

Session Report 2: CPT Interpretation

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1 OVERVIEW

Session 2 on interpretation of cone penetration test (CPT) data included nearly 60 papers covering a wide variety of sites, analysis methods, and engineering parameters. This session report focuses on a summary of the different types of information presented in these papers and will provide details from select papers that highlight various aspects of soil behaviour. Papers within this session are referenced in italics, with no year, to distinguish them from other references.

Measurements of soils data at a given site can be broken down into in situ testing, laboratory index testing, and laboratory performance testing. Laboratory performance tests are those which measure a specific soil engineering property, such as strength (s_u or ϕ'), stiffness (G), compressibility (κ , λ , C_c , C_s , or M), and water flow characteristics (k , c_v , or c_h). Conversion of laboratory or in situ test measurements to engineering parameters that can be used for design requires some degree of interpretation. Interpretation methods can include analytical solutions, numerical analyses, probabilistic methods, experimental studies, as well as empiricism (judgment). Experimental studies are separated in this discussion of interpretation methods to highlight papers that focused on differences in response due to changes in specific variables, rather than comparison of different measurement techniques for standard test methods employed.

Interpretation of soil properties based on the results of laboratory or in situ tests each have their own advantages and disadvantages. Laboratory testing has controlled boundary conditions and an extensive and rational basis for interpretation, although it is time consuming and only tests a limited volume of soil that has been disturbed to varying degrees by sampling. On the contrary, in situ testing is rapid and tests a large volume of soil, but there are poorly known boundary condition (particularly drainage, rate effects, and horizontal effective stress), installation or performance of the test disturbs the soil to varying degrees and the basis for interpretation is somewhat limited and often inconsistent between sites. Both laboratory and in situ test results are presented for about 60% of the papers in this session to aid development and verification of interpretation methods. A summary of the distribution of content in Session 2 is presented in Figures 1 and 2.

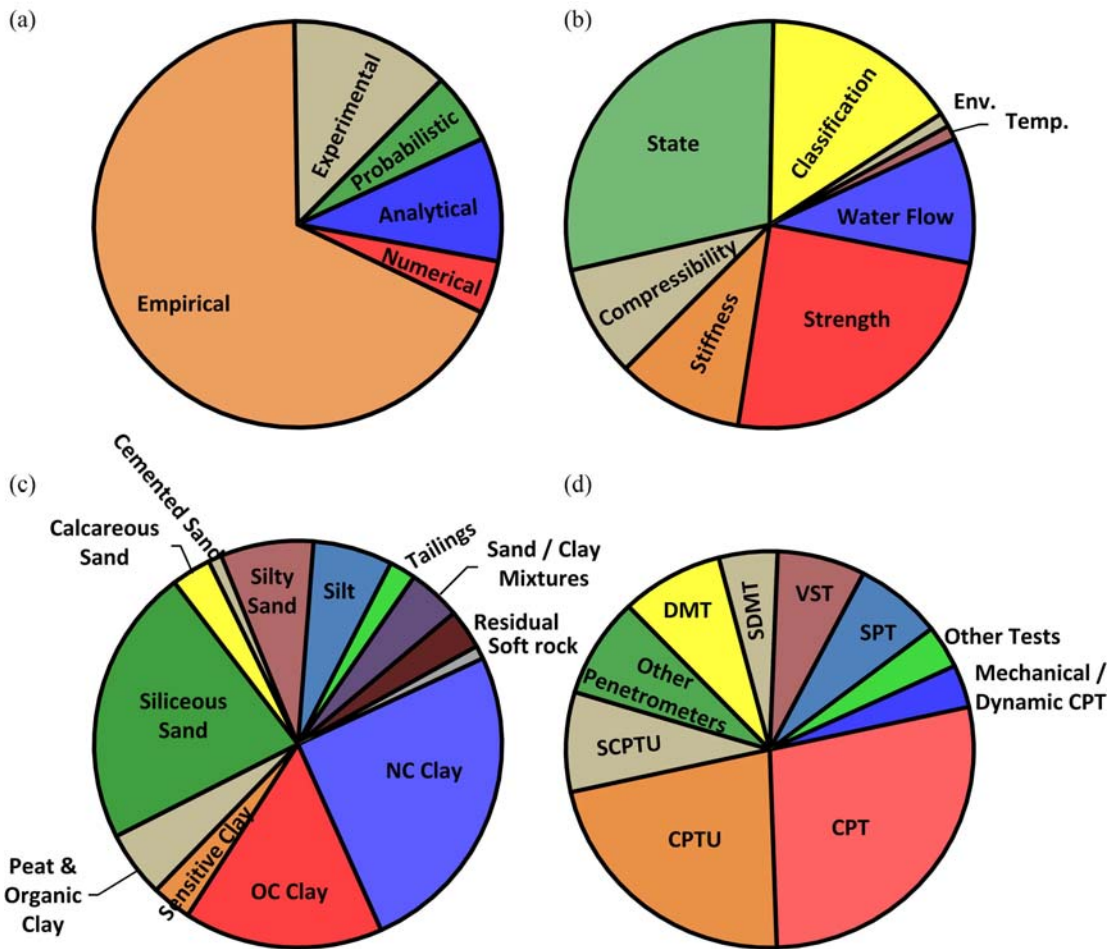


Figure 1. Distribution of papers within interpretation session (a) analysis methods; (b) soil characteristic evaluated; (c) soil types discussed; and (d) in situ tests used

