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Piezocone profiling of clays for maritime site investigations

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ABSTRACT: Site investigations for ports, bridges, and offshore facilities benefit from the use of piezocone tests in the field exploration program since multiple continuous readings with depth are obtained by the soundings. The interpretation of the piezocone resistances is discussed in context with geostratigraphy and the interpretation of strength parameters needed in geotechnical design. All three piezocone readings can be used to advantage in profiling site conditions and soil properties. The classical method of directly assessing undrained shear strength of clays can alternately be directed at a stress history approach using either critical state soil mechanics or normalized strength ratios which (s_u/σ_{vo}') to overconsolidation ratio (OCR). Case studies involving an offshore site and two nearshore sites with port expansions are presented in this context.

1. INTRODUCTION

Maritime projects involve both onland sites with high groundwater tables and nearshore to offshore locations with freestanding water over the seabed. The utilization of piezocone penetration tests (CPTu or PCPT) are therefore appropriate for obtaining quick profiles of the subsurface soil stratigraphy. The CPTu obtains the measured cone tip resistance (q_t), sleeve friction (f_s), and induced penetration porewater pressures (u_m) continuously and expediently with depth (z). These readings (q_t, f_s, u_m) can be post-processed and interpreted to provide relevant geotechnical parameters needed for the design of foundations, pilings, walls, fill embankments, and ground modification works as they relate to the construction of bridges, ports, wharfs, and offshore platform structures. The CPTu results can be utilized to supplement limited information and discrete data obtained from conventional soil test borings used to procure samples for laboratory testing. An excellent and detailed summary of site investigative methods for geotechnical practice on nearshore and offshore projects is given by Kolk, et al. (2005).

For site investigations encountering soft to firm clays, in-situ field vane shear tests (VST) are commonly employed to ascertain the undrained shear strength of these sediments (Peuchen & Mayne, 2007). The VST also provides a direct assessment of the sensitivity of fine-grained soils. Depending upon the specific conditions, additional in-situ devices can be brought into the field testing program, such as the offshore flat dilatometer blade (Lunne, et al. 1990), pressuremeter probe (Richards & Zuidberg, 1985), and tapered piezoprobes (Whittle, et al. 2001), as well as geophysical mapping technologies (Peuchen & NeSmith, 2004).

Of recent utilization, a series of full-flow penetrometers (i.e., T-bar, plate, and ball) have been introduced for investigations in very soft soils to improve the measurement of loads and increase their resolution at low stresses (Randolph, et al. 1999, Randolph 2004). In actuality, the T-bar and ball penetrometers are modified versions of the cone penetrometer but using a larger "head" with 100-cm^2 cross-sectional area, rather than the standard small size 10-cm^2 head with 60º angled tip.

With the cone penetrometer in clays, practice has centered on the use of the cone tip resistance for ascertaining the undrained shear strength. As the device provides three separate measurements, all three readings in fact can be used towards this purpose. In this paper, the author shares some experiences with the use and interpretation of CPTu results on two recent port projects and other maritime sites.

2. DEPLOYMENT

During the CPT, the penetrometers are pushed vertically at a constant rate of 20 mm/s. This is normally accomplished using hydraulic actuators and pump systems mounted on various types of vehicles. Onshore test locations underlain by firm ground can move about with truck-mounted equipment, although many coastal areas have soft ground conditions, thus the hydraulics are better positioned on vehicles mobilized on tracks, all-terrain wheels, or marsh buggies (Figure 1). When overwater soundings are required for bridge piers or dock facilities, the CPT vehicles can be driven onto barges, floating rafts, or jackup platforms in order to provide a stable working area (Figure 2).

Figure 1a. Cone truck at Port of Los Angeles, California (courtesy of Fugro Geosciences).

Figure 1b. All-terrain vehicle for CPTs in Charleston, South Carolina (courtesy S&ME)

Figure 1c. Track-mounted CPT system, Alberta (courtesy ConeTec Investigations).

Figure 1d. Marsh-buggie CPT for swampy ground in New Orleans area (Terracon).

Fig. 2. Jackup platform (SeaCore) used for CPTU soundings at Port of Anchorage, Alaska (courtesy of ConeTec Investigations)

Fig. 3. Portable remote operated drill (PROD) for offshore work

The effects of tidal variations need to be considered when conducting CPTu probings in nearshore environments, since the reference hydrostatic porewater pressures (u_0) and associated calculated parameters can be influenced over the duration of the sounding.

In offshore site investigations, a number of innovative deployment strategies have been devised (Richards & Zuidberg, 1985; Kolk, et al. 2005). These include special CPT systems based on: (a) downhole wireline technology, (b) automated seabed supported frames, and (c) dynamic positioning ships. Seabed systems include proprietary designs such as the Wison and Seasprite that include structural frames that are lowered to the mudline for operation (Peuchen & NeSmith, 2004). The portable remote operated drill (PROD) is a complete stand-alone remote-controlled robotic drilling platform that rests on three extended footers and operates on a 5-km long tether. PROD is capable of drilling soil, taking samples, pushing cones, ball-penetrometers, and vanes, as well as coring rock to 150 m depths below the seabed (see Figure 3).

In some arrangements (Sage system), a true continuous push up to 15 m long can be attained using a special set of steel tubing (in lieu of rods) that is unfurled from an initially coiled cable spool. The front end of the tubing utilizes a miniature size penetrometer to take CPT readings. Both onshore and offshore versions are available (Tumay, et al. 1998).

In the wireline type system, the CPT is pushed in sequential set increments of 1 to 3 m at a time in order (termed "downhole CPT) to achieve the desired final test depths. The follow-up is to drill-out this tested depth before proceeding with the next phase of CPT pushing. Details on this operation are given by Lunne, et al. (1997).

Recent research using an electric screw motor to replace the hydraulic pushing system has advantages in allowing a "twitch" test whereby the penetrometer rate is successively stepped down by several orders of magnitude. The resulting changes in recorded cone tip resistance and porewater pressures can be evaluated to ascertain the boundary between undrained response and partial drainage effects, as well as the viscous rate effect on undrained strength of the geomaterial (Randolph, 2004).

3. CLASSICAL s_u INTERPRETATIONS

When CPTu results encounter clay soils, the conventional practice is to directly evaluate the in-situ undrained shear strength ($s_u = c_u$). The classical route is to adopt an inverted bearing capacity form, whereby:

$$s_u = \frac{q_t - \sigma_{vo}}{N_{kt}} \tag{1}$$

where q_t = total (corrected) cone tip resistance (Lunne, et al. 1997), σ_{vo} = total vertical overburden stress, and N_{kt} is a bearing factor dependent upon the theoretical basis (e.g., Konrad & Law, 1987; Yu & Mitchell, 1998). In practice, this is most often the sole interpretation performed, perhaps using an assumed value for N_{kt} or else a fitted value to an adopted reference value of s_u (e.g, vane tests or laboratory shear test). In some cases, a couple of s_u profiles may be shown, using two adopted values of N_{kt} in equation (1). Common ranges for N_{kt} in soft intact clays are generally taken to be between 10 and 20, yet are mode-dependent (Lunne, et al. 1997).

