

Interpretation of in-situ tests – some insights

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ABSTRACT: The use and application of in-situ testing has continued to expand in the past few decades. This paper focuses on some major in-situ tests (SPT, CPT and DMT) and presents selected insights that the geotechnical engineering profession may find helpful. Many of the recommendations contained in this paper are focused on low to moderate risk projects where empirical interpretation tends to dominate. For projects where more advanced methods are more appropriate, the recommendations provided in this paper can be used as a screening to evaluate critical regions/zones where selective additional in-situ testing and sampling may be appropriate.

1 INTRODUCTION

The objective of this paper is not to provide a detailed summary on the interpretation of all in-situ tests, but to focus on some major tests and to present selected insights that the geotechnical engineering profession may find helpful.

The use and application of in-situ testing for the characterization of geomaterials have continued to expand over the past few decades, especially in materials that are difficult to sample and test using conventional methods. Mayne et al (2009) summarized the key advantages of most in-situ tests as:

- improved efficiency and cost effectiveness compared to sampling and laboratory testing,
- large amount of data, and,
- evaluation of both vertical and lateral variability.

Table 1 presents a summary of the current perceived applicability of the major in-situ tests. It is fitting that this J.K. Mitchell lecture/paper should start with this table, since it was first published in 1978 by Professor Mitchell (Mitchell et al., 1978). Professor Mitchell carried out research and published on a wide range of topics ranging from fundamentals of clay behavior to the use and interpretation of in-situ tests, with the Apollo moon landing one of his first in-situ test experiments. In-situ testing was only a part of his extensive and impressive research record.

Table 1 illustrates that the Cone Penetration Test (CPT), and its recent variations (e.g. CPTu and SCPTu), have the widest application for estimating geotechnical parameters over a wide range of materials from very soft soil to weak rock. This explains the continued growth in the use and application of the CPT worldwide. Hence, much of this paper will focus on the use and interpretation of the CPT.

2.0 ROLE OF IN-SITU TESTING

Before discussing in-situ tests, it is appropriate to briefly identify the role of in-situ testing in geotechnical practice. Hight and Leroueil (2003) suggested that the appropriate level of sophistication for a site characterization program should be based on the following criteria:

- Precedent and local experience
- Design objectives
- Level of geotechnical risk
- Potential cost savings

The evaluation of geotechnical risk was described by Robertson (1998) and is dependent on the hazards (what can go wrong), probability of occurrence (how likely is it to go wrong) and the consequences (what are the outcomes).

Traditional site investigation in many countries typically involves soil borings with intermittent standard penetration test (SPT) N-values at regular depth intervals (typically 1.5m) and occasional thin-

Group	In-situ Test	Geotechnical Parameter													Ground Type					
		Soil type	Profile	u ₀	OCR	D _{R-ψ}	φ'	s _u	G ₀ -E	σ-ε	M-C _c	k	c _v	hard rock	soft rock	gravel	sand	silt/clay	peat-organic	
Penetrometer/ Direct Push	Dy. Probing (DP)	C	B	-	C	C	C	C	C	-	-	-	-	-	C	B	A	B	B	
	SPT	B	B	-	C	B	C	C	C	-	-	-	-	-	C	B	A	B	B	
	CPT	B	A	-	B	B	B	B	B	C	C	C	-	-	B	B	A	A	A	
	CPTu	A	A	A	B	A	B	A	B	C	B	A	A	-	B	B	A	A	A	
	SCPTu	A	A	A	A	A	B	A	A	B	B	A	A	-	B	B	A	A	A	
	DMT	B	B	B	B	C	B	B	B	C	B	C	B	-	C	C	A	A	A	
	SDMT	B	B	B	A	B	B	B	A	B	B	C	B	-	C	C	A	A	A	
	Full-flow (T/ball)	C	B	B	B	C	C	A	C	C	C	C	C	-	-	-	C	B	A	
	Field vane (FVT)	B	C	-	B	-	-	A	-	-	-	-	-	-	-	-	-	A	B	
Pressuremeter	Pre-bored	B	B	-	C	C	C	B	B	C	C	-	C	A	A	B	B	B	B	
	Self-bored	B	B	A ¹	B	B	B	B	A	A	B	B	A ¹	-	C	-	B	A	B	
	Full-displacement	B	B	B	C	C	C	B	A	A	B	B	A	-	C	-	B	A	A	
Other	Screw/plate load	C	-	-	B	C	C	B	B	B	B	C	C	C	A	B	B	B	B	
	Borehole shear	C	-	-	-	-	B	C	-	-	-	-	-	C	B	C	C	C	-	
	Permeameter	C	-	A	-	-	-	-	-	-	-	A	B	A	A	A	A	A	B	
	Borehole seismic	C	C	-	B	C	-	-	A	C	-	-	-	A	A	A	A	A	B	
	Surface seismic	-	C	-	B	C	-	-	A	C	-	-	-	A	A	A	A	A	A	
	Hydraulic fracture	-	-	B	-	-	-	-	-	-	-	C	C	B	B	-	-	B	C	

Applicability: A = high, B = moderate, C = low, - = none

Geotechnical parameters: u₀ = in-situ static pore pressure, OCR = over-consolidation ratio, D_{R-ψ} = relative density and/or state parameter, φ' = peak friction angle, s_u = undrained shear strength (peak and/or remolded), G₀-E = small strain shear and/or Young's modulus, σ-ε = stress-strain relationship, M-C_c = constrained modulus and/or compression index, k = permeability, c_v = coefficient of consolidation

φ' will depend on soil type; ¹ only when pore pressure sensor fitted.

Table 1 Perceived applicability of in-situ tests (updated from Mitchell et al., 1978 and Lunne et al, 1997)

walled tube samples for subsequent laboratory testing. Additional specific tests, such as field vane tests (FVT) in soft clay layers and/or intermittent pre-bored pressuremeter tests (PMT) in harder layers are added for higher risk, larger projects. Increasingly a more efficient and cost effective approach is the utilization of direct-push methods using multi-measurement in-situ devices, such as the seismic cone penetration test with pore pressure measurements (SCPTu) and the seismic flat dilatometer test (SDMT). Since the CPTu is about 3 to 4 times faster, collects more frequent data and is less expensive than the DMT, the CPTu is increasingly the preferred primary in-situ test. The continuous nature of the CPTu results provide valuable information about soil variability that is difficult to match with sampling and laboratory testing. Given the above statements, emphasis in this paper will be placed on the interpretation of the CPTu.

For low risk projects direct-push logging tests (e.g. CPT and CPTu) and index testing on disturbed samples combined with conservative design criteria are often appropriate. For moderate risk projects, the above can be supplemented with additional specific in-situ testing, such as the SCPTu with pore pressure dissipation combined with selective sampling and laboratory testing to develop site specific correlations. For high risk projects, the above can be used for screening to identify potentially critical regions/zones appropriate to the design objectives, followed by possible additional in-situ tests, selective high quality sampling and advanced laboratory test-

ing. The results of the often limited laboratory testing are correlated to the in-situ test results to extend and apply the results for other regions/zones within the project.

A common complaint with most direct-push in-situ tests, such as the CPT and DMT, is that they do not provide a soil sample. Although it is correct that a soil sample is not obtained during either the CPT or DMT, most commercial operators carry simple direct-push samplers that can be pushed using the same direct-push installation equipment to obtain a small (typically 25 to 50 mm in diameter) disturbed soil sample of similar size to that obtained from the SPT. The preferred approach, and often more cost effective solution, is to obtain a detailed continuous stratigraphic profile using the CPT, then to move over a short distance (< 1m) and push a small diameter soil sampler to obtain discrete selective samples in critical layers/zones identified by the CPT. The push rate to obtain samples can be significantly faster (up to 20 times) than the 20 mm/s used for the CPT and sampling can be rapid and cost effective for a small number of discrete samples.

Many of the recommendations contained in this paper are focused on low to moderate risk projects where empirical interpretation tends to dominate. For projects where more advanced methods are more appropriate, the recommendations provided in this paper can be used as a screening to evaluate critical regions/zones where selective additional in-situ testing and sampling may be appropriate.

3.0 BASIC SOIL BEHAVIOUR

Mayne et al (2009) and others have identified the need for an interpretative framework within which to assess the results of tests and assign parameters or properties based on the measured response. Increasingly that framework is critical state soil mechanics (CSSM). Mayne et al (2009) presented a short summary of the main points in CSSM and showed that the basics of CSSM lie in the definition of only a few soil constants: effective friction angle (ϕ'_{cv}), compression index (C_c) and swelling index (C_s) as well as the initial state (e_0 , σ'_{v0} and either OCR or ψ).

Since most soils are essentially frictional in their behavior, they can be classified into either coarse-grained (e.g. sands) or fine-grained (e.g. silts and clays). The classification based on grain size is linked to drainage conditions during loading, where coarse-grained soils tend to respond drained during most static loading and fine-grained soils tend to respond undrained during most loading. Soils experience volume change during shear that can be either dilative or contractive. Hence, in a general sense, soil behavior can be classified into four broad and general groups: drained-dilative, drained-contractive, undrained-dilative and undrained-contractive. Hence, it is helpful if any in-situ test can identify these broad behaviour types.

Although there are a large number of potential geotechnical parameters and properties, the major ones used most in practice are in general terms: in-situ state, strength, stiffness, compressibility and conductivity. In-situ state represents quantification of the density and compactness of the soil, as well as factors such as cementation. For most soils, in-situ state is captured in terms of either relative density (D_r) or state parameter (ψ) for coarse-grained soils and over-consolidation ratio (OCR) for fine-grained soils. These 'state' parameters essentially identify if soils will be either dilative or contractive in shear. Sands with a negative state parameter ($-\psi$, i.e. 'dense') and clays with high OCR ($OCR > 4$) will generally dilate at large strains in shear, whereas sands with a positive state parameter ($+\psi$, i.e. 'loose') and normally to lightly over-consolidated clays ($OCR < 2$) will generally contract in shear at large strains. The tendency of a soil to either dilate or contract in shear often defines if the key design parameters will be either the drained shear strength (ϕ') or the undrained shear strength (s_u).

4.0 STANDARD PENETRATION TEST (SPT)

Although in-situ testing has evolved and improved over the past 25 years, several old and inadequate tests remain in common use in many parts of the

world. One of the oldest in-situ tests is the standard penetration test (SPT), remaining a staple in many site investigations around the world. Mayne et al (2009) correctly questions the "false sense of reality in the geotechnical engineer's ability to assess each and every soil parameter from the single N-value". Geotechnical engineers in the 21st century should progressively abandon this crude, unreliable in-situ test.

Reasons frequently given as to why the SPT continues to be used in some parts of the world are: (a) other better tests (e.g. CPT and DMT) are not locally available, (b) the SPT provides a soil sample for visual identification, (c) the SPT is inexpensive, and, (d) direct-push tests are not possible in the local soils. The lack of local availability in some parts of the developed world is often a circular argument where local site investigation contractors do not offer better tests because engineers are not requesting the better tests and engineers are not requesting the better tests because local contractors are not offering these tests. This cycle needs to be broken by geotechnical engineers requiring (demanding) better in-situ testing. Local contractors will then make the capital investment to provide these tests if local geotechnical engineers require them. There are also many alternate, cost effective and efficient direct-push methods to obtain small disturbed samples for visual identification and for index testing, as described above. The inexpensive nature of the SPT is also incorrect. The SPT is very expensive based on a per data point basis. Typically the cost of the SPT in the USA is about \$20 to \$25 per N-value (data every 1.5m), compared to about \$0.50 per data point for the CPT, because the CPT obtains more channels of data (typically 3) at more frequent intervals (typically every 20 to 50mm). It is frequently less expensive to push the CPT followed by a small number of discrete direct-push samples at selective depths, than it is to do conventional drilling and the SPT.

