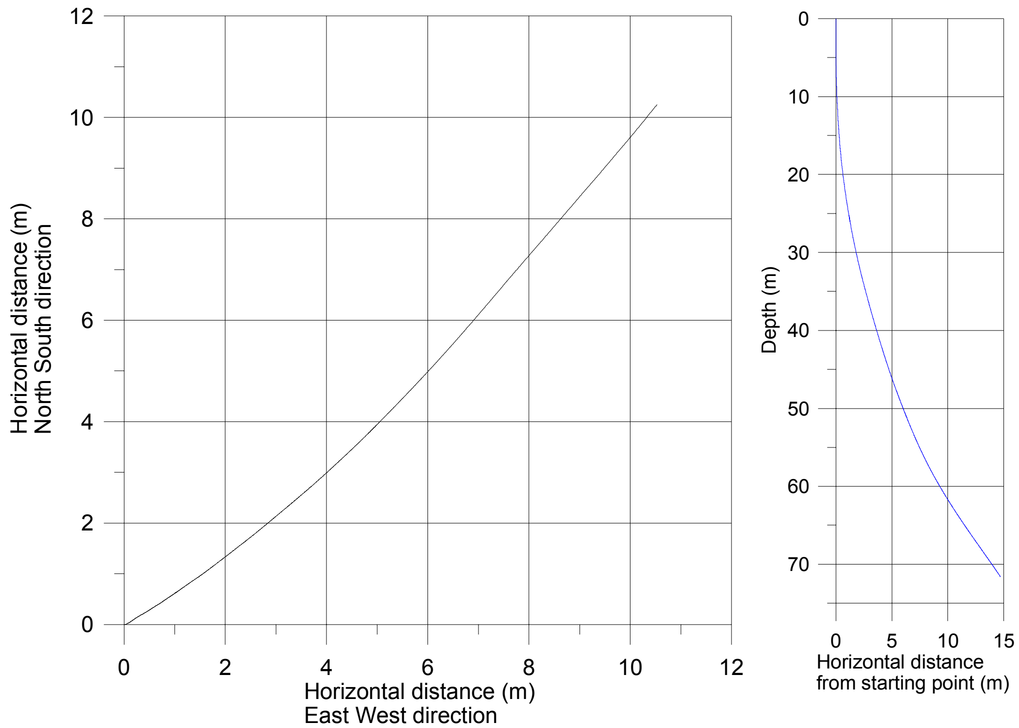


Fig. 7b. Effect of correcting penetration depth for inclination, deep profile

However, for most routine CPT work the maximum depth without a slope sensor, for which negligible error in recorded depth might be assumed, is about 15 m (although this may not always be the case, see below). For soundings deeper than about 15 m it is advisable to record inclinations on a continuous basis to allow corrections to be made to the depth of penetration. Figure 7a shows the effect of inclination correction on a deep sounding in clays and sands; although the total error in depth measurement is only about 3 m in 70 m the effect at depth of the perceived layering is quite evident. Figure 7b shows the relative position of the cone penetrometer at 70 m penetration when the cone is some 15 m away from the vertical line below the entry point.

When using the latest coiled rod systems inclination must be recorded at all times as the more flexible nature of the push rods and their potential for memory means that deviation from the vertical is much more important in knowing their true depth below reference level. Figure 8 illustrates the effect the correction can have on the plotted results, in this rather extreme case and error of 1 m in 12 m.

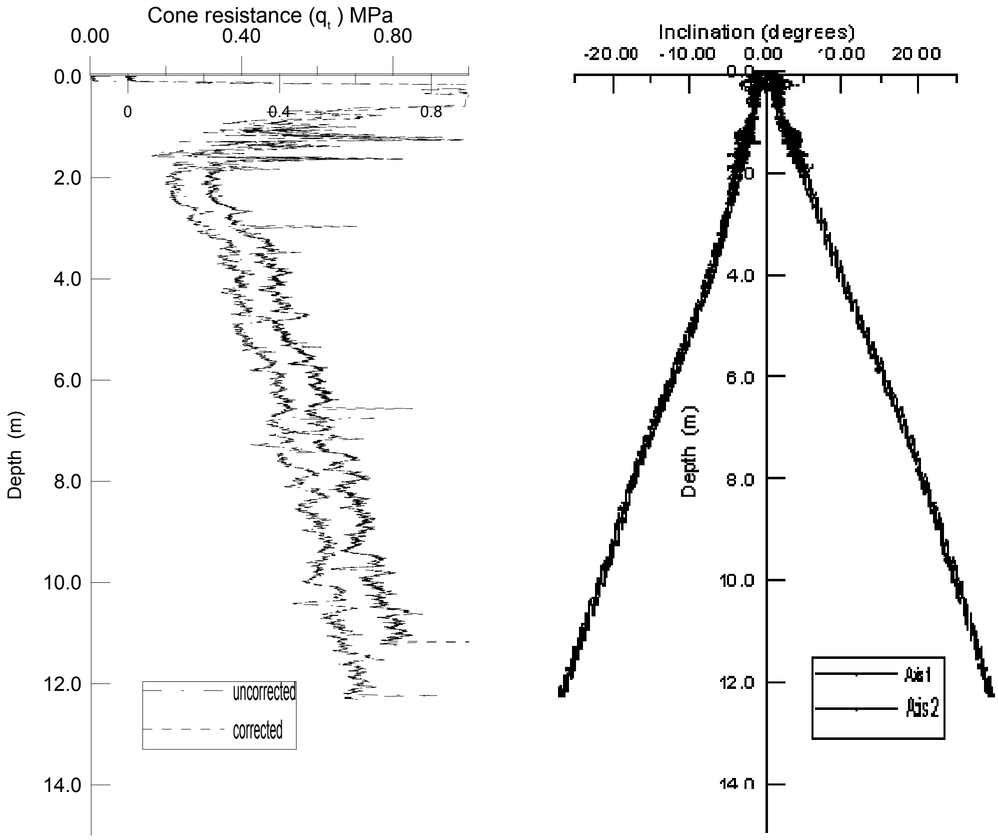


Fig. 8. Effect of correcting penetration depth for inclination, shallow depths coiled system (note axis shift in q_c to separate plots)

3. INTERPRETATION OF CPTU RESULTS

3.1. GENERAL INTRODUCTORY REMARKS

The use of the CPT/CPTU in engineering practice has three main applications:

a) to define sub-surface stratigraphy and identify the materials present,

b) to evaluate geotechnical parameters for use in foundation design,

c) to provide results for direct geotechnical design.

As mentioned in the introduction the last aspect will not be covered in this paper.

For soil stratification and identification of soil type the following text will be more general in nature and valid for a range of soils, whereas the coverage of soil design parameters will be restricted to clayey or fine grained soils. For other soil types the reader is referred to LUNNE et al. [25] and other relevant publications, e.g. JAMIOLKOWSKI et al. [14], PREVIATELLO and COLA [38].

It should not be forgotten that before analysing any CPT/CPTU data then it is important to ensure, as discussed above, that the quality of data are good and suitable for their expected application.