In fact, it is possible to independently evaluate a profile of s_u entirely from the porewater pressure measurements. While this may be best handled by a mid-face element (designated u_1), the shoulder element (u_2) reading is a more suitable measurement from a pragmatic standpoint because it is required in the correction of measured tip resistance (q_c) to the total tip resistance (q_t), as detailed by Jamiolkowski et al. (1985). The expression for undrained strength here is given by:

$$s_u = \frac{u_2 - u_o}{N_{\Delta u}} \tag{2}$$

where $N_{\Delta u}$ = porewater bearing factor (e.g., Tavenas & Leroueil, 1987).

The idea of redundancy and the ability to produce two separate profiles of undrained shear strength in the clay formation is actually quite nice, since hopefully the two will support each others findings. If the two profiles agree, then a higher degree of reliance might be afforded in the value used in design. If the two profiles do not agree, then the results may serve as a warning that a higher level scrutiny needs to be undertaken by the engineer. For instance, perhaps the porewater porous filter or cone assembly was not properly saturated prior to the sounding. In that case, one would expect poor quality u_2 measurements. In another scenario, poor agreement might suggest electrical baseline shifts of one or the other channels, otherwise damage to the load cell or transducer. In other possibilities, the soil formation itself may have unusual aspects and therefore not behave as "clay". Perhaps the fine-grained soil under examination has a rather substantial sand fraction, unusual mineralogy, or an abnormal structure and fabric that would place it within the domain of "nontextbook geomaterials" (Schnaid, 2005).

Of additional note, the measured sleeve friction resistance (f_s) can be considered as a remoulded shear strength of clays (Gorman, et al. 1975), expressed:

$$f_s \approx s_{u\,(rem)} \tag{3}$$

As such, since equations (1) and (2) are directed toward evaluating the peak strength of the clay, (3) can serve as a lower bound in assessing the s_u profile (Figure 4).

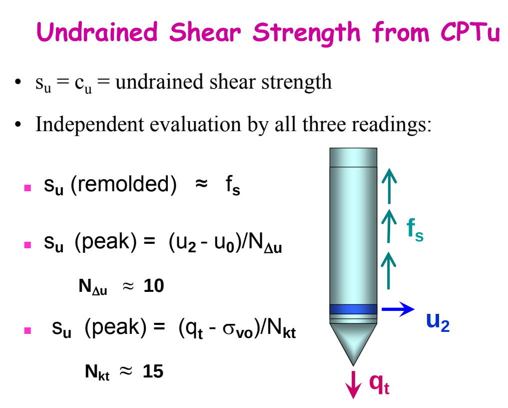

Fig. 4. Direct methods for evaluating peak and remoulded strengths in clays.

To illustrate the use of all three CPT readings in assessing undrained strengths, an example sounding from the restoration studies from the New Orleans levee system is presented in Figure 5. Here the profile is comprised of shallow soft peaty ground from the surface to about 4 m, underlain by soft organic silts with some fine sand lenses to 8 m, overlying soft silty clay extending to at least the termination depths of the sounding at 21 meters. A simple post-processing of CPT results in terms of the friction ratio (FR = f_s/q_t in %) can be helpful in delineating the changes in these three basic strata at the site. Porewater pressure development is not particularly strong in either of the upper two layers of peat and organic silt, but rather significant in the lower clay unit at depths of 8 to 21 m.

For reference, results from field vane tests (VST) were obtained at the New Orleans location using a Geotech AB type electro-vane system. Adopting N_{kt} = 15 and $N_{\Delta u}$ = 10, Figure 6a shows good agreement between the derived profiles of peak s_u for the lower silty clay sediments and vane shear data. The upper organic silt is also adequately represented by the values obtained from the net cone tip resistance profile ($q_{tnet} = q_t - \sigma_{vo}$). Note that the peat layer is more complicated and thus is a geomaterial requiring considerably more attention. Peats are not likely to be easily characterized using piezocone tests, nor by other in-situ methods. Some guidance on peaty soils and their evaluation is given by den Haan & Kruse (2007).

In Figure 6b, the results from the remoulded vane data in the lower silty clay compare quite well with the measured sleeve friction resistances. Therefore, all three CPT readings can serve a purpose in profiling strengths in clays. For silts, some reasonable results were obtained from tip and sleeve channels. On the other hand, special procedures would need to be engaged for peaty soils.

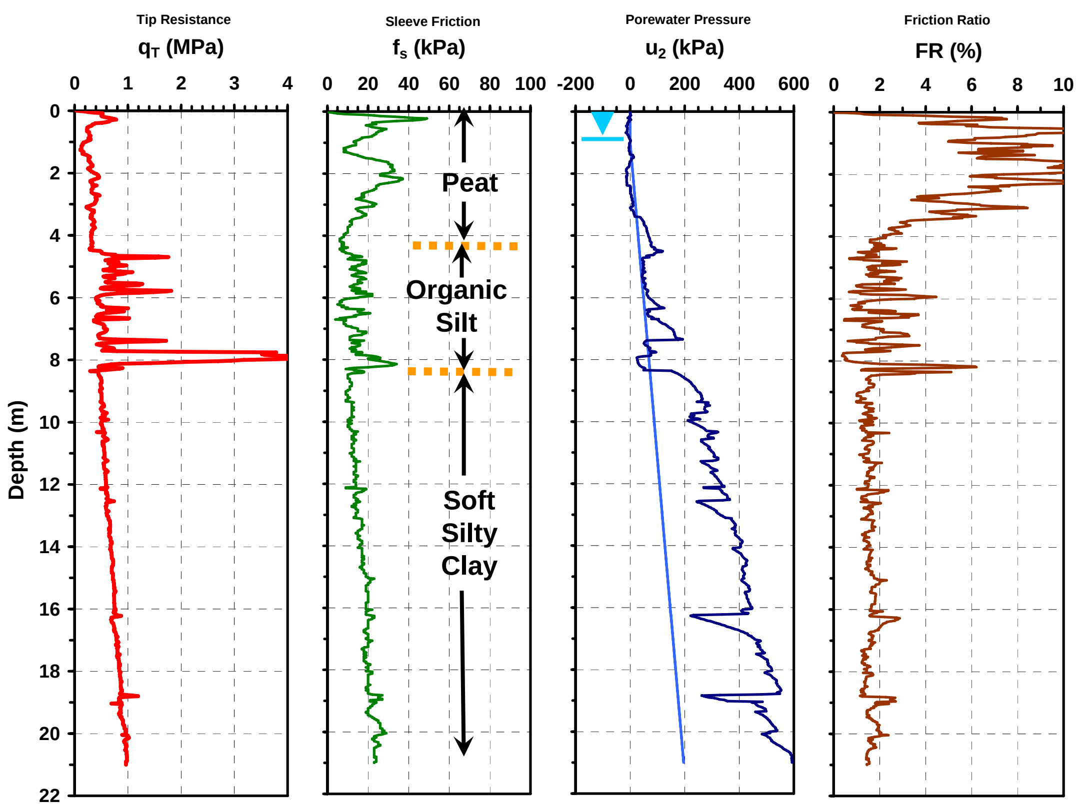

Fig. 5. Representative piezocone tests in soft ground at New Orleans levees.