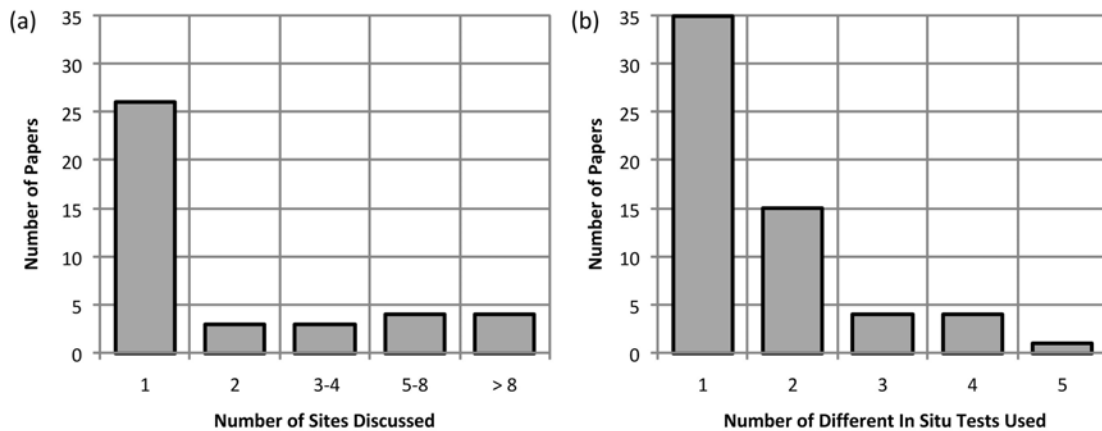


Figure 2. Distribution of (a) number of sites discussed in each paper; and (b) number of in situ tests used for papers in Session 2

The largest proportion of papers in this session focus on increasing the number of soils types and geological conditions with which the engineering community has documented experience. Most papers focus on comparing results from one type of

testing to another. This may include comparisons between in situ test methods (i.e., CPT, DMT, PMT), as well as comparison of laboratory tests (index or performance) with results from in situ tests. The use of probabilistic (*Uzielli et al.*, *Kurup et al.*, *Kurup & Griffin*, *van der Wal et al.*, and *Ng & Zhou*), numerical (*Butlanska et al.*, *Ghafghazi & Shuttle*, *Kumar et al.*), and analytical (*Larsson et al.*, *Isaev & Ryzhkor*, *Löfroth*, *Russell et al.*, *Ghafghazi & Shuttle*) models have a comparatively low proportion of papers in this session. This may result from difficulties in accurately modelling the penetration process using numerical methods with realistic soil models, as many constitutive models do not account for soil structure and aging effects, high stress compressibility and particle crushing, stress dependent dilation, as well as stress and strain dependence on soil stiffness. These features of soil behaviour are being addressed in analytical and numerical modelling of in situ tests, and we should expect to see more application in future conferences (e.g., *Salgado et al. 1997*, *Fahey 2005*, *Zhou & Randolph 2009*, *Einav et al. 2010*).

A wide variety of soil types are discussed, although most papers still focus on normally consolidated (NC) clays, overconsolidated (OC) clays, or siliceous sands. Unsurprisingly, over half of the in situ tests discussed were cone, piezocone, or seismic piezocone tests. Other penetrometers discussed included temperature, resistivity, environmental (ROST/UVOST), water injection penetrometers, as well as full flow penetrometers (T-bar and Ball). Other in situ tests discussed included pressuremeter testing, flat dilatometer (DMT) and seismic dilatometer (SDMT), vane shear testing (VST), and plate load testing.

The remainder of the session report highlights certain aspects of soil response that were observed during review of these papers. This discussion is written around interpretation of (i) strength; (ii) stiffness; (iii) compressibility; (iv) water flow characteristics; (v) soil state; and (vi) soil classification. Interpretation of temperature characteristics (*Isaev & Ryzhkov*, *Trevor et al.*) and environmental issues (*Smith & Gleason*) made up a relatively minor portion of this session and will not be addressed further.

2 STRENGTH

Selection of appropriate strength parameters for design depends on the rate of loading in relation to the size of the loaded object and the rate of water flow through the loaded soil. For interpretation of CPT and CPTU of standard diameters and standard rates of penetration, most sandy soils will behave as drained while most clayey soils will behave as undrained.

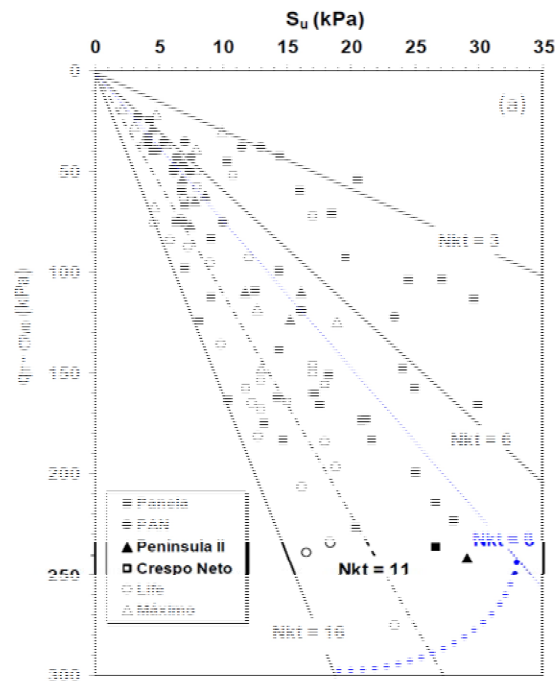


Figure 4. Comparison of vane shear strength and net cone tip resistance for Brazilian Marine clays (*Almeida et al.*)