3.2. LAYERING AND STRATIFICATION

Figure 2 illustrates the excellent profiling capability of the CPTU. The continuous monitoring of pore pressure during cone penetration can improve the identification of soil stratigraphy. The definition of layering should be based on all three parameters being measured. As a guide in identifying the layers the following general observations with regard to response to penetration of a piezocone are made:

- In sandy soils, drained conditions are expected with pore pressure close to hydrostatic, high cone resistance, frequently quite irregular, and low friction ratio.
- In clayey soils, undrained conditions are expected with high pore pressures in soft normally to moderately overconsolidated clays, low cone resistance and relatively high sleeve friction. In more heavily overconsolidated soils then only the pore pressure in u_1 can be expected to be significant (LUNNE et al. [25])

Hight et al. (2002) showed that the u_1 pore pressure in heavily overconsolidated aged clays aided the identification of stratigraphic layers much better than u_2 which was often in suction or very small. Powell and Quarterman (1995) showed that significant detail could be achieved in defining both lithological units and facies features in soft soils with good quality testing.

The penetration pore pressure depends on the location of the filter. The pore pressure on the cone face will in most cases be a better detector of thin layers (smearing is far less prevalent in this location). Also cone resistance and pore pressure on cones of smaller diameters will generally be better for definition of very thin layers (Hird et al., 2002).

In dense sands, negative penetration pore pressures can be observed in many cases in the u_2 position. This is mainly because of the dilative behaviour of such soils. An example of this kind of behaviour is shown in figure 9 from the Dese site in a Venetian Lagoon (after PREVIATELLO and COLA [38]).

3.3. SOIL TYPE

Since BEGEMANN [3] introduced the measurement of sleeve friction the CPT has been used successfully for identification of soil type being penetrated. With the introduction of pore pressure measurements the ability of the CPT to predict soil type has

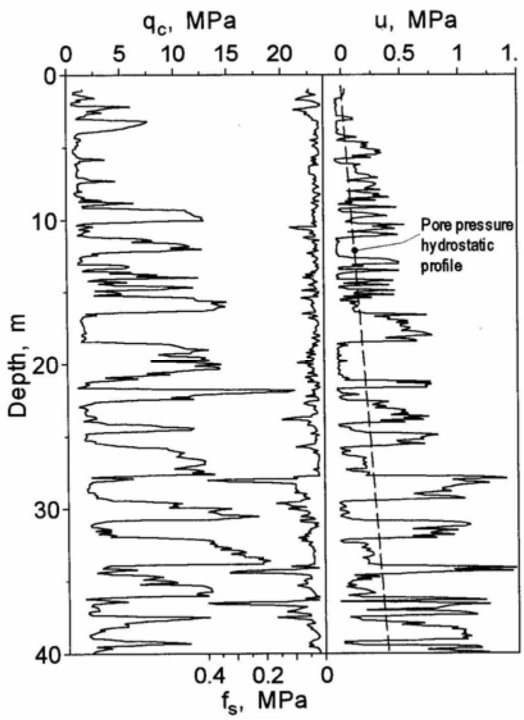


Fig. 9. CPTU profile for Venetian soils (PREVIATELLO and COLA [38])

been further enhanced. Based mainly on experience from CPTUs and parallel soil borings ROBERTSON et al. [41] developed the soil classification chart using both friction ratio and the pore pressure ratio B_q as shown in figure 10. This chart is used extensively all over the world. One problem with the chart shown in figure 10 is that soils can change their apparent classification as cone penetration increases with increasing depth. This is due to the fact that q_t, f_s and u all tend to increase with increasing overburden stress. To improve this situation several researchers have attempted to use normalised parameters. WROTH [51] suggested that CPTU data should be normalised using the following parameters:

- Pore pressure ratio:

$$B_q = \Delta u/(q_t - \sigma_{v0}),$$

where $\Delta u = u_2 - u_0$.

- Normalised cone resistance:

$$Q_t = (q_t - \sigma_{v0})/ - \sigma'_{v0},$$

where σ'_{v0} is the effective vertical stress $\sigma_{v0} - u_0$.

- Normalised friction ratio:

$$F_r = f_s/(q_t - \sigma_{v0}).$$

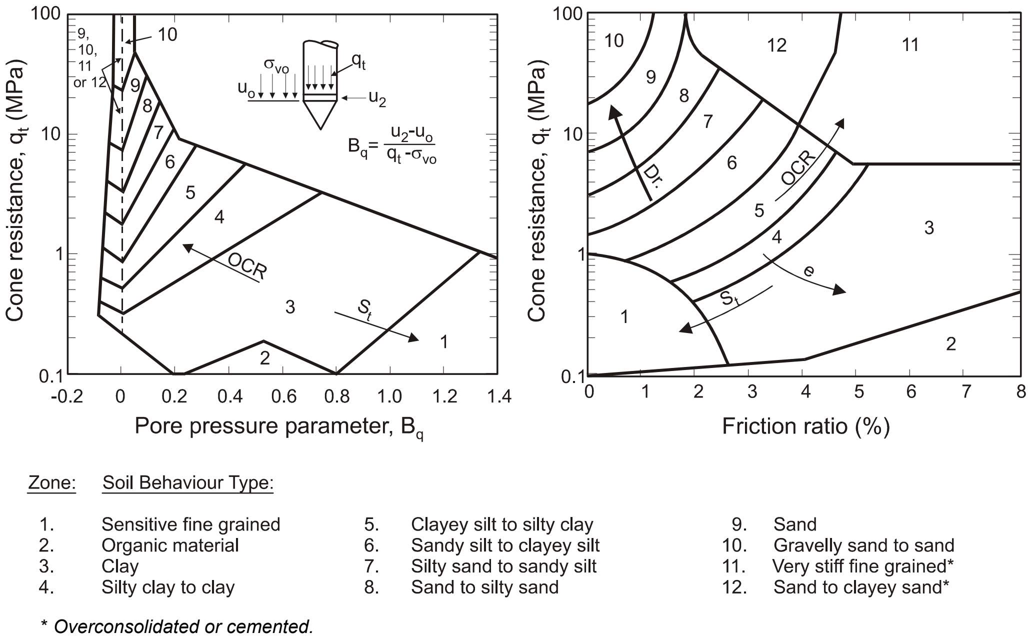


Fig. 10. Soil behaviour type classification chart (after ROBERTSON et al. [41])

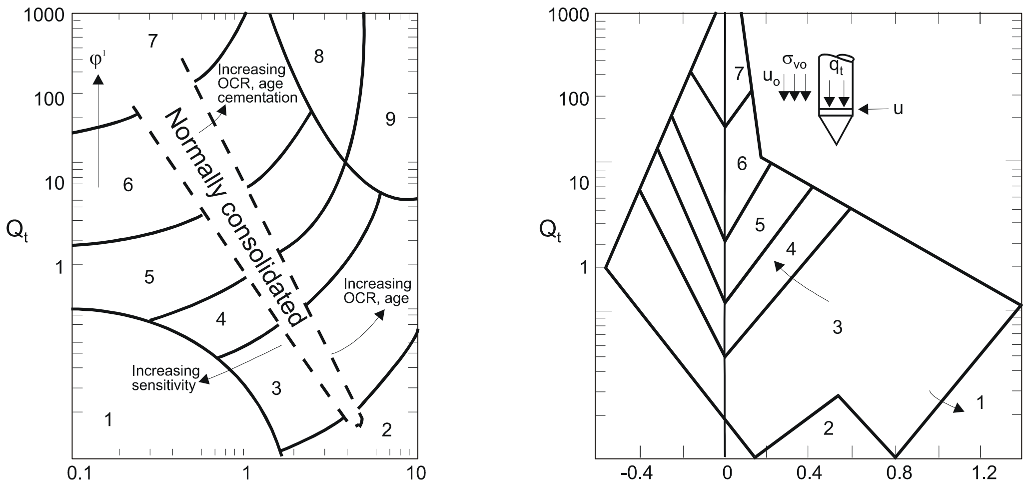
Based on these normalised parameters and using the extensive CPTU data base available at the time ROBERTSON [39] published the soil classification chart shown in figure 11. The two charts shown in figure 11 represent a three-dimensional classification system that incorporates all three pieces of CPTU data. For basic CPT data where only q_c and f_s are measured the left-hand part of the chart should be used. The error in using q_c instead of q_t is larger for clays with lower Q_t values.

