Some issues related to applications of the CPT

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ABSTRACT: This paper reviews some issues related to the use of Cone Penetration Testing for geotechnical applications. Some of the areas that are considered include:

a) The advantages and disadvantages of Cone Penetration Testing (CPT)
b) The advantages of integrating CPT with laboratory testing.
c) Identification of similar geological units using statistics.
d) A review of published classification/behaviour charts, using a diverse and highly dependable database.
e) The importance of using correct cone calibration and cone zero values in normally consolidated fine-grained soils.

1 INTRODUCTION

The Burland Triangle (Burland, 1987), shown in Figure 1, provides a useful framework for the majority of geotechnical problems. The Cone Penetration Test (CPT) can provide valuable input to this framework, by providing cost-effective and useful information for the "Ground profile" and "Soil behaviour" aspects of the triangle.

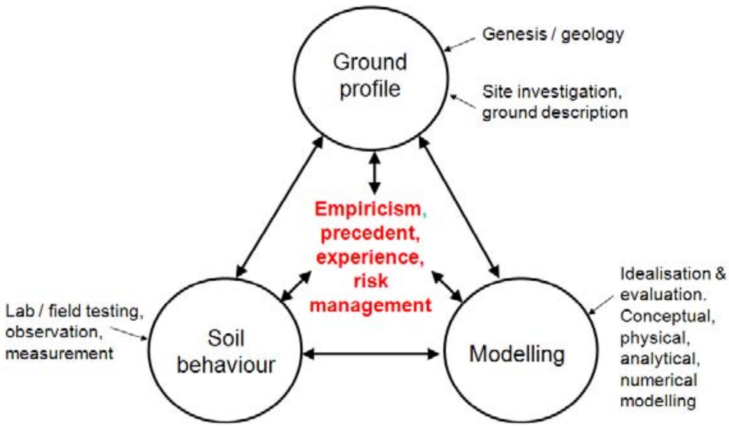

Figure 1: The Burland Triangle (Burland, 1987)

The main purposes of this paper are to review the contribution of the CPT, in terms of ground profiling and the assessment of soil behaviour. The paper concentrates primarily on sites containing normally consolidated (NC) fine grained soils, as these soils tend to be relatively difficult to analyse, and because published correlations can be less reliable in these soils. Practical examples are presented from several widely distributed sites, brief details of which are presented in Table 1.

Site	Location	Idealised soil description	Water Content (%)	Liquid Limit (%)	Plasticity Index (%)	Fines Content (%)
X	Offshore Europe	loose to medium dense silty sand	-	-	-	<35
Y	Offshore North Africa	NC clay	80 - 90	90 - 100	50-60	100
Z	Offshore West Africa	NC clay	70 - 80	70 - 80	40-50	100
V	Onshore Australia	NC clay over OC clay	75 – 85 25 – 45	70 – 90 40 - 80	50 – 60 20 – 50	90 – 100 80 - 100
S	Onshore Australia	FILL overlying NC clay	- 55-60	- 55-60	- 30-40	- 25-35
Note: NC denotes “normally consolidated” and OC denotes “overconsolidated”						

Table 1: Approximate details of example sites

2 CPT FOR ASSESSING SOIL LAYERING AND VARIABILITY

The capability of the CPT (and CPTu) for assessing soil layering and variability is well documented (e.g. Lunne et al., 1997). As a practical example, Figure 2 presents CPT data from Site V, indicating sand, overlying normally consolidated clay, overlying overconsolidated clay. As well as highlighting the strength of the CPT for differentiating soil layers and transition zones, Figure 2 also illustrates the CPT’s ability to identify variations in soil macro-fabric.

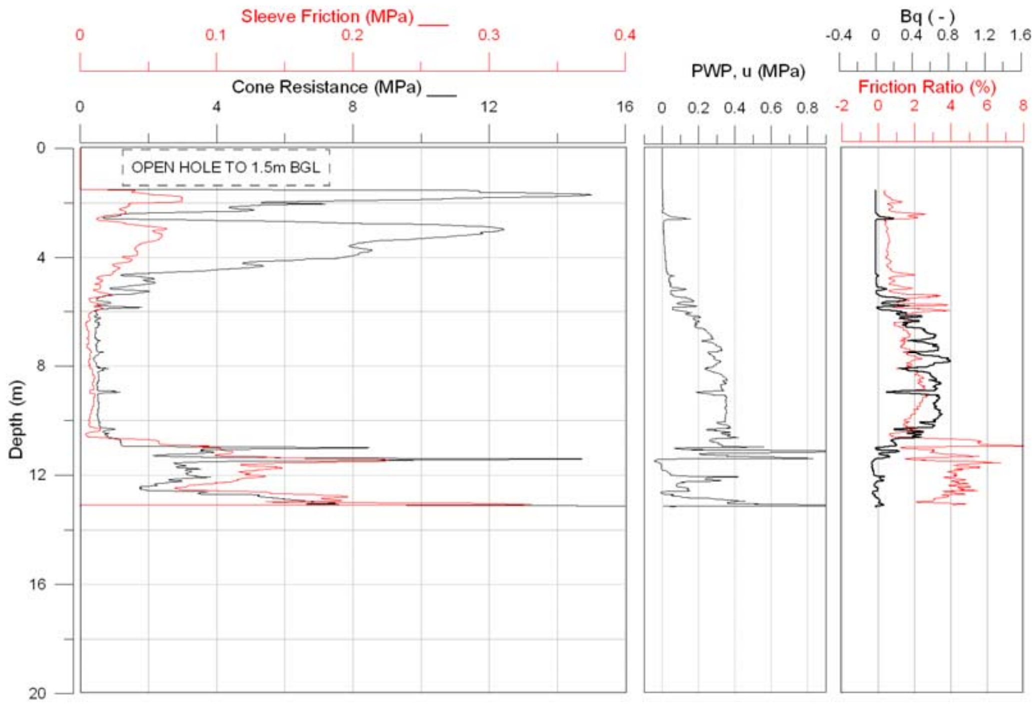

Figure 2: Site V - Differentiation of soil layers and identification of macro-fabric

The value of the CPT for identifying variability within similar soil units is illustrated in Figure 3, using data from Site Z. Although the eight CPTs at this site were separated laterally by approximately 25km, the cone resistance profiles at seven of the eight locations are similar – indicating a relatively uniform depositional environment over the area of interest. The anomalous cone resistance profile at the eighth site was later attributed to the presence of a nearby salt intrusion (diapir), which had increased the lateral effective stresses in the vicinity of the test.

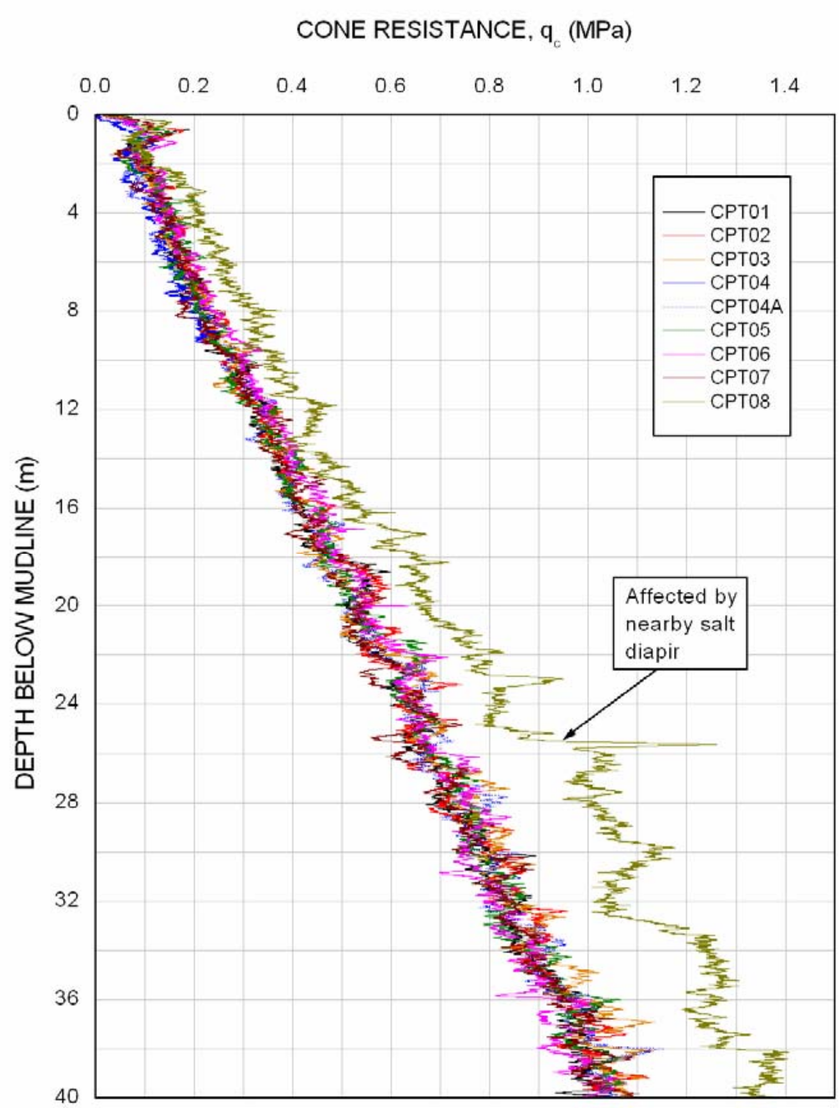

Figure 3: Site Z - Identification of anomalies

Figure 4 illustrates the ability of the CPT to differentiate even relatively subtle differences is geological units. Site Y is located offshore North Africa in an area containing a large variety of geohazards, including mud volcanoes and areas that have previously experienced major mass movements. As a consequence, the

depositional environment is extremely variable. To provide a reference framework, a statistical approach was used to assess individual soil units and grouping of similar soils. Although the four locations shown in Figure 4 are spread over an area of approximately 4km by 8km. the results clearly suggest that the depositional environment, at all four locations, is similar.