The SPT N-value becomes meaningless when $N > 50$ due to the limitation in the hammer energy. Generally, if direct-push reaction of at least 150 kN (15 tons) is available, the CPT can be pushed in soils with $N > 50$. With 200 kN (20 tons) reaction it is generally possible to push the CPT into most soils with $N > 100$.

Although direct-push in-situ tests such as the CPT and DMT are more efficient using customized, large pushing equipment, it is also possible to carry out these direct-push methods using conventional drilling equipment. Treen et al (1992) showed how the CPT can be carried out in a cost effective manner in stiff glacial soils using a very simple down-hole CPT pushed with a drill-rig. It is very easy to push a cone (either wireless or with a cable) into the bottom of borehole for a stroke of about 1.5m or more, similar to the way an SPT is performed but at a constant (non-dynamic) rate of penetration and recording

several channels of data, e.g. tip resistance (q_c) and sleeve friction (f_s). The 1.5m distance over which the CPT is pushed is then drilled (and sampled, if necessary) before repeating the procedure. This form of incremental down-hole CPT can provide a near continuous profile of CPT data in a cost effective and more reliable manner, compared to the SPT. Additional measurements, such as pore pressure (penetration pore pressure, u , and rate of dissipation, t_{50}) and shear wave velocity (V_s) can also be recorded using either a down-hole CPTu or SCPTu. Hence, up to five (5) independent measurements can be made in a cost effective manner using conventional, well proven equipment and procedures, compared to the single crude N-value. The drill-rig can also be used to obtain either small diameter direct-push samples or larger diameter undisturbed (thin-walled tube) samples in the critical soils identified by the CPT. The drill-rig can also be used to drill through hard layers (e.g. gravel) where direct-push methods may reach refusal. Hence, perceived low cost and the need for samples should no longer be used as an excuse for the continued use of the unreliable SPT.

Jefferies and Davies (1993) correctly suggested that the most reliable way to obtain SPT N values was to perform a CPT and convert the CPT to an equivalent SPT. Jefferies and Davies (1993) suggested a method to convert the CPT cone resistance, q_t , to an equivalent SPT N value at 60% energy, N_{60} , using a soil behaviour type index, $I_{c,JD}$. The method was modified slightly by Lunne et al (1997), based on the simpler soil behaviour type index defined by Robertson and Wride (1998), as follows:

$$(q_t/p_a)/N_{60} = 8.5 [1 - (I_c/4.6)] \quad (1)$$

Where q_t is the corrected cone tip resistance and I_c is the soil behaviour type index defined by Robertson and Wride (1998), that will be defined in detail later.

The above method has been shown to work effectively in a wide range of soils, although recent experience in North America has shown that equation (1) tends to under predict the N_{60} values in some clays. Figure 1 compares various relationships of $(q_t/p_a)/N_{60}$ as a function of I_c and presents a suggested updated relationship that can be defined by the following:

$$(q_t/p_a)/N_{60} = 10^{(1.1268 - 0.2817I_c)} \quad (2)$$

Equation 2 produces slightly larger N_{60} values in fine-grained soils than the previous Jefferies and Davies (1993) method. In fine-grained soils with high sensitivity, equation 2 may over estimate the equivalent N_{60} .

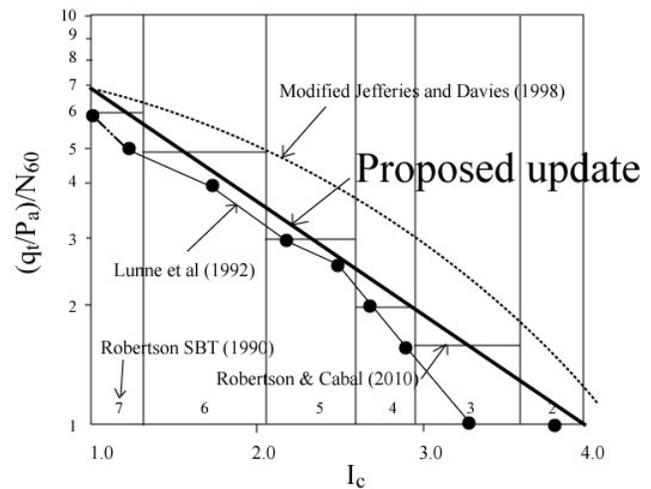


Figure 1. SPT-CPT correlations in terms of $(q_t/p_a)/N_{60}$ and CPT-based SBT index I_c

5.0 CONE PENETRATION TEST (CPT)

The electric cone penetration test (CPT) has been in use for over 40 years. The CPT has major advantages over traditional methods of field site investigation such as drilling and sampling since it is fast, repeatable and economical. In addition, it provides near continuous data and has a strong theoretical background. These advantages have produced a steady increase in the use and application of the CPT in many parts of the world.

Significant developments have occurred in both the theoretical and experimental understanding of the CPT penetration process and the influence of various soil parameters. These developments have illustrated that real soil behaviour is often complex and difficult to accurately capture in a simple soil model. Hence, semi-empirical correlations still tend to dominate in CPT practice although most are well supported by theory.

5.1 Equipment and Procedures

Lunne et al (1997) provided a detailed description on developments in CPT equipment, procedures, checks, corrections and standards, which will not be repeated here. Most CPT systems today include pore pressure measurements (i.e. CPTu) and provide CPT results in digital form. The addition of shear wave velocity (Robertson et al, 1986) is also becoming increasingly popular (i.e. SCPTu). If dissipation tests are also performed, the rate of dissipation can be captured by t_{50} (time to dissipate 50% of the excess pore pressures). Hence, it is now more common to see the combination of cone resistance (q_c), sleeve friction (f_s), penetration pore pressure (u), rate of dissipation (t_{50}) and, shear wave velocity (V_s), all measured in one profile. The addition of shear wave velocity has provided valuable insight into correla-

tions between cone resistance and soil modulus that will be discussed in a later section.

There are several major issues related to equipment design and procedure that are worth repeating and updating. It is now common that cone pore pressures are measured behind the cone tip in what is referred to as the u_2 position (ASTM D5778, 2007; IRTP, 1999). In this paper, it will be assumed that the cone pore pressures are measured in the u_2 position. Due to the inner geometry of the cone the ambient pore water pressure acts on the shoulder behind the cone and on the ends of the friction sleeve. This effect is often referred to as the unequal end area effect (Campanella et al, 1982). Many commercial cones now have equal-end area friction sleeves that essentially remove the need for any correction to f_s and, hence, provide more reliable sleeve friction values. However, the unequal end area effect is always present for the cone resistance q_c and there is a need to correct q_c to the corrected total cone resistance, q_t . This correction is insignificant in sands, since q_c is large relative to the water pressure u_2 and, hence, $q_t \sim q_c$ in coarse-grained soils (i.e. sands). It is still common to see CPT results in terms of q_c in coarse-grained soils. However, the unequal end area correction can be significant (10 - 30%) in soft fine-grained soil where q_c is low relative to the high water pressure around the cone due to the undrained CPT penetration process. It is now common to see CPT results corrected for unequal end area effects and presented in the form of q_t , f_s and u_2 , especially in softer soils. The correlations presented in this paper will be in terms of the corrected cone resistance, q_t , although in sands q_c can be used as a replacement.

Although pore pressure measurements are becoming more common with the CPT (i.e. CPTu), the accuracy and precision of the cone pore pressure measurements for on-shore testing are not always reliable and repeatable due to loss of saturation of the pore pressure element. At the start of each CPTu sounding the porous element and sensor are saturated with a viscous liquid such as silicon oil or glycerin (Campanella et al, 1982) and sometimes grease (slot element). However, for on-shore CPTu the cone is often required to penetrate several meters through unsaturated soil before reaching saturated soil. If the unsaturated soil is either clay or dense silty sand the suction in the unsaturated soil can be sufficient to de-saturate the cone pore pressure sensor. The use of viscous liquids, such as silicon oil and grease, has minimized the loss of saturation but has not completely removed the problem. Although it is possible to pre-punch or pre-drill the sounding and fill the hole with water, few commercial CPT operators carry out this procedure if the water table is more than a few meters below ground surface. A further complication is that when a cone is pushed

through saturated dense silty sand or very stiff over consolidated clay the pore pressure measured in the u_2 position can become negative, due to the dilative nature of the soil in shear, resulting in small air bubbles coming out of solution in the cone sensor pore fluid and loss of saturation in the sensor. If the cone is then pushed through a softer fine-grained soil where the penetration pore pressures are high, these air bubbles can go back into solution and the cone becomes saturated again. However, it takes time for these air bubbles to go into solution that can result in a somewhat sluggish pore pressure response for several meters of penetration. Hence, it is possible for a cone pore pressure sensor to alternate from saturated to unsaturated several times in one sounding. It can be difficult to evaluate when the cone is fully saturated, which adds uncertainty to the pore pressure measurements. Although this appears to be a major problem with the measurement of pore pressure during a CPTu, it is possible to obtain good pore pressure measurements in suitable ground conditions where the ground water level is close to the surface and the ground is predominately soft. In very soft, fine-grained soils, the CPTu pore pressures (u_2) can be more reliable than q_t , due to loss of accuracy in q_t in very soft soils. It is interesting to note that when the cone is stopped and a pore pressure dissipation test performed below the ground water level, any small air bubbles in the cone sensor tend to go back into solution (during the dissipation test) and the resulting equilibrium pore pressure can be accurate, even when the cone may not have been fully saturated during penetration before the dissipation test. CPTu pore pressure measurements are almost always reliable in off-shore testing due to the high ambient water pressure that ensures full saturation.

Even though pore pressure measurements can be less reliable than cone resistance for on-shore testing, it is still recommended that pore pressure measurements be made for the following reasons: any correction to q_t for unequal end area effects is better than no correction in soft fine-grained soils, dissipation test results provide valuable information regarding equilibrium piezometric profile and penetration pore pressures provide a qualitative evaluation of drainage conditions during the CPT as well as assisting in evaluating soil behaviour type.

It has been documented (e.g. Lunne et al., 1986) that the CPT sleeve friction is less accurate than the cone tip resistance. The lack of accuracy in f_s measurement is primarily due to the following factors (Lunne and Anderson, 2007);

- Pore pressure effects on the ends of the sleeve,
- Tolerance in dimensions between the cone and sleeve,
- Surface roughness of the sleeve, and,
- Load cell design and calibration.

ASTM, D5778 (2007) specifies the use of equal end-area friction sleeve to minimize the pore pressure effects. Boggess and Robertson (2010) showed that cones that have unequal end-area friction sleeves can produce significant errors in f_s measurement in soft fine-grained soils and during offshore testing. All standards have strict limits on dimensional tolerances. Some cones are manufactured to have sleeves that are slightly larger than the cone tip, but within standard tolerances, to increase the measured values of f_s . The IRTP (1999) has clear specifications on surface roughness. In the early 1980's subtraction cone designs became popular for improved robustness. In a subtraction cone design the sleeve friction is obtained by subtracting the cone tip load from the combined cone plus sleeve friction load. Any zero load instability (shifts) in each load cell results in a loss of accuracy in the calculated sleeve friction. Cone designs with separate tip and friction load cells are now equally as robust as subtraction cones. Hence, it is recommended to use only cones with independent load cells that have improved accuracy in the measurement of f_s . ASTM D5778 (2007) and the IRTP (1999) specify zero-load readings before and after each sounding for improved accuracy. With good quality control it is possible to obtain repeatable and accurate sleeve friction measurements, as illustrated by Robertson (2009a). However, f_s measurements, in general, will be less accurate than tip resistance in most soft fine-grained soils.

The accuracy for most well designed, strain gauged load cells is 0.1% of the full scale output (FSO). Most commercial cones are designed to record a tip stress of around 100 MPa. Hence, they have accuracy for q_t of around 0.1 MPa (100 kPa). In most sands, this represents an excellent accuracy of better than 1%. However, in soft, fine-grained soils, this may represent an accuracy of less than 10%. In very soft, fine-grained soils, low capacity cones (i.e. max. tip stress < 50 MPa) have better accuracy.