Included on the normalised soil behaviour type classification chart is a zone that represents approximately normally consolidated soil behaviour. A guide is also provided to indicate the variation of normalised CPT and CPTU data for changes in: overconsolidation ratio (OCR), age and sensitivity (S_t) for fine grained soils, where cone penetration is generally undrained, and OCR, age, cementation and friction angle (φ') for cohesionless soils, where cone penetration is generally drained.

Generally, soils that fall in zones 6 and 7 represent approximately drained penetration, whereas soils in zones 1, 2, 3 and 4 represent approximately undrained penetration. Soils in zones 5, 8 and 9 may represent partially drained penetration. An advantage of pore pressure measurements during cone penetration is the ability to evaluate drainage conditions more directly.

ROBERTSON [39] suggested that the charts in figure 11 are still global in nature and should be used as a guide to define soil behaviour type based on CPT and CPTU data. Factors such as changes in stress history, in situ stresses, sensitivity, stiffness, macrofabric, mineralogy and void ratio will also influence the classification.

The problems mentioned above with friction measurements in soft soils can of course have significant implications in both the f_s and R_f charts and the better consistency of pore-water pressure measurements may prove to be a better and more consistent alternative for use in interpretation in soft soils when based on the B_q charts.



$$Q_t = \frac{q_t - \sigma_{vo}}{\sigma'_{vo}} \qquad B_q = \frac{u_2 - u_o}{q_t - \sigma_{vo}} \qquad F_r = \frac{f_s}{q_t - \sigma_{vo}} \times 100\%$$

Zone	Soil behaviour type	Zone	Soil behaviour type	Zone	Soil behaviour type
1.	Sensitive, fine grained;	4.	Silt mixtures clayey silt to silty clay	7.	Gravelly sand to sand;
2.	Organic soils-peats;	5.	Sand mixtures; silty sand to sand silty	8.	Very stiff sand to clayey sand
3.	Clays-clay to silty clay;	6.	Sands; clean sands to silty sands	9.	Very stiff fine grained

Fig. 11. Normalised soil behaviour type classification chart (after ROBERTSON [39])

Occasionally, soils will fall within different zones on each chart; in these cases judgement is required to correctly classify the soil behaviour type. Often the rate and manner in which the excess pore pressures dissipate during a pause in the cone penetration will significantly aid the classification (see LUNNE et al. [25]).

The normalised charts may become slightly misleading at shallow depths and low effective stresses, especially in the offshore environment.

3.4. STRESS HISTORY

Over the last 20 years a number of correlations have been proposed to relate derived piezocone parameters to preconsolidation stress p'_c, or overconsolidation ratio OCR through theoretical considerations or through empirical correlations. Table 2 gives a summary of the relationships that have been most frequently referred to.

Table 2

Most used relationships between piezocone parameters and p'_c or OCR

Correlation No.	p'_c OCR	Piezocone parameter	References giving background for proposed correlation
1	OCR	$B_q = \Delta u_2/(q_t - \sigma_{v0})$	SENNESET et al. [45], WROTH [51]
2		$Q_t = (q_t - \sigma_{v0})/\sigma'_{v0}$	WROTH [51]
3		$Q_u = (q_t - u_2)/\sigma'_{v0}$	HOULSBY [11], MAYNE [26]
4	p'_c	Δu_1 or Δu_2	MAYNE and HOLZ [27]
5		$q_t - \sigma_{v0}$	TAVENAS and LEROUEIL [48]
6		$q_t - u_2$	KONRAD and LAW [17]

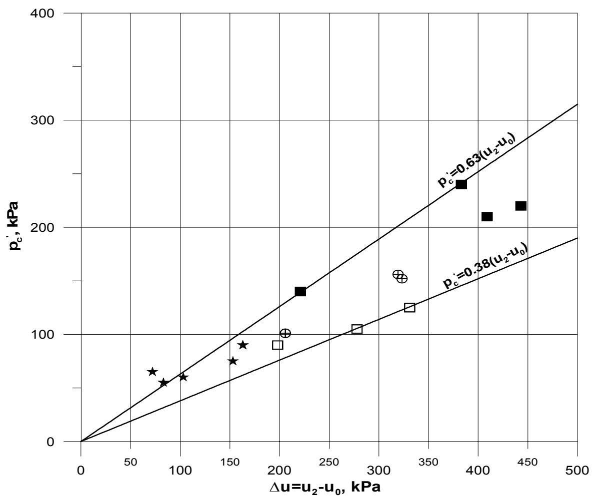
CHEN and MAYNE [7] compiled a large database containing CPTU results from 205 clay sites all over the world. They found that using the correlations to p'_c (Nos 4, 5 and 6) resulted in higher coefficients of determination (r^2) compared to the correlations to OCR (Nos 1, 2 and 3). Average relationships found by Chen and Mayne were:

$p'_c = k_1(u_2 - u_0)$; $k_1 = 0.53$; $n = 811$; $r^2 = 0.722$ (n = number of data sets),

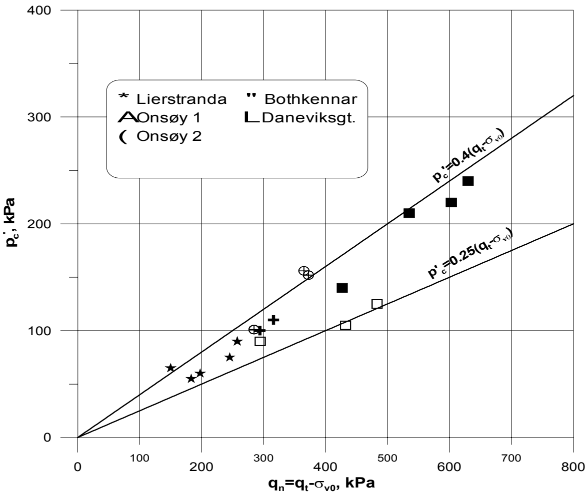
$p'_c = k_2(q_t - \sigma_{v0})$; $k_2 = 0.305$; $n = 1256$; $r^2 = 0.82$,

$p'_c = k_3(q_t - u_2)$; $k_3 = 0.50$; $n = 884$; $r^2 = 0.797$.

a)



b)



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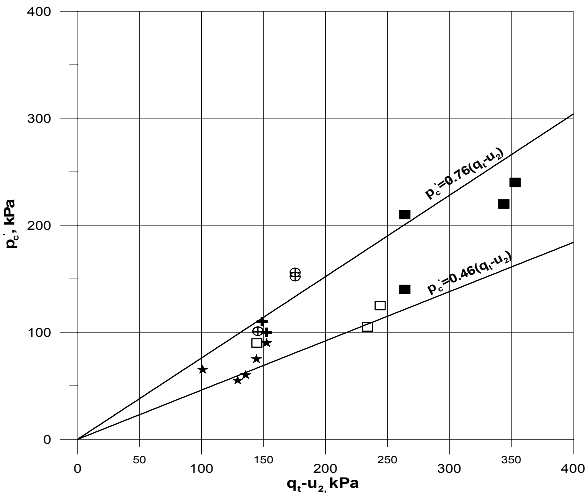

Fig. 12. Correlations between CPTU parameters and p_c' for 5 soft clay test bed sites:
a) p_c' vs. $u - u_0$, b) p_c' vs. $q_t - \sigma_{v0}$, c) p_c' vs. $q_t - u_2$

LEROUEIL et al. [20] also preferred the direct correlation to p_c' and found $k_2 = 0.28$ for eastern Canada clays. MESRI [29] based on general evaluation of relationships for soft inorganic clays and silts found the same relationship as Leroueil et al. He also argued that for organic soft clays and silts the relationship $k_2 = 0.24$ should apply.