Use of the undrained strength (s_u) implies that changes in mean effective stress during loading are solely due to shear induced pore pressures. For this case, laboratory measurements of undrained strength performed at the current in situ effective stress (and state) are appropriate for design ($\tau_f = s_u$). There is no volume change during undrained loading, therefore the dilation angle (ψ) is equal to zero. Cone tip resistance is strongly controlled by soil strength such that:

$$s_u = \frac{q_t - \sigma_{v0}}{N_{kt}} = \frac{q_{cnet}}{N_{kt}} \quad (1)$$

Interpretation of undrained strength from CPT and CPTU was the most commonly discussed parameter in this session, and was the focus of 14 papers. Values of N_{kt} are widely reported in the literature, and ranges on the order of 8 to 27 are commonly discussed (e.g., Denver 1995). A large range of N_{kt} values was also presented in this session. An example of this range is shown in Figure 3 for marine clays from coastal Brazil (*Almeida et al.*). Typical reasons for variation in N_{kt} are associated with (i) mode of strength ($s_{u,DSS}$, $s_{u,CIUC}$, $s_{u,UU}$, $s_{u,VST}$, $s_{u,PMT}$, etc); (ii) soil rigidity (G/s_u); (iii) viscous rate effects; (iv) partial consolidation during penetration, testing, or between installation and testing; (v) sample disturbance; (vi) spatial variability between CPT and samples; (vii) CPTU area ratio correction factors; (viii) drift in CPT zero load readings during testing; as well as (ix) soil structure and fabric, such as fissuring as sensitivity. Use of full flow penetrometers, as presented in this session by *Cargill & Camp*, may reduce some of the uncertainty in N (N_{kt} , N_{t-bar} , N_{ball}) values (e.g., Randolph 2004).

Strength parameters for drained analysis are typically the peak friction angle (ϕ'_{pk}), which is a combination of the critical state friction angle (ϕ'_{cv}) and the dilation angle (ψ'). Due to stress dependent dilation of sands and crushing of particles at high stresses typical of cone penetration testing, practice tends to rely on empirical correlations to ϕ'_{pk} . For interpretation of strength of sands, 4 papers discussed friction angle (*Kumar et al.*, *Aykin et al.*, *McNulty & Harney*, *Robertson*) and one discussed dilation angle (*Kumar et al.*). Most interpretation in sands focused on state or relative density (discussed later in this report).

An interesting case history was presented by *McNulty & Harney*, which included CPTU, DMT, and triaxial test results. The focus of this discussion is the CPTU data and triaxial test results. Figure 4 attempts to reproduce part of Figure 1 of *McNulty & Harney* using data from their Table 1. The correlation between CPT data and friction angle used by *McNulty & Harney* was not stated, and whether a 'fines content correction' was applied to CPTU data for their silty and clayey sands was also unclear. Re-interpretation in Figure 4 used no fines content correction and the correlation between stress normalized CPT data and friction angle that was published by *Kulhawy & Mayne* (1990), and seems to agree with the authors' original interpretation:

$$\phi'_{pk} = 17.6 + 11 \cdot \log \left[\frac{q_c / p_{ref}}{(\sigma'_{v0} / p_{ref})^{0.5}} \right] = 17.6 + 11 \cdot \log(q_{cIN}) \quad (2)$$

For additional comparison, peak friction angles calculated using the method of *Robertson* are included in the figure. The method discussed by *Robertson* is based on

the concept of a state parameter for sands (Been & Jefferies 1985) and includes a ‘fines content correction’ estimated from normalized cone tip resistance and friction ratio (F_r). A constant volume friction angle of 30° was assumed for all layers when using the analysis procedures discussed by *Robertson* for this profile.

Two questions arise from comparison of the laboratory and in situ test data (i) is a correlation developed from drained calibration chamber data on clean siliceous sands applicable to silty and clayey sands; (ii) is the laboratory measured friction angle of 20° (at 15m depth) realistic? To address the first question, the degree of drainage during cone penetration is assessed using standard CPTU classification charts (e.g., Robertson 1990, Schneider et al. 2008). In the Schneider et al. (2008) charts, it appears that 6 of the layers classify as drained sands (Zone 2) where the ϕ'_{pk} correlation is likely applicable, 3 of the layers classify as ‘transitional soils’ (Zone 3) where pore pressures during penetration likely have some effect on interpretation, and high penetration pore pressures in 4 layers indicate the material is undrained (Zone 1) and an interpretation of undrained strength is more applicable. The reasonable friction angles of 30 to 35 degrees for the Zone 1 soils are considered to have little to do with penetration resistance and soil response. Agreement is considered to be more likely a result of compensating effects than application of an appropriate correlation. Higher friction angles in Zone 1 soils calculated using the method discussed by *Robertson* resulted from an increase in normalized tip resistance due to the ' K_c ' correction.

With respect to the low measured friction angle in the overconsolidated clays, *McNulty & Harney* attributed this response to apparent cohesion at stress levels lower than the preconsolidation stress (p'_c). It is correct that adding an apparent cohesion intercept will reduce the interpreted friction angle from laboratory tests, but this does not reflect the actual soil behaviour in the laboratory test or in the field. An increase in resistance and occurrence of peak friction angles at effective stresses below the preconsolidation stress results from interlocking and resistance in addition to that from friction. Misinterpretation of this behaviour as caused by ‘cohesion’ does not reflect the mechanical behaviour of the soil and may be unconservative for certain design situations (e.g., Santamaria 1997, Schofield 2005).