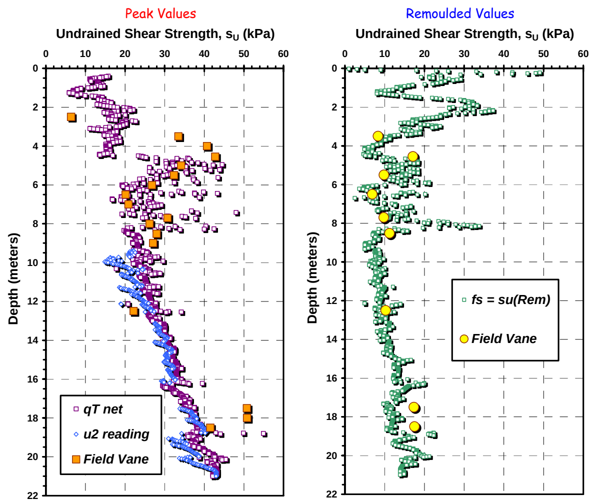

Fig. 6. Piezocone and vane interpretations of (a) peak strength; (b) remoulded strength from south New Orleans.

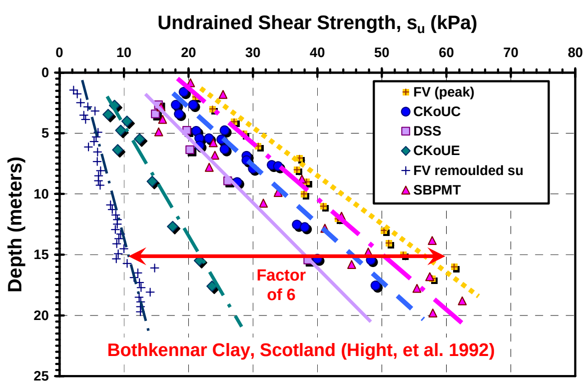

Fig. 7. Various measured undrained shear strengths for well-documented Bothkennar clay, Scotland (data from Nash et. al 1992 and Hight et al. 2003)

4. UNDRAINED SHEAR STRENGTHS

The undrained shear strength is not a unique value for a given soil deposit, but depends upon the type of test used to measure its value. This creates difficulties when trying to assess a representative s_u in clays, as the user will already have selected a bias in his or her choice of a reference value for s_u.

For example, various sets of undrained shear strengths measurements for the well-documented Bothkennar soft clay site in the UK are presented in Fig. 7. These include results from in-situ pressuremeter tests (PMT) and vane shear tests (VST), as well as laboratory series from triaxial compression, direct simple shear, and triaxial extension tests on undisturbed samples (Hight, et al. 2003). At a given depth, these various values might range up to a factor of six, depending upon the particular test mode and strength under consideration.

Similar variations can be reported for other well-characterized clays, such as the listed values of undrained shear strength ratios S = $(s_u/\sigma_{vo}')_{NC}$ for normally-consolidated Boston Blue Clay given in Table 1. The results vary from a low of 0.14 for unconfined compression (UC) testing to a high value of S = 0.42 for pressuremeter tests (PMT) on the same soil.

Observed ranges in the various undrained shear strengths can be attributed to the differences in the test modes, strength anisotropy, boundary conditions, strain rates, initial stress state, and other factors. The non-uniqueness of s_u must be addressed if a clear and consistent framework is to be adopted by the geotechnical community in its practice.

Table 1. Undrained strength ratios (S) for normally-consolidated Boston Blue Clay (data from Ladd et al. 1980; Ladd 1991).

Test Method/Mode	$(s_u/\sigma_{vo}')_{NC}$
Self - boring type pressuremeter test (SBPMT)	*0.42*
Plane strain compression (PSC)	*0.34*
Triaxial compression (CK_0UC)	*0.33*
Unconsolidated Undrained (UU)	*0.275*
Field vane shear test (FV)	*0.21*
Direct simple shear (DSS)	*0.20*
Plane strain extension (PSE)	*0.19*
Triaxial extension (CK_0UE)	*0.16*
Unconfined compression (UC)	*0.14*

5. STRESS HISTORY APPROACH

An alternative means to the aforementioned approach involves the utilization of the piezocone data to profile the stress history of the clay deposits. The stress history of soils can be represented by a geologic model of sedimentation, unloading (erosion, glaciation), reloading, ageing, and groundwater changes, as well as other effects (cementation, cyclic loading, and wet-dry seasonal changes). Such complexities in the geological setting of a soil deposit are usually not known with any degree of certainty, therefore the stress history is commonly evaluated through one-dimensional consolidation tests on undisturbed samples.

The effective preconsolidation stress ($P_c' = \sigma_{vmax}' = \sigma_p'$) is the definitive yield point from plotting the void ratio vs. log effective stress (e-logσ_v') from one-dimensional consolidation testing. The magnitude of yield stress is delineated by graphical techniques, such as procedures reviewed by Grozić et al. (2003).

The degree of preconsolidation can be represented in normalized form, termed the overconsolidation ratio (OCR = σ_p'/σ_{vo}'). If a simple unloading mechanism or reduction in effective stress state has occurred in the deposit, then the overconsolidation difference (OCD = $\sigma_p' - \sigma_{vo}'$) or prestress ($\Delta\sigma_v'$) can be useful to represent these changes (Locat, et al. 2003). The advantage here is that OCD will be a constant in an erosional soil profile, whereas the associated profile of OCR will decrease with depth.

One means to address the various modes is via the MIT SHANSEP approach (Ladd, 1991; Ladd & DeGroot, 2003) that assigns an appropriate mode value to each relevant part of the stability surface failure arc (Figure 8). The value of $S = (s_u/\sigma_{vo}')_{NC}$ is experimentally found in the laboratory by extensive testing for large projects (Lunne & Andersen, 2007), else on small projects is estimated from empirical relationships. The effect of overconsolidation is considered from:

$$(s_u/\sigma_{vo}')_{NC} = S \cdot OCR^m \tag{4}$$

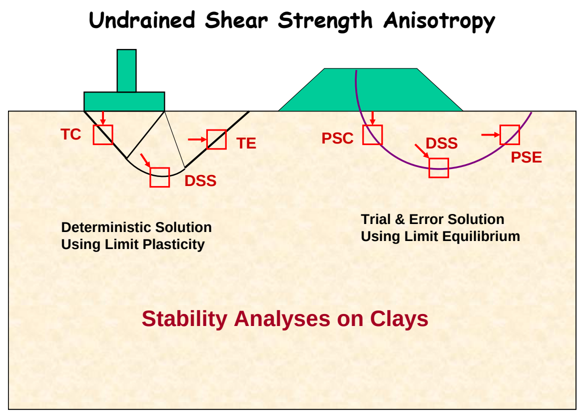

Fig. 8. Applicability of strength modes to foundation and embankment stability

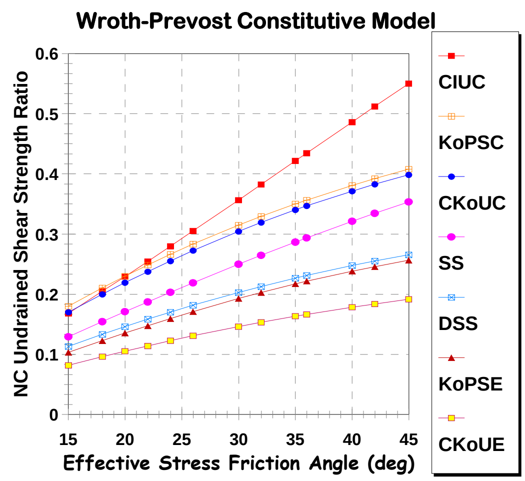

Fig. 9. Undrained shear strengths (S) from Wroth-Prevost hybrid constitutive model.

where the exponent m can be determined experimentally and has been found to be on the order of 0.8 ± 0.1.