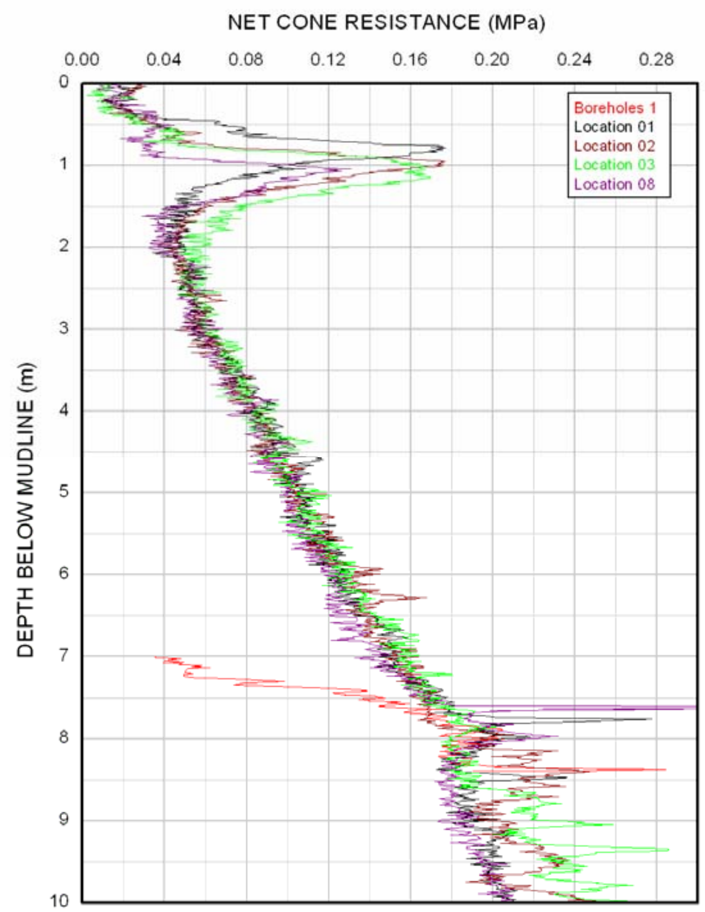

Figure 4: Site Y - Variation of net cone resistance

At the site, the quantity of site investigation was sufficient to enable statistical characterization of the CPT results to identify statistical signatures for each soil unit.

The statistical signature was based on:

- The average normalised cone resistance, Q_t.
- The coefficient of variation of Q_t.
- The calculated "best fit" slope of the net cone resistance, q_{net}, values with respect to depth.
- The regression coefficient, "r", representing the closeness of the depth and q_{net} data to a straight line.

This approach enabled:

- A quantitative objective method of recognising individual units and unit boundaries to be established.
- A quantitative objective method for assessing soils that may have been subjected to mass-movement to be established.
- A calibrated estimate of soil parameter values from cone penetration tests, at locations where no geophysical or laboratory data were available - including the undrained shear strength, remoulded shear strength, overconsolidation ratio, carbonate content, constrained modulus and small strain shear modulus (G_{max}).

The ability of CPTs to assess variations in soil fabric has already been mentioned. Published information normally suggests that the order of precedence for maximising resolution and reliability is typically; pore-pressure ratio, B_q, followed by friction ratio, R_f, followed by cone resistance, q_c. To illustrate, the capability of these sensors, Figures 5 to 7 present a sample of composite results between 2m and 6m below mudline at Site Z. It may be seen, by inspection, that the B_q profile can be used to detect thin layers of relatively permeable soil that cannot be discerned using the other the other sensors. These results confirm that the B_q parameter is the premier parameter for assessing variations in soil macro-fabric, at least for offshore sites, where sensor saturation is more reliable.

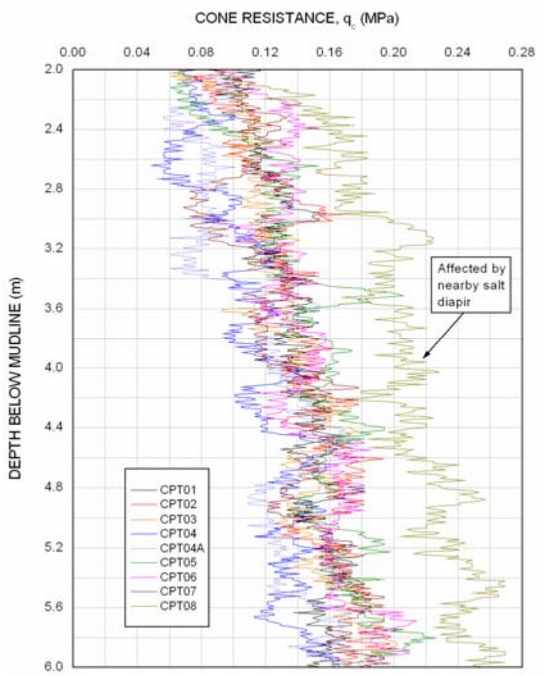

Figure 5: Site Z, q_c profiles

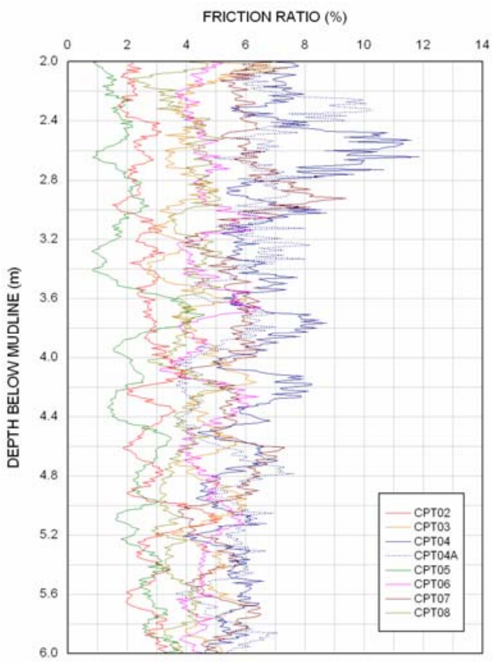

Figure 6: Site Z, R_f profiles

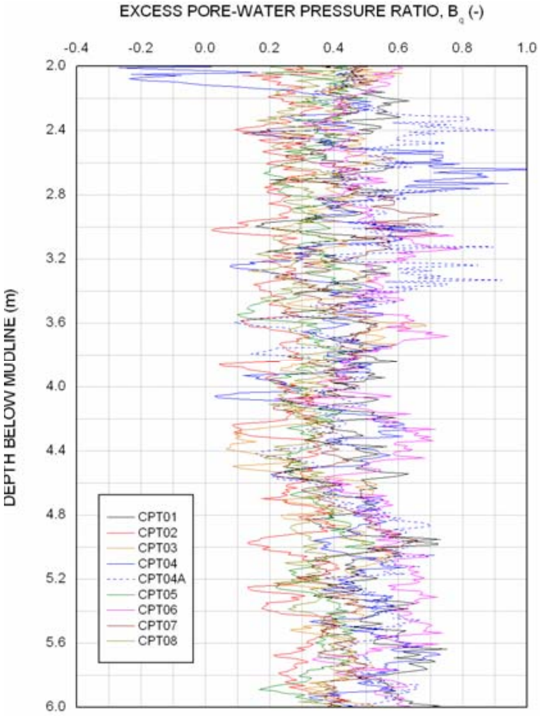

Figure 7: Site Z, B_q profiles

One of the limitations of performing CPTs in fine-grained soils containing granular inclusions, such as coarse-gravel and cobbles, is that these inclusions can distort the soil interpretation by causing sharp reductions in pore-water pressure (pwp) that temporarily impair the performance of the cone sensor, when the cone sensor is located on the cone shoulder. These rapid reductions in pwp are caused by the inclusion being pushed aside by the cone, thus creating local suctions adjacent to the pwp sensor. The time taken for the sensor to return to normal performance depends on the stiffness and permeability of the soil being tested, and on the magnitude of the reduction in pwp. Typically, the pwp sensor recovers within 0.5m to 1m penetration, but in severe cases, the pwp sensor may take metres of penetration to recover, or may not recover for the whole test – this behaviour is often referred to as "cavitation". When this behaviour is caused by coarse gravel or cobbles, it is common to observe complementary "spikes" in the friction sleeve readings, which are caused by local increases in lateral stress, as the inclusion is pushed to one side. If the inclusion is dilatant sand, spikes in the friction sleeve readings are seldom observed. Figure 8 presents cavitation data obtained from a variety of sites, with water depths ranging from less than 20m to almost 250m. It may been that, in general, the pwp sensor appears to cavitate when the pwp reduces to the ambient pressure at the cone shoulder, regardless of whether glycerine or silican oil has been used to saturate the cone sensor.

Another disadvantage of performing CPTs is the limited thrust that can be applied to the cone, either due to the limitation on the magnitude of reactive force, or due to the risk of damage to the cone. As a consequence, CPTs tend not to be the tool of choice for accurately assessing the depth to rock-head (i.e. bedrock), or deep soil profiling in coarse-grained soils, or heavily overconsolidated fine grained soils. In

volcanic soils, such as the basaltic clays found around Melbourne, Australia, the problem is exacerbated by the presence of isolated boulders (floaters) of relatively unweathered material in a soil matrix. It would be wrong to suggest, however, that boreholes always produce better results. This is illustrated in Figure 9, where a series of boreholes performed over a site failed to identify a zone of relatively unweathered basalt beneath a critical part of a proposed structure. In this case, geophysics would have been the most reliable approach for assessing this boundary.