Figure 12 shows CPTU parameters plotted against p_c' as found from CRS oedometer tests performed on high-quality samples obtained with the Sherbrooke block sampler (LEFEBVRE and POULIN [19]). The following well investigated test bed sites are included: Onsøy (1 and 2), Lierstranda, Drammen (Daneviksgate), all Norway and Bothkennar, UK.

It can be observed from figure 12 that the upper and lower bounds of the data from the research sites are quite similar to the average values found by Chen and Mayne. It is suggested that for soft clays similar to the ones at the reference sites the upper and lower relationships indicated in figure 12 can be used to estimate the range of p'_c values. It is recommended to use all three relationships.

It should be noted that for overconsolidated clays where there is a tendency towards negative Δu_2-values the correlation using this parameter will not work.

For comparison equations 2 & 5 and 3 & 6 are directly compatible with the only difference being the inclusion of σ'_{v0} in 2 & 3. It is therefore not surprising that the factor k_2 above is very similar to the constant of 0.3 given by LUNNE et al. [25] for equation 2 as a starting value for assessment of OCR (the range from 0.2 to 0.5), but with the warning that higher values will be needed for aged heavily overconsolidated clays (POWELL et al. [36]). More recently Trevor and Mayne (2004) used a version of equation 3 based on critical state concepts and raising the equation to a power of 1.33; they found that whilst it profiled the variation of OCR with depth well, there was a need for a "correction" factor varying from 0.4 to 0.6 to get "fit".

An alternative that is often seen is the use of the Pore Pressure Difference Parameter (PPD) (SULLY et al. [46]) where: PPD = $(u_1 - u_2)/u_0$.

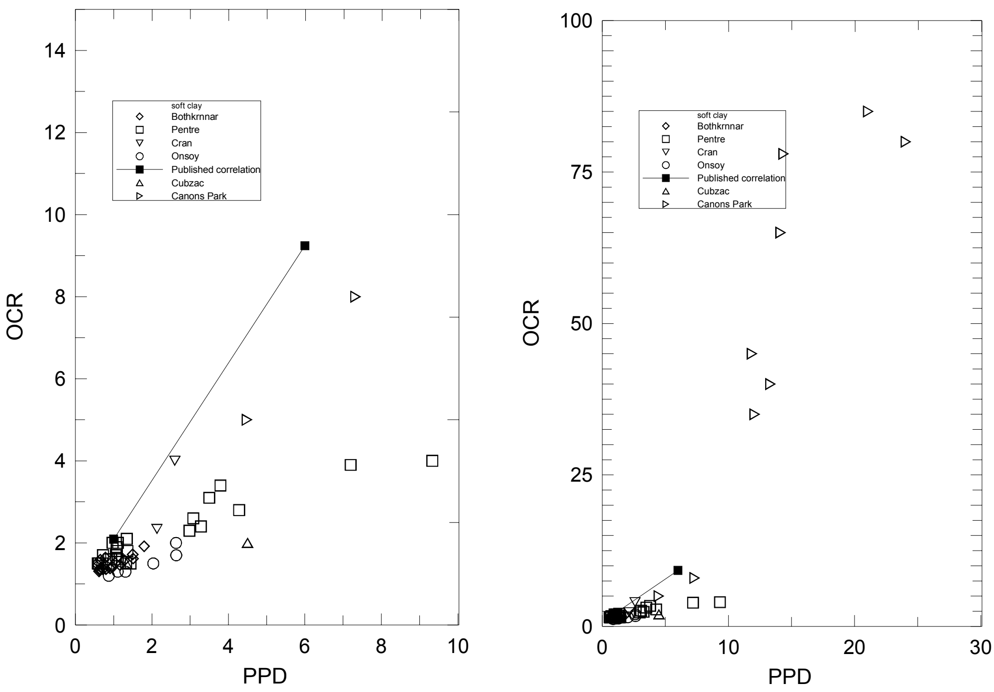

Fig. 13. OCR against PPD: a) lightly overconsolidated soils, b) high OCR

Whilst Sully et al. showed that for clays with OCRs less than about 6 a strong correlation existed this broke down for more heavily overconsolidated clays. Figure 13 shows data from a range of well categorised European testbed sites (ref to BRITE Euram) where it is seen that for lightly overconsolidated soils then there appears to be site specific correlations which fall significantly below the Sully et al. line, whilst for the heavily overconsolidated aged clays they fall significantly above the line. It is worth noting that the data for Canons Park, where the upper few metres of material has been reworked and reconsolidated, falls within both groups separated by the geological change at an interface due to reworking in a past glaciation (the reworked material falling with the more lightly overconsolidated soils). One disadvantage of this method is that the procedure obviously requires two measurements of pore pressure and this is not generally available.

3.5. LATERAL STRESS RATIO

In the authors' opinion there are presently no truly reliable methods of determining the in situ horizontal stress (σ'_h) or the coefficient of lateral earth pressure at rest (K_0) from piezocone test results in fine grained soils.

However, some show some promise, to arrive at a rough estimate of σ_h or K_0 methods are available based on:

- OCR,
- pore pressure difference,
- measurement of lateral stress or sleeve friction.

The first of these uses the OCR, s_u and σ'_{v0} and established correlations to arrive at K_0. Several authors have also proposed a similar approach to calculating OCR from normalised cone resistance, considerable scatter can result but in any one deposit the variation of K_0 down a profile can often be very well defined (POWELL [31]).

The use of pore pressure difference normalised by the vertical effective stress PPSV ($= (u_1 - u_2)/\sigma'_{v0}$; not to be confused with PPD above) was suggested by SULLY & CAMPANELLA [46] and obviously again requires two measurements of pore pressure and so is generally not available; they showed a generally linear relationship between K_0 and PPSV and although they suggested a general correlation more site specific ones appeared to exist. Figure 14 shows a collection of data from European testbed sites as well as the Sully and Campanella general line. Again the Sully line does not seem to fit, but the option of site specific versions is evident, The method does seem capable of profiling the variations of K_0 within a deposit in soft soils (POWELL [31]).

The third technique appears to be the least used and least reliable owing to the difficulties of measurements of lateral stress and the reliability of friction sleeve data and is seeing little application.