A more fundamental avenue is afforded through critical-state soil mechanics (CSSM). In this case, a constitutive model can provide the hierarchy of the strength modes (i.e., S values), such as presented for the Wroth-Prevost formulations given in Figure 9 or the Ohta et al. (1985) model in Figure 10.

In CSSM, the exponent m is actually derived from theoretical considerations and thus, $m = \Lambda \approx 1 - C_s/C_c$ where C_s = swelling index and C_c = compression index. In the CSSM version, equation (4) can be used with the appropriate shear mode for S selected and

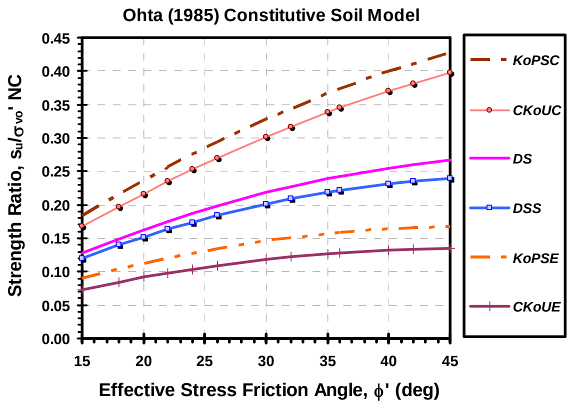

Fig. 10. Undrained shear strengths (S) for NC clay from Ohta constitutive model.

Λ taken to be 0.8 for many practical purposes (e.g., Kulhawy & Mayne, 1990). Additional considerations such as strain rate effects can also be included if desired.

The CSSM and SHANSEP procedures allow for a distinct and separate evaluation of each of the various strength modes, therefore helping to sort out the confusion and scatter normally found in s_u plots. The role of the in-situ testing therefore becomes focused on the assessment of the stress history profile.

If the geotechnical problem has not yet been fully established, then the simple shear (SS) mode is probably a best value to hone in on. The SS represents a middle representative value of the compression, shear, and extension modes. In the SS, pure shear is applied to top and sides of the specimens, whereas in commercial system, a direct simple shear (DSS) is a close approximation. For OCR=1, the corresponding S can be obtained from:

$$S_{DSS} = (s_u/\sigma_{vo}')_{NC}[DSS] = \tfrac{1}{2}\sin\phi' \qquad (5)$$

Available data from a variety of clays support (5), as shown in Figure 11. These data also follow the general trends found from empirical relationships between S_{DSS} and liquid limit (Fig. 12) and with plasticity index (Fig. 13).

As a consequence, these graphs are at odds with the notion that ϕ' decreases with plasticity index, as suggested by old studies based on remoulded clays and artifically produced soils. In fact, for natural clays, the effect of clay constituents and mineralogy play an extremely important role in defining the frictional characteristics of geomaterials (see Locat, et al. 2003).

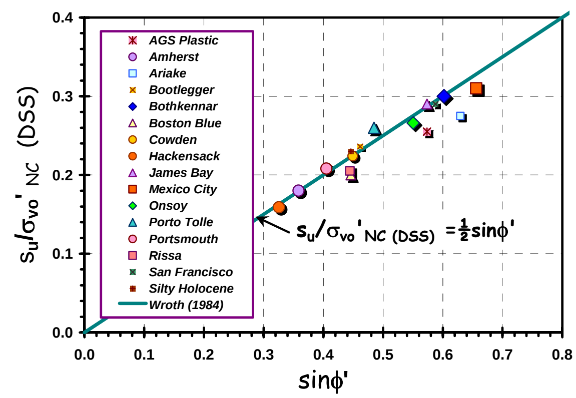

Fig. 11. Undrained ratio S_{DSS} with effective ϕ' (proposed by Wroth, 1984).

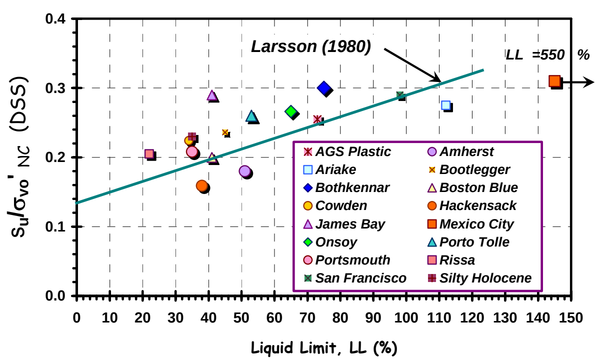

Fig. 12. Undrained ratio S_{DSS} with liquid limit (as suggested by Larsson, 1980)

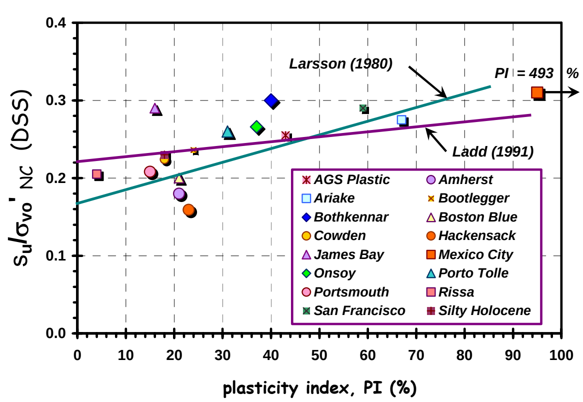

Fig. 13. Undrained ratio S_{DSS} with plasticity index (as suggested by Ladd, 1991)

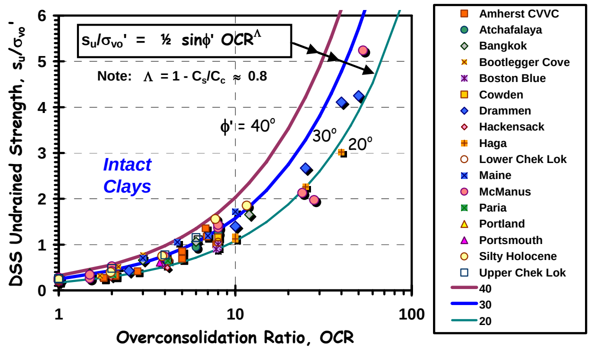

Fig. 14. Undrained ratio S_{DSS} with OCR and effective friction angle (after Mayne 2005)

Combining eqns (4) and (5), the general expression for the undrained shear strength corresponding to the SS mode for a range of overconsolidation ratios is given by:

$$(s_u/\sigma_{vo}')_{OC}[DSS] = ½ \sin\phi' \, OCR^{\Lambda} \qquad (6)$$

where $\Lambda \approx 1 - C_s/C_c \approx 0.8$ for clays of low to medium sensitivity, increasing perhaps to 0.9 for sensitive and structured soils.