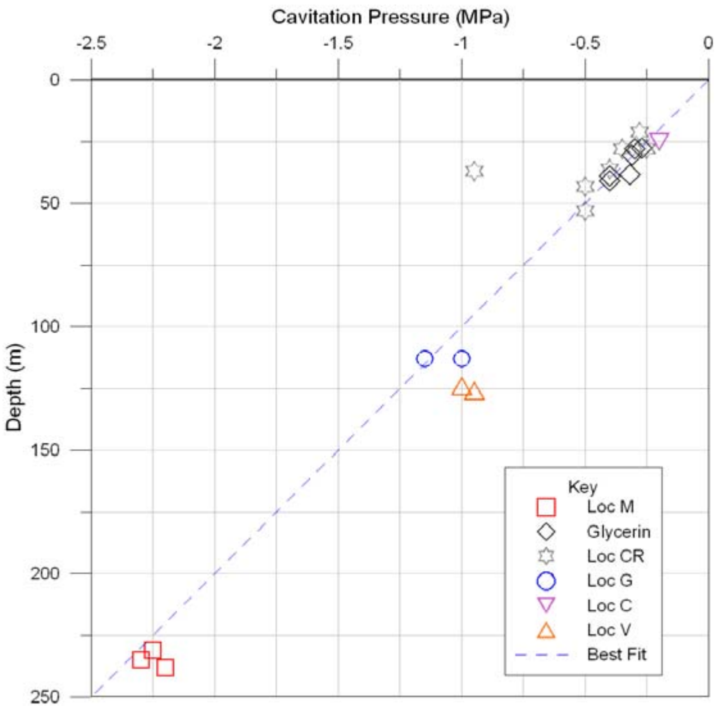

Figure 8: Measured whole-test cavitation pressure versus depth of water for pwp sensor on cone shoulder

Figure 9: Photo showing unweathered Basalt rock near the surface at a site in Victoria, Australia

3 PREDICTING SOIL PARAMETER VALUES

Published correlations provide a useful means of predicting soil property values from CPTs, and there are many excellent papers on this subject, most recently Robertson (2009). Robertson (2009) correctly points out that for all but low-risk projects CPT should be supplemented with other types of in-situ testing and/or more advanced project-specific laboratory testing. Two examples are provided in Figure 10 and Figure 11 to illustrate why this approach is recommended. Figure 10 presents remoulded shear strength profiles from Site Y – including a profile generated from the design undrained shear strength profile using sensitivity calculated using the approach recommended by Robertson (2009), where sensitivity is calculated using Equation 1.

$$S_t = 7.1/R_f \tag{1}$$

It may be seen that there is good agreement between the measured remoulded strengths and the generated profile. It is also interesting to note that the author's typical approach, of assuming that the remoulded strength is two-thirds of measured sleeve friction, also produces good agreement.

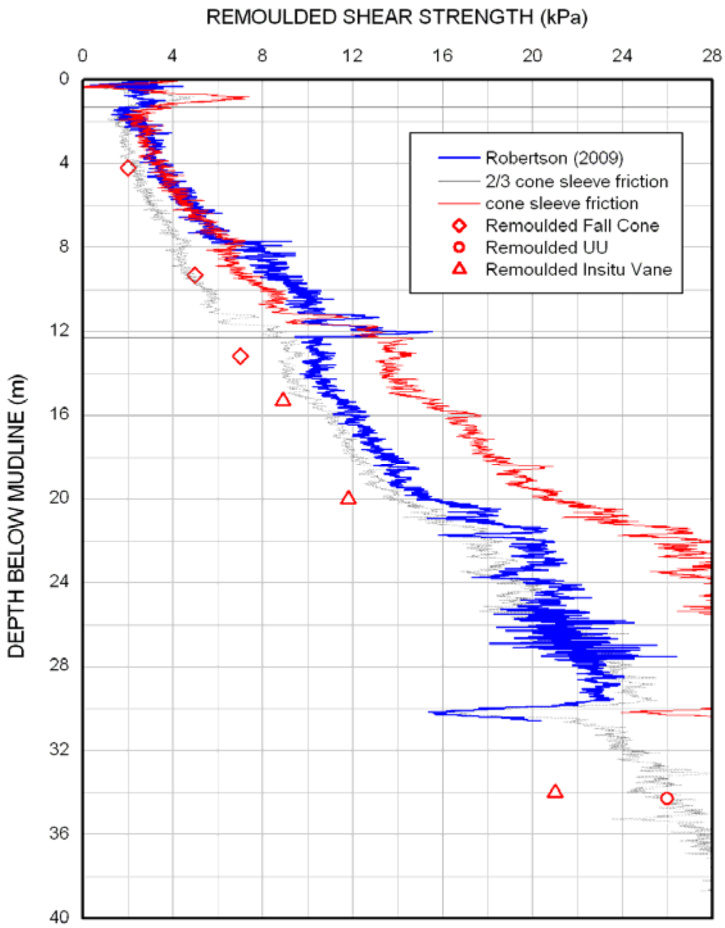

Figure 10: Site Y – Remoulded shear strength compared to CPT sleeve friction

In contrast, at the same site, the estimation of G_{max}/q_{net} directly from CPTs, using the approach suggested by Robertson (2009) and shown in Equation 2, is not in good agreement with values inferred from laboratory tests (Figure 11). Note that the laboratory tests were performed on test specimens inferred to be of good quality, so sample disturbance is not considered to be the cause of the discrepancy between the measured and inferred values. It is also worthy of note that Robertson (2009) points out that the formula presented in Equation 2 is less reliable in NC fine-grained soils.

$$G_{max} = [10^{(0.55\, I_c + 1.68)}]\,(q_{net} - \sigma_v) \qquad (2)$$

where:

$$I_c = [3.47 - \log Q_t)^2 + (\log R_f + 1.22)^2]^{0.5} \qquad \text{(Robertson, 2009)} \qquad (3)$$

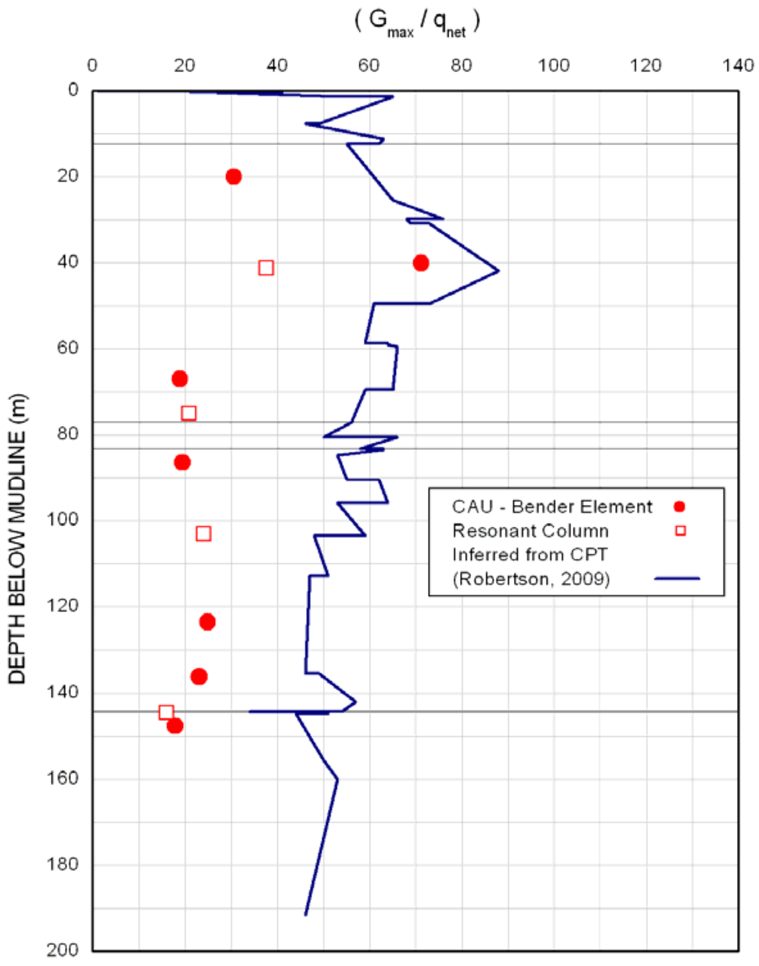

Figure 11: Site Y - Comparison between estimated G_{max}/q_{net} based on Robertson (2009) and measured values on laboratory samples

4 INTEGRATING IN-SITU AND LABORATORY TESTING

The advantages and disadvantages of CPTs and laboratory testing are summarised in Table 2. It may be seen that the pros and cons of the two techniques are, in most cases, complementary. As a consequence, the reliability of each is significantly increased when performed in combination with the other.

	CPT	Lab Testing
Pros	• Provide precise boundary elevations oftern missed by drilling techniques. • Provide accurate profiling information, including the proportion of soil inclusions and the proportion of intermittent layers of higher permeability. • Provide information on natural soil variability within geological units. • Provide information on the characteristics of the soil matrix.	• Enable the effects of increasing strain levels to be assessed on soil behavior – particularly changes in effective stress, strength and stiffness. • Soil samples can be reconsolidated to take account of the increases in effective stresses caused by the foundation loading. • Tests are performed in a controlled environment of strain-rate, drainage and temperature. • Soil behavior under cyclic loading conditions can be assessed.
Cons	• Do not provide information on the effect of the foundation on soil behavior. • Do not provide information on the effect of different rates of loading on soil behavior.. • Do not provide information on soil behavior under cyclic loading conditions. •	• A much smaller amount of soil is being tested so macro effects may not be representative of macro-behaviour • Sensitive to the quality of the test specimen and the method of specimen preparation (particularly sand soils).