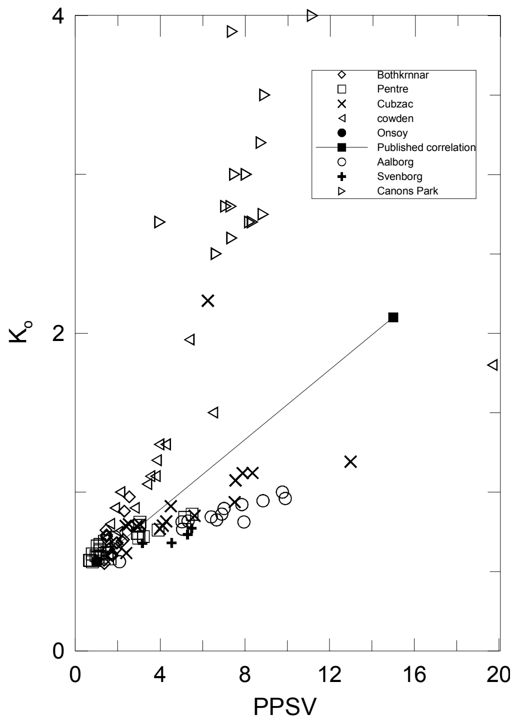

Fig. 14. PPSV against K_0

3.6. UNDRAINED SHEAR STRENGTH

For a soil no single undrained shear strength exists. The in situ undrained shear strength s_u depends on the mode of failure, soil anisotropy, strain rate and stress history. The s_u value to be used in analysis therefore depends on the design problem. Hence it is very important that any interpretation of CPT/CPTU in terms of undrained shear strength should state which undrained shear strength it refers to; e.g. s_u corresponding to that measured in an anisotropically consolidated undrained triaxial test sheared in compression – CAUC.

A large amount of work has been reported in the literature on the interpretation of undrained shear strength of clays from CPT or CPTU-results. There are two main approaches of interpretation, one based on theoretical solutions and the other based on empirical correlations.

3.6.1. THEORETICAL SOLUTIONS

The theoretical available solutions can be grouped under the following 5 classes:

1. Classical bearing capacity theory.

2. Cavity expansion theory.

3. Conservation of energy combined with cavity expansion theory.

4. Analytical and numerical approaches using linear and non-linear stress–strain relationships.

5. Strain path theory.

All the theories result in a relationship between cone resistance and s_u, of the form

$$q_c = N_c \cdot s_u + \sigma_0,$$

where N_c is a theoretical cone factor and σ_0 is the in situ total pressure. Depending on the theory used, σ_0 may be σ_{v0}, σ_{h0} or σ_{mean} as summarised by LUNNE et al. [25].

Since cone penetration is a complex phenomenon, all the theoretical solutions make several simplifying assumptions regarding soil behaviour, failure mechanism and boundary conditions. The theoretical solutions need to be verified from actual field and/or laboratory test data. Theoretical solutions have limitations in modelling the real soil behaviour under conditions of varying stress history, anisotropy, strain softening, sensitivity, ageing and macrofabric. Hence, empirical correlations are generally preferred, although the theoretical solutions have provided a useful framework of understanding.

3.6.2. EMPIRICAL CORRELATIONS

The empirical approaches available for interpretation of s_u from CPT/CPTU results can be grouped under 3 main categories as follows:

1. s_u estimation using “total” cone resistance.
2. s_u estimation using “effective” cone resistance.
3. s_u estimation using excess pore pressure.

• *s_u estimation using total cone resistance*. Estimation of s_u from CPT using cone resistance is made from the following equation:

$$s_u = \frac{(q_t - \sigma_{v0})}{N_{kt}}.$$

Over the years, a large number of studies have been performed, many of them resulting in N_k (or N_{kt}) factors of about 10–20 (see ESOPT 1974 and 1982, ISOPT 1988, CPT95). However, the method of determining s_u may vary from one study to another. It is again emphasised that a consistent reference s_u should be used.

In addition to the type of laboratory tests, the effect of sample disturbance can be important. Obviously the less disturbed the sample, the higher the undrained shear strength. Recent work in Norway (KARLSRUD et al. [16]) has shown that in soft clays the Sherbrooke block sampler (LEFEBVRE and POULIN [19]) gives superior sample quality, especially in low plasticity clays. According to Karlsrud et al. CAUC tests on Sherbrooke block samples give s_u-values that are as representative for in situ conditions as one can. A new generation of cone factors are therefore under development based on high quality block samples.

Figure 15a shows a good correlation between N_{kt} and B_q with N_{kt} values in the range of 6–15 and increasing with decreasing OCR. For comparison the data reported by LUNNE et al. [23] is also included, the data gives on average higher N_{kt} values with considerably more scatter. The reason for the larger scatter is thought to reflect more variation in sample quality.

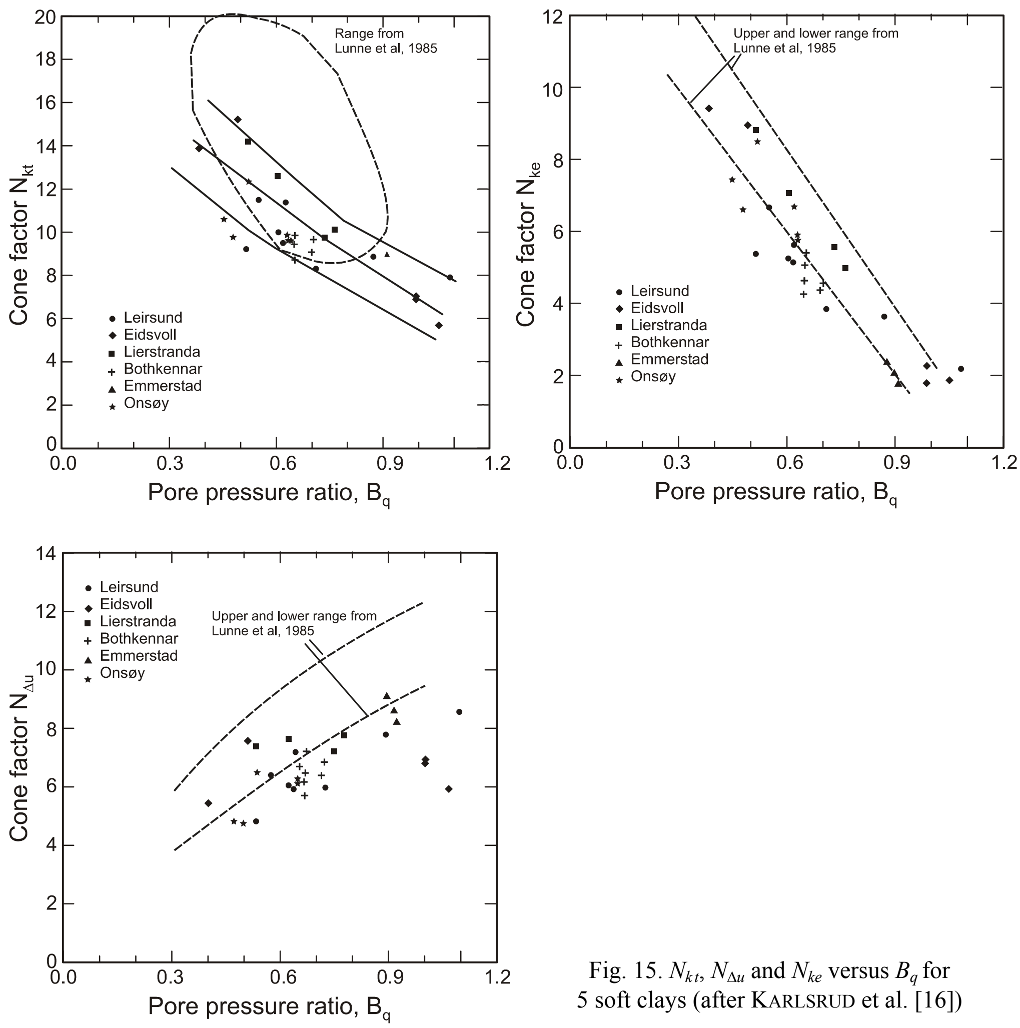

Fig. 15. N_{kt}, $N_{\Delta u}$ and N_{ke} versus B_q for 5 soft clays (after KARLSRUD et al. [16])

The s_u value determined as a function of cone resistance q_c in highly overconsolidated clays must be treated with caution due to uncertainty about the effects of fissures on the response of the clays (POWELL and QUARTERMAN [35]). For overconsolidated

UK clays N_{kt} factors have been found to vary between 15 and 30, depending on the degree of fissuring and the source of s_u for comparison and can be related to relative "scale effects" for the size of test.