6. PIEZOCONE PROFILING OF σ_p'

The uniqueness of the preconsolidation stress (σ_p') helps to focus the results of the in-situ and laboratory testing programs towards one single common goal. All results are consistent to defining the level of prestress ($\Delta\sigma_v'$) or OCD, or the site-specific profile of σ_p' with depth. A clear and certain benchmark reference for obtaining σ_p' is assured through one-dimensional consolidation testing.

The link between the soil stress history (in terms of OCR) and piezocone measurements has been established via a hybrid cavity-expansion and critical-state theory (Mayne, 1991). In this formulation, the OCR can be mapped using either the normalized cone tip resistance, $Q = (q_t - \sigma_{vo})/\sigma_{vo}'$, or alternatively with the normalized penetration porewater pressure, $U^* = \Delta u/\sigma_{vo}'$. Both expressions require a knowledge of the effective friction angle (ϕ'), undrained rigidity index ($I_R = G/s_u$), and $\Lambda = 1 - C_s/C_c$, where G = shear modulus. The friction angle is expressed in terms of $M = 6 \cdot \sin\phi'/(3 - \sin\phi')$. The expressions are given in Figure 15.

Of additional interest is to combine those two expressions based on Q and U* to obtain an independence from I_R, thereby relating OCR directly to the normalized piezocone parameter $Q_u = (q_t - u_2)/\sigma_{vo}'$ and only two soil parameters: ϕ' and Λ (Burns & Mayne, 2002).

For a practical approach, two levels of simplification can be taken (Fig. 15). First, the adopted value of $\Lambda = 1$ can be used to reduce these three expressions and represent the preconsolidation stress in terms of simply M and I_R, net cone tip resistance, excess porewater pressure, and effective cone tip resistance. Secondly, representative "typical" values of $\phi' = 30$ (i.e., M = 1.2) and $I_R = 100$ can be adopted to provide direct first-order expressions:

$$\sigma_p' \approx 0.33 \, (q_t - \sigma_{vo}) \qquad (7a)$$

$$\sigma_p' \approx 0.54 \, (u_2 - u_o) \qquad (7b)$$

$$\sigma_p' \approx 0.60 \, (q_t - u_2) \qquad (7c)$$

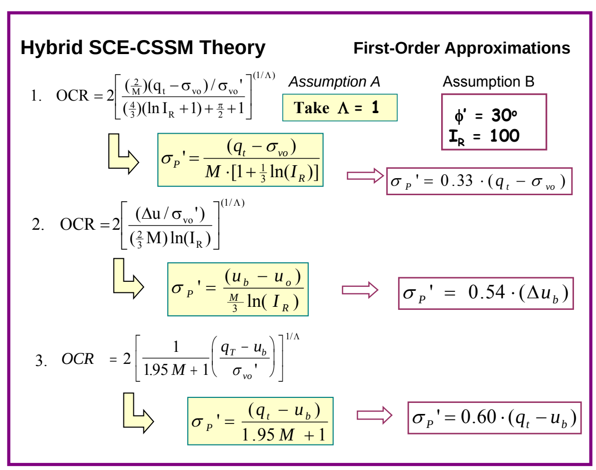

Fig. 15. Simplification of cavity-expansion and critical-state formulations to obtain first-order estimates on preconsolidation from piezocone.

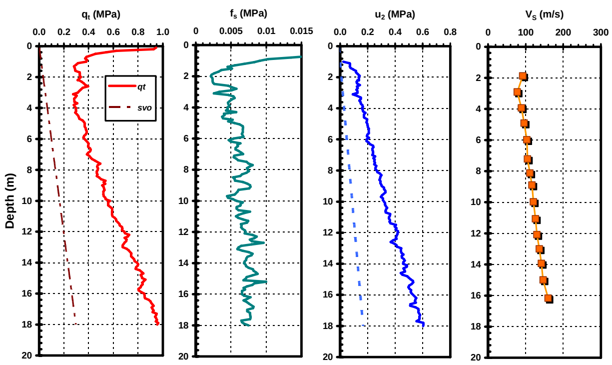

Fig. 16. Representative piezocone sounding from Bothkennar soft clay site, U.K. (data from Nash, et al. 1992).

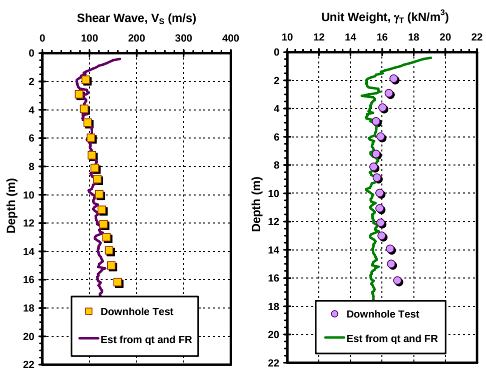

Fig. 17. Estimated shear wave velocity and unit weight profiles at Bothkennar.

An illustrated example of the approach is afforded from laboratory and in-situ data taken from the Bothkennar soft clay site. Figure 16 presents results of a seismic piezocone test (SCPTu), re-digitized from Nash et al. (1992).

In order to capture the stress history profile, an evaluation of overburden stresses will be necessary. In the case shear wave data are available, the results can be used to estimate the soil total unit weight from the expression (Mayne, 2001; Mayne 2007b):

$$\gamma_T \text{ (kN/m}^3) = 8.32 \log(V_s) - 1.61 \log(z) \qquad (8)$$

where V_s = shear wave velocity (m/s) and z = depth (m). In the cases where downhole results are not available, the CPT readings might be used to provide a rough estimate of V_s, for example (Hegazy & Mayne, 1995):

$$V_s = [10.1 \cdot \log(q_t) - 11.4]^{1.67} \cdot (FR)^{0.3} \qquad (9)$$

where relevant units are: V_s (m/s), q_t (kPa), and FR = f_s/q_t = friction ratio as a percentage.

Using these expressions with the Bothkennar site data (Figure 17), the profiles of shear wave velocity and unit weight are seen to be slightly underestimated.