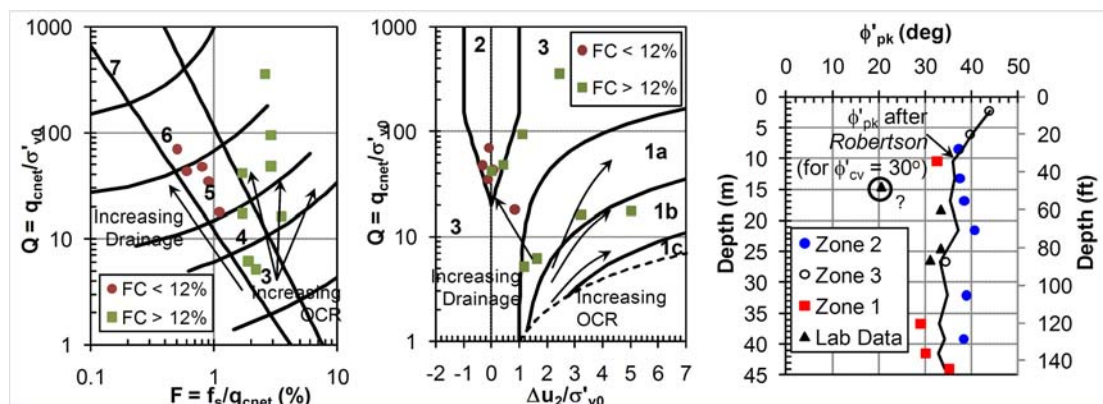


Figure 4. Soil classification and friction angle correlations (data from *McNulty & Harney*, for details on classification charts see Robertson 1990, Schneider et al. 2008)

3 STIFFNESS

To evaluate deformation of materials, two elastic parameters are needed; (i) the elastic modulus (E) and Poisson ratio (ν); or (ii) the shear modulus (G) and bulk modulus (K). It is more appropriate to consider G and K for soil mechanics applications, as this separates the effects of shearing (change in shape due to application of a deviator stress) from compression (change in size due to changes in mean effective stress) (e.g., Atkinson 2007). This section will focus on evaluation of shear modulus, while the next section will discuss compression characteristics of soils.

The stiffness of soil is characterized using the shear modulus, G . Our ability to characterize stiffness of soils from cone penetration tests has greatly increased since the development of the seismic cone (SCPT) and seismic piezocone penetration (SCPTU) tests (e.g., Robertson et al. 1986). Measured values of shear wave velocity (V_s) can be used to calculate stiffness, or more specifically the small strain shear modulus (G_0):

$$G_0 = \rho \cdot V_s^2 \quad (3)$$

where ρ is the total mass density of the soil. A majority of papers within this session that relate to soil stiffness focus on shear wave velocity (*Uzielli et al., Paoletti et al., Prakoso, Aykin et al., Trevor et al.*), while 3 papers discuss small strain shear modulus (*Kavur et al., Schneider & Lehane, Kumar et al.*), and 1 paper discusses larger strain stiffness measurements (G_{sec}) from a self boring pressuremeter test; (*Schneider & Lehane*). It is common to quantify shear modulus as a multiple of CPT tip resistance, and G/q_{cnet} is presented in this section:

$$\frac{G}{q_{cnet}} \approx K_G \cdot \left[\frac{q_c/p_{ref}}{(\sigma'_{v0}/p_{ref})^{0.5}} \right]^{-n} = K_G \cdot q_{c1N}^{-n} \quad (4)$$

where ' K_G ' is a soil deposit dependent coefficient and ' n ' is a stress exponent.

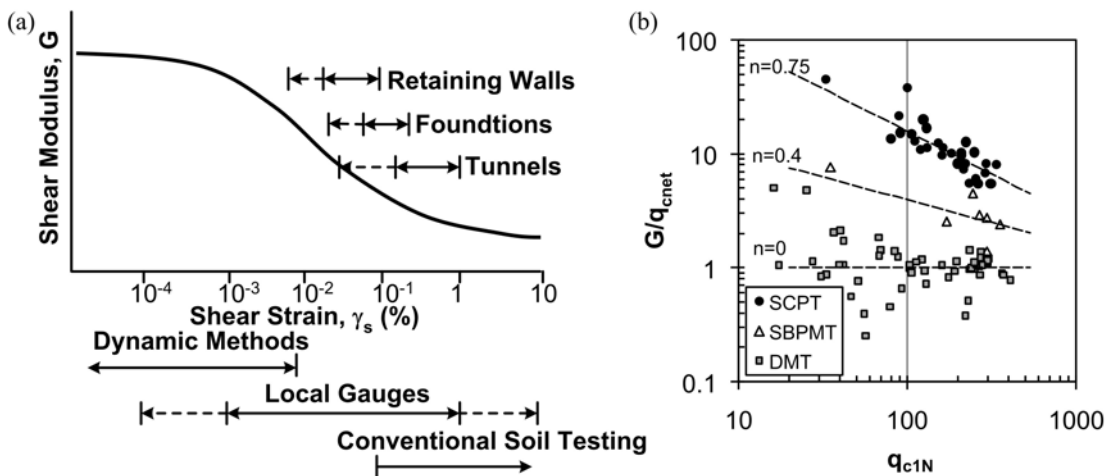


Figure 5. Influence of measurement technique on soil stiffness (a) laboratory tests and design situations (after Atkinson 2000); (b) in situ test method (for a calcareous sand, data from Schneider & Lehane)