• *s_u estimation using effective cone resistance*. SENNESET et al. [45] suggested the use of an "effective" cone resistance q_e to determine s_u, where q_e is defined as the difference between the measured cone resistance and pore pressure, measured immediately behind the cone (u_2). The corrected cone resistance q_t should be used, s_u can then be found using the relation:

$$s_u = \frac{q_e}{N_{ke}} = \frac{(q_t - u_2)}{N_{ke}}.$$

SENNESET et al. [45] indicated that the value of $N_{ke} = 9 \pm 3$. LUNNE et al. [23] showed that N_{ke} varied between 1 and 13 and appears to correlate with the pore pressure parameter B_q.

KARLSRUD et al. [16] used CAUC triaxial tests on high-quality block samples to obtain reference s_u values as described above. Their resulting N_{ke} versus B_q plot gives in a rather narrow band, again with slightly lower N_{ke} values, the mean falling close to the lower bound line of LUNNE et al. (figure 15b). The above correlations for N_{ke} were derived for normally to lightly overconsolidated clays and should not be extrapolated to heavily overconsolidated deposits where B_q is small or even negative (POWELL et al. [36]).

• *s_u estimation using excess pore pressure*. Using theoretical and semi-theoretical approaches based on cavity expansion theory, a number of relationships have been proposed between excess pore pressure Δu and s_u (e.g. BATTAGLIO et al. [2]). The relationships have the form of

$$s_u = \frac{\Delta u}{N_{\Delta u}} \qquad (\Delta u = u_2 - u_0).$$

Based on cavity expansion, $N_{\Delta u}$ is theoretically shown to vary between 2 and 20.

LUNNE et al. [23] found $N_{\Delta u}$ to correlate well with B_q and to vary between 4 and 10, for North Sea clays taking triaxial compression (CAUC) strength as the reference strength.

Using s_u values from CAUC tests on block samples, KARLSRUD et al. [16] obtained $N_{\Delta u}$ values varying between 5 and 9 with no clear tendency to be dependent on B_q (figure 15c). The above correlations for $N_{\Delta u}$ were derived for normally to lightly overconsolidated clays and should not be extrapolated to heavily overconsolidated deposits where B_q is small or even negative (POWELL et al. [36]).

The correlations referred to the above relate to the excess pore pressure measured immediately behind the cone (u_2). Use of site specific empirical correlations still seems to be the best procedure for interpretation of s_u from CPT/CPTU.

Based on the above discussion it is recommended to evaluate s_u in cohesive fine grained soils from CPT/CPTU data as follows:

1. For deposits, where little experience is available, estimate s_u using the total cone resistance (q_t) and preliminary cone factor values (N_{kt}) from 15 to 20. For a more conservative estimate, select a value close to the upper limit. For normally and lightly overconsolidated clays, N_{kt} can be as low as 10, and in stiff fissured clay it can be as high as 30. In very soft clays, where there maybe some uncertainty with the accuracy of q_t, estimate s_u from the excess pore pressure (Δu_2) measured behind the cone using $N_{\Delta u}$ from 7 to 10. The approach using N_{ke} can also be used in soft clays. For a more conservative estimate select a value close to the upper value.

2. If previous experience is available in the same deposit, the values suggested above should be adjusted to reflect this experience.

3. For larger projects, where high quality field and laboratory data may be available, site specific correlations should be developed based on appropriate and reliable values of s_u.

POWELL [31] showed that the use of the above correlations can give remarkably consistent results, using all three correlations and the derived shear strength falls very close to those obtained from high-quality block samples, whilst using the older correlations for N_{kt} related to plasticity index gives strengths much closer to those from routine piston sampling and laboratory testing.

HERMANN and JENSEN [9] and JENSEN [15] presented a very interesting case history illustrating the successful use of the CPTU for an important infrastructure project in Norway. The Nykirke railway track(1.5 km long) is part of the modernization project of the Norwegian Railway south of Oslo. The terrain along the track is rather hilly. The soil conditions are dominated by outcropping bedrock and soft silty clay deposits, which locally are found to be very sensitive and quick. Results from soil investigations, with 54 mm piston tube samples, and shear strength assessments based on fall cone tests carried out in the early part of the project were used by the client to develop the technical solution of an important fill. The strengths were between 0.2 and 0.3 σ'_{v0}. Steel pipes to bedrock had to be used.

Based on experience NGI evaluated that the 54 mm samples were severely influenced by sample disturbance and the strength and deformation characteristics of the clay were very likely to be underestimated. In the tendering phase, NGI carried out three CPTU profiles and interpreted the results of these tests using the correlations in figure 16. As with the Bothkennar example much higher shear strength values were obtained compared to the laboratory tests on the routine 54 mm samples. Based on the upgraded undrained shear strength profile an alternative and cost saving solution was recommended (see JENSEN [15]). In the engineering phase, the soil parameters interpreted from the CPTUs were confirmed by taking high-quality block samples and performing anisotropically consolidated undrained triaxial tests sheared in compression (CAUC). Figure 16 shows the results from these triaxial tests together with the undrained shear strength interpreted from the CPTUs. The measurements after building the fill confirmed the predictions made in the engineering phase (JENSEN [15]).

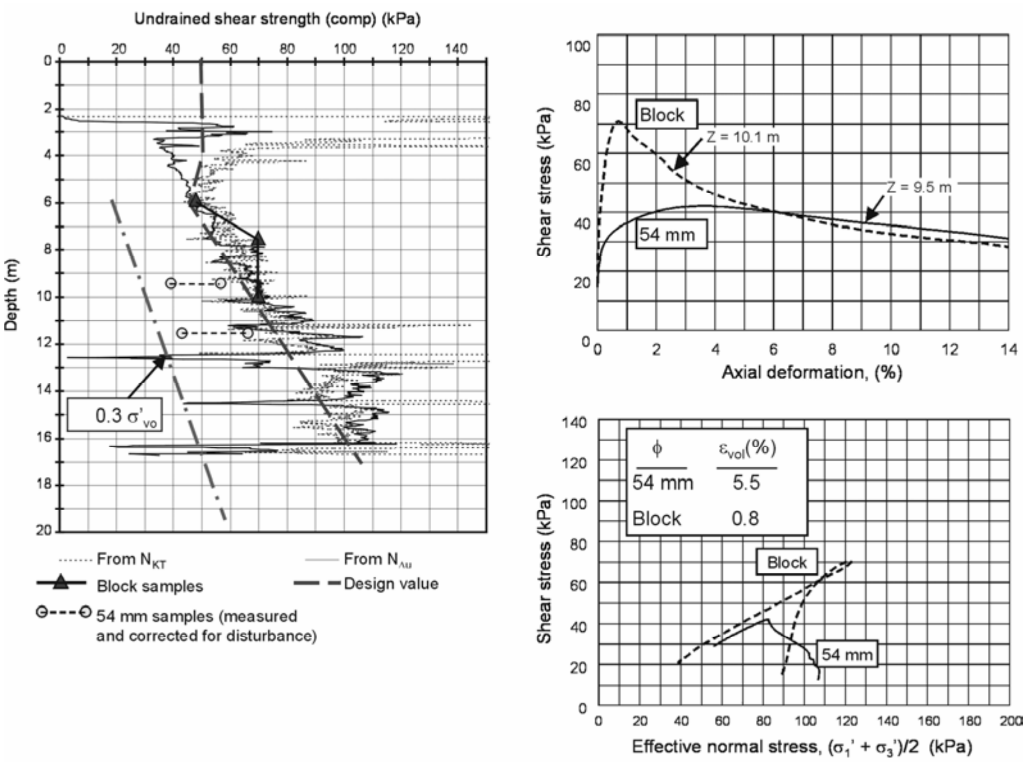

Fig. 16. Above: Undrained shear strength derived from cone penetration tests and compared with laboratory tests results on 54-mm and 250-mm block samples. Nykirke railway link (JENSEN [15])