All three piezocone relationships given by Equation (7) can be used to provide separate assessments of the preconsolidation stress profile at Bothkennar. In this case, the redundancy is a good means to check the validity and reliability of the results. Figure 18 shows that a consistent and compatible profile of σ_p' is attained by all three piezocone parameters. The results are comparable to the laboratory series of consolidation tests, though perhaps a tad on the high side. Three types of consolidation test series are available here including: conventional incremental loading (IL), constant rate of strain (CRS), and restricted flow (RF) type. Moreover, the effects of sampling disturbance are to be considered in all laboratory testing. For consolidation testing, the tendency of lower sample quality is to reduce the apparent magnitude of σ_p' as well as decrease the apparent C_c (e.g., Lacasse et al. 1985).

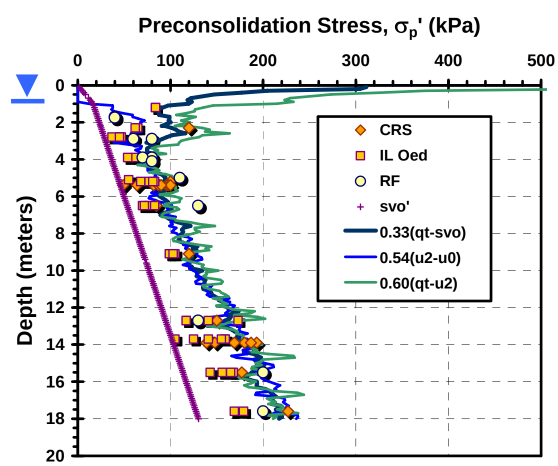

Fig. 18. Derived profiles of preconsolidation stress at Bothkennar.

As the results are rather similar in this case, a single stress history profile derived from (7a) can be used in further analyses. Figure 19 indicates the OCR profile using:

$$OCR \approx Q/3 \quad (9)$$

The final step for defining the family of undrained shear strengths for this soft clay is to utilize a set of normalized undrained strength ratios for each desired mode: $S = 0.30$ (triaxial compression), 0.23 (simple shear), and 0.13 (triaxial extension). Combined with the OCR interpretation from the CPTu, a consistent set of s_u profiles can be derived, as shown by Figure 20. The results can be seen to be comparable and reasonable with respect to lab-measured values on high-quality samples reported by Hight et al. (2003).

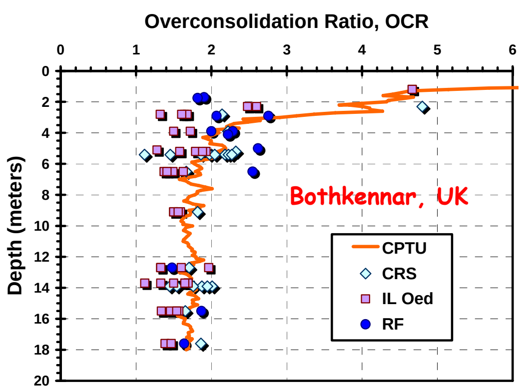

Fig. 19. Profile of OCR from consolidation and piezocone tests at Bothkennar

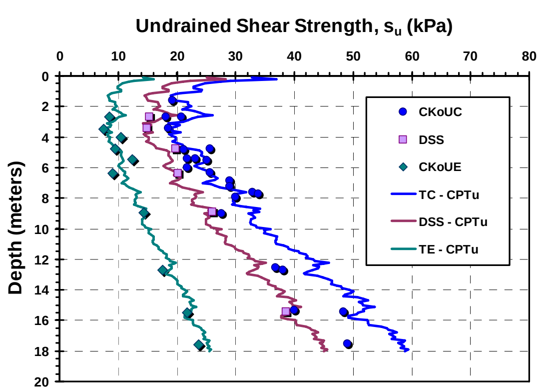

Fig. 20. Undrained strength profiles at Bothkennar for TC, SS, and TE modes (lab data from Hight et al. 2003)

7. CASE STUDIES

Three case studies involving in-situ testing at two port facilities and an offshore platform are presented to document the results. The clays include a very soft normally consolidated deposit (Brisbane), lightly overconsolidated clay (Troll), and overconsolidated Bootlegger Cove Formation (Anchorage).

7.1 Port of Brisbane

The development at the Port of Brisbane (PoB) involves filling in large areas of a bay to form new areas of land reclamation. The coastal nearshore is underlain by soft Holocene clays with low natural prestresses. Dredged sand is pulled from the seabed of the shipping lanes of the intercoastal waterways and deposited over the clays to form a working surface platform layer for new port facility expansion. Some trial areas are being investigated to utilize dynamic compaction to compact these sands prior to construction of on-site buildings and loading facilities.

Fig. 21. Large bay to be infilled with dredged sand to expand Port of Brisbane, Australia

Piezocone tests are routinely conducted in conjunction with the site exploration studies at PoB in order to evaluate the extent, thickness, and consistency of the soft clays that relates to the magnitude of expected settlements and rates of consolidation during reclamation operations. These underlying soft clays are forced into normally-consolidated states due to the added weight of the sands fills. A representative CPTu for PoB after infilling is shown in Fig. 22.

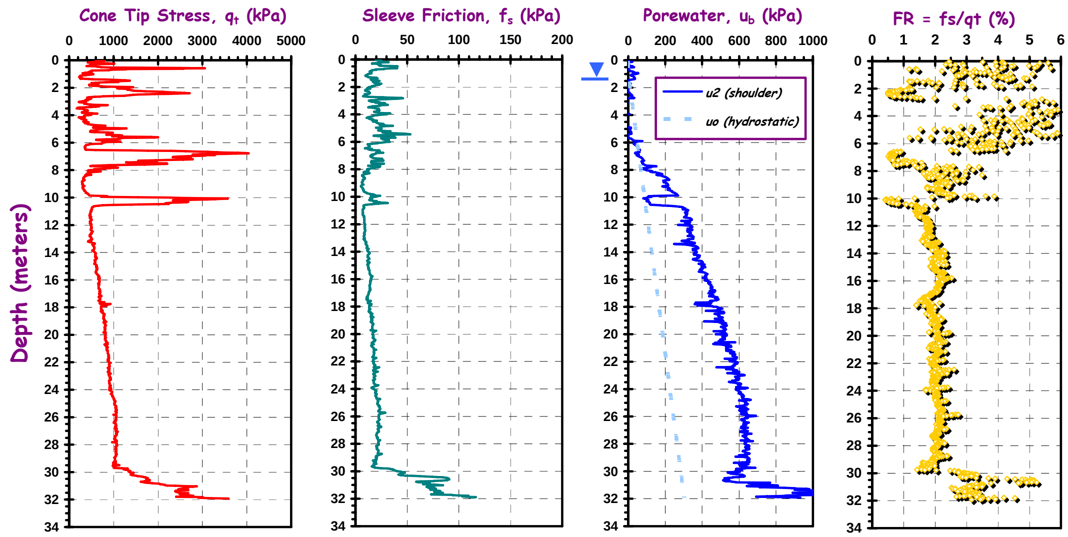

Fig. 22. Piezocone P379 after dredge sand placement at Port of Brisbane

The thickness of the sand fill can be seen to be about 10.5 m at this location, particularly clear in the signatures of the q_t, f_s, and FR plots. The soft clay layer is evident from depths between 10.5 and 30 meters by the uniform profile of readings in all channels.