Table 2: Advantages and disadvantages of CPT and laboratory testing

The improved reliability associated with combining CPTs and laboratory testing is illustrated in Figure 12 Both conventional CPTs and seismic CPTs (SCPT) were used to estimate and measure G_{max} in a loose to medium dense silty sand layer, beneath a proposed large Concrete Gravity Structure (CGS) at Site X in the North Sea. The seismic cone results were considered to provide a reliable measure of the in-situ G_{max} prior to the CGS installation. However, due to the critical nature of G_{max} for foundation design, it was considered appropriate to use laboratory testing to assess G_{max} after the CGS had been installed. As a consequence, bender element and resonant column laboratory tests were performed on soil specimens reconstituted to

relative densities inferred from the CPT data and then consolidated to take account of the additional effective stresses induced by the structure loading. The results from these tests indicated an increase in G_{max}, which was used in design. In addition, the laboratory test results were in good agreement with a modified G_{max} profile, calculated using the procedure recommended by Lunne et al (1997) to account for the effect of increased foundation stresses. The good agreement between the two approaches was considered to increase the reliability of the chosen G_{max} design profile.

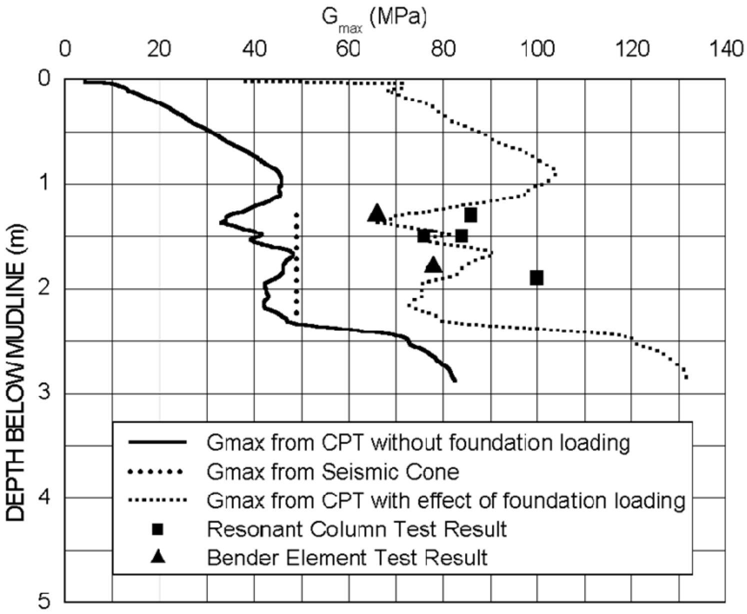

Figure 12: Site X – G_{max}, profiles

5 SOIL CLASSIFICATION METHODS

A crucial component for successful foundation design is accurate interpretation of soil type to enable the practising design engineer make reliable assessments of soil behaviour. To assist in this assessment a comparison of several published approaches is presented in Figures 15 to 22.The assessment has been made using the author's database which comprises 439 data points, with tests depths ranging from less than 0.3m to greater than 100m below ground level, at 101 testing and sampling locations, spread over a wide geographical area. The database is taken only from offshore piezocone data, to ensure that all tests were performed in saturated soils.

The Ramsey (2002) database was divided into nine soil categories, separated according to the criteria presented in Table 3:

Category	General Description	S_t (-)	OC (% by weight)	s_u / p_o' (-)	Fines Content (%)	Clay Content (%)	Relative to A' Line
1	Extra sensitive clay soils, i.e. soils with a sensitivity greater than 8.	>8	-	-	>35	>12	Above
2	Organic soils and peat, i.e. soils with an organic content greater than 5% by weight.	-	>5	-		-	-
3	Clay soils with a ratio of s_u / p_o' ≤ 1 (approximately corresponding to normally consolidated to slightly overconsolidated clay)	-	-	≤ 1	>35	>12	Above
4	Clay soils with a ratio of s_u / p_o' > 1 (approximately corresponding to significantly overconsolidated clay)	-	-	> 1	>35	>12	Above
5	clayey SAND	-	-	-	≤35%	≥5 - 12%	Below
6	sandy very clayey SILT	-	-	-	>35% - 65%	≥5 - 12%	Below
7	sandy SILT	-	-	-	>35 – 65%	<5%	-
8	silty SAND	-	-	-	>12 – 35%	<5%	-
9	"clean" to slightly silty SAND/GRAVEL	-	-	-	≤12%	<5%	-

Notes:
(1) Zones 1 and 2 are based directly on the Robertson (1990) classification model.
(2) S_t = Sensitivity
(3) OC = Organic Content (by weight).
(4) 'A' line indicates the plotting position of Atterberg Limits results relative to the 'A' line

Table 3: Soil Zone Categorisation (Ramsey, 2002)

The Ramsey (2002) Model (Figure 13 and 14) evolved during the 1990s, when the author was working at Fugro Limited in the UK. The database, presented in this paper, was not used to develop the model; it was merely used to corroborate its effectiveness in a quantitative way. This was achieved by asking a person with no previous experience with CPT to look through Fugro's archives to find situations where CPT data were available adjacent to laboratory test results, then compare the predicted soil category with the measured soil category. As a consequence, the database is considered to be highly reliable. Full details of the methodology used to verify the database, and the results of the assessment, are presented in Ramsey (2002).

The database is presented on the following figures:

- Figure 15: Schmertmann (1969)
- Figure 16: Douglas and Olsen (1981)
- Figure 17: Jefferies and Davies (1991)

- Figure 18: Schneider (2008)
- Figure 19: Robertson R_f plot (1990)
- Figure 20: Robertson B_q plot (1990)
- Figure 21: Ramsey R_f plot (2002)
- Figure 22: Ramsey B_q plot (2002)

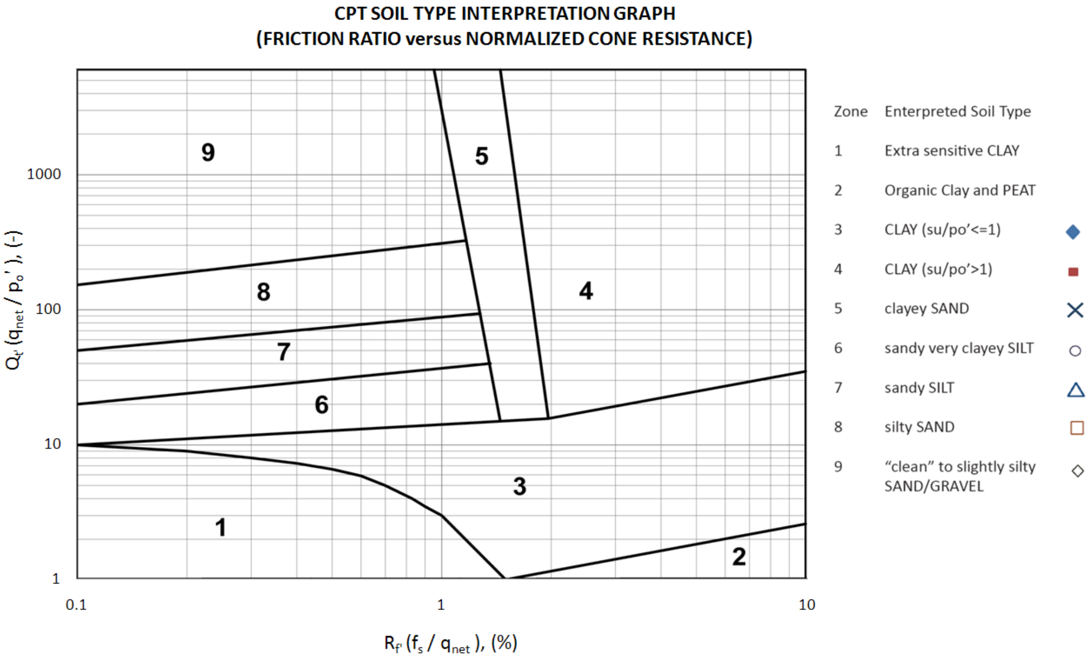

Figure 13: Ramsey (2002) R_f classification plot

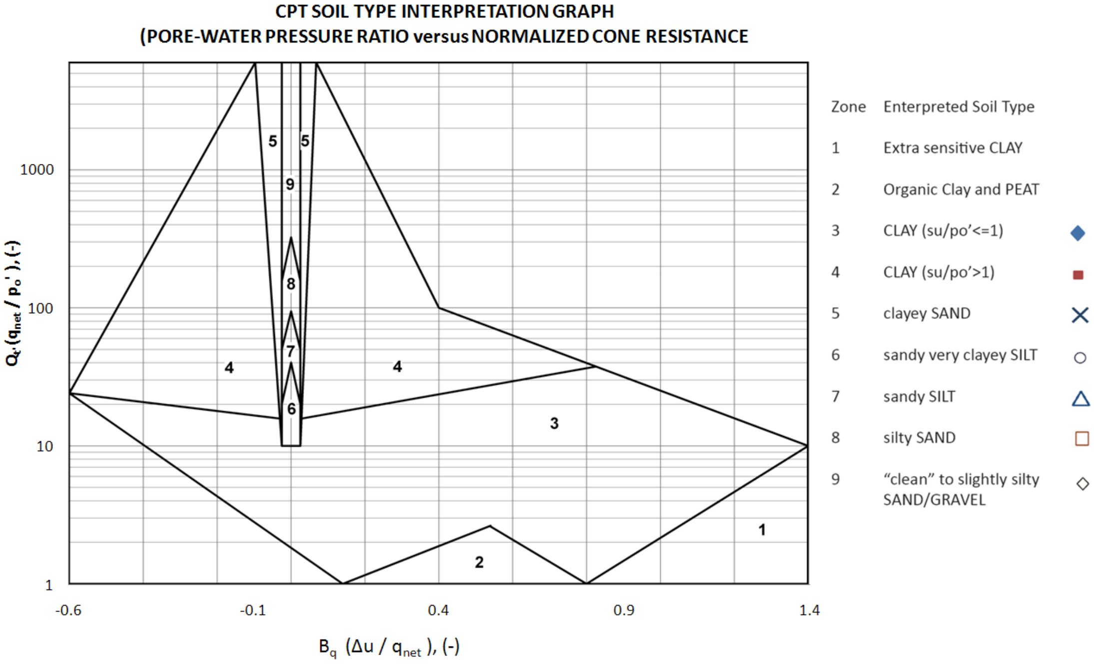

Figure 14: Ramsey (2002) B_q classification plot

It is not the purpose of this paper to make detailed evaluations of the relative merits of each model, and it is acknowledged that each of the models has strengths, weaknesses and limits of application. Instead, it is left to the readers to assess the information, and make their own decisions. However, some general comments are:

a) Methods that use appropriately corrected and normalized cone parameters, tend to have a greater range of application than methods that apply no correction or normalization..