3.7. SENSITIVITY AND REMOULDED SHEAR STRENGTH

The sensitivity of a clay is defined as the ratio of undisturbed undrained shear strength to totally remoulded undrained strength. It has long been recognised that the sleeve friction stress from an electrical CPT is a function of the remoulded shear strength and is approximately equal to the stress f_s. An example comparing the CPT sleeve stress to the remoulded strengths from UU remoulded, fall cone remoulded and residual ring shear strengths is given in figure 17 (KVALSTAD et al. [18]). Remarkably good agreement can be seen. As a result it is suggested that the sensitivity of a clay can be estimated by calculating the peak s_u from either site-specific or general correlations with q_t and then the remoulded s_u from sleeve friction measurements. The accuracy of the sleeve measurements is crucial to the assessment and this is an area most susceptible to error (see above).

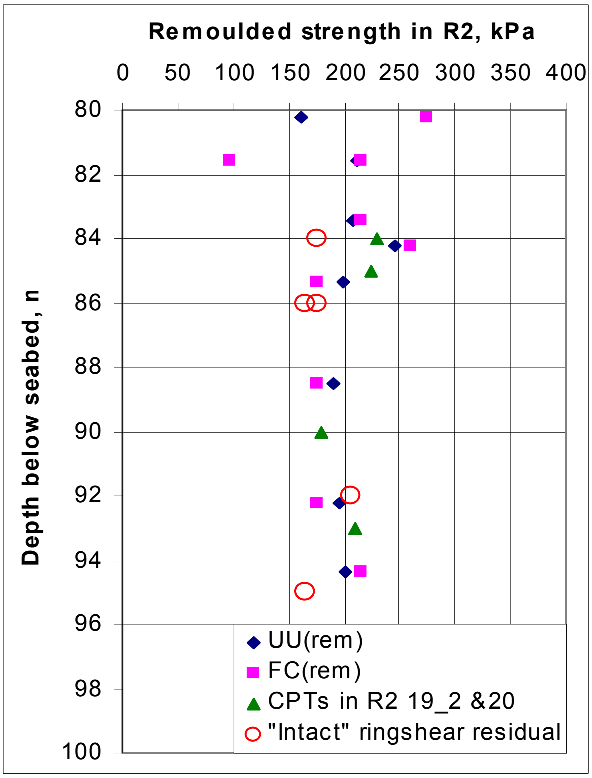

Fig. 17. Remoulded strength compared to sleeve friction

3.8. SMALL STRAIN SHEAR MODULUS

The shear modulus is largest at very low strains and decreases with increasing shear strain. It has been found that the initial maximum shear modulus is constant for strains less than 10^{-3}% although this will vary in clays with plasticity index (I_p) of the soil – the constant range being to higher strains with higher I_p. This initial, small strain modulus is often denoted by G_0.

Various authors have tried to correlate q_c or q_t with G_0 with varying success. MAYNE and RIX [28] suggested that the small strain shear modulus varied with void ratio e and cone penetration resistance q_c for a wide range of clays and can be expressed as:

$$G_0 = 99.5\ (p_a)^{0.305}\ \frac{(q_t)^{0.695}}{(e)^{1.130}},$$

where p_a is the atmospheric reference stress in the same units as G_0 and q_c.

As their data was not always in terms of q_t the above correlation is strictly only valid for q_c and this may explain the significant scatter. However this correlation would appear to be only slightly better than their simpler one based solely on q_c.

The implied dependence of G_0 upon void ratio e would require that CPT q_c is only successful as a profiler of G_0 if comparison profiles of e_0 are known. This is not usually the case. Simonini and Cola (2000) suggest that the use of B_q as an additional parameter in the correlation could be used to replace void ratio. They show that when considering relatively lightly overconsolidated mixed deposits in Venice then a better correlation between q_c and G_0 was obtained when incorporating B_q.

Care must always be taken when using any of these correlations as it should be remembered that G_0 is not independent of the direction of shear (POWELL and BUTCHER [33]). BUTCHER and POWELL [4] showed that the shear wave velocity in clays, and therefore the G_0 value deduced, was dependent on the stresses in the directions of propagation and polarisation of the shear waves.

More recently POWELL and BUTCHER [34] showed that the correlations with CPT data in clays varied with the shear modulus measured. Figure 18 shows their data in a similar way to that of Mayne and Rix, but with two main differences, namely the use the corrected cone resistance q_t instead of q_c (in all cases) and the G_0 as G_{vh}, G_{hv} and G_{hh} (v and h refer to vertical and horizontal, respectively; the first subscript refers to the direction of propagation – where the wave is travelling, and the second – to the direction of polarisation – is the wave moving up and down or side to side). In figure 18a, it is not possible to see a single correlation for q_t with G_{vh} for all the clays although two distinct groups appear to sit above and below the simplified correlation of MAYNE and RIX [28] based on G_{vh} (the work by Simonini and Cola, 2000, was also related to G_{vh}). Powell and Butcher found that the correlation with G_{hv} showed a merging of the two groups but still with considerable scatter. However, as shown in figure 18b, they found a very strong and simple correlation between q_t and G_{hh} which looks very promising; why this should be better than that for G_{vh} is not fully understood and needs further study. It would imply a strong dependence of q_t on the horizontal stress. The interrelation (differences) between field and laboratory assessments of G_0 should also be taken into account when developing correlations (BUTCHER and POWELL [5]).

At this stage any use of correlations of this type should only be used as an indication of likely G_0 and its variation with depth, direct measurement by say seismic cone testing should be undertaken for reliable assessment (G_{vh}), always remembering the potential effects of test method and orientation.