Since shear modulus measured by dynamic methods is much larger than operational levels for typical design situations (e.g., Figure 5a) measurement of G_0 is useful but must be reduced appropriately. Modulus reduction schemes are not unique, and measurement of stiffness from a number of different in situ tests can aid in the assessment of modulus reduction for design (e.g., Lehane & Fahey 2004). Figure 5b shows small strain shear modulus measured by seismic methods (SCPTU), as well as that from self boring pressuremeter tests (SBPMT) and that estimated from flat plate dilatometer [$G \approx E_D/(2 \cdot (1+\nu))$]. Shear strain levels associated with each tests are approximately 10^{-4} % for SCPTU, 10^{-1} % for SBPMT, and 1% for DMT. It is clear that the shear modulus reduces as characteristic strain levels associated with various testing methods increase, but it is also interesting to see that the stress exponent 'n' appears to reduce with this increasing strain level. The implications of these trends in stress exponents, and whether they are actually a function of soil behaviour rather than scatter in the measurements, requires further study.

4 COMPRESSIBILITY

The change in size of an element of soil with increasing mean effective stress can be considered as compression. Compressibility is essentially the inverse of the bulk modulus, $1/K = \Delta\varepsilon_v/\Delta p'$. For geotechnical engineering purposes the one dimensional (1D, no horizontal strain) case is typically considered and the stress dependence on compressibility is handled using the compression index, $C_C = \Delta e/\Delta \log \sigma'_v$. When developing correlations to in situ tests the 1D constrained modulus is typically quantified where, $M = D' = 1/m_v = \Delta \sigma'_v/\varepsilon_v = \Delta \sigma'_v/\varepsilon_a = 2.3 \cdot (1+e_0) \cdot \sigma'_{v,avg}/C_C$. Within the papers presented as part of this session, three methods were used to evaluate constrained modulus (i) oedometer tests (*Lee, Choi, et al., Bihs et al., Ozer et al.*); (ii) correlations to DMT (*Bihs et al., Kavur et al., Mlynarek et al., Aykin et al.*); and (iii) results from a large scale test embankment (*Tonni et al.*). Most sites tested were clays and organic clays, although data for a cemented sand (*Lee, Choi, et al.*) and layered sand, silt, and clay site (*Tonni et al.*) were also presented.

Based on elastic theory, the constrained modulus should be approximately 0.9E and therefore essentially equal to the dilatometer modulus E_D (e.g., Marchetti 1980). A unique correlation between E_D and M has not been observed (e.g., Marchetti 1980, Powell & Uglow 1988) highlighting the need to separate change in shape of soil element (G or E) from change in size of a soil element (M or 1/K).

Difficulties also exist when attempting to select the appropriate stress range for M when correlating to laboratory or field test data, as well as selecting whether cone tip resistance is an appropriate parameter for correlations to M. Figure 6 shows two correlations between CPT parameters and M (or D'). Typically, the in situ vertical effective stress level is used for correlations to M (e.g., Mayne 2007). Within this session *Ozer et al.* utilized the value of M at the preconsolidation stress, while *Tonni et al.* evaluated M for a stress range starting from σ'_{v0} and increasing by up to 110kPa. Since M increases with effective stress (for a given soil in an overconsolidated or normally consolidated state), variations in effective stress range for assessment of M will increase uncertainty in correlations. While incorporation of measured pore pres-

tures improved CPT-M correlations for the Treporti site, complex pore pressure response that includes both shear and octahedral components as well as reduction in total stress behind the cone tip (e.g. *Ha et al.*) make application of this correlation to a wider range of soil types and states uncertain.

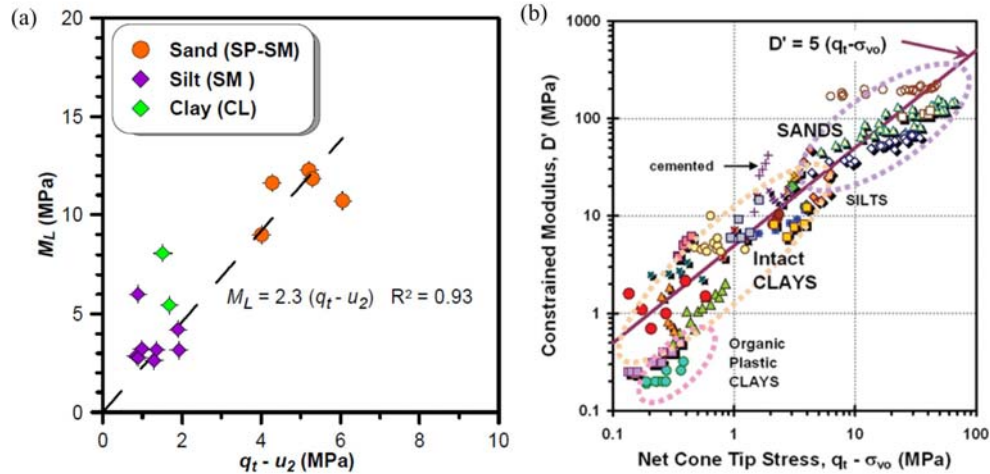


Figure 6. CPT – constrained modulus correlations for loading of (a) sands silts and clays loaded by 110kPa at the Treporti site (*Tonni et al.*); (b) global database for oedometer tests on sand silts and clays with M taken at σ'_{v0} (*Mayne 2007*)

5 WATER FLOW CHARACTERISTICS

Water flow characteristics were addressed in 7 papers, 3 focusing on coefficient of consolidation (c_h or c_v ; *Kim, Prezzi, et al., Almeida et al., Jaeger et al.*), 2 focusing on coefficient of permeability (k ; *Vienken et al., Boone & Frietas*), and 2 discussing both parameters (*Cai et al., Robertson*). Direct methods for interpretation of c_h and k include analytical solutions for piezocone dissipation tests (e.g., *Teh & Houlsby 1991*), variable rate penetration testing (e.g., *House et al. 2001, Randolph 2004*), as well as injection methods (e.g., *Vienken et al.*). Indirect methods typically involve correlations to soil behaviour type (I_c) and/or fines content (*Boone & Frietas, Robertson*).