b) Most of the models are relatively good at differentiating between clean sand and normally consolidated to slightly overconsolidated clay ($s_u/p_o' < 1$).

c) Intermediate soils tend to be much more difficult to differentiate.

d) None of the published friction ratio soil classification models, presented in this paper apply for tests performed using cones with non-standard projected areas of $2cm^2$ or $1cm^2$, as friction ratios measured using these cones tend to differ considerably from values obtained using standard $10cm^2$ cones.

e) None of the models presented in this paper give reliable soil classification in calcareous soils, due to the variations in grain compressibility, carbonate content and degree of cementation, that often occur in these soils over very short distances.

f) None of the models reliably predict peat or highly organic materials.

One area in which the Ramsey (2002) model differs from the other models is that the soil classification zones were purposely chosen so that, if there is a discrepancy between soil classifications according to the two models, the decision-making process to assess the matrix soil classification is based on a single criterion. This criterion is simply that whenever the models predict different zones then the zone with the lower numerical value should be chosen. This is because it is generally more conservative to interpret a soil with a higher fines content. So, for example, if the Q_t/R_f model indicates Zone 4 and the Q_t/B_q model indicates Zone 5 (or any zone higher than 5), then the matrix soil classification should be classified as Zone 4. It should be noted, however, that in keeping with previous discussions, the B_q parameter, and hence the Q_t/B_q model, is much better at detecting thin secondary layers or inclusions within a primary soil matrix, provided the pwp sensor remains saturated. Therefore, if it is deemed important to define the extent and thickness of thin layers of coarser material, within a fine grained soil layer, then the Q_t/B_q model data should be reassessed and, if appropriate, used to over-ride the Qt/R_f model.