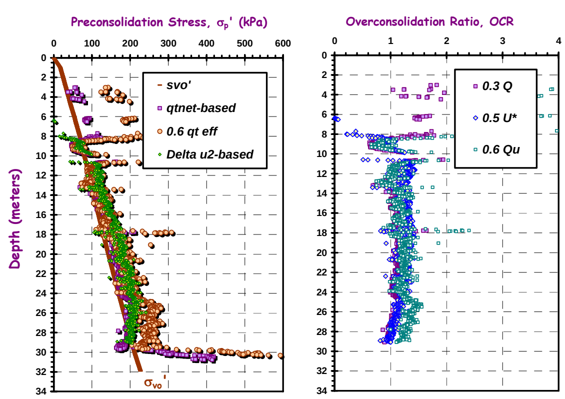

Fig. 23. Derived σ_p' and OCR for CPTu P379

The CPTu data can be post-processed to ascertain the degree of consolidation per all three parts of eqn(7). Figure 23 shows these results in terms of interpreted preconsolidation stresses and corresponding OCRs. All three methods show similar findings with nearly normally-consolidated clays throughout the profile. In the case of underconsolidated clays, apparent excess porewater pressures (and effective cone resistances) measured by piezocones may cause differences in the derived profiles per (7) due to elevated hydrostatic values (Tanaka & Sakagami, 1989).

Of course, the installation of piezometers, observation wells, and open standpipes, as well as piezocone dissipation phases, would be warranted to confirm and verify the degree of consolidation, yet the quick approach discussed herein offers some initial and preliminary results to the field engineers.

7.2 Troll, North Sea

The Troll platforms in the North Sea have seen quite extensive and thorough site investigation programs to detail the subsurface soil conditions for foundation design involving gravity structures. In addition to soil drilling, sampling, and laboratory testing, considerable exploration studies have been carried out by piezocone tests using the vessel *Bucentaur* (Fig. 24) which has special dynamically-operated field positioning systems (Amundsen, et al. 1985), as well as seabed deployment capabilities. Water depths at Troll are on the order of 330 m.

Fig. 24. The vessel Bucentar used at Troll (courtesy of Fugro Geosciences)

Various types of soundings have been conducted in the Troll fields using single element piezocones, including type 1 (midface) and type 2 (shoulder) positions as well as triple element soundings with simultaneous measurements of u_1, u_2, and u_3 (behind sleeve) porewater pressures readings (Skomedal & Bayne, 1988).

A representative type 2 sounding reported by Lunne & By (1989) is presented in Fig. 25 that indicates a top plastic clay layer (0 to 18 m) overlying a thicker lean clay stratum, within the final termination depths of 44 m. The upper clay has a sensitivity of around $S_t \approx$ 5 to 6 and PI ≈ 40% while the lower clay exhibits a $S_t \approx 2$ and PI ≈ 20%

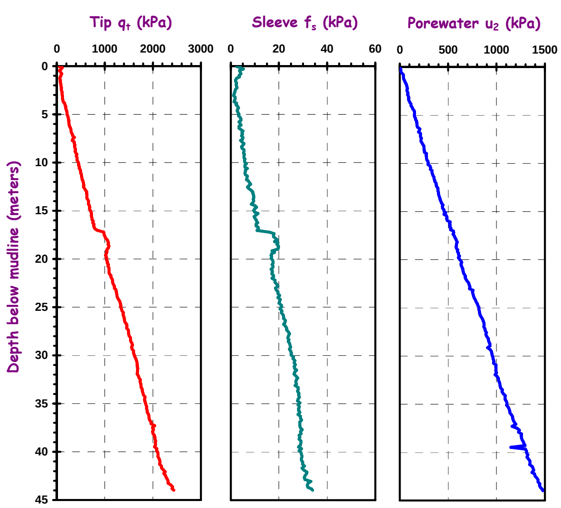

Fig. 25. CPTu 89-05 at offshore Troll site

Using eqn (7) to post-process these data provides a consistent interpretation of OCR ≈ 1.5 in the Troll clays that compares favourably with the reference values obtained from series of consolidation tests (Figure 26).

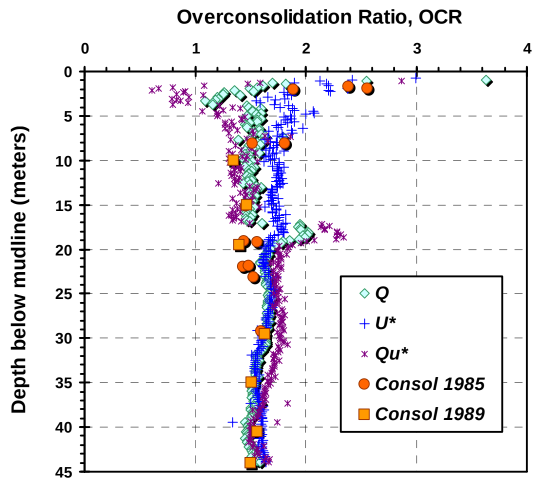

Fig. 26. Derived OCR profiles at Troll site

7.3 Port of Anchorage

The expansion of the Port of Anchorage in the Cooke Inlet, Alaska required a large investigation with soil borings and piezocone soundings that were conducted in a heavy tidal area. Low to high tides vary by up to 11 m elevation change in this region and this required use of a jackup platform (Fig. 27 and 28). The SeaCore was shipped in from the UK to use for this purpose.

The area is underlain by variable strata of gravelly sands and generally thick layers of silty clays of the Bootlegger Cove Formation (BCF). Lab testing on undisturbed samples included index, CIUC triaxial shear, direct simple shear (DSS), and one-dimensional consolidation tests (Mayne & Pearce, 2005). In addition to some 20 soil test borings, the field testing program included 58 piezocone soundings, as well as limited vane shear tests and downhole shear wave velocity profiles. A representative piezocone sounding from the investigation is shown in Figure 29.

Fig. 27. Jackup platform SeaCore during high tide conditions at Port of Anchorage

Fig. 28. View of jackup platform during low tide conditions in Cooke Inlet, Anchorage