While discussed since the early 1970's, the prominence of variable rate penetration testing has increased in research (and in practice) since about 2004 (e.g., *Randolph 2004*). Two similar experimental studies of variable rate penetration testing performed independently but on similar materials (sand with 25% Kaolin) are presented in this session; chamber tests of *Kim, Prezzi et al.* and centrifuge tests of *Jaeger et al.* Normalized CPTU parameters (Q , $\Delta u_2/\sigma'_{v0}$, B_q) are plotted in Figure 7 against normalized velocity ($V = vd/c_v$, where v is the penetrometer velocity, d is the penetrometer diameter, and c_v is the vertical coefficient of consolidation). Similar response is observed for both studies; (i) Drained behaviour occurs for V less than about 0.05 and is characterized by relatively high Q values and essentially zero values of $\Delta u_2/\sigma'_{v0}$ and B_q ; (ii) Undrained behaviour occurs for V greater than about 15 and is characterized by relatively low values of Q and 'high' values of $\Delta u_2/\sigma'_{v0}$ and B_q . Differences in B_q are observed between the two studies during undrained penetration and seem to be

related to very low values of Q recorded by *Jaeger et al.*. Results from supplemental undrained triaxial tests on the sand-clay mixtures would help clarify these differences.

It is difficult to characterize water flow characteristics in aquifers and some levees since penetration in sands produces essentially no excess pore pressures ($\Delta u_2 \approx 0$). *Vieken et al.* discuss a number of direct methods for aquifer characterization, including active source penetrometers, while *Boone & Frietas* discuss application of an indirect method to a wide range of soil types.

A summary of indirect and direct methods for characterization of water flow characteristics is given by *Robertson*. The importance of performing piezocone dissipation tests when the degree of drainage is unclear is highlighted in this paper, and it is suggested that if t_{50} from a piezocone dissipation test takes longer than 30 seconds ($t_{95} \approx 20$ minutes) then tip resistance during penetration is essentially equal to the undrained value. Recording piezocone dissipation response in questionable layers for only one to three minutes can provide invaluable information for assessing degree of drainage during penetration, and supplementing those results with variable rate penetration tests (for rates of 0.1mm/s to 200 mm/s) can help quantify the change in resistance to changing drainage conditions.

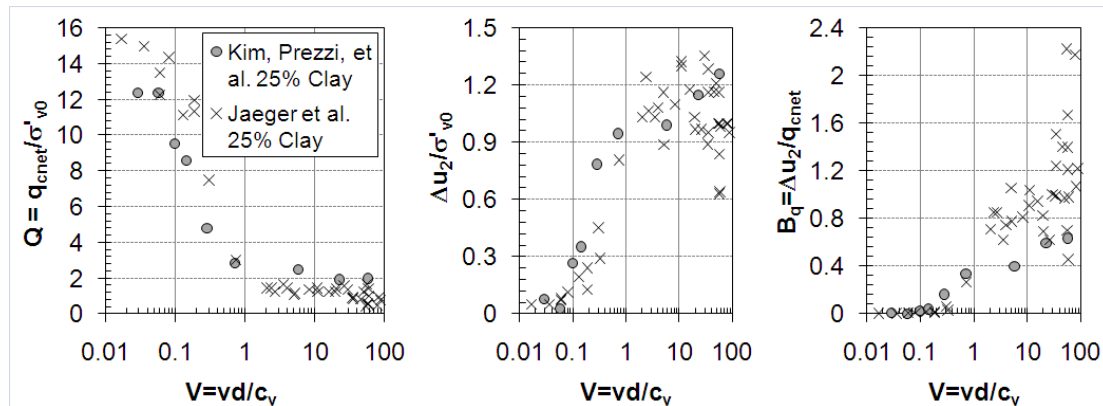


Figure 7. Influence of normalized velocity on penetration resistance in clayey sands (data from *Kim, Prezzi, et al.*, and *Jaeger et al.*)

6 SOIL STATE

A large portion of the papers in this session (18) focused on evaluation of soil state. This typically revolved around OCR or preconsolidation stress (p'_c) in clayey soil (9 papers) and relative density in sands (8 papers). Additionally, *Ghafghazi & Shuttle* and *Robertson* discuss assessment of the state parameter for sands, ψ_0 . Soil state is an indirect measure that can aid in prediction of soil response, and will have a strong influence of strength, stiffness, and compressibility.

While the state of sands is defined by current void ratio and effective stress in relation to the critical state line (e.g., *Been & Jefferies 1985*, *Coop 2005*), evaluation of the state of sands typically revolves around relative density, D_r . Since CPT tip resistance is strongly controlled by in situ horizontal effective stress, shear modulus, friction angle, (stress dependent) dilation angle, and high stress compressibility (crush-

bility), there are large uncertainties in evaluation of D_r and ψ_0 . Two papers are focused on here in relation to relative density evaluation; (i) *Butlanska et al.* who attempt to better understand the relationship between q_c and D_r through DEM numerical modelling, and (ii) *Senders* who evaluates shallow embedment factors through centrifuge testing. Selected results from the two studies are highlighted in Figure 8. Results of the *Butlanska et al.* study may lead to a better understanding of CPT behaviour in sands, although additional study into scale effects between penetrometer and particles as well as the near surface effects presented by *Senders* would be useful in future work. It is noted that the ratio of penetrometer diameter (71.2mm) to particle mean grain size ($d_{50} = 28\text{mm}$) appears to be 2.5 for the study of *Butlanska et al.*. Bolton et al. (1999) recommend that this ratio be at least 20 such that the soil-penetrometer interaction behaves closer to a continuum, which is more representative of typical field conditions.

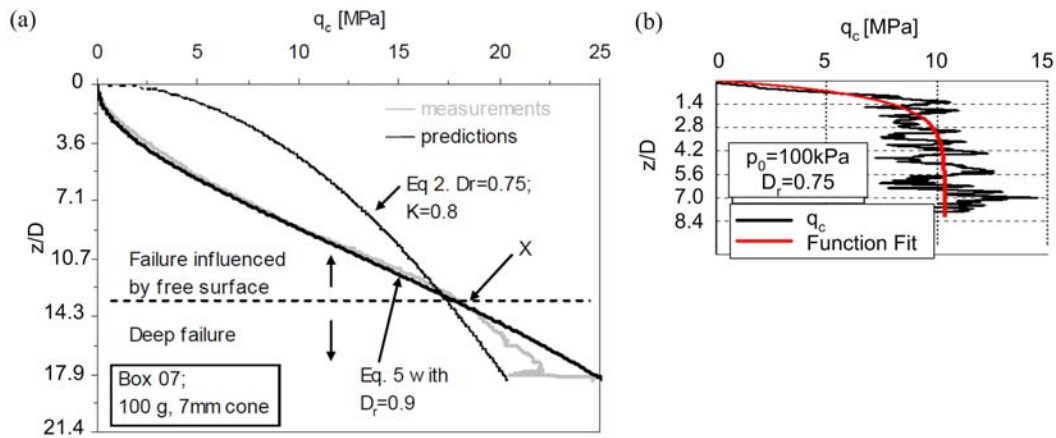


Figure 8. Comparison of sand relative density evaluations from (a) centrifuge testing (*Senders*) and (b) DEM numerical modelling (*Butlanska et al.*)

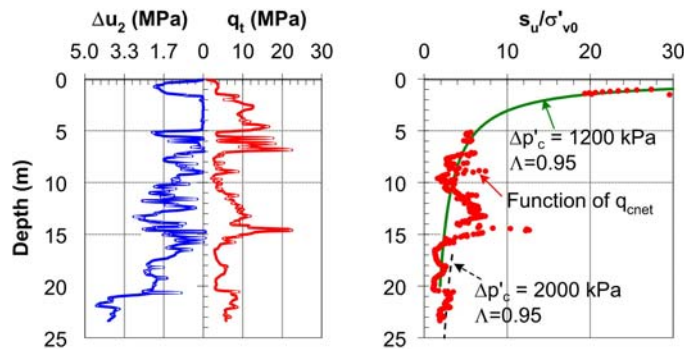


Figure 9. Plot of normalized undrained strength based on CPT q_{cnet} and estimated from stress history (data from *Liao et al.*)

Soil state for clays, represented by the overconsolidation ratio ($OCR = p'_c/\sigma'_{v0}$) or vertical yield stress ratio ($YSR = \sigma'_{vy}/\sigma'_{v0}$), is a vital parameter for linking known geologic history to anticipated mechanical behaviour at a given site. The relationship between undrained shear strength ratio and OCR, based on the concepts of critical

state soil mechanics, is a powerful tool for understanding soil behaviour and predicting engineering response.

$$\frac{s_u}{\sigma'_{v0}} = \left(\frac{s_u}{\sigma'_{v0}} \right)_{NC} \cdot OCR^\Lambda \quad (5)$$

where $(s_u/\sigma'_{v0})_{NC}$ is the normally consolidation undrained strength ratio [$\approx (\sin\phi'_{cv})/2$ for DSS] and Λ is the plastic volumetric strain ratio (about 0.8) (e.g., Mayne 2001). The application of this concept to a glacial soil site in Eastern Wisconsin discussed by *Liao et al.* is presented in Figure 9.

Liao et al. present a case history where a 35m high lakeshore bluff was removed to build a coal power plant. The underlying glacial clay and sandy clay till soils tended to have a preconsolidation stress predicted by a range of empirical correlations that was higher than the stress of the overburden removed. If the normalized undrained strength ratio (using $N_{kt}=12$) is plotted for clayey layers with $\Delta u_2/\sigma'_{v0}$ greater than about 1, trends match an estimated preconsolidation difference ($\Delta p'_c = p'_c - \sigma'_{v0}$) of 1200kPa in the upper 21m and 2000kPa below 21m. These values of $\Delta p'_c$ are consistent with the regional glacial geology and in excess of those related to bluff removal. Knowledge of local geology is often invaluable for interpretation of in situ tests. Additional validation of correlations through complementary laboratory oedometer testing is recommended for these large projects, although, applied vertical stresses up to 10MPa may be necessary for a clear definition of p'_c and the slope of the normal compression line.