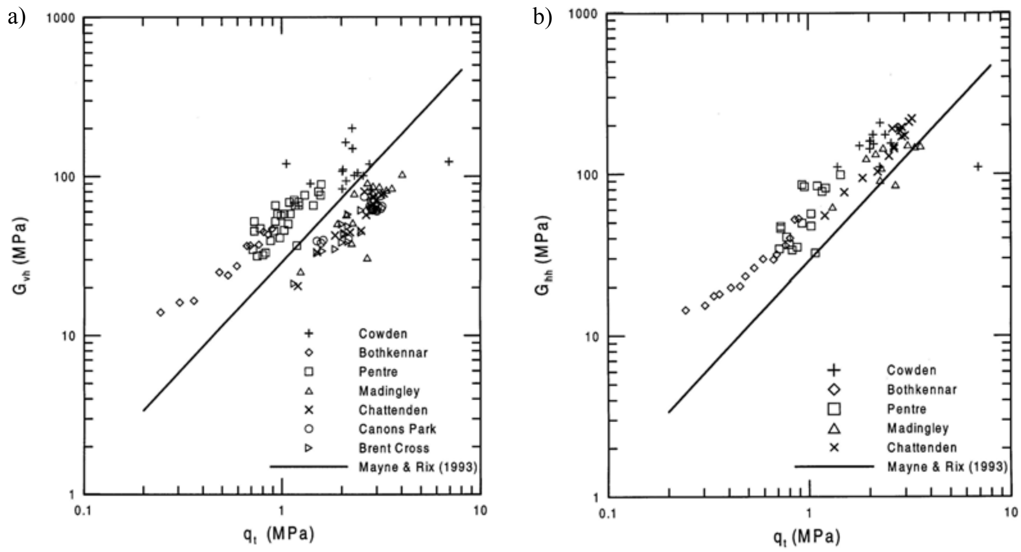

Fig. 18. Comparison of G_0 with q_t: a) using G_{vh}, b) using G_{hh}

3.9. COEFFICIENT OF CONSOLIDATION

Much has been written about the interpretation of coefficient of consolidation and permeability. Rate of consolidation parameters may be assessed from the piezocone test by measuring the dissipation or decay of pore pressure with time after a stop in penetration.

ROBERTSON et al. [40] reviewed dissipation data from piezocone tests to predict coefficient of consolidation using HOULSBY and TEH'S [12] solutions with reference values from laboratory tests and field observations. The review showed that the Teh and Houlsby solution provided reasonable estimates of c_h. Results were evaluated for pore pressure data from different filter locations and the least scatter was obtained with the pore pressure element location immediately above the cone (u_2). Figure 19 shows some of the results presented by ROBERTSON et al. [40].

The procedure to be followed in estimating the coefficient of consolidation recommended by the authors is to use dissipation data from the filter location behind the cone (u_2); however, other filter locations may be used even though the data may be somewhat less consistent.

The recommended procedure is as follows:

a) Plot the early part of the dissipation (less than 10% dissipation) at an enlarged scale, either log or square root time, and evaluate the initial pore pressure u_i.

b) Define u_0 from available data on ground water level, piezometric readings or data from piezocone tests in adjacent sand layers.

c) Plot normalised excess pore pressure ($U = (u_t - u_0)/(u_i - u_0)$, where u_t is the pore pressure at time t and u_0 is the in situ equilibrium) against time on a log and/or $\sqrt{t}$ scale.

d) Define time for 50% dissipation (t_{50}).

e) Use t_{50} and the curves/lines in figure 19 to predict c_h. If no other data is available, use an average rigidity index ($I_r = G/s_u$) in the range given in figure 19.

f) If dissipation has not proceeded sufficiently long to define t_{50}, the slope of the straight line from the first part of u vs. $\sqrt{t}$ plot (m) may be used to predict c_v using values suggested by Houlsby and Teh.

Figure 19 shows a simplified diagram that can be used to estimate c_h using the HOULSBY and TEH [12] solution and shows the 3 sets of lines for u_1, u_2 and u_3 positions (ROBERTSON et al. [40]). Data is plotted in figure 19 for two soft clay sites, where detailed decay curves from all three positions were available; it can be seen that very similar estimate ranges for c_h are obtained from all three positions for each site – this is encouraging as it implies consistency in the Houlsby and Teh model between the different filter positions.

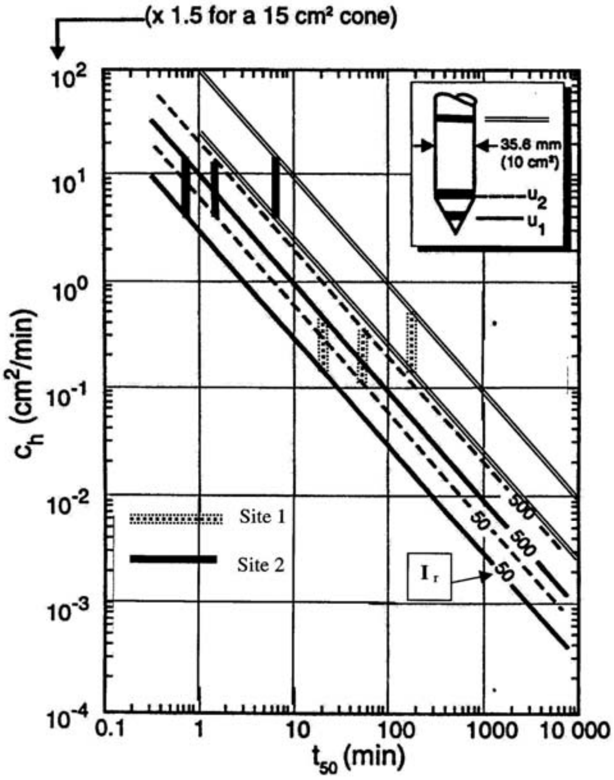

Fig. 19. Simplified plot for determining c_h from piezocone dissipation

Based on available experience, this recommended procedure should provide estimates of c_h within ± half an order of magnitude. However, the technique is repeatable and provides an accurate measure of the changes in the consolidation characteristics within a given soil profile.

3.10. IN SITU PORE PRESSURE

Reliable measurement of in situ pore pressure can be important in many design applications, pressures in excess of or below hydrostatic have implications for ground water flow. As discussed above the pore water pressure generated in a CPTU test can be allowed to dissipate to the equilibrium value to assess in situ pore pressure. Whilst in sands this may be quite rapid, in clays it can take may hours or even days. This makes the use of the standard CPTU unattractive for this purpose and as a result investigators have looked at alternatives. RUST [42] suggests a method of curve fitting incomplete dissipation tests to predict equilibrium pore water pressures; he showed significant success when using the method in mine tailings, but in less permeable deposits still needs in excess of 50% dissipation as well as background information on the idealised curve which seems to require a typical full dissipation curve to start with.

An alternative approach has been to develop smaller-diameter, but less robust, probes (e.g. MOKKELBOST and STRANDVIK [30]) where dissipation will be significantly quicker. These probes can only be penetrated relatively short distances and so need the use of boreholes. These devices are still being evaluated but show promise, their interpretation can be confused by the effect of stresses generated by the larger diameter of the main shaft of the device. SUTABUTR [47] developed a method for interpreting the devices allowing this interaction but this still needs further evaluation.

4. SUMMARY AND CONCLUSIONS

The CPTU is used more and more for deriving soil parameters for foundation design, in addition to profiling and soil identification. For this enhanced use of CPTU data it is of vital importance that the test results are accurate and reliable. The new International Reference Test Procedure (IRTP) gives the requirements for equipment and procedures according to which accuracy class is specified. The most strict accuracy class is applicable when the CPTU results are to be used to derive soil design parameters in soft clay. It is a requirement to measure inclination in addition to cone resistance, sleeve friction and pore pressure.

Methods are now available for interpreting CPTUs in clay as regards to soil layering and soil type, and also the following soil parameters: stress history, undrained shear strength, small strain shear modulus and coefficient of consolidation. Our increased knowledge and understanding of the factors affecting the results of CPT/CPTU data is allowing more consistent results to be obtained. Correlations developed in the past may well have been affected by lack of understanding of the factors affecting the measured results. More recently the continued development of high quality databases of CPTU results and soil properties is enabling a new or improved set of correlation to be developed.

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