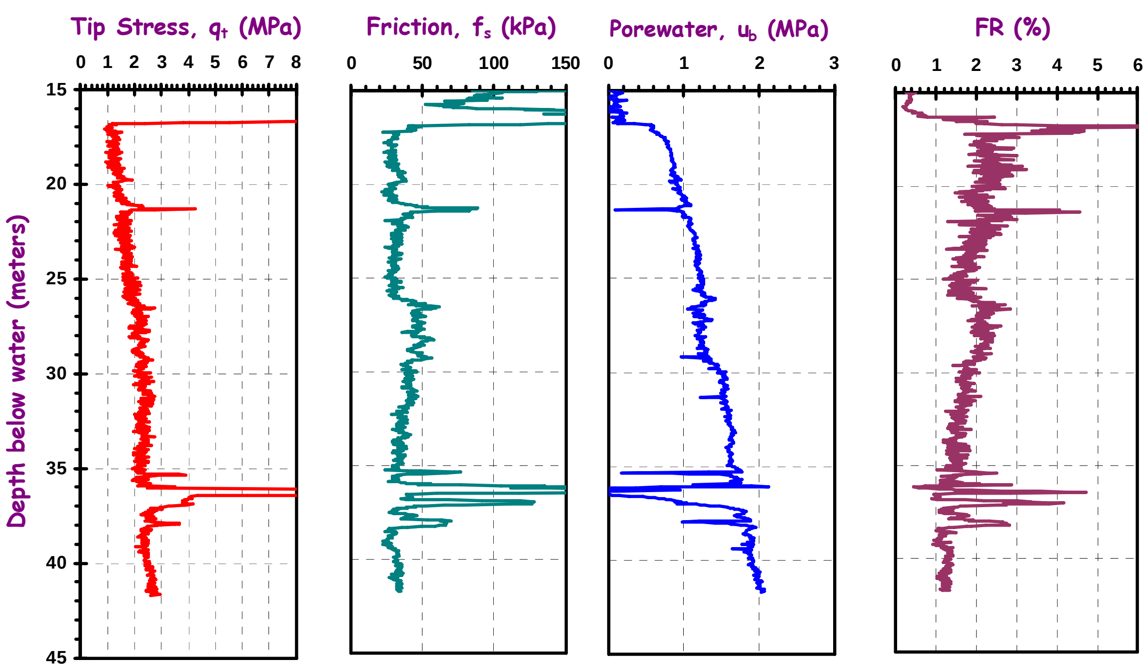

Fig. 29. Representative CPTu C-18 from Port of Anchorage site investigation program

The application of equations (7) to detail the stress history of Bootlegger Cove clay indicates moderately strong levels of prestress represented by a mean OCR = 450 kPa. Figure 29 shows the results from the CPT post-processing in general agreement with consolidation values obtained using the Casagrande method to define the σ_p' values from e-logσ_v' curves. Moreover, supplementary results from vane shear tests and V_s measurements can also be evaluated to indicate these levels of preconsolidation, as detailed by Mayne (2007a).

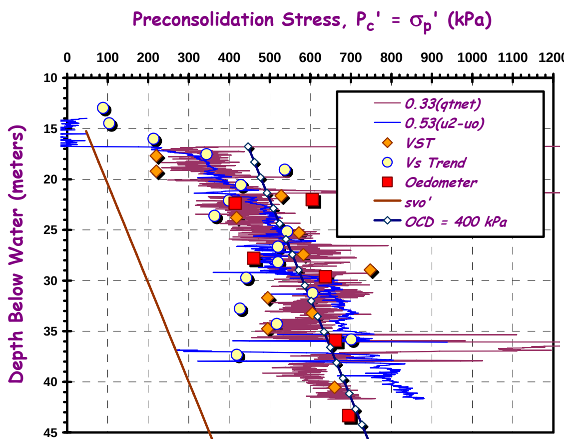

Fig. 29. Preconsolidation stress of stiff clays at Port of Anchorage

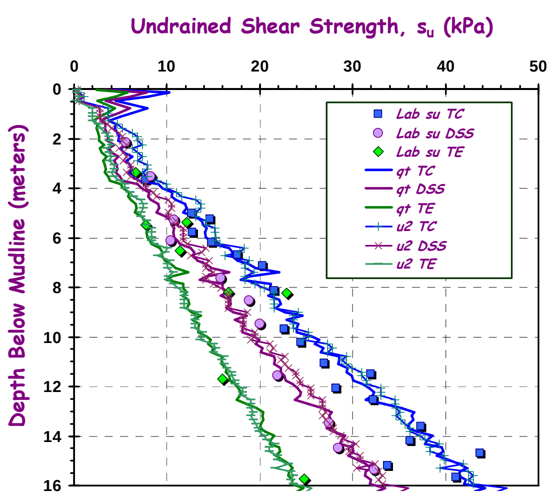

Fig. 30. Tri-modal profiles of undrained shear strengths from piezocone results at Troll.

8. DISCUSSION

When post-processing CPTu data, a focus on the derived stress history profiles (either in terms of preconsolidation stress σ_p' or prestress = OCD = σ_p' - σ_{vo}') is advantageous since a unique soil parameter is sought during the interpretation phase. When seeking a direct assessment of undrained shear strength from CPT results, a major difficulty lies in that a large suite of bearing factors (N_{kt}, $N_{\Delta u}$, and/or N_{qeff}) must be considered because so many different modes exist for s_u (i.e, CIUC, PSC, CK_0UC, DSS, PSE, CK_0UE, UU, UC, etc.).

Instead, the s_u is obtained as a second step process. After the in-situ test results are utilized for stress history profiling, then the s_u is found via SHANSEP (Ladd & DeGroot, 2003) or CSSM (Wroth, 1984) for the relevant mode(s) of s_u that are desired.

To illustrate the procedure, the results from the Troll site can be used. Here, the OCR profile appears rather constant with depth, likely due to ageing mechanisms. Fig. 30 shows three sets of s_u profiles corresponding to triaxial compression (TC), simple shear (SS), and triaxial extension (TE) modes. For each strength mode, two comparable profiles are presented based on the OCRs obtained from the net cone tip resistance (OCR ≈ 0.3 Q) and the other from measured excess porewater pressures (OCR ≈ 0.5 U*). A third profile can also be derived with equally good comparisons using OCR ≈ 0.6 Q_u.

A second example is afforded from the Anchorage program. Results from the laboratory series of CIUC and DSS testing are compared in Figure 31 with those derived using the OCR profiles obtained from CPTu interpretations with S = 0.32, 0.23, and 0.19 for TC, DSS, and TE modes, respectively. This approach allows for the direct consideration of strength anisotropy when conducting stability analyses involving embankments, walls, and foundations on clay subsoils. Most commercial stability packages now allow an option for consideration of strength anisotropy.

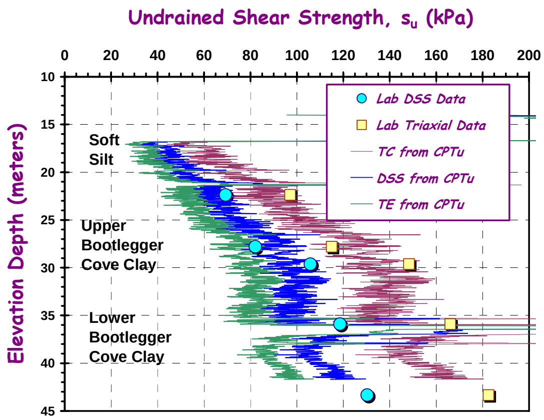

Fig. 31. Modal profiles of undrained strength for Bootlegger Cove clay at Anchorage.

9. SUMMARY

The use of piezocone testing for geotechnical strength characterization of clays in maritime projects is documented with three case studies involving port facilities in Australia and Alaska and an offshore North Sea site reported in the literature. The post-processing of CPTu results is best handled by first emphasizing the evaluation of soil stress history (OCR or OCD) which can then be utilized to profile various selected modes of undrained shear strength, such as plane strain compression, triaxial compression, and simple shear, as well as plane strain and triaxial extension loading.

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