7 CLASSIFICATION

Soil classification was a major component of approximately 13 papers. Most papers focused on cone and piezocone response in a range of soil conditions (*Kurup et al.*, *Dasenbrock et al.*, *Lo Presti et al.*, *Mateos*, *Robertson*, *Tonni et al.*), while others focused primarily on characterization of clays (*Bihs et al.*, *Rømoen et al.*, *Eslami et al.*, *Ng & Zhou*), calcareous sands (*Schneider & Lehane*), residual soils (*Mio et al.*), or clayey sands (*Jaeger et al.*).

To rationally assess soil behaviour and classification from piezocone response normalized parameters must be used (e.g., Wroth 1988, Robertson 1990). *Robertson* points out that for typical stress ranges related to onshore projects (50 to 150kPa), normalization does not significantly shift points within the classification charts and an equivalent non-normalized chart (and I_{SBT} parameter) can be used with similar levels of reliability to normalized charts. When dealing with larger stress ranges and normalized parameters, response due to effects of soil state must be separated from those due to water flow characteristics. This can most easily be done using normalized tip resistance ($Q = q_{cnet}/\sigma'_{v0}$) and penetration pore pressures ($\Delta u_2/\sigma'_{v0} = B_q \cdot Q$) (e.g., *Schneider et al.* 2008), although use of Q and friction ratio ($F = f_s/q_{cnet}$) is common due to difficulties in obtaining high quality pore pressure data as well as when testing soils above the water table.

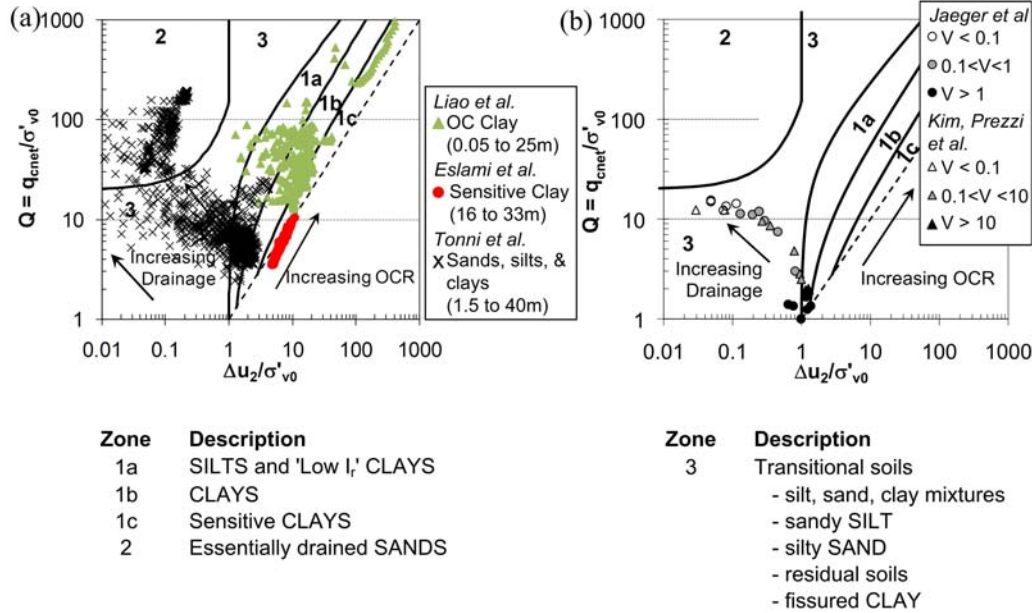


Figure 10. Separation of partial consolidation during penetration from overconsolidation using classification charts of Schneider et al. (2008) for (a) natural soils; and (b) variable rate penetration testing (values of Q below one not plotted)

Figure 10 highlights separation of soil state from water flow characteristics for an OC Clay, a sensitive clay, and layered sands, silts, and clays. Site data comes from papers within this session. It should be noted that soil state and partial consolidation during penetration can not clearly be distinguished in $Q - B_q$ classification charts, despite using the same data as Figure 10 ($\Delta u_2/\sigma'_{v0} = Q \cdot B_q$). In Figure 10b, results from variable rate penetration tests shown in Figure 7 are also plotted in a $Q - \Delta u_2/\sigma'_{v0}$ classification chart. Trends in behaviour are consistent with those for natural soils; undrained penetration tends to plot in Zone 1 – undrained clays; as V decreases, measurements move through Zone 3 – transitional soils towards Zone 2 – drained sands; for V less than about 0.1 $\Delta u_2/\sigma'_{v0}$ is less than 0.1 and penetration is essentially drained. The greater high stress compressibility of the clayey sands as compared to soils that would have drained penetration for standard velocities and diameters lead to relatively low Q values and data plotting below Zone 2. When comparing these variable rate tests to others performed in Kaolin, it appears that the high stress compressibility affects drained penetration resistance (e.g., Robertson & Campanella 1983) as well as the ratio of drained to undrained penetration resistance.

8 CONCLUSIONS

CPT data at a wide variety of new sites is presented in this session along with complementary laboratory and in situ test data for interpreting a range of soil properties. Most interpretations build on previously developed methods and discuss similarities or differences due to specific site conditions. These studies are important for developing judgment for a wider range of soil conditions. It is noted that these data should be viewed within a systematic engineering framework such that experiences from a sin-

gle site can be applied to differing soil conditions that will be encountered in the future. Specifically, one must be sure to separate partial consolidation during loading from increases in overconsolidation ratio, separate stiffness and compressibility, as well as utilize critical state soil mechanics to couple strength with stress history. In addition to comparison of data from new sites, new interpretation methods (analytical, numerical, probabilistic) still need to be developed to aid in consistent understanding of a wide range of soil conditions, as well as spatial variability of interpreted engineering parameters.

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