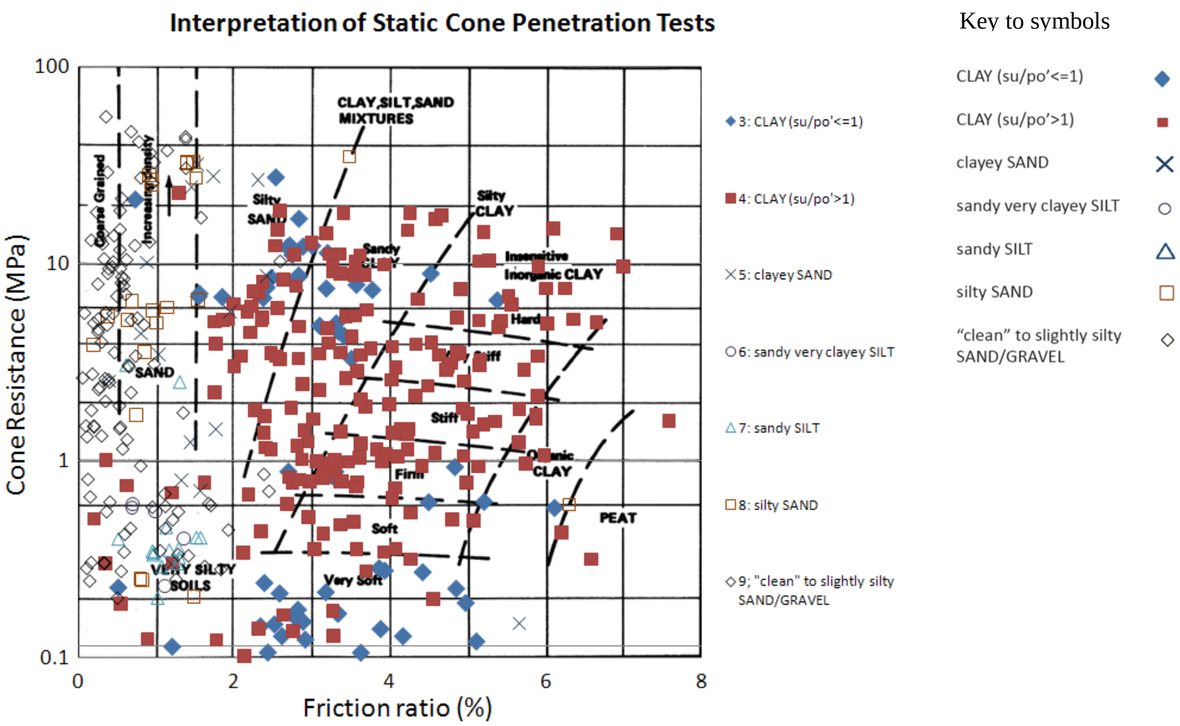

Figure 15: Schmertmann (1969) Soil Classification Plot

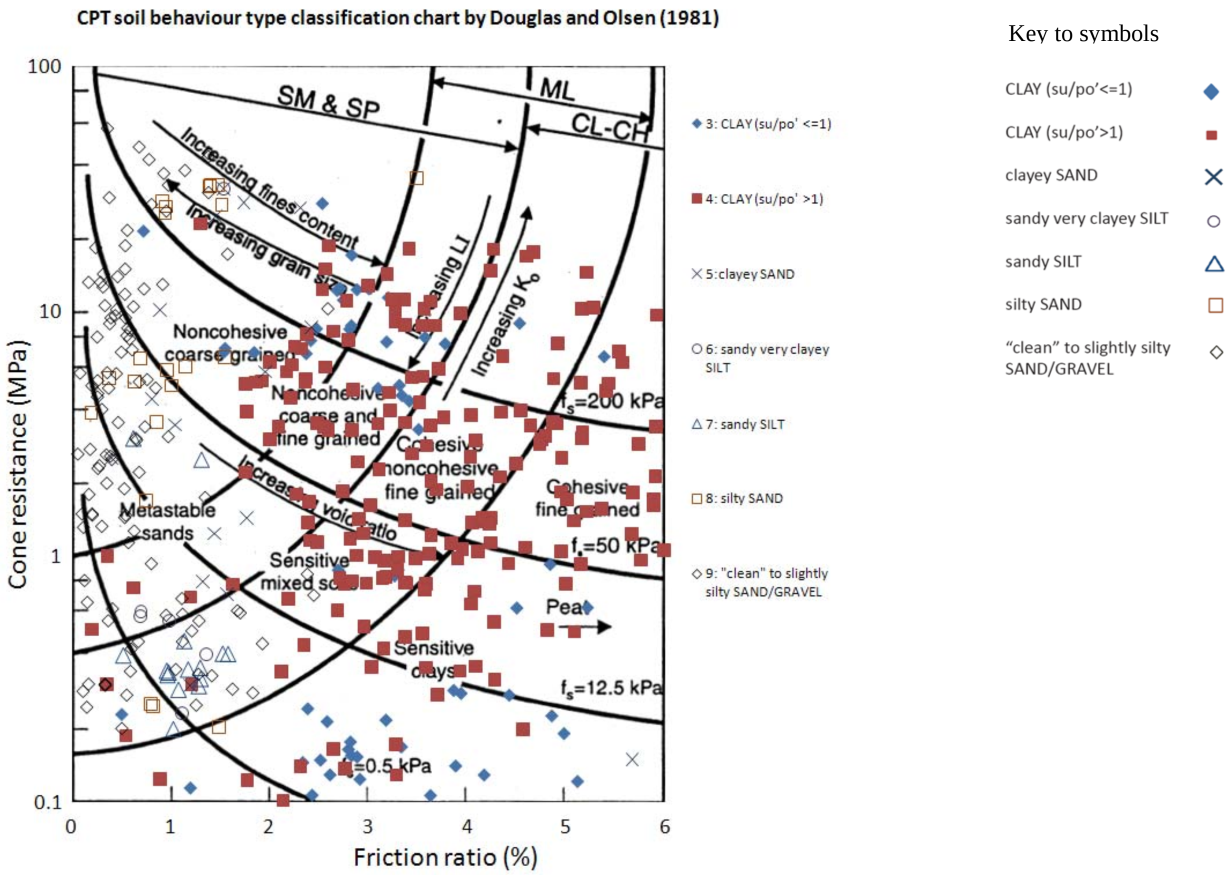

Figure 16: Douglas and Olsen (1981) Soil Classification Plot

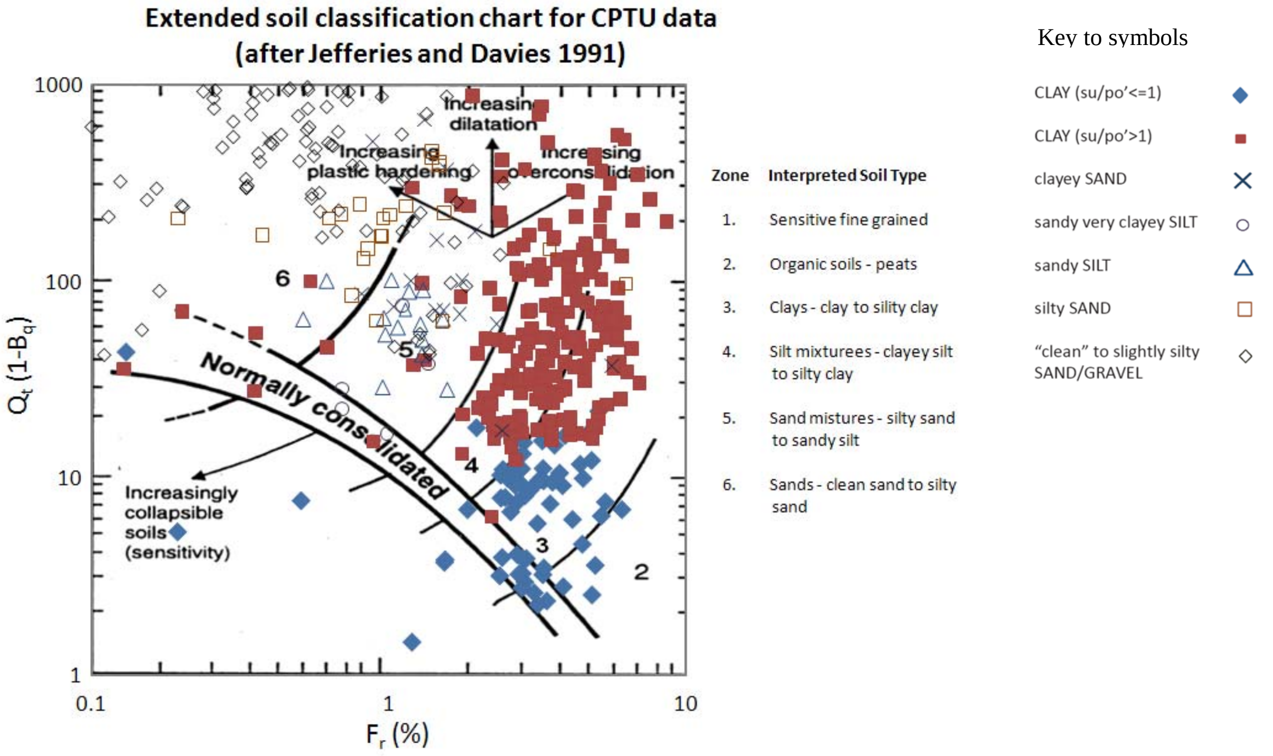

Figure 17: Jefferies and Davies (1991) Soil Classification Plot

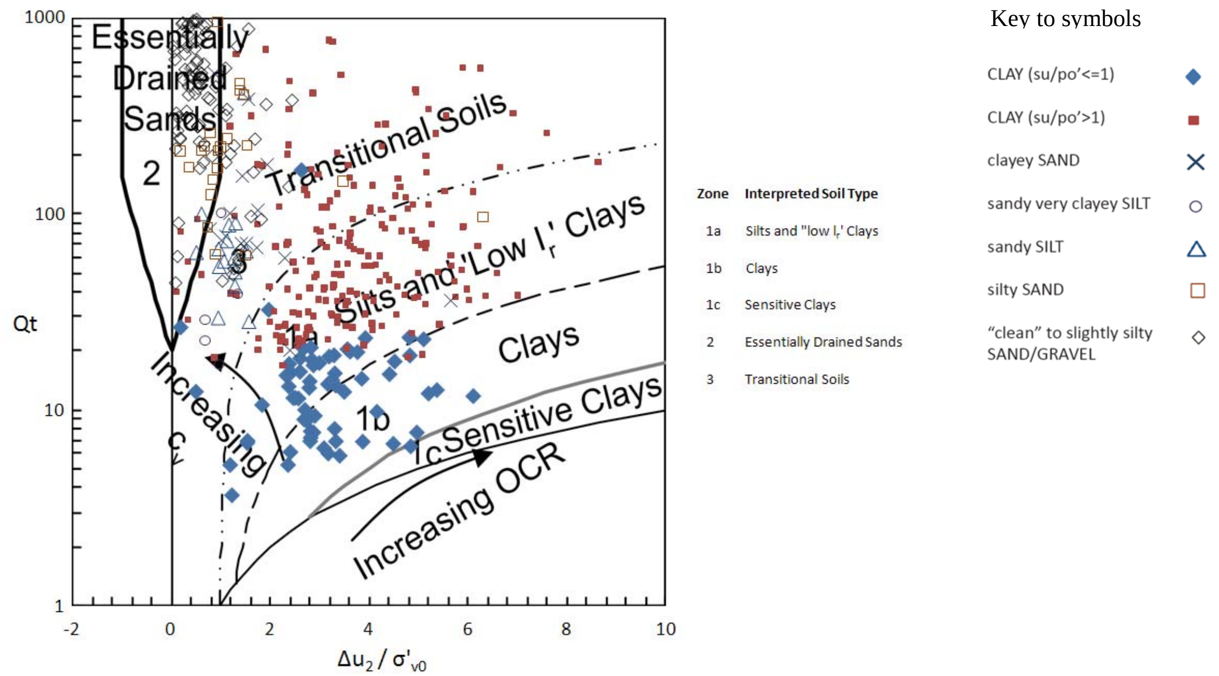

Figure 18: Schneider (2008) Soil Classification Plot

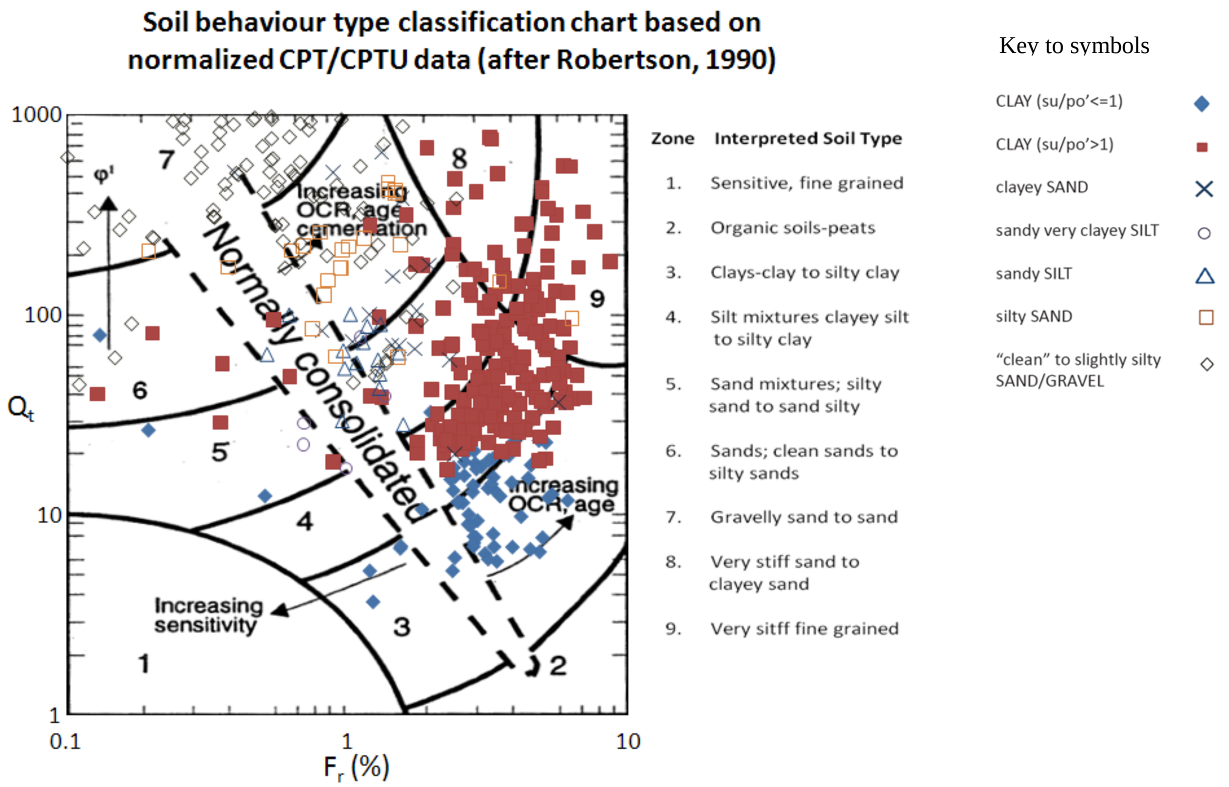

Figure 19: Robertson (1990) Soil Classification R_f plot

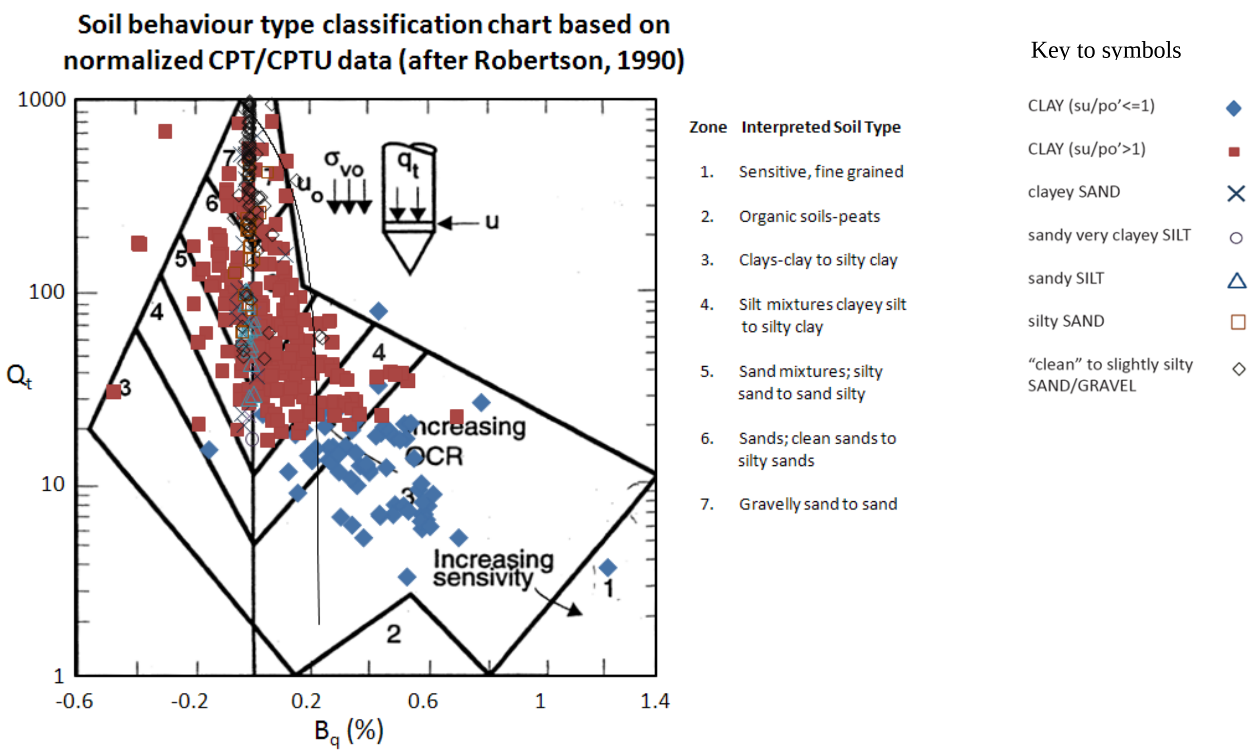

Figure 20: Robertson (1990) Soil Classification B_q plot

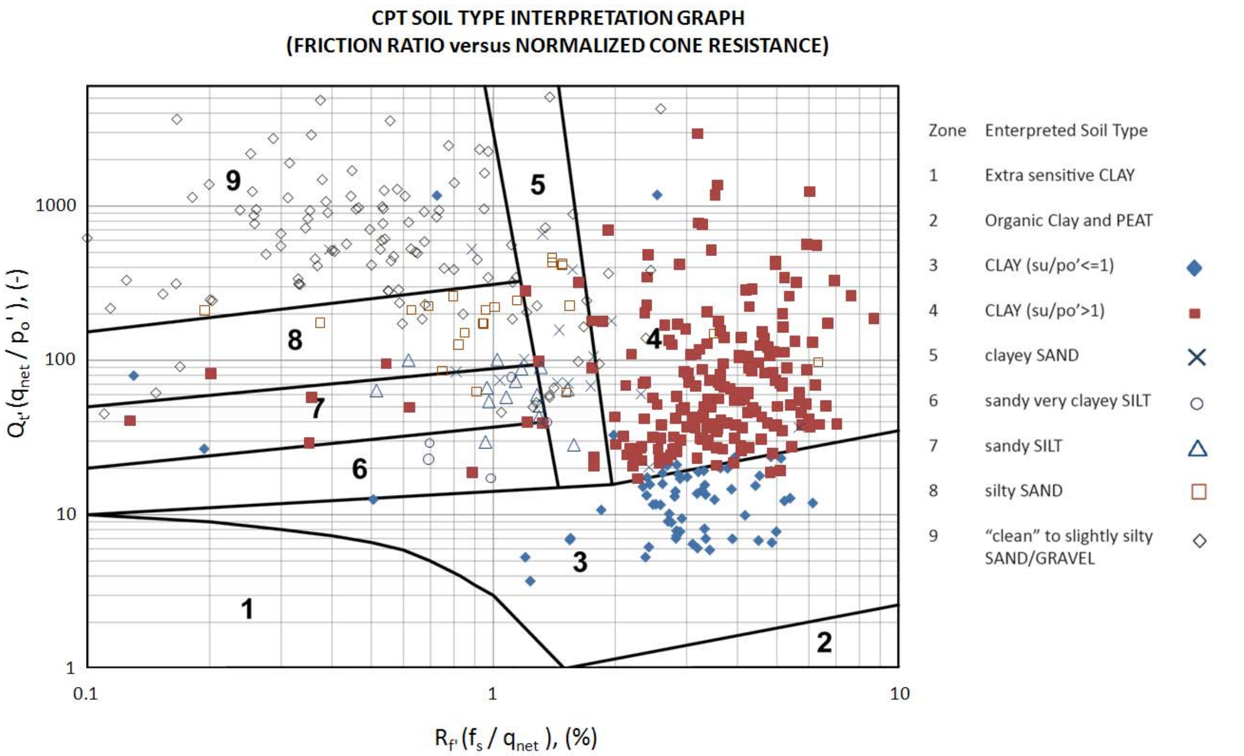

Figure 21: Ramsey (2002) Soil Classification R_f plot

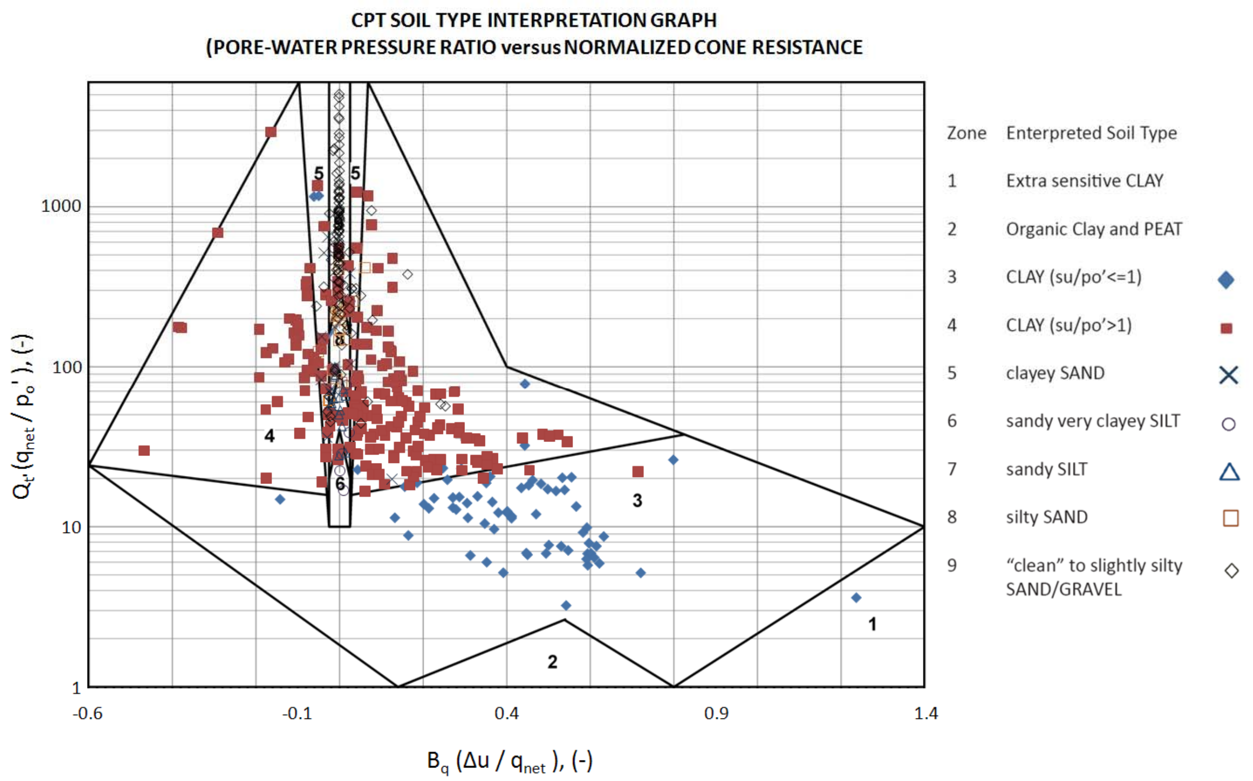

Figure 22: Ramsey (2002) Soil Classification B_q plot

6 CPT IN SOFT SOILS

Figure 23 presents a CPT performed at Site S, in South Australia, in predominantly normally consolidated clay soils. It may be seen that, at this site, calculating the net cone resistance, using vertical total stress based on measured soil unit weights, results in negative cone resistance between 12m and 17m depth. It is the author's experience that there are other similar cases in very soft normally consolidated soils. In these instances, the author's standard practice has been to recalculate the net cone resistance using half the total stress, rather than the total stress. It is the author's experience that this approach generally results in much better correlations with other in-situ and laboratory test results. This approach is in keeping with Wroth (1988) who pointed out that the correct equation for net cone resistance should be as shown in Equation 4. It is the author's suggestion that it would be better to adopt $K_o = 0.5$, as standard practice, as the only soils likely to be significantly affected by this change would be those soils with Q_t values less than 5, i.e. those soils where the K_o value would tend to be closer to 0.5, rather than the standard value of 1.0 that is currently used.

$$q_{net} = q_t - K_o.\sigma_v \tag{4}$$

The discussion above is explicitly based on the assumption that the measured cone sensor readings are accurate. In the author's experience, if the measured cone resistance is zero, or lower, it is almost always due to one of the following:

a) Incorrect calibration.

b) Insufficient sensor resolution.

c) Incorrect zero load reading at the start of the test.

The only other obvious alternatives are that the soil layer is still consolidating after the recent addition of overburden (i.e. under consolidated), or there are artesian conditions.