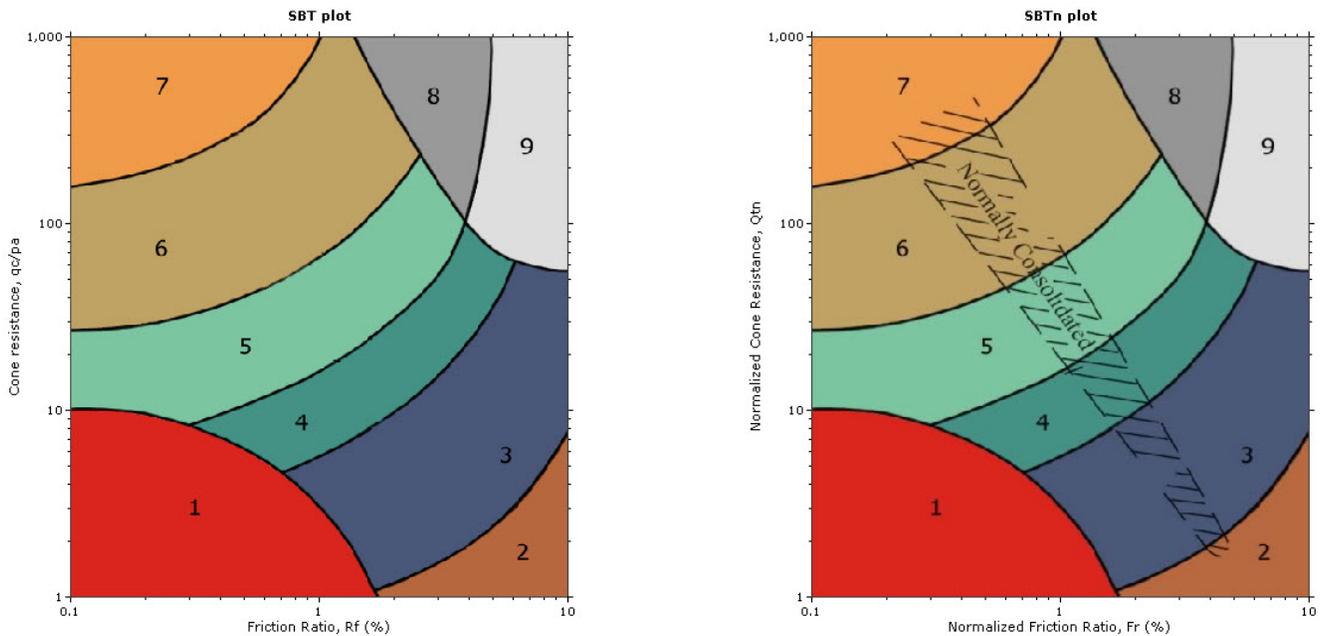
Throughout this paper use will be made of the normalized soil behaviour type (SBT) chart using normalized CPT parameters. Hence, accuracy in both q_t and f_s are important, particularly in soft fine-grained soil. Accuracy in f_s measurements requires that the CPT be carried out according to the standard (e.g. ASTM D5778) with particular attention to cone design (separate load cells and equal-end area friction sleeves), tolerances, and zero-load readings.

5.2 Soil Type

One of the major applications of the CPT has been the determination of soil stratigraphy and the identification of soil type. This has been accomplished using charts that link cone parameters to soil type.

Early charts using q_c and friction ratio ($R_f = 100 f_s/q_c$) were proposed by Douglas and Olsen (1981), but the charts proposed by Robertson et al. (1986) and Robertson (1990) have become very popular (e.g. Long, 2008). The non-normalized charts by Robertson et al (1986) defined 12 soil behaviour type (SBT) zones, whereas, the normalized charts by Robertson (1990) defined 9 zones. This difference caused some confusion that led Robertson (2010a) to suggest an update on the charts, as shown in Figure 2. The updated charts, that are dimensionless and colour coded for improved presentation, define 9 consistent SBT zones.

Robertson et al (1986) and Robertson (1990) stressed that the CPT-based charts were predictive of soil behaviour, and suggested the term 'soil behaviour type', because the cone responds to the in-situ mechanical behavior of the soil (e.g. strength, stiffness and compressibility) and not directly to soil classification criteria, using geologic descriptors, based on grain-size distribution and soil plasticity (e.g. Unified Soil Classification System, USCS). Grain-size distribution and Atterberg Limits are measured on disturbed soil samples. Fortunately, soil classification criteria based on grain-size distribution and plasticity often relate reasonably well to in-situ soil behavior and hence, there is often good agreement between USCS-based classification and CPT-based SBT (e.g. Molle, 2005). However, several examples can be given when differences can arise between USCS-based soil types and CPT-based SBT. For example, a soil with 60% sand and 40% fines may be classified as 'silty sand' (sand-silt mixtures) or 'clayey sand' (sand-clay mixtures) using the USCS. If the fines have high clay content with high plasticity, the soil behavior may be more controlled by the clay and the CPT-based SBT will reflect this behavior and will generally predict a more clay-like behavior, such as 'silt mixtures - clayey silt to silty clay' (Fig. 2, SBT zone 4). If the fines were non-plastic, soil behavior will be controlled more by the sand and the CPT-based SBT will generally predict a more sand-like soil type, such as 'sand mixtures - silty sand to sandy silt' (SBT zone 5). Very stiff, heavily overconsolidated fine-grained soils tend to behave more like a coarse-grained soil in that they tend to dilate under shear and can have high undrained shear strength compared to their drained strength and can have a CPT-based SBT in either zone 4 or 5. Soft saturated low plastic silts tend to behave more like clays in that they have low undrained shear strength and can have a CPT-based SBT in zone 3 (clays - clay to silty clay). These few examples illustrate that the CPT-based SBT may not always agree with traditional USCS-based soil types based on samples and that



SBT zone	Proposed common SBT description
1	<i>Sensitive fine-grained</i>
2	<i>Clay - organic soil</i>
3	<i>Clays: clay to silty clay</i>
4	<i>Silt mixtures: clayey silt & silty clay</i>
5	<i>Sand mixtures: silty sand to sandy silt</i>
6	<i>Sands: clean sands to silty sands</i>
7	<i>Dense sand to gravelly sand</i>
8	<i>Stiff sand to clayey sand*</i>
9	<i>Stiff fine-grained*</i>

* Overconsolidated or cemented

Figure 2: Updated Soil Behaviour Type (SBT) charts based on either non-normalized or normalized CPT (after Robertson, 2010a)

the biggest difference is likely to occur in the mixed soils region (i.e. sand-mixtures & silt-mixtures). Geotechnical engineers are often more interested in the in-situ soil behavior than a classification based only on grain-size distribution and plasticity carried out on disturbed samples, although knowledge of both is helpful. The geotechnical profession has a long history of using simplified classification systems with geologic descriptors, and it will likely be some time before the profession fully accepts and adopts the more logical framework based on mechanical response measurements directly from the in-situ tests.

Robertson (1990) proposed using normalized (and dimensionless) cone parameters, Q_{t1} , F_r , B_q , to estimate soil behaviour type, where;

$$Q_{t1} = (q_t - \sigma_{v0}) / \sigma'_{v0} \quad (3)$$

$$F_r = [(f_s / (q_t - \sigma_{v0}))] 100\% \quad (4)$$

$$B_q = (u_2 - u_0) / (q_t - \sigma_{v0}) = \Delta u / (q_t - \sigma_{v0}) \quad (5)$$

Where:

σ_{v0} = in-situ total vertical stress

σ'_{v0} = in-situ effective vertical stress

u_0 = in-situ equilibrium water pressure

Δu = excess penetration pore pressure = $(u_2 - u_0)$

In the original paper by Robertson (1990) the normalized cone resistance was defined using the term Q_{t1} . The term Q_{t1} is used here to show that the cone resistance is the corrected cone resistance, q_t and the stress exponent for stress normalization is 1.0 (further details are provide in a later section).

In general, the normalized charts provide more reliable identification of SBT than the non-normalized charts, although when the in-situ vertical effective stress is between 50 kPa to 150 kPa there is often little difference between normalized and non-normalized SBT. The term SBTn will be used to distinguish between normalized and non-normalized SBT. The above normalization was based on theoretical work by Wroth (1984). Robertson (1990)

suggested two charts based on either $Q_{t1} - F_r$ or $Q_{t1} - B_q$ but recommended that the $Q_{t1} - F_r$ chart was generally more reliable.

Schneider et al (2008) have shown that $\Delta u/\sigma'_{v0}$ is a better form for normalized pore pressure parameter than B_q . The chart by Schneider et al (2008) is shown in Figure 3. Superimposed on the Schneider et al chart are contours of B_q to illustrate the link with $\Delta u/\sigma'_{v0}$. Also shown on the Schneider et al chart are approximate contours of OCR (dashed lines). Figure 3 shows that the use of $\Delta u/\sigma'_{v0}$ is not a good approach for estimating OCR. Application of the Schneider et al chart can be problematic for some onshore projects where the CPTu pore pressure results may not be reliable, due to loss of saturation. However, for offshore projects, where CPTu sensor saturation is more reliable, and onshore projects in soft fine-grained soils with high groundwater, the chart can be very helpful. In near surface very soft fine-grained soils, where the accuracy of the cone resistance (q_t) may be limited, the Schneider et al chart can become a useful check on the data (e.g. if the cone resistance values are too low, the data can plot below the limit of the chart, $B_q > 1.0$). Also the Schneider et al chart is focused primarily on fine-grained soils where excess pore pressures are recorded and Q_{t1} is small.

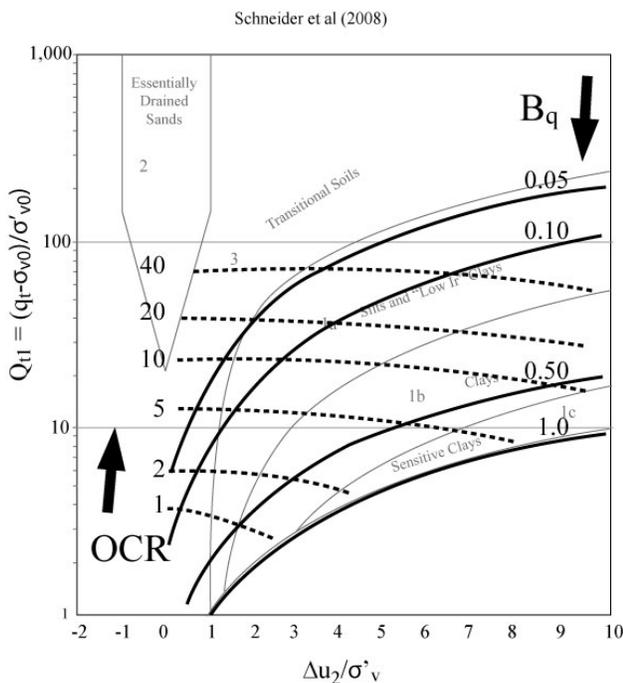


Figure 3: CPT classification chart from Schneider et al (2008) based on $(\Delta u_2/\sigma'_v)$ with contours of B_q and OCR

Since 1990 there have been other CPT soil type charts developed (e.g. Jefferies and Davies, 1991, Olsen and Mitchell, 1995, Eslami and Fellenius, 1997). The chart by Eslami and Fellenius (1997) is based on non-normalized parameters using effective cone resistance, q_e and f_s , where $q_e = (q_t - u_2)$. The

effective cone resistance, q_e , suffers from lack of accuracy in soft fine-grained soils, as will be discussed in a later section. Zhang and Tumay (1999) developed a CPT based soil classification system based on fuzzy logic where the results are presented in the form of percentage probability (e.g. percentage probability of either clay silt or sand). Although this approach is conceptually attractive (i.e. provides some estimate of uncertainty for each SBT zone) the results are often misinterpreted as a grain size distribution.

Jefferies and Davies (1993) identified that a soil behaviour type index, $I_{c,JD}$, could represent the SBTn zones in the $Q_t - F_r$ chart where, $I_{c,JD}$ is essentially the radius of concentric circles that define the boundaries of soil type. Robertson and Wride, (1998) modified the definition of I_c to apply to the Robertson (1990) $Q_t - F_r$ chart, as defined by:

$$I_c = [(3.47 - \log Q_t)^2 + (\log F_r + 1.22)^2]^{0.5} \quad (6)$$

Contours of I_c are shown in Figure 4 on the Robertson (1990) $Q_{t1} - F_r$ SBTn chart. The contours of I_c can be used to approximate the SBT boundaries and also extent interpretation beyond the chart boundaries (e.g. $F_r > 10\%$). Jefferies and Davies (1993) suggested that the SBT index I_c could also be used to modify empirical correlations that vary with soil type. This is a powerful concept and has been used where appropriate in this paper. Also shown on Figure 4 are lines that represent normalized sleeve friction $[(f_s/\sigma'_{v0}) - \text{dashed lines}]$, to illustrate the link between f_s and F_r .

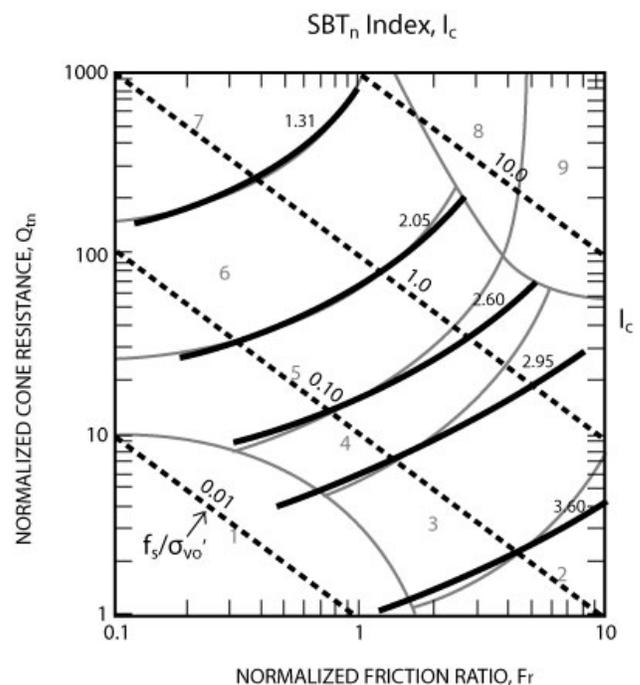


Figure 4: Contours of soil behaviour type index, I_c (thick lines) and normalized sleeve friction (dashed lines) on normalized SBTn $Q_{tm} - F_r$ chart. (SBT zones based on Fig 2)

The form of equation 6 and the shape of the contours of I_c in Figure 4 illustrate that I_c is not overly sensitive to the potential lack of accuracy of the sleeve friction, f_s , but is more controlled by the more accurate tip stress, q_t . Research (e.g. Long, 2008) has sometimes questioned the reliability of the SBT based on sleeve friction values (e.g. $Q_t - F_r$ charts). However, numerous studies (e.g. Molle, 2005) have shown that the normalized charts based on $Q_t - F_r$ provide the best overall success rate for SBT compared to samples. It can be shown, using equation 6, that if f_s vary by as much as +/- 50%, the resulting variation in I_c is generally less than +/- 10%. For soft soils that fall within the lower part of the $Q_t - F_r$ chart (e.g. $Q_t < 20$), I_c is relatively insensitive to f_s .

5.4 Stress normalization

Conceptually, any normalization to account for increasing stress should also account for the important influence of horizontal effective stresses, since penetration resistance is strongly influenced by the horizontal effective stresses (Jamiolkowski and Robertson, 1988). However, this continues to have little practical benefit for most projects without a prior knowledge of in-situ horizontal stresses. Even normalization using only vertical effective stress requires some input of soil unit weight and groundwater conditions. Fortunately, commercial software packages have increasingly made this easier and unit weights estimated from the non-normalized SBT charts appear to be reasonably effective for many applications (e.g. Robertson, 2010b).

Jefferies and Davies (1991) proposed a normalization that incorporates the pore pressure directly into a modified normalized cone resistance using: $Q_{t1} (1 - B_q)$. Recently, Jefferies and Been (2006) updated their modified chart using the parameter $Q_{t1} (1 - B_q) + 1$, to overcome the problem in soft sensitive soils where $B_q > 1$. Jefferies and Been (2006) noted that:

$$Q_{t1}(1-B_q) + 1 = (q_t - u_2) / \sigma'_{vo} \quad (7)$$

Hence, the parameter $Q_{t1} (1 - B_q) + 1$ is simply the effective cone resistance, $(q_t - u_2)$, normalized by the vertical effective stress. Although incorporating pore pressure into the normalized cone resistance is conceptually attractive it has practical problems. In stiff fine-grained soils, the excess pore pressure also can be sensitive to the exact location of the porous sensor. Accuracy is a major concern in soft fine-grained soils where q_t is small compared to u_2 . Hence, the difference $(q_t - u_2)$ is very small and lacks accuracy and reliability in most soft soils. For most commercial cones the precision of Q_{t1} in soft fine-grained soils is about +/- 20%, whereas, the precision for $Q_{t1} (1 - B_q) + 1$ in the same soil is about +/- 40%, due to the combined lack of precision in $(q_t$

- $u_2)$. Loss of saturation further complicates this parameter.

Robertson and Wride (1998), as updated by Zhang et al (2002), suggested a more generalized normalized cone parameter to evaluate soil liquefaction, using normalization with a variable stress exponent, n ; where:

$$Q_{tn} = [(q_t - \sigma_{vo})/p_a] (p_a/\sigma'_{vo})^n \quad (8)$$

Where:

$(q_t - \sigma_{vo})/p_a$ = dimensionless net cone resistance,

$(p_a/\sigma'_{vo})^n$ = stress normalization factor

n = stress exponent that varies with SBT

p_a = atmospheric pressure in same units as q_t and σ_v

Note that when $n = 1$, $Q_{tn} = Q_{t1}$. Zhang et al (2002) suggested that the stress exponent, n , could be estimated using the SBT Index, I_c , and that I_c should be defined using Q_{tn} .

In recent years there have been several publications regarding the appropriate stress normalization (Olsen and Malone, 1988; Zhang et al., 2002; Idriss and Boulanger, 2004; Moss et al, 2006; Cetin and Isik, 2007). The contours of stress exponent suggested by Cetin and Isik (2007) are very similar to those by Zhang et al, (2002). The contours by Moss et al (2006) are similar to those first suggested by Olsen and Malone (1988). The normalization suggested by Idriss and Boulanger (2004) incorrectly used uncorrected chamber test results and the method only applies to sands where the stress exponent varies with relative density with a value of around 0.8 in loose sands and 0.3 in dense sands. Robertson (2009a) provided a detailed discussion on stress normalization and suggested the following updated approach to allow for a variation of the stress exponent with both SBT I_c (soil type) and stress level using:

$$n = 0.381 (I_c) + 0.05 (\sigma'_{vo}/p_a) - 0.15 \quad (9)$$

where $n \leq 1.0$

Robertson (2009a) suggested that the above stress exponent would capture the correct in-situ state for soils at high stress level and that this would also avoid any additional stress level correction for liquefaction analyses in silica-based soils.

The approach used by Jefferies and Been (2006) (equation 7) is the only method that uses a stress exponent $n = 1.0$ for all soils. This has significant implications at shallow depth ($z < 3m$) and great depth ($z > 30m$). Recent research (e.g. Boulanger, 2003) shows that the stress exponent is a function of soil state. Recent major projects that have applied the detailed approach suggested by Jefferies and Been (2006) also support the observation that the stress exponent is a function of soil state.

5.5 In-situ state and shear strength

For fine-grained soils, in-situ state is usually defined in terms of overconsolidation ratio (OCR), where OCR is defined as the ratio of the maximum past effective consolidation stress (σ'_p) and the present effective overburden stress (σ'_{vo}):

$$\text{OCR} = \sigma'_p / \sigma'_{vo} \quad (10)$$

For mechanically overconsolidated soils where the only change has been the removal of overburden stress, this definition is appropriate. However, for cemented and/or aged soils the OCR may represent the ratio of the yield stress and the present effective overburden stress. The yield stress will also depend on the direction and type of loading.

The most common method to estimate OCR and yield stress in fine-grained soils was suggested by Kulhawy and Mayne (1990):

$$\text{OCR} = k (q_t - \sigma_v) / \sigma'_{vo} = k Q_{t1} \quad (11)$$

$$\text{or } \sigma'_p = k (q_t - \sigma_{vo}) \quad (12)$$

Where k is the preconsolidation cone factor and σ'_p is the preconsolidation or yield stress.

Kulhawy and Mayne (1990) showed that an average value of $k = 0.33$ can be assumed, with an expected range of 0.2 to 0.5. Although it is common to use the average value of 0.33, Been et al (2010) suggested that the selection of an appropriate value for 'k' should be consistent with other parameters, as discussed below.

In clays, the peak undrained shear strength, s_u and OCR are generally related. Ladd and Foott, (1974) empirically developed the following relationship based on SHANSEP concepts:

$$s_u / \sigma'_{vo} = (s_u / \sigma'_{vo})_{\text{OCR}=1} (\text{OCR})^m = S (\text{OCR})^m \quad (13)$$

The term S varies as a function of the failure mode (testing method, strain rate). Ladd and De Groot (2003) recommended $S = 0.25$ with a standard deviation of 0.05 (for simple shear loading) and $m = 0.8$ for most soils. Equation 13 is also supported by Critical State Soil Mechanics (CSSM) where $S = 1/2 \sin \phi'$ in direct simple shear (DSS) loading, and, $m = 1 - C_s / C_c$, where C_s is the swelling index and C_c is the compression index.

The peak undrained shear strength is estimated using:

$$s_u = (q_t - \sigma_{vo}) / N_{kt} \quad (14)$$

Where N_{kt} is the cone factor that depends on soil stiffness, OCR and soil sensitivity, but experience has shown that soil sensitivity has the largest influ-

ence. N_{kt} can be linked to soil sensitivity via the normalized friction ratio, F_r , and can be represented approximately by:

$$N_{kt} = 10.5 + 7 \log(F_r) \quad (15)$$

Been et al (2010) showed that for consistency the following should hold:

$$(Q_{t1})^{1-m} = S N_{kt} (k)^m \quad (16)$$

Where:

$$\text{OCR} = k (Q_{t1}) \quad (\text{when } Q_{t1} < 20)$$

$$s_u / \sigma'_{vo} = Q_{t1} / N_{kt} = S (\text{OCR})^m$$

$$\text{and, } S = (s_u / \sigma'_{vo})_{\text{OCR}=1}$$

For most sedimentary clays, silts and organics fine-grained soil, $S = 0.25$ for average direction of loading and $\phi' \sim 26$, and, $m = 0.8$. Hence, the constant to estimate OCR can be automatically estimated based on CPT results using:

$$k = [(Q_{t1})^{0.2} / (0.25 (10.5 + 7 \log F_r))]^{1.25} \quad (17)$$

Then,

$$\text{OCR} = (2.625 + 1.75 \log F_r)^{-1.25} (Q_{t1})^{1.25} \quad (18)$$

This compares very closely to the form suggested by Karlsrud et al (2005) based on high quality block samples from Norway (when soil sensitivity, $S_t < 15$) and that resulting from CSSM:

$$\text{OCR} = 0.25 (Q_{t1})^{1.2} \quad (19)$$

Equation 18 represents a method to automatically estimate the in-situ state (OCR) in fine-grained soils based on measured CPT results, in a consistent manner.

The state parameter (ψ) is defined as the difference between the current void ratio, e and the void ratio at critical state e_{cs} , at the same mean effective stress for coarse-grained (sandy) soils. Based on critical state concepts, Jefferies and Been (2006) provide a detailed description of the evaluation of soil state using the CPT. They describe in detail that the problem of evaluating state from CPT response is complex and depends on several soil parameters. The main parameters are essentially the shear stiffness, shear strength, compressibility and plastic hardening. Jefferies and Been (2006) provide a description of how state can be evaluated using a combination of laboratory and in-situ tests. They stress the importance of determining the in-situ horizontal effective stress and shear modulus using in-situ tests and determining the shear strength, compressibility and plastic hardening parameters from laboratory testing on reconstituted samples. They also show how the problem can be assisted using

numerical modeling. For high risk projects a detailed interpretation of CPT results using laboratory results and numerical modeling can be appropriate (e.g. Shuttle and Cunning, 2007), although soil variability can complicate the interpretation procedure. Some unresolved concerns with the Jefferies and Been (2006) approach relate to the stress normalization using $n = 1.0$ for all soils, as discussed above, and the influence of soil fabric in sands with high fines content.

For low risk projects and in the initial screening for high risk projects there is a need for a simple estimate of soil state. Plewes et al (1992) provided a means to estimate soil state using the normalized soil behavior type (SBT) chart suggested by Jefferies and Davies (1991). Jefferies and Been (2006) updated this approach using the normalized SBT chart based on the parameter $Q_{t1}(1-B_q) + 1$. Robertson (2009) expressed concerns about the accuracy and precision of the Jefferies and Been (2006) normalized parameter in soft soils. In sands, where $B_q = 0$, the normalization suggested by Jefferies and Been (2006) is the same as Robertson (1990). The contours of state parameter (ψ) suggested by Plewes et al (1992) and Jefferies and Been (2006) were based primarily on calibration chamber results for sands.

Based on the data presented by Jefferies and Been (2006) and Shuttle and Cunning (2007) as well the measurements from the CANLEX project (Wride et al, 2000) for predominantly coarse-grained uncemented young soils, combined with the link between OCR and state parameter in fine-grained soil, Robertson (2009a) developed contours of state parameter (ψ) on the updated SBTn $Q_{tn} - F$ chart for uncemented Holocene age soils. The contours, that are shown on Figure 5, are approximate since stress state and plastic hardening will also influence the estimate of in-situ soil state in the coarse-grained region of the chart (i.e. when $I_c < 2.60$) and soil sensitivity for fine-grained soils. An area of uncertainty in the approach used by Jefferies and Been (2006) is the use of Q_{t1} rather than Q_{tn} . Figure 5 uses Q_{tn} since it is believed that this form of normalized parameter has wider application, although this issue may not be fully resolved for some time. The contours of ψ shown in Figure 5 were developed primarily on laboratory test results and validated with well documented sites where undisturbed frozen samples were obtained (Wride et al., 2000). Jefferies and Been (2006) suggested that soils with a state parameter less than -0.05 (i.e. $\psi < -0.05$) are dilatative at large strains.

Over the past 40 years significant research has been carried out utilizing penetration test results (initially SPT and then CPT) to evaluate to resistance to cyclic loading (e.g. Seed et al., 1983, and Robertson & Wride, 1998). The most commonly accepted approach (often referred to as the Berkeley approach) uses an equivalent clean sand penetration resistance

to correlate to the cyclic resistance of sandy soils. The approach is empirical based primarily on observations from past earthquakes and supported by laboratory observations.

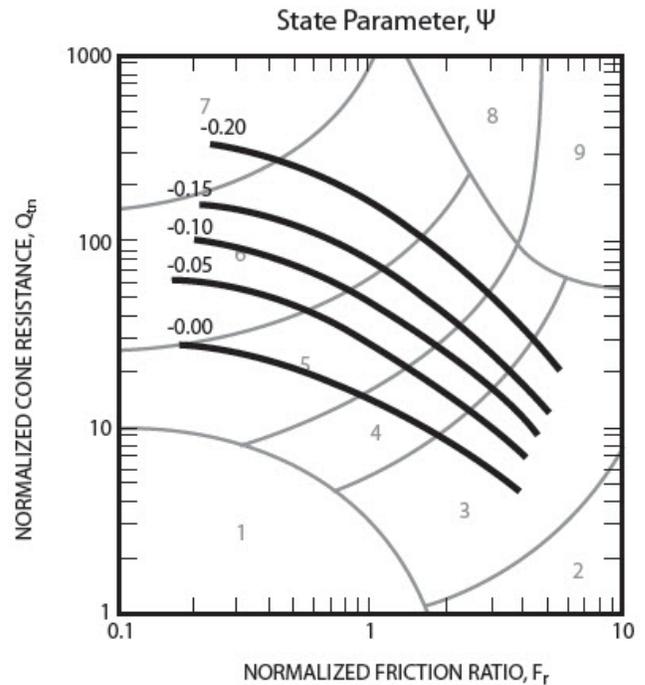


Figure 5: Contours of estimated state parameter, ψ (thick lines), on normalized SBTn $Q_{tn} - F_r$ chart for uncemented Holocene-age sandy soils (after Robertson, 2009a)

Robertson and Wride (1998), based on a large database of liquefaction case histories, suggested a CPT-based correction factor to correct normalized cone resistance in silty sands to an equivalent clean sand value ($Q_{tn,cs}$) using the following:

$$Q_{tn,cs} = K_c Q_{tn} \quad (20)$$

Where K_c is a correction factor that is a function of grain characteristics (combined influence of fines content, mineralogy and plasticity) of the soil that can be estimated using I_c as follows:

$$\text{if } I_c \leq 1.64 \quad (21) \\ K_c = 1.0$$

$$\text{if } I_c > 1.64 \quad (22) \\ K_c = 5.581I_c^3 - 0.403I_c^4 - 21.63I_c^2 + 33.75I_c - 17.88$$

Figure 6 shows contours of equivalent clean sand cone resistance, $Q_{tn,cs}$, on the updated CPT SBT chart. The contours of $Q_{tn,cs}$ were developed based on liquefaction cased histories.

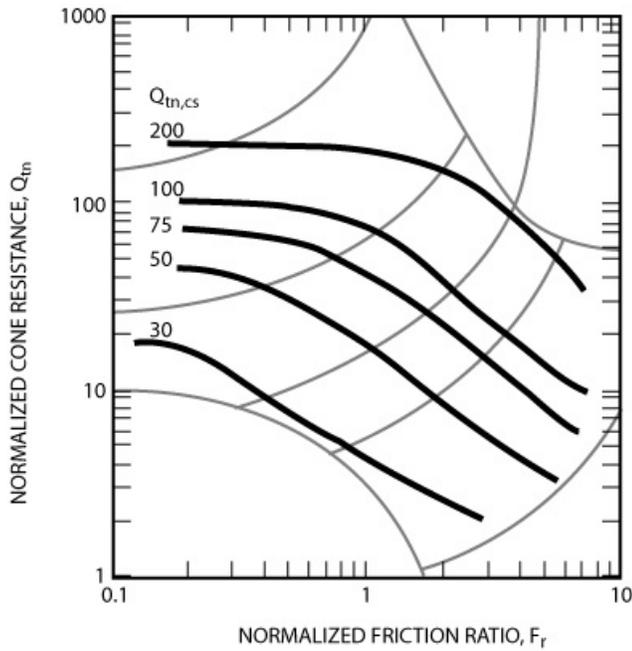


Figure 6: Contours of clean sand equivalent normalized cone resistance, $Q_{tn,cs}$, based on Robertson and Wride (1998) liquefaction method

The contours of ψ , shown in Figure 5, are supported by CSSM theory, extensive calibration chamber studies and high quality frozen sampling. The contours of $Q_{tn,cs}$, shown in Figure 6, are supported by an extensive case history database.

Comparing Figures 5 and 6 shows a strong similarity between the contours of ψ and the contours of $Q_{tn,cs}$. The observed similarity in the contours of ψ and $Q_{tn,cs}$, support the concept of a clean sand equivalent as a measure of soil state in sandy soils. Based on Figures 5 and 6, Robertson (2010b) suggested a simplified and approximate relationship between ψ and $Q_{tn,cs}$, as follows:

$$\psi = 0.56 - 0.33 \log Q_{tn,cs} \quad (23)$$

Equation 23 provides a simplified and approximate method to estimate in-situ state parameter for a wide range of sandy soils.

Based on Figure 5 and acknowledging that coarse-grained soils with a state parameter less than -0.05 and fine-grained soils with an OCR > 4 are dilative at large strains, it is possible to define a region based on CPT results that identifies soils that are either dilative or contractive, as shown on Figure 7. Included on Figure 7 is a region (dashed lines) that defines the approximate boundary between drained and undrained response during a CPT. Robertson (2010c) reviewed case histories of flow (static) liquefaction that confirmed the dilative/contractive boundary shown in Figure 7. Figure 7 represents a simplified chart that identifies the four broad groups of soil behaviour (i.e. drained-dilative, drained-

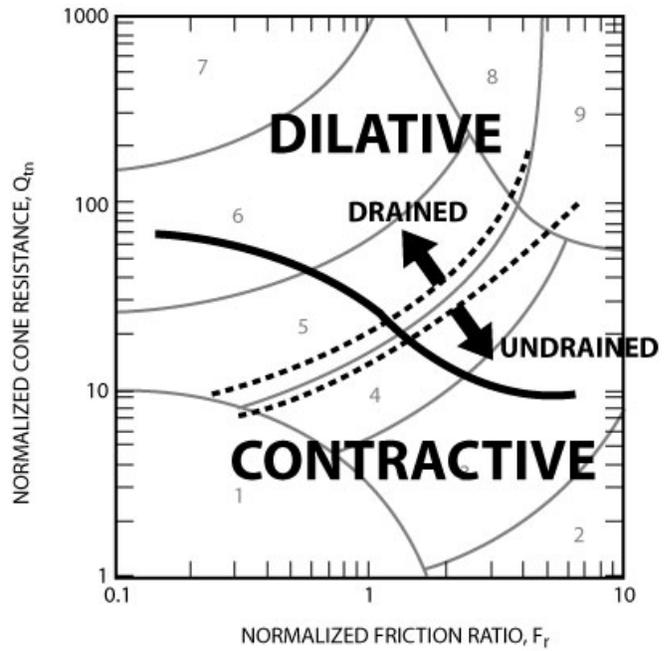


Figure 7: Approximate boundaries between dilative-contractive behaviour and drained-undrained CPT response on normalized SBTn $Q_{tn} - F_r$ chart

contractive, undrained-dilative and undrained-contractive).

Jefferies and Been (2006) showed a strong link between ψ and the peak friction angle (ϕ') for a wide range of sands. Using this link, it is possible to link $Q_{tn,cs}$ with ϕ' , using:

$$\phi' = \phi'_{cv} - 48 \psi \quad (24)$$

Where ϕ'_{cv} = constant volume (or critical state) friction angle depending on mineralogy (Bolton, 1986), typically about 33 degrees for quartz sands but can be as high as 40 degrees for felspathic sand. Hence, the relationship between $Q_{tn,cs}$ and ϕ' becomes:

$$\phi' = \phi'_{cv} + 15.84 [\log Q_{tn,cs}] - 26.88 \quad (25)$$

Equation 25 produces estimates of peak friction angle for clean quartz sands that are similar to those by Kulhawy and Mayne (1990). However, equation 25 has the advantage that it includes the importance of grain characteristics and mineralogy that are reflected in both ϕ'_{cv} , as well as soil type through $Q_{tn,cs}$. Equation 25 tends to predict ϕ' values closer to measured values in calcareous sands where the CPT tip resistance can be low for high values of ϕ' .

5.6 Stiffness and compressibility

Eslaamizaad and Robertson (1997) and Mayne (2000) have shown that the load settlement response for both shallow and deep foundations can be accurately predicted using measured shear wave velocity, V_s . Although direct measurement of V_s is preferred over estimates, relationships with cone resistance are

useful for smaller, low risk projects where V_s measurements are not always taken. Schneider et al (2004) showed that V_s in sands is controlled by the number and area of grain-to-grain contacts, which in turn depend on relative density, effective stress state, rearrangement of particles with age and cementation. Penetration resistance in sands is also controlled by relative density, effective stress state and to a lesser degree by age and cementation. Thus, although strong relationships between V_s and penetration resistance exist, some variability should be expected due to age and cementation. There are many existing relationships between cone resistance and V_s (or small strain shear modulus, G_0), but most were developed for either sands or clays and generally relatively young deposits. The accumulated 20 years of experience with SCPT results enables updated relationships between cone resistance and V_s to be developed for a wide range of soils, using the CPT SBT_n chart ($Q_{tn} - F_r$) as a base. Since V_s is a direct measure of the small strain shear modulus, G_0 , there can also be improved linkages between CPT results and soil modulus.

Based on over 100 SCPT profiles from 22 sites in California combined with published data, Robertson (2009a) developed a set of contours of normalized shear wave velocity, V_{s1} on the normalized SBT $Q_{tn} - F_r$ chart, as shown on Figure 8, where;

$$V_{s1} = V_s (p_a / \sigma'_{vo})^{0.25} \quad \text{m/s} \quad (26)$$

where V_s is in m/s.

Since the CPT measurements are normalized in terms of Q_{tn} and F_r , the resulting shear wave velocity values are also normalized. The deposits ranged predominately from Holocene to Pleistocene age and were mostly uncemented, although cementation was possible in some soils. Andrus et al. (2007) showed that most Holocene age deposits have V_{s1} values less than 250 m/s. In general, the Holocene age data tends to plot in the center-left portion of the SBT_n chart, whereas the Pleistocene age data tends to plot in the upper-right portion of the chart.

The contours of V_{s1} in Figure 8 can be approximated using the following equations:

$$V_{s1} = (\alpha_{vs} Q_{tn})^{0.5} \quad \text{m/s} \quad (27)$$

or

$$V_s = [\alpha_{vs} (q_t - \sigma_v) / p_a]^{0.5} \quad \text{m/s} \quad (28)$$

Since the shape of the contours for α_{vs} is similar to those for I_c , α_{vs} can be estimated using:

$$\alpha_{vs} = 10^{(0.55 I_c + 1.68)} \quad \text{in units of (m/s)}^2 \quad (29)$$

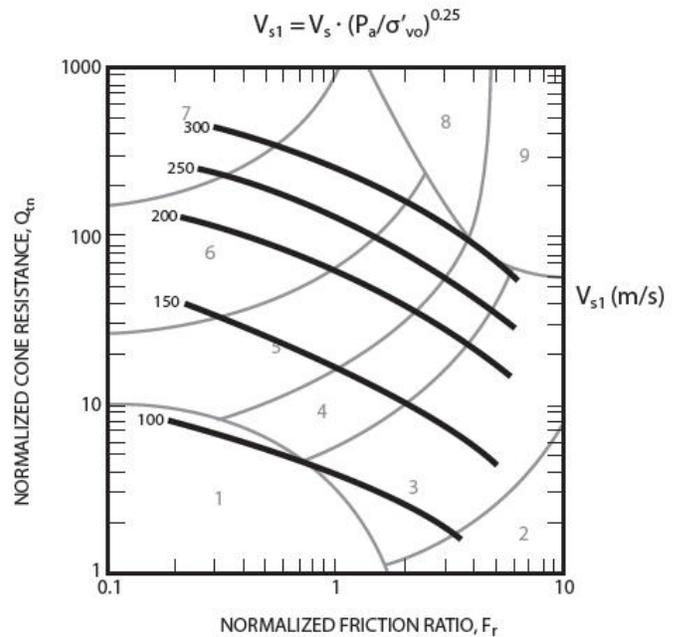


Figure 8: Contours of normalized shear wave velocity, V_{s1} (thick lines), on normalized SBT_n $Q_{tn} - F_r$ chart for uncemented Holocene- and Pleistocene-age sandy soils (after Robertson, 2009a)

At low shear strain levels (less than about 10^{-4} %), the shear modulus in soils is constant and has a maximum value, G_0 . This small strain shear modulus is determined from shear wave velocity using the equation:

$$G_0 = \rho (V_s)^2 \quad (30)$$

where ρ is the mass density (or total unit weight divided by the acceleration of gravity) of the soil. Using the V_s contours, Figure 9 shows the associated contours of the small strain shear modulus number, K_G , where:

$$G_0 = K_G p_a (\sigma'_{vo} / p_a)^n \quad (31)$$

where n is a stress exponent that has a value of about 0.5 for most coarse-grained soils.

Relationships between soil modulus and cone resistance can have the general form:

$$G_0 = \alpha_G (q_t - \sigma_{vo}) \quad (32)$$

where α_G is the shear modulus factor for estimating small strain shear modulus (G_0) from net cone resistance ($q_t - \sigma_{vo}$). Since the stress exponent is similar for the normalization of both Q_{tn} and G_0 in the sand region, it follows that:

$$\alpha_G = K_G / Q_{tn} \quad (33)$$

Hence, it is possible to develop contours of α_G that are also shown in Figure 9.

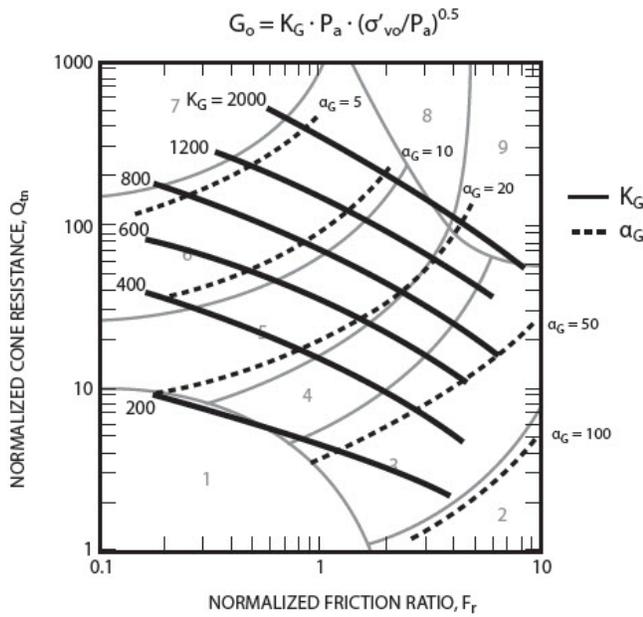


Figure 9: Contours of small strain modulus number, K_G (thick solid lines) and modulus factor α_G , on normalized SBTn $Q_m - F_r$ chart for uncemented Holocene- and Pleistocene-age sandy soils (After Robertson, 2009a)

Eslaamizaad and Robertson (1996a) and Schnaid (2005) showed that it is possible to identify cemented soils using the ratio of G_0/q_t . Hence, if the measured $G_0/(q_t - \sigma_{vo})$ (i.e. α_G) is significantly larger than estimated using Figure 9, the soils are likely either cemented and/or aged.

It is also possible to estimate the appropriate value of α_G from I_c based on the link with α_{vs} using:

$$\alpha_G = (\rho/p_a) \alpha_{vs} \quad (34)$$

Where (ρ/p_a) is in units of $(s/m)^2$

For an average unit weight, $\gamma = 18 \text{ kN/m}^3$ ($\rho = 1.84$), it follows that:

$$\alpha_G = 0.0188 [10^{(0.55I_c + 1.68)}] \quad (35)$$

Hence, the small strain shear modulus, G_0 for young, uncemented soils can be estimated using:

$$G_0 = 0.0188 [10^{(0.55I_c + 1.68)}] (q_t - \sigma_{vo}) \quad (36)$$

Figure 9 and equation 36 provide a simplified means to estimate the small strain shear modulus over a wide range of soils using CPT data. The relationships shown in Figures 8 and 9 are less reliable in the region for fine-grained soils (i.e. when $I_c > 2.60$) since the sleeve friction f_s and hence F_r , are strongly influenced by soil sensitivity. The relationships are generally better in the coarse-grained region (i.e. when $I_c < 2.60$) and are primarily for un-

cemented, predominately silica-based soils of Holocene and Pleistocene age.

For some applications engineers require an estimate of the Young's modulus, E' . The Young's modulus, E' is linked to the shear modulus via:

$$E' = 2(1+\nu) G \quad (37)$$

Where:

ν = Poisson's ratio, which ranges from 0.1 to 0.3 for most soils under drained conditions. Hence, for most coarse-grained soils, $E' \sim 2.5 G$.

Since the small strain shear modulus G_0 applies only at very small strains there is a need to soften G_0 to a strain level appropriate for design purposes. Eslaamizaad and Robertson (1997) showed that the amount of softening required for design was a function of the degree of loading. Fahey and Carter (1993) suggested a simple approach to estimate the amount of softening using:

$$G/G_0 = 1 - f(q/q_{ult})^g \quad (38)$$

Where:

q = applied load (e.g. net bearing pressure for foundations); q_{ult} = ultimate or failure load (e.g. ultimate bearing capacity for foundations); q/q_{ult} = degree of loading; f and g are constants depending on soil type and stress history.

Fahey and Carter (1993) and Mayne (2005) suggested that values of $f = 1$ and $g = 0.3$ are appropriate for uncemented soils that are not highly structured. For a degree of loading from 0.2 to 0.3, the ratio G/G_0 ranges from 0.30 to 0.38. Hence, for many design applications the appropriate Young's modulus for application in simplified elastic solutions is approximately;

$$E' \sim 0.8 G_0 \quad (39)$$

Equation 39 represents an important observation, in that it shows that the Young's modulus for many design applications can be either measured or estimated directly from the in-situ shear wave velocity (V_s) to get G_0 . Using this ratio, it is possible to create contours of Young's modulus number, K_E , on the CPT SBT chart as shown on Figure 10, where:

$$E' = K_E p_a (\sigma'_{vo} / p_a)^n \quad (40)$$

Where n is a stress exponent that has a value of about 0.5 for most coarse-grained soils.

Since the application of Young's modulus, E' is generally only applicable to drained soils, the contours on Figure 10 are therefore limited to the region defined by $I_c < 2.60$.

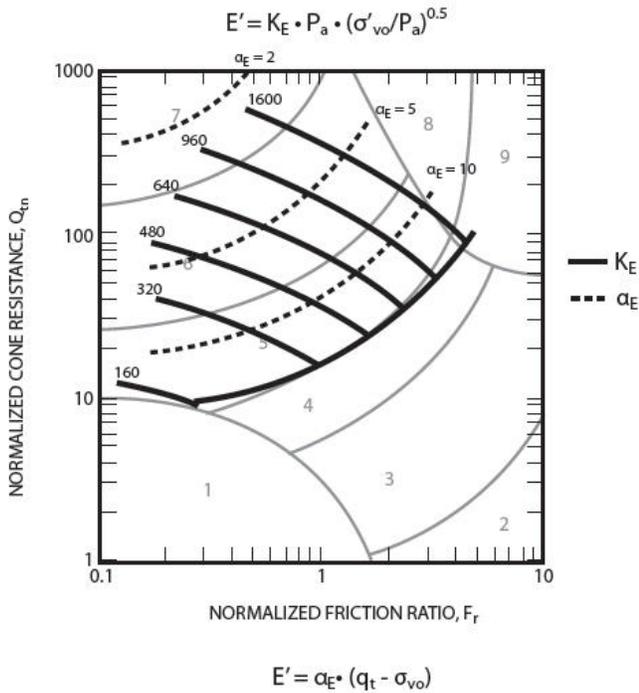


Figure 10: Contours of Young's modulus number, K_E (thick solid lines) and modulus factor α_E , on normalized SBTn $Q_{tn} - F_r$ chart for uncemented Holocene- and Pleistocene-age sandy soils (After Robertson, 2009a)

Some existing relationships between soil modulus and cone resistance have the form:

$$E' = \alpha_E (q_t - \sigma_{vo}) \quad (41)$$

Where:

α_E is the modulus factor for estimating Young's modulus (E') from net cone resistance ($q_t - \sigma_{vo}$).

Most existing relationships use q_c , whereas they should use $(q_t - \sigma_{vo})$, although the error is generally small in sands, where $q_t \gg \sigma_{vo}$ and $q_c \sim (q_t - \sigma_{vo})$. Since the stress exponent is similar for the normalization of both Q_{tn} and E' in the sand region, it follows that:

$$\alpha_E = K_E / Q_{tn} \quad (42)$$

Hence, it is possible to develop contours of α_E that are also shown in Figure 10.

It is possible to estimate the appropriate value of α_E from I_c using the following equation:

$$\alpha_E = 0.015 [10^{(0.55I_c + 1.68)}] \quad (43)$$

From this, the Young's modulus, E' for uncemented, predominately silica-based soils of either Holocene or Pleistocene age (when $I_c < 2.60$) can be estimated using:

$$E' = 0.015 [10^{(0.55I_c + 1.68)}] (q_t - \sigma_{vo}) \quad (44)$$

Bellotti et al, (1989) showed that the ratio E'/q_c varied between 3 and 12 for aged, normally consolidated sands and between 5 and 20 for over consolidated sands and was a function of normalized cone resistance. The relationship shown in Figure 10 indicates that a more appropriate ratio should be $E'/(q_t - \sigma_{vo})$ and that the range shown in Figure 10 is consistent with previous work.

Figure 10 and equation 44 provide a simplified means to estimate the equivalent Young's modulus using CPT data for a wide range of coarse-grained soils. Since the appropriate value for E' is a function of the degree of loading, it is also possible to vary α_E as a function of degree of loading. The values of α_E shown in Figure 10 are for an average degree of loading of about 0.25 (i.e. factor of safety around 4). As the degree of loading increases the associated value of α_E will decrease. To incorporate degree of loading into the estimate of E' , the final form would be:

$$E' = 0.047 [1 - (q/q_{ult})^{0.3}] [10^{(0.55I_c + 1.68)}] (q_t - \sigma_{vo}) \quad (45)$$

For low risk projects the simpler form shown in equation 44 would generally be adequate. Robertson (2009a) showed that equation 45 provided very good prediction of settlements for shallow foundation on coarse-grained (sandy) soils.

Consolidation settlements (at the end of primary consolidation) can be estimated using the 1-D Constrained tangent Modulus, M , (Lunne et al., 1997) where;

$$M = 1/m_v = \delta\sigma_v / \delta\varepsilon = 2.3 (1+e_0) \sigma'_{vo} / C_{c/r} \quad (46)$$

Where:

m_v = equivalent oedometer coefficient of compressibility.

$\delta\sigma_v$ = change in vertical stress

$\delta\varepsilon$ = change in vertical strain

e_0 = initial void ratio

$C_{c/r}$ = compression index, either C_c or C_r , depending on σ'_{vo}

Mayne (2007) has shown that the ratio of M/G_0 varies from 0.02 to 2 from soft clays to sands. Using the link between normalized cone values and G_0 as a starting point, it is possible to develop contours of constrained modulus number, K_M on the normalized soil behaviour type (SBT) chart, $Q_{tn} - F_r$, as shown on Figure 11, where:

$$M = K_M p_a (\sigma'_{vo} / p_a)^a \quad (47)$$

Where: a = stress exponent.

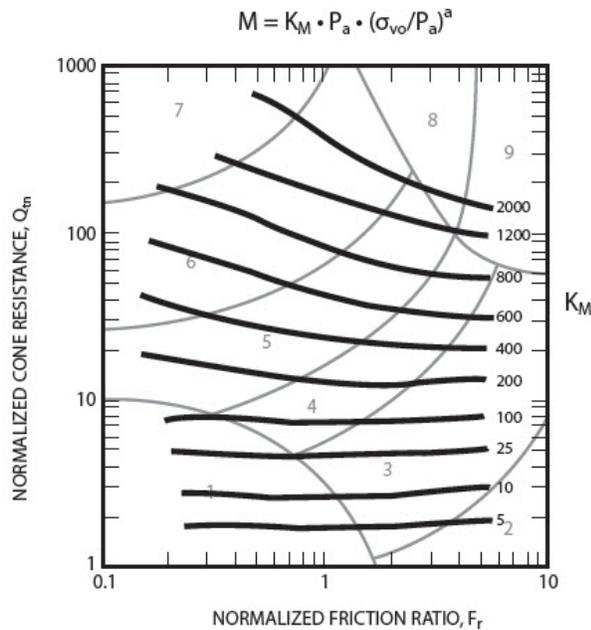


Figure 11: Contours of 1-D modulus number, K_M (thick lines), on normalized SBTn $Q_{tn} - F_r$ chart (After Robertson, 2009a)

Janbu (1963) showed that the stress exponent (a) was equal to 1.0 for stresses above the preconsolidation stress and zero below the preconsolidation stress (i.e. M is approximately constant below the preconsolidation stress). Hence, at stresses less than the preconsolidation stress:

$$M = K_M p_a \quad (48)$$

The flat dilatometer test (DMT) has often been shown to provide excellent estimates of settlement using predicted values of the 1-D constrained modulus (Monaco et al, 2006). The shape and location of the contours of K_m were also guided by recent correlations between normalized DMT and CPT parameters (Robertson, 2009b). The shape of the contours was also guided by existing relationships between M and net cone resistance ($q_t - \sigma_{vo}$).

Existing correlations between constrained modulus and cone resistance typically have the form:

$$M = \alpha_M (q_t - \sigma_{vo}) \quad (49)$$

Sanglerat (1972) suggested that α_M varies with soil plasticity and natural water content for a wide range of fine-grained and organic soils, although the data were based on q_c . Mayne (2007) showed that α_M varied with soil type and net cone resistance with values from 1 to 10, where the low values apply to soft clays.

Based on the contours shown in Figure 11 and equation 49, the following simplified correlation is suggested:

When $I_c > 2.2$ use:

$$\begin{aligned} \alpha_M &= Q_{tn} & \text{when } Q_{tn} < 14 \\ \alpha_M &= 14 & \text{when } Q_{tn} > 14 \end{aligned} \quad (50)$$

When $I_c < 2.2$ use:

$$\alpha_M = 0.03 [10^{(0.55I_c + 1.68)}] \quad (51)$$

Equation 49 shows that when $Q_{tn} < 14$, α_M varies from about 2 to 14 which is similar to that observed by Mayne (2007), but there is a clearer link on how to select the appropriate value for α_M . Robertson (2009a) showed that equations 49 and 50 provided very good prediction of 1-D settlement for several published case histories.

In countries where settlement calculations are carried out using the compression index (either C_c or C_r), it is possible to combine equations 46 and 50 in fine-grained soils ($I_c > 2.2$) to get:

$$C_{c/r} = 2.3 (1+e_0)/(Q_{t1})^2 \quad \text{when } Q_{t1} < 14 \quad (52)$$

$$C_{c/r} = 2.3 (1+e_0)/(14 Q_{t1}) \quad \text{when } Q_{t1} > 14 \quad (53)$$

In general, estimates of 1-D constrained modulus, M , and compression index (C_c) from undrained cone penetration will be approximate. Estimates can be improved with additional information about the soil, such as plasticity index and natural moisture content, where α_M is lower for organic soils.

5.7 Conductivity/Flow (permeability)

Soil permeability (k) can be estimated from CPTu pore pressure dissipation tests. The dissipation of pore pressures during a CPTu dissipation test is controlled by the coefficient of consolidation in the horizontal direction (c_h) which is influenced by a combination of the soil permeability (k_h) and compressibility (M), as defined by the following:

$$k_h = (c_h \gamma_w)/M \quad (54)$$

Where: M is the 1-D constrained modulus and γ_w is the unit weight of water, in compatible units.

Schmertmann (1978), Parez and Fauriel (1988) and Robertson et al (1992) suggested methods to estimate soil permeability (k) using the time for 50% dissipation (t_{50}) from a CPTu dissipation test. These simplified relationships are approximate, since the relationship is also a function of the soil compressibility (M), as shown in equation 54. An alternate and better approach is to estimate the coefficient of consolidation from a dissipation test then combine this with an estimate of the soil compressibility (M)

to obtain an improved estimate of the soil permeability (k).

The simplified relationship presented by Robertson et al (1992), based on the work of Teh and Hously (1991), for the coefficient of consolidation in the horizontal direction (c_h) as a function of the time for 50% dissipation (t_{50} , in minutes) for a 10 cm² cone can be approximated using:

$$c_h = (1.67 \times 10^{-6}) 10^{(1 - \log t_{50})} \quad \text{m}^2/\text{s} \quad (55)$$

For a 15cm² cone, the values of c_h are increased by a factor of 1.5.

Combining the estimated 1-D constrained modulus, given in equations 49 and 50, in compatible units (i.e. net cone resistance, $(q_t - \sigma_{vo})$ in kPa and $\gamma_w = 9.81 \text{ kN/m}^3$) it is possible to develop contours of k versus t_{50} for various values of Q_{t1} and σ'_{vo} , as shown on Figure 12.

The relationship shown in Figure 12 can be applied to data from standard 10 cm² and 15 cm² cones pushed into soft to stiff, fine-grained soils, where the penetration process is essentially undrained (i.e. $I_c > 2.60$).

Robertson et al (1992) also presented a summary of CPTu data where laboratory derived values of horizontal coefficient of permeability results were also available and these are included on Figure 12. Sites where normalized cone resistance values were also available confirm that the observed scatter in test results is due to the variation in soil stiffness reflected in the normalized cone resistance.

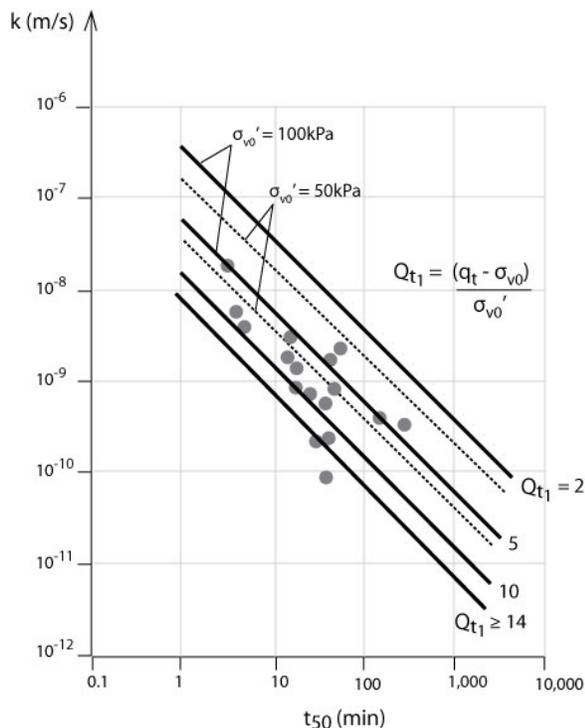


Figure 12: Relationship between CPTu t_{50} (in minutes) and soil permeability (k) and normalized cone resistance, Q_{t1} (data from Robertson et al, 1992)

The degree of consolidation during cone penetration depends on the penetration rate (v), cone diameter (d_c), and the coefficient of consolidation of the soil (c_h) (Finnie and Randolph, 1994). These factors can be used to obtain a normalized, dimensionless penetration rate, V :

$$V = v d_c / c_h \quad (56)$$

According to a number of researchers (e.g. Finnie and Randolph, 1994, Chung et al., 2006, Kim et al., 2008) the transition from fully undrained to partially drained conditions is approximately when $V \sim 10$. Therefore, for CPT using a standard 10 cm² cone carried out at the standard rate of 20 mm/s, undrained penetration can be expected in soils with c_h values less than about $7 \times 10^{-5} \text{ m}^2/\text{s}$. Because of the offsetting effect of rate-dependence shear strength, Kim et al (2010) showed that the cone resistance is unchanged for $V > 1$, which corresponds to a $c_h \sim 7 \times 10^{-4} \text{ m}^2/\text{s}$. Based on the relationship between t_{50} and c_h , (equation 55) this corresponds to a $t_{50} < 0.5$ min (30 sec). Hence, a simple method to evaluate if CPT penetration is occurring undrained or partially drained is to perform a dissipation test. If $t_{50} > 30$ seconds, cone penetration for either a 10 cm² or 15 cm² cone is likely undrained and the measured cone resistance can be used to estimate undrained shear strength. If $t_{50} < 30$ seconds, the measured cone resistance may be slightly high due to partial drainage. This is consistent with the observation made by Robertson et al (1992).

6.0 FLAT DILATOMETER TEST (DMT)

The flat dilatometer test (DMT) was developed in Italy by Professor Silvano Marchetti in the 1980's and has become popular in some parts of the world. The DMT is simple, robust, repeatable and economical. Marchetti (1980) provided a detailed description of the DMT equipment, the test method and the original correlations. Various international standards and manuals are available for the DMT. Marchetti (2001) also prepared a comprehensive report on the DMT for Technical Committee 16, ISSMGE.

The flat dilatometer is a stainless steel blade with a flat, circular steel membrane mounted flush on one side. The test involves two readings A and B that are corrected for membrane stiffness, gage zero offset and feeler pin elevation in order to determine the pressures p_0 and p_1 . Readings are taken every 200 mm during a pause in the penetration and the corrected pressures p_0 and p_1 are subsequently used for interpretation. The original correlations (Marchetti, 1980) were obtained by calibrating DMT results with high quality soil parameters from several test sites in Europe. Many of these correlations form the

basis of current interpretation, having been generally confirmed by subsequent research.

The interpretation evolved by first identifying three "intermediate" DMT parameters (Marchetti 1980):

$$\text{Material index, } I_D = (p_1 - p_0)/(p_0 - u_0) \quad (57)$$

$$\text{Horizontal stress index, } K_D = (p_0 - u_0) / \sigma'_{v0} \quad (58)$$

$$\text{Dilatometer modulus, } E_D = 34.7(p_1 - p_0) \quad (59)$$

Where:

u_0 = pre-insertion in-situ equilibrium water pressure

σ'_{v0} = pre-insertion in-situ vertical effective stress

The dilatometer modulus E_D can also be expressed as a combination of I_D and K_D in the form:

$$E_D / \sigma'_{v0} = 34.7 I_D K_D \quad (60)$$

The key DMT design parameters are I_D and K_D . Both parameters are normalized and dimensionless. I_D is the difference between the corrected lift-off pressure (p_0) and the corrected deflection pressure (p_1) normalized by the effective lift-off pressure ($p_0 - u_0$). K_D is the effective lift-off pressure normalized by the in situ vertical effective stress. Although alternate methods have been suggested to normalize K_D , the original normalization suggested by Marchetti (1980) using the in-situ vertical effective stress is still the most common. It is likely that a more complex normalization for K_D would be more appropriate, especially in sands, but most of the available published records of K_D use the original normalization suggested by Marchetti (1980).

The DMT is harder to push in very stiff ground compared to the CPT and the DMT is carried out every 200 mm whereas CPT readings are taken every 20 to 50 mm. The DMT requires a pause in the penetration to perform the test. Hence, the DMT produces less data than the CPT and is also slower than the CPT. Both tests do not include a soil sample, although it is possible to take small diameter soil samples using the same pushing equipment used to insert either the CPT or DMT. Robertson (2009b) suggested a preliminary set of correlations that links the key DMT parameters (I_D , K_D , and E_D) to normalized CPT parameters (Q_t and F_r). The proposed correlations are approximate and will likely be influenced by variations in in-situ stress state, soil density, stress history, age, cementation and soil sensitivity. The correlations are unlikely to be unique for all soils but the suggested relationships form a framework for future refinements. The resulting correlations are shown in Figure 13, in the form of contours of I_D , K_D on the CPT normalized SBTn chart.

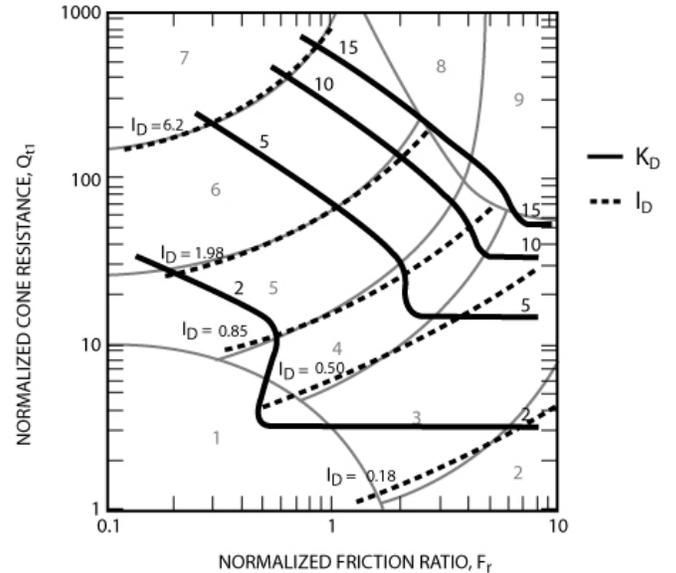


Figure 13: Approximate correlation between DMT K_D and I_D and CPT normalized parameters for uncemented soils (After Robertson, 2010)

Comparing Figure 6 with Figure 13, shows a similarity between the contours of CPT clean sand equivalent cone resistance ($Q_{t,cs}$) and DMT K_D . This link was also observed by Tsai et al (2009) and Kung et al (2010) related to correlations to evaluate soil liquefaction. Based on the data presented by Tsai et al (2009) for sandy soils ($I_D > 1.2$) and $2 < K_D < 6$, a simplified relationship can be given by:

for $2 < K_D < 6$ and $I_D > 1.2$

$$Q_{t,cs} = 25 K_D \quad (61)$$

By combining equation 61 with equations 23 and 25, it is possible to link the DMT K_D to state parameter (ψ) and peak friction angle (ϕ') to get:

$$\psi = 0.56 - 0.33 \log (25 K_D) \quad (62)$$

$$\phi' = \phi'_{cv} + 15.84 [\log (25 K_D)] - 26.88 \quad (63)$$

Equation 62 predicts smaller (i.e. denser) values for state parameter (ψ) than that suggested by Yu (2004). Yu (2004) would suggest that a $K_D = 4$ in a very loose sand ($K_0 = 0.5$) when $\psi = 0$, whereas, equation 62 suggests a more reasonable value of $K_D = 2$ in a very loose sand when $\psi = 0$. Equation 63 correctly predicts values for ϕ' that are slightly larger than the current method suggested by Marchetti et al (2001) for estimating the lower bound peak friction angle. Equation 63 has the advantage that it incorporates the importance of soil mineralogy via ϕ'_{cv} .

Based on equation 61, it is also possible to update the link between the DMT K_D and the cyclic resis-

tance ratio ($CRR_{7.5}$) for evaluating soil liquefaction in sandy soils. Using the CPT-based relationship between CRR and $Q_{tn,cs}$, suggested by Robertson and Wride (1998), and equation 61, an updated DMT relationship becomes:

for $2 < K_D < 6$ and $I_D > 1.2$

$$CRR_{7.5} = 93(0.025K_D)^3 + 0.08 \quad (64)$$

Figure 14 compares the proposed correlation between $CRR_{7.5}$ and K_D and those suggested by Monaco et al (2005) and Tsai et al (2009). The proposed correlation is very similar when $K_D < 4$ and falls between the others when $K_D > 4$.

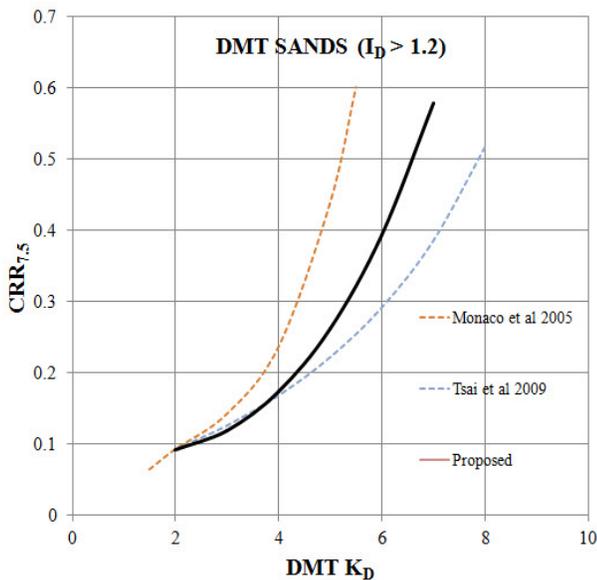


Fig. 14: Proposed correlation between cyclic resistance ratio ($CRR_{7.5}$) and DMT K_D for uncemented sandy soils

The intent of presenting a link between CPT and DMT results is not to infer that one test is better than the other, since each test has advantages and limitations, but to seek similarities and linkages between the two in-situ tests so that to these linkages can be used to expand and improve correlations and applications by applying existing experience and databases from one test and extrapolating to the other test.

7 SUMMARY

The objective of this paper was to focus on some major in-situ tests and to present some selected insights that the geotechnical engineering profession may find helpful. The use and application of in-situ testing for the characterization of geomaterials has continued to expand over the past few decades, especially in materials that are difficult to sample and test using conventional methods.

A short discussion is provided in an effort to encourage geotechnical engineers to progressively abandon the SPT because it is a crude, unreliable in-situ test. As suggested by Jefferies and Davies (1993), the most reliable way to obtain SPT N values is to perform a CPT and convert the CPT to an equivalent SPT. An update on the CPT-SPT correlation is presented based on the CPT soil behaviour type index, I_c .

A discussion is provided on recent developments with the CPT including some elements on equipment and procedures, evaluation of soil type and estimation of key geotechnical design parameters. Updated SBT charts with 9 consistent SBT zones, that are dimensionless and colour coded for improved presentation, are presented. The normalized SBT chart, based on $Q_{tn}-F_r$, is used to illustrate correlations for soil state (ψ and OCR), stiffness (G_0 , and E) and compressibility (M and C_c). A discussion is also provided to illustrate the usefulness of the clean sand equivalent cone resistance, $Q_{tn,cs}$. Although this parameter evolved out of liquefaction evaluation methods based on case histories, it is shown that the resulting correlations are remarkably similar to those that have been developed independently using CSSM concepts and theory.

A brief discussion is also provided regarding a possible correlation between DMT K_D and CPT $Q_{tn,cs}$, and its application for possible new correlations between the K_D and soil state, peak friction angle and resistance to cyclic loading.

8 ACKNOWLEDGEMENTS

This research could not have been carried out without the support, encouragement and input from John Gregg, Kelly Cabal and other staff at Gregg Drilling and Testing Inc. The sharing of ideas and data by Paul Mayne and Silvano Marchetti is also appreciated.

This author is grateful for the teaching, guidance and encouragement from Professors Campanella and Mitchell. Professor Dick Campanella was one of the first graduate students of Professor J.K. Mitchell at UC Berkeley and this author was one of the first graduate students of Professor Campanella at the University of British Columbia.

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