In stiffer soils ($Q_t > 20$), small errors in the assessment of zero load readings often have a negligible influence on the inferred soil parameters, but in the case of soft soils a small error can cause major errors. As an example of the implications of incorrect calibration or incorrect zero load reading, Figure 24 presents two CPTs performed within a distance of less than 2m, by two different CPT operators at Site S. The cone resistance values measured by Operator E resulted in an anomalously low net cone profile. In contrast, undrained shear strength profiles, interpreted from a CPT and a T-bar test performed by Operator B, indicated remarkably similar results. On the basis of the discrepancies between the operators' data, the calibration and zero readings were rechecked, and Operator E confirmed that it had inadvertently used an incorrect procedure to record the zero load reading.

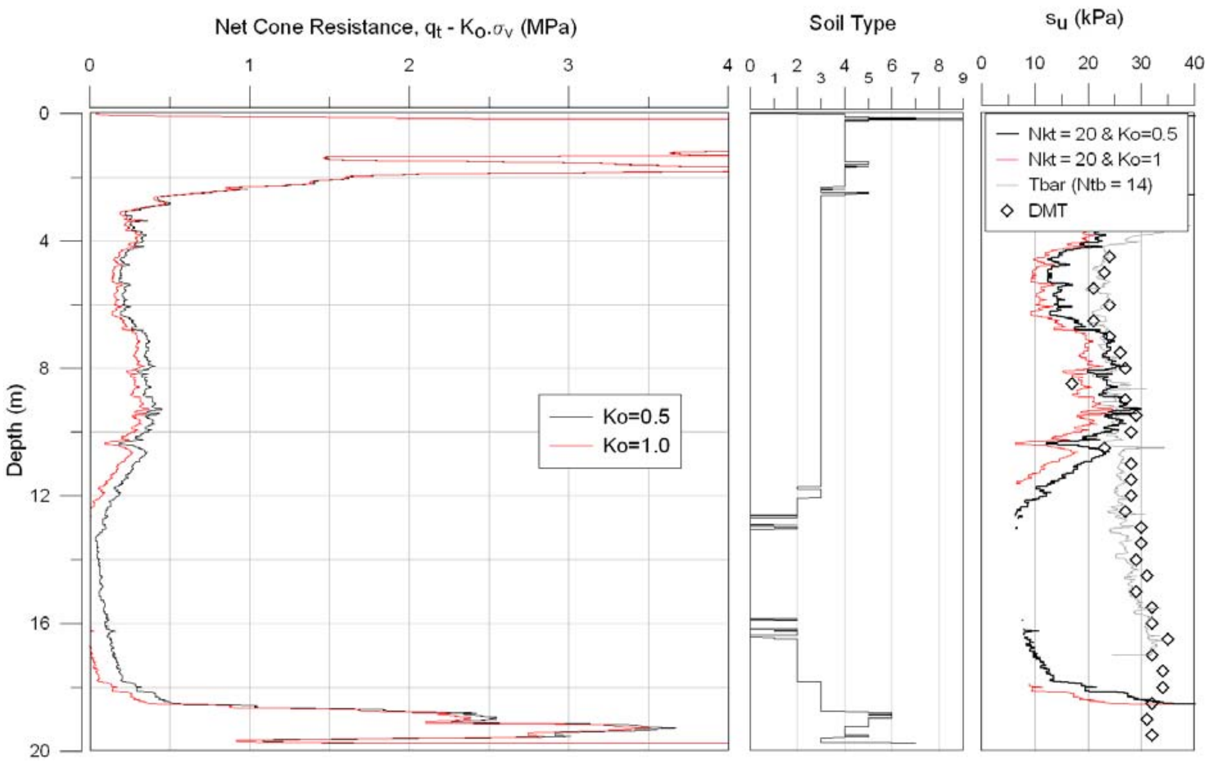

Figure 23: Site S – Net cone resistance, soil type and shear strength (based on K_o = 1.0 and 0.5) and associated undrained shear strength profiles

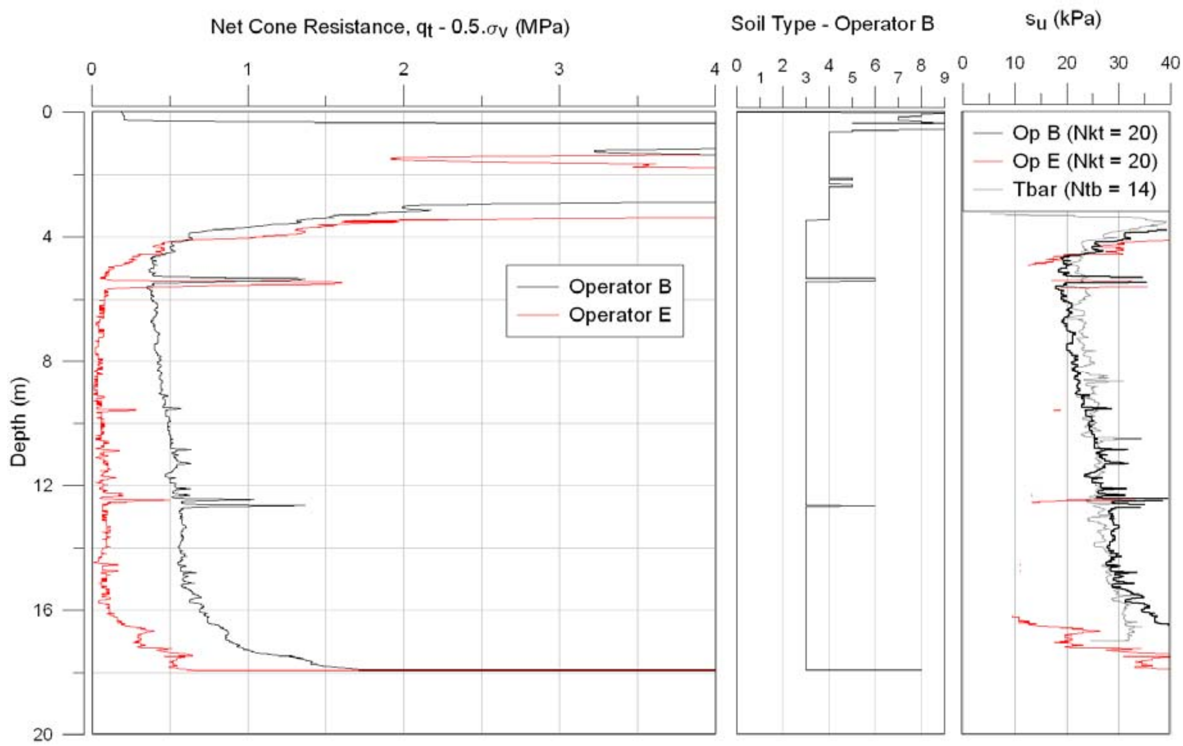

Figure 24: Site S - Comparison of results from two different CPT operators

7 CONCLUSIONS

This paper presents and discusses a few selected issues related to the application of CPTs. Emphasis has been placed on CPT profiles in predominantly normally consolidated fine-grained soils. On the basis of the data and discussions presented, the following observations and conclusions are considered appropriate.

1) The CPT is an excellent tool for assessing soil layers and soil variability.

2) The pore-pressure sensor, in particular, provides valuable information on thin layers of more, or less, permeable material, within a soil matrix.

3) The CPT has limitations, in terms of guaranteeing target penetration in very dense granular soils or in fine-grained soils containing coarse gravel-sized, or larger, inclusions.

4) Granular inclusions, within a generally fine-grained soil matrix, can cause temporarily impaired pore-pressure sensor readings.

5) Laboratory testing significantly enhances the benefits of CPT.

6) In some cases, statistical assessments can be used to identify similar geotechnical units over wide geographical areas and, in these cases, soil parameter values interpreted from CPTs, can be used with a good degree of reliability.

7) The choice of soil classification model has a significant bearing on the interpreted soils, if no complementary soil sampling and testing have been performed.

8) In soft soils, in particular, it is extremely important to ensure that the cones have sufficient resolution and have been correctly calibrated and zeroed prior to testing.

9) There are some grounds for defining the net cone resistance as the total cone resistance minus one half of the total overburden pressure, rather than the current standard practice of subtracting the total overburden pressure.

8 ACKNOWLEDGEMENTS

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8 KEY TO SYMBOLS AND TERMS

- B_q is the excess pore-water pressure ratio, defined as the ratio of the measured excess pore-water pressure to the net cone resistance.
- Clay content is defined as the % of particles with a diameter less than 2 microns.
- Coefficient of Variation (COV) of Q_t (expressed as %) is obtained by dividing the standard deviation of the Q_t values by the average Q_t value. It therefore indicates the uniformity of the Q_t values about the average.
- Fines content is defined as the % of soil grains passing a 63 micron sieve.
- f_s is the measured sleeve friction
- p_o' is the effective overburden pressure, also referred to as σ_v'.
- p_o is the total overburden pressure, also referred to as σ_v.
- q_c is the measured cone resistance
- q_t is the total cone resistance, i.e. the measured cone resistance, corrected for the effects of cone shape and pore-water pressure distribution around the cone tip.
- q_{net} is normally defined as the total cone resistance minus the total overburden pressure relative to ground level.
- Q_t is the normalized cone resistance, defined as the ratio of net cone resistance to effective overburden pressure.
- "r" coefficient - The "r" value is the regression coefficient representing the closeness of the depth and q_{net} data to a straight line. In general, an "r" value greater than +/-0.8 represents an excellent fit, greater than +/-0.6 represents a good fit, and a value greater than +/-0.4 represents a moderate fit.
- R_f is the friction ratio, defined as the ratio of the measured sleeve friction to the net cone resistance.
- S_t is the sensitivity, defined as the ratio of the undisturbed undrained shear strength to the remoulded undrained shear strength.
- s_u is the undrained shear strength, as measured by unconsolidated undrained triaxial compression tests.
- σ_v' is the effective overburden pressure, also referred to as p_o'.
- σ_v is the total overburden pressure, also referred to as p_